DRAINAGE OF WATER FROM PAVEMENT STRUCTURES

Final Report

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1.0 INTRODUCTION

1.1 Overview

The structural capacity and integrity of a highway pavement is influenced to a significant degree by the drainability of, and average moisture levels in, the aggregate and soil materials underlying the asphaltic or concrete wearing surface, as well as the wearing surface itself. This relationship has been recognized for many years, and has been typically accounted for by the provision of special drainage facilities, such as edge drains, and/or increased pavement structure thicknesses. It is now acknowledged, however, that both pavement performance and longevity are always reduced when the drainage characteristics of the materials from which the pavement structure are constructed are insufficient to permit water to drain from them both quickly and nearly completely. That is, trying to solve the highway pavement subdrainage problem by the mere provision of subdrainage features and/or increased structural thicknesses is not enough to guarantee the long-term performance and serviceability of roadway surfaces. This is particularly disturbing when one also considers the incremental costs of design, construction, and maintenance that are associated with these types of solutions.

Since 1986, the AASHTO Guide for the Design of Pavement Structures (AASHTO, 1993) has included subsurface drainage parameters as essential ingredients in its recommended procedures for design of pavement structures. Within the design procedures, the drainage characteristics of the various construction materials from which the pavement layers are constructed are considered by introducing empirical drainage coefficients which modify the structural layer coefficients (for flexible pavements) and the load transfer coefficient (for rigid pavements). While it is recognized that the magnitudes of the empirical drainage coefficients are dependent on (1) the quality of drainage, i.e., the time required for the pavement to drain, and (2) the percent of time that the pavement structure is exposed to moisture levels approaching saturation, it is left to the users of the AASHTO Guide to determine the appropriate values of the drainage coefficients that should be used (AASHTO, 1993, p. I-5). The pavement design procedures are rather sensitive to the drainage coefficients in that small changes in them can have a significant influence on the required thicknesses of the various pavement structural components (J.K. Lindly, personal communication, 1994).

It is clear that the drainage coefficients which are used in the pavement design procedure, because of the dependencies noted above, are strongly related to environmental factors, and especially precipitation characteristics. Locally high groundwater tables and soil types making up the natural subgrade materials are also important environmental considerations. It is also evident that the drainage coefficients are dependent on the actual materials used for construction of subbase, base course, and/or drainage layers in a pavement structure. This is true not only because some construction materials are more permeable than others, and hence are capable of transmitting water more easily, but also because of the influence of the pore sizes in the various materials on the phenomenon of capillary retention, which prevents a certain fraction of the pores from draining, even after prolonged periods of time.

Because of variations in natural meteorological and soil conditions from one state to another (and even within a given state), and because of variations in the available and economical construction materials that are indigenous to various locales, it is necessary that determinations of suitable design values for the drainage coefficients in the AASHTO design procedures be accomplished on a regional basis. It is the intent of this report to present the findings of a study which has been directed to this issue for the State of Alabama. Flexible pavements only are considered, and recommendations of drainage coefficient design values are made for use in that state. Additional research is needed to identify suitable design values for rigid pavements in Alabama.

1.2 Background

For the purposes of the design procedure given in the AASHTO Guide for flexible pavements, the effects of subsurface drainage characteristics on the pavement design process are embodied in drainage coefficients denoted m_i, where the subscript i refers to a specific layer of either aggregate or asphaltic materials within the pavement structure. Numerical values of these coefficients, which have been recommended for use by AASHTO, are presented in Table 1.1. It may be seen from that table that the numerical values of the coefficients depend on the quality of drainage of the respective layer materials (i.e. whether drainage is excellent, good, fair, poor, or very poor) and on the percent of time that the materials in the pavement structure are exposed to moisture levels near saturation. The quality of drainage of a layer material is a measure of

TABLE 1.1

Recommended m_i Values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements
(Source: AASHTO, 1993)

	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
Quality of Drainage	Less Than 1%	1–5%	5–25%	Greater Than 25%
Excellent	1.40–1.35	1.35–1.30	1.30–1.20	1.20
Good	1.35–1.25	1.25–1.15	1.15–1.00	1.00
Fair	1.25–1.15	1.15–1.05	1.00–0.80	0.80
Poor	1.15–1.05	1.05–0.80	0.80–0.60	0.60
Very poor	1.05–0.95	0.95–0.75	0.75–0.40	0.40

TABLE 1.2

Relationship Between Quality of Drainage and Water Removal Times
(Source: AASHTO, 1993)

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very poor	(water will not drain)

the amount of time that is required for drainage from that layer to occur, and is quantified in Table 1.2.

The water removal times referred to in Table 1.2 are rather ambiguous in the sense that they do not specify how much of the water must be removed within the given time frames. In order to eliminate this ambiguity, it is usually considered that the times given in the table are associated with the time to 50 percent drainage. This is the amount of time that would be required, if the pavement section were initially saturated, for 50 percent of the drainable water to be removed from the pavement section. Note the use of the terminology "drainable water" here; as noted earlier, not all water in a porous medium is in fact drainable because of capillary retention. To illustrate and reinforce the point that is being made here, an extreme case might consist of one in which a pavement section "drains" very quickly (say in a few hours), but in which at the completion of "drainage" it is still nearly saturated.

It is clear from this discussion that there are at least two issues that need to be considered when evaluating the subdrainage characteristics of a pavement structure. The first of these issues is the rate at which drainage will occur, and is closely related to the hydraulic conductivities, or permeabilities, of the construction materials. Table 1.2 effectively accounts for hydraulic conductivity, but does so by considering the drainage time as a surrogate for it. The second type of issue that needs to be considered concerns the water retention and saturation characteristics of the pavement materials. These characteristics relate closely to the distribution of pore sizes in a material, and they also relate to the presence of high or perched groundwater tables. That is, they relate to the concern of much water remains in a material even after it has "drained", and with how frequently the material is exposed to internal moisture conditions approaching saturation. There is again some ambiguity here, because it is not entirely clear what is meant by "conditions approaching saturation", but presuming that this can be resolved, Table 1.1 accounts for this second issue through its indication of the effect of saturation levels on the drainage coefficients.

It is interesting to observe the sensitivity of drainage coefficient values obtained from Table 1.1 to the quality of drainage and saturation levels on which they depend. Consider, for instance, a layer that is saturated greater than 25 percent of the time and which has poor drainage so that its m_i value is 0.40. If the hydraulic conductivity of that material could be somehow improved so as to result in excellent drainage, then the drainage coefficient would become 1.20, a three-fold increase. On the other hand, if the percentage of time for which the material approached saturated conditions could somehow be reduced to less than 1, then the drainage coefficient would become about 1.00, or 2.5 times greater. Note that if a layer were to have excellent drainage, then the gain that would be realized by reducing the percentage of time of saturated conditions from 25 to 1 percent would only be a factor of about 1.15. Similarly, for a material having a saturation time of less than 1 percent, increasing the drainage quality from very poor to excellent would yield an increase of about 40 percent. These observations tend to reinforce the fact that both saturation times and drainage quality are important parameters in pavement design.

1.3 Objectives of Study

Recognizing that both the quality of drainage and the amount of time for which a pavement section has moisture levels near saturation are quantities that need to be studied on a regional basis, the Alabama Department of Transportation (ADOT) (formerly the Alabama Highway Department) issued a letter (AHD, 1992) soliciting proposals to study the pavement drainage problem and to make recommendations for its resolution within the State of Alabama. A contract was subsequently entered into between the ADOT and the University of Alabama at Birmingham (Dr. Robert Pitt, Department of Civil and Environmental Engineering). An additional participant in this project consisted of The University of Alabama (Dr. Rocky Durrans, Department of Civil and Environmental Engineering). Actual work on the project commenced in the late Spring of 1993, and is summarized by this report.

The main objective of this study was to develop a methodology which would allow Alabama highway pavement designers to select appropriate drainage coefficient values for flexible pavement designs. Field tests were conducted to determine values of critical parameters needed for the selection process, and to verify the process for Alabama conditions.

1.4 Scope of Work and Outline of Report

The scope of the effort that was proposed to be performed as a part of this project entailed four distinct, but related, tasks (Pitt, 1992). An outline and brief description of each of the specific tasks is contained in the following subsections. More detailed descriptions of the work performed and the results obtained for each task are provided in later sections of this report.

1.4.1 Task 1: Rainfall Analyses. It is clear that the percentage of time for which a pavement structure will have internal moisture conditions at or near the saturation level depends at least in part on precipitation characteristics. In most highway locations with flexible pavements, water supplied by precipitation enters the pavement structure by a combination of downward movement through cracks and/or joints in the wearing surface, and by infiltration through the asphaltic layer, though movement through cracks and joints is often the dominant mechanism. Where asphaltic surfaces are exceptionally porous, however, infiltration rates through the asphaltic layer itself can be extraordinarily high, and can approach or even exceed the rates frequently employed by rainfall-runoff modelers for porous soils. In other locations where the groundwater table is shallow, moisture in pavement structures may also derive from seasonal fluctuations in the water table, as well as from capillary rise.

Precipitation characteristics of primary importance for the pavement drainage problem consist of both the magnitudes and frequencies of precipitation events. The magnitude of an event may be expressed in terms of either the total depth or the average intensity of the precipitation. These quantities are often conditioned on, or expressed as a function of, the event durations as well. In the context of the pavement drainage problem, the magnitudes of precipitation events are significant in that they are the amounts of water that are available for potential infiltration and downward movement into the pavement structure. The frequency of rainfall events, as the term is used here, relates to the average number of them that occur within a given time horizon. Equivalently, this can also be interpreted in terms of the amount of time that elapses between the occurrence of sequential storm events (the inter-event times). This quantity is significant for pavement drainage in that it is the amount of time that is available for a pavement to drain before it is loaded again by the next storm event.

Results of rainfall analyses performed for the State of Alabama, and focusing on event magnitudes (depths and intensities) and inter-event periods, are summarized in Section 3.0 of this report.

1.4.2 Task 2: Infiltration and Percolation Testing. Paved surfaces such as roadways and parking lots are generally considered to be impervious, implying no infiltration, by runoff modelers. There is now a considerable amount of evidence, however, that paved surfaces can indeed experience significant amounts of water infiltration. Smooth, steep pavements tend to experience the smallest amounts of infiltration because of the rapidity with which surface drainage occurs, while rough, flat pavements tend to experience larger amounts of infiltration.

As already noted, the dominant mechanism in the movement of water through pavement surface layers is usually that of downward movement through cracks and joints in the wearing surface. Except in the case of exceptionally porous pavements, the phenomenon of infiltration through the pore spaces of the asphaltic wearing surface is often negligible in comparison. It is evident, therefore, that the primary factors controlling infiltration of water into pavement sublayers is the general condition of the pavement (cracking, porosity, etc.) and the spacing of joints in the pavement surface.

In this project, controlled infiltration and percolation tests of flexible pavements in Alabama were conducted using infiltrometers, and were performed over a range of pavement conditions. Results of these field tests provided a basis for the development of a relationship to predict the amount of infiltration/percolation that would occur in a given pavement. A description of the activities performed to accomplish these tests and to develop the predictive relationship are given in Section 4.0.

1.4.3 Task 3: Pavement Drainage Analyses. Excluding environmental factors such as precipitation amounts or groundwater depths, the drainability of a pavement structure is governed primarily by a combination of its geometry and the materials of which it is constructed. The predominant geometric variables are the length and slope of the drainage path, though the thicknesses of the base course and/or drainage layers can have some effect as well. As a general rule of thumb, and all other things being equal, pavements with steep slopes, short travel distances, and thick sublayers can be expected to be better drained than those with flat slopes, long travel lengths, and thin sublayers. The important aspects of pavement construction materials are their hydraulic properties. Since these materials are porous, and since flow in them may be under either fully saturated or unsaturated conditions, the hydraulic properties of interest relate not only to the ability of a material to transmit water (its hydraulic conductivity relationship), but also to its water storage and retention characteristics, such as its porosity, pore sizes, and residual saturation level.

Both field monitoring and mathematical modeling of moisture in pavement structures have been performed as a part of this project to study the internal drainage characteristics of pavement structures. Both 1-dimensional and 2-dimensional mathematical formulations of the problem have been addressed, as have both event-based and continuous simulations. Results of the mathematical models were compared with precipitation and soil moisture data which were obtained by installing monitoring instruments and data recorders at various sites in the Birmingham metropolitan area. Descriptions of these modeling and data collection activities are presented in Section 5.0.

1.4.4 Task 4: Implementation of Results. The performance of this project involved a number of meetings with the ADOT staff in Montgomery, at which progress was reported and strategic planning took place.

A summary of the findings and conclusions of this study are presented in Section 6.0, as are recommendations pertaining to the selection of drainage coefficients for use in design of Alabama roadways.

2.0 LITERATURE REVIEW

2.1 Overview

Whereas the subject of pavement drainage is one that involves the integration of knowledge and experience from both the transportation and water resources disciplines, literature pertaining to that subject may be found in a wide array of locations. The review presented here is not intended to be an extensive nor comprehensive one, but rather is included to establish the general framework within which the present project applies. Additional citations to the literature are given in other sections of this report as well and as the need or opportunity presents itself. The review presented here is given in several parts, each of which relates to one of the specific project tasks that were identified in Section 1.4.

2.2 Previous Precipitation Studies

The volume of literature dealing with the precipitation process, because of its importance to a broad field of inquiry and applications, is tremendous and could not possibly be reviewed in its entirety here. In order to restrict the scope of this review, it has been chosen to focus on those characteristics of the precipitation process which are thought to be of the greatest relevance to the problem of pavement drainage. In Alabama, where most precipitation occurs in the form of rainfall, the primary characteristics of interest are the total depths, durations, and average intensities of individual events, as well as the inter-event times. The variabilities of these characteristics from one location to another throughout the state are also of interest.

At least among civil engineers, the most widely known publications relating to precipitation are the U.S. Weather Bureau Technical Paper No. 40 (TP-40) (Hershfield, 1961) and the National Weather Service publication commonly known as HYDRO 35 (Frederick et al., 1977), which partially supersedes TP-40. Those publications are both rather dated, and a tremendous amount of additional precipitation data has been collected since they were published. The information presented in those publications is also of primary utility in terms of the concept of a "design storm", where one may be interested in the design of a hydraulic structure or conveyance system capable of handling a T-year runoff event. Because of that intended use of those publications for design purposes, their focus was on the presentation of the characteristics of precipitation extremes, i.e., on the characteristics of the most severe storm that is expected to occur within a given year.

The U.S. Army Corps of Engineers (Allen, 1991) employs the 2-year, 1-hour rainfall rate as the design criterion for the design of subsurface drainage systems. The AASHTO and FHWA guidelines for subsurface drainage system design employ the 1-year, 1-hour precipitation event.

In terms of the pavement drainage problem considered in this report, where the focus is on the estimation of average subsurface conditions as opposed to the actual design of subdrainage systems, one is more interested in normal rather than extreme rainfall conditions, and hence the TP-40 and HYDRO 35 publications have limited utility. What one needs for the type of pavement drainage problem considered here are descriptors of individual precipitation events (as opposed to just the annual maxima), as well as the elapsed times between the events. Data of this nature, which can be used as a basis for the development of either average descriptors or more complicated stochastic models, such as that presented by Lytton et al. (1990), can be obtained on CD-ROM diskettes from EarthInfo, Inc., Boulder, Colorado, and from the NOAA/NWS National Climatic Data Center, Asheville, North Carolina. Other private and public organizations are also involved in precipitation data collection efforts, but usually only to support their own missions.

In Alabama, there seems to be a relative dearth of studies that have been conducted to specifically address the types of precipitation properties needed for this project. One exception to this is that the University of Alabama at Birmingham has extensively evaluated Birmingham precipitation patterns over the period from 1970 to 1990. A result of these studies is that the probability distributions of rainfall depths, intensities, and inter-event periods have been estimated for each month in the Birmingham area. The present study extends the scope of that effort to span the entire State of Alabama.

2.3 Previous Infiltration/Percolation Studies

Unfortunately, impervious area runoff loss estimates, and especially pavement infiltration, are assumed to be much more accurate than warranted. When extensive field studies have been conducted simultaneously with modeling efforts, major differences in "actual" and modeled infiltration parameters have been noted (Pitt, 1987). Current prediction methods used to estimate the amount of water infiltrating into pavements have serious problems.

When rain falls on an impervious surface, most of it will normally flow off the surface and contribute to surface runoff. The remainder of it will be "lost" in various ways, including: interception by overhanging vegetation before it reaches the ground surface (not likely important for paved highway surfaces); evaporation caused by the heat of the ground surface and other surroundings; depression storage, where the water is caught in surface depressions, such as potholes, and/or is retained on the surface by surface tension effects, and is later infiltrated and/or evaporated; and infiltration of the water into the pavement. These losses are primarily associated with the initial portions of a rainfall event and are termed initial abstractions. Surface runoff begins after the initial abstractions have been satisfied. Infiltration through the pavement surface, as well as through cracks and/or joints in the pavement, continues as long as does the presence of free water at the pavement surface unless it is halted by the complete saturation of the pavement above some sort of impermeable barrier. Infiltration through pavement surfaces is assumed by most rainfall-runoff modelers to be zero, but this infiltration is the major source of water affecting the subsurface characteristics of pavement structures.

2.3.1 Initial Abstractions. Brater (1968) summarized values of initial abstractions that have been used for most rainfall-runoff modeling studies. Tholin and Kiefer (1960) suggested initial abstraction values of 1.6 mm for pavements, and Viessman (1966) recommended initial abstractions ranging from 1.0 to 2.5 mm for small paved areas. Aron (1982) reported that the Denver Regional Council of Governments used initial abstraction values of 2.5 mm for large paved areas. Since these initial abstractions do not significantly affect computed peak flow rates when they are used in rainfall-runoff modeling efforts, they have usually not been evaluated in much detail.

Falk and Niemczynowicz (1978) measured initial abstraction values ranging from 0.13 to 1.75 mm for paved surfaces. The lowest value was for a site having little traffic, while the largest value was for a site having the "most complicated geometry" with high traffic volumes and deep pools of water along the gutter during rainfall. They also found a correlation between slope and initial abstraction. Lazaro (1979) reported that depression storage might best be estimated by calculating actual volumes for small incremental areas and surface roughness heights.

Pitt (1987) directly measured initial runoff losses during special street water infiltration tests in Toronto and from monitoring many rainfall events at two large paved areas in Milwaukee. He also directly determined surface depression storage by measuring surface roughness and slope. The directly measured depression storage and the initial abstraction values agreed well, indicating that depression storage was the most important initial loss mechanism. Values estimated were also used with different surface slopes to estimate depression storage values for flat to steep pavements.

2.3.2 Evaporation Losses. Flash evaporation occurs when rainfall strikes a hot surface and evaporates on contact, or evaporates within the first few minutes after falling as it travels to a drainage system inlet. Longer term evaporation, as well as infiltration, is responsible for the depletion of water held in depression storage.

Diniz (1980) reported a peak evaporation rate of about 20 mm/hr for Austin, Texas. This peak evaporation rate occurred only for a short time during the early afternoon and decreased to nearly zero during the night. Grimmond et al. (1986) and Grimmond and Oke (1986) reported a total peak evaporation potential of about 5 mm/day, and a typical evaporation rate of 1 to 3 mm/day for a Vancouver urban study area. Only about 0.3 mm, or 3 percent, of the rainfall was lost to evaporation during a typical 3 hr - 10 mm rainfall event.

Evaporation as a direct component of initial abstractions may be small, but Diniz (1980) reported that evaporation may be a significant loss mechanism of ponded water after a storm, especially in arid areas. Evaporation can also play an important role in drying out saturated pavements.

2.3.3 Infiltration into Pavements. Paved surfaces are usually considered to be impervious, implying no infiltration, by rainfall-runoff modelers. There is now a considerable amount of evidence, however, that paved surfaces can indeed experience significant amounts of infiltration. Falk and Niemczynowicz (1978) found that smooth paved surfaces had the lowest losses, excluding depression storage (about 0.2 percent of the total rainfall depth), while poorly maintained paved surfaces had the largest losses (about 7 percent of the total rainfall). They therefore concluded that these other losses were mostly due to infiltration through the pavement. Pratt and Henderson (1981) found that infiltration through the joints between concrete pavement sections and along the drainage gutters was the principal mechanism in runoff losses.

Cedergren (1974) extensively studied and analyzed infiltration through pavement and through pavement cracks in highway and airport pavements. His studies were directed towards methods to encourage water that had infiltrated through pavement surfaces to pass through the pavement base layers. Transportation engineers are constantly troubled by failures of pavement surfaces because of inadequate drainage of the underlying layers. Cedergren found that the compacted pavement bases typical of most U.S. highways have very little permeability and hence offer little chance of draining well between rainfall events.

Cedergren (1974) also conducted infiltration experiments along pavement cracks. He found that crack sealing procedures were ineffective and that substantial pavement seepage was quite common both during and for up to 20 hours after rainfall events. He measured infiltration rates through typical sealed joints of about 20 mm/hr (with pavement joints located about every 8 meters). He also examined infiltration through typical pavements. Measured rates ranged from nearly zero for new, well-sealed pavements or older pavements that had been overlaid many times, to a few hundred feet per day (about 3 mm/s) for unsealed asphaltic concrete mixtures. Ridgeway (1976), Moulton (1980), and Markow (1982), as well as Lytton et al. (1990) and the U.S. Army Corps of Engineers (U.S. Army, 1988) have suggested various equations which may be used to estimate infiltration rates for subdrainage system design purposes. A summary of these equations is presented by Allen (1991).

Singh and Buapeng (1977) found that errors in estimates of infiltration may be large and may therefore be responsible for major errors in runoff predictions and pavement drainage rates. One of the possible sources of errors is the general lack of consideration of the apparent relationship between infiltration rate and rainfall intensity. Hawkins (1982) and Kumar and Jain (1982) recognized that infiltration rates vary with the rainfall intensity; the higher the rainfall intensity, the higher the infiltration rate. However, few infiltration estimation procedures account for this relationship. Hawkins reported that the rainfall intensity effects on infiltration have not been observed during rain simulator experiments because almost all rainfall simulations have been conducted within a relatively narrow range of intensities, usually near 75 mm/hr.

Pitt (1987) conducted many infiltration tests on roads with flexible pavements to determine the significant factors that affected water infiltration through the pavements. The factors investigated included pavement material, texture and condition, and rainfall intensity and duration. It was found that both pavement condition and rainfall intensity were the most important factors in controlling the infiltration rate for the different pavement sites investigated. The total rainfall depth was the determining factor related to the time at which the pavements would become saturated and infiltration would effectively halt. Pitt also observed that infiltration rates on large paved parking areas were substantially less than those observed on typical city streets, but this observation was probably because of the geometry of the pavement structure. When the initial abstractions become satisfied and the pavement becomes saturated, all additional rainfall apprears as direct surface runoff.

2.4 Previous Pavement Drainage Studies

Water that infiltrates into a pavement either through cracks and joints, or through the asphaltic layer itself, will move through the various layers making up the pavement structure and the underlying natural subgrade materials. The actual movement of the infiltrated water is quite complicated as there are several different layers of porous media involved, the flow is often unsaturated in some or all of the layers, and the flow may in some cases be affected by thermal conditions and evaporation.

The literature on flow through porous media is extensive, and has been contributed to by a wide array of professional disciplines. Civil engineers, which no doubt make up the majority of transportation engineering officials, are exposed to flow through porous media primarily through studies in soil mechanics and groundwater hydrology. It is perhaps unfortunate, however, that the emphasis in these studies is usually on saturated flow, where the voids in the porous medium contain water only. The phenomenon of unsaturated flow, where the voids contain both water and air (which are immiscible fluids and do not mix), is much more relevant to the problem of pavement drainage, but also represents a situation that is much more difficult to treat on an analytical basis. The primary contributors in this area have come from the fields of petroleum and agricultural engineering, as well as those of agronomy and soil science.

One of the earliest studies performed to study the movement of water in pavement structures was performed by Cedergren (1956), who applied classical flow-net types of techniques to solve idealized problems of steady-state flow in saturated soils and roadway bases. Casagrande and Shannon (1951) addressed the movement of moisture in pavement base courses as a transient flow problem. They assumed that the flow in base courses occurred under saturated conditions and with a linear piezometric surface. Based on their work, the time required for a base course material to attain 50 percent drainage may be expressed as

$$t = \frac{n_e D^2}{2880 K H_0} \tag{2-1}$$

where t is the drainage time in days, n_e is the effective porosity of the base material, K is the hydraulic conductivity of the base material in ft/min, D is the pavement width, or flow travel distance, in feet, and H_0 is the vertical distance, in feet, from the bottom of the base layer at the pavement edge to the top of the base layer at the pavement centerline. The effective porosity is defined as the ratio of the drainable volume of the pores in the base to the total volume of the base. The effective porosity is less than the actual porosity of the base material. Both the AASHTO and the Corps of Engineers (Allen, 1991) have continued to employ Equation (2-1).

Liu et al. (1983) developed a pavement drainage model based on the assumption that the piezometric surface was parabolic instead of linear. They also considered the case of a permeable subgrade underlying the base course by combining a simplified one-dimensional model for vertical infiltration into the subgrade with a two-dimensional model for the base course. Because of this additional complexity, numerical methods had to be used to solve the flow equations. A computer program to perform this task, and its application to a study of moisture effects on pavement structures was presented by Pufahl et al. (1990).

The actual movement of moisture in pavement structures frequently, if not nearly always, occurs under at least partially unsaturated conditions, where the presence of air in the soil and aggregate voids, and the consequent suction forces arising due to capillarity, tend to inhibit the moisture movement. Because of this retarding effect, the use of procedures based on the assumption of saturation can result in grossly misleading results. Wallace (1977) may have been the first to acknowledge this and to attempt to model the unsaturated flow process in pavement structures. Wallace assumed a grossly simplified cross-sectional geometry consisting of an impervious subgrade and a perfectly sealed and flat pavement slab. The sole source of water to the base layer was through unsealed shoulder areas. Dempsey and Elzeftawy (1977) presented a model for predicting moisture conditions in pavement sections for non-isothermal conditions. An implicit finite difference scheme was developed to solve the governing partial differential equations.

Espinoza et al. (1993) have also developed a numerical model for solution of the unsaturated flow problem in pavement bases. They considered more than a single soil layer in which flow could take place, but they neglected the influences of heat effects. They concluded that procedures such as those used by the AASHTO and the Corps of Engineers (Eqn. (2-1)), which do not take into consideration the unsaturated flow phenomenon, tend to lead to an overestimation of the drainage efficiency of a given roadway base. The net result is that pavement designs based on the use of Eqn. (2-1) may not attain the desired drainage characteristics.

McEnroe and Zou (1993) have developed two numerical models for evaluation of the unsaturated flow problem in pavement bases. The first of these assumes that the base is initially saturated, and simulates the drainage of the layer. The second model, which is a generalization of the first, is continuous in nature and can be used to simulate both the drainage and infiltration processes over time periods spanning several years. Either historically observed or synthetically generated rainfall time series are used as a model input, and probability distributions of average saturation levels in the base are output for each month of the year. Both models neglect heat effects, and both consider a single base course layer only.

The U.S. Geological Survey (Jeffcoat et al., 1992) has prepared a report on the effectiveness of highway edgedrains, based on an experimental study performed in 10 states. Jeffcoat et al. measured precipitation, edge drain outflow rates, subsurface temperatures, piezometric water levels, and soil moisture levels. They also performed tracer tests which could provide information on the travel times through the subsurface materials. In only one case, however, was dye detected in the edge drain discharges. Perhaps the most significant conclusion reached by Jeffcoat et al. is that most of the subsurface movement of water is "piping" through voids and channels that develop within the materials making up the pavement section. This conclusion was supported by the observation at virtually every site tested that the edge drain outflow response to precipitation was almost immediate. If the water were truly percolating through the subsurface soils and aggregates, then a delayed response would be expected. Their conclusion was also supported by the inability to detect dye in the edge drain discharges (except in one case), and by frequent observations of pumping of both water and fines, as well as pavement slab corner "punch-out" in rigid pavements.

White and Ahmed (1991) also performed an instrumentation study similar to that of Jeffcoat et al. (1992), but concentrated on a single site in central Indiana. Their general conclusions were essentially the same as those of Jeffcoat et al., but a delay of about 1 day was observed between the edge drain outflow response and precipitation. It is not clear why this difference was observed, but it is likely due to differences in the hydraulic properties of the subsurface materials.

2.5 Design of Pavement Drainage Facilities

There are in existence a number of fairly comprehensive documents which deal with the design of pavement drainage facilites, including such issues as rainfall and pavement infiltration estimation, and the sizing and appurtenances associated with subdrainage collection systems. Notable among these are a FHWA document entitled Highway Subdrainage Design (Moulton, 1980), and an Army Corps of Engineers document entitled Subsurface Drainage of Pavement Structures (Allen, 1991). Design information is also provided in the AASHTO Guide for the Design of Pavement Structures (AASHTO, 1993). The FHWA also has a Drainage Analysis and Modeling Program (DAMP) (Carpenter, 1990), which is a microcomputer implementation of the procedures set forth by Moulton (1980).

In Alabama, previous work related to the establishment of design parameters for longitudinal geotextile-lined subdrainage systems has been conducted at The University of Alabama by Ball et al. (1979).

3.0 RAINFALL CHARACTERISTICS IN ALABAMA

3.1 Overview

The main objective of this portion of this pavement research project was to gain insight about the variation of rainfall depth, antecedent period (or previous dry period), and intensity of rains throughout Alabama. The first phase of the analysis involved performing a broad analysis of yearly and monthly rain depths throughout Alabama and portions of surrounding states to identify a typical and representative rainfall year for further analyses. The second phase involved analyzing this typical rainfall year, on an hourly basis, to show how rainfall depth, dry period between rains (antecedent period), average rainfall intensity, and peak rainfall intensity varied throughout Alabama.

The importance of rainfall for the evaluation and design of highway pavement structures was discussed previously in Sections 1.4.1 and 2.2. AASHTO (1993) recommends that the percentage of time that the pavement drainage layer is exposed to near saturated conditions affects the m_i factor used to modify the structural layer coefficients of untreated base and subbase materials in flexible pavements (see Table 1.1). The rainfall return period and drainage time will affect the "near-saturation" criteria. If it requires 10 days to drain a pavement to the 50% level, but rains occur every 3 to 5 days (likely in most of Alabama), then the pavement will remain near-saturated most of the time. However, if the pavements drain within six hours, then the percentage of time at critical moisture conditions would be much less.

Table 3.1 is a summary of the rain conditions for 1976 in Birmingham. The year 1976 was found to have typical rainfall characteristics throughout the state, as will be discussed later. The minimum inter-event period is defined as six hours (0.25 day) with no rain. Therefore, rain periods separated with fewer than six hours are considered as the same rain event.

Inter-event period analyses set the maximum time available for pavement drainage. Most of the inter-event periods are less than 3 days, and 90 percent are less than 7 days. Only two rains during 1976 in Birmingham had inter-event periods of more than ten days. Statistical analyses of these inter-event data found no significant trends over the year: no months or seasons had significantly longer or shorter inter-event periods.

As noted in the Section 4, small rain depths are capable of saturating typical pavement structures. Rains of 20 mm (0.8 inch), or greater, occurred about 25 percent of the time in Birmingham during 1976. Smaller rains are much more common: 10 mm (0.4 inch) rains, were exceeded about 40 percent of the time. The largest rains occurred during the spring months of March through May, while the late simmer and fall months of September through November had much smaller rains.

TABLE 3.1

Birmingham Rain Characteristics for 1976

	Total Rain (in)	Number of Rains	Range of Inter-event Periods (days)	Average Inter-event Periods (days)
January	4.13	9	0.25 to 3.8	2.3
February	1.84	8	0.33 to 6.0	3.1
March	14.07	14	0.25 to 12.0	2.1
April	2.14	6	1.9 to 7.3	4.4
May	8.83	10	0.25 to 5.5	1.9
June	2.80	9	0.33 to 7.3	3.3
July	4.93	11	0.42 to 8.6	2.5
August	4.74	10	0.25 to 8.0	2.9
September	3.54	15	0.25 to 6.8	1.6
October	1.60	7	0.25 to 7.8	3.5
November	2.28	7	0.42 to 11.8	3.6
December	4.33	6	2.6 to 7.0	4.4
Total Annual	55.23	112	0.25 to 12.0	2.7

The average rain depth is about 12.5 mm (0.5 inches). In many U.S. locations, the average inter-event period is much greater than three days. These other areas could therefore experience longer pavement drainage times and still have many fewer annual hours at saturated conditions. Therefore, typical U.S. conditions would lead to very misleading results if applied to Alabama. Therefore, specific local rainfall analyses are needed when examining pavement drainage conditions, especially where the rainfall is so unusual, as in Alabama.

These analyses show that it is very important to consider the saturation period directly and not an arbitrary drainage time when selecting drainage conditions. As an example, it rains during about 15 percent of all hours during a typical year in Alabama. In most areas of the U.S., rain occurs only during about 5 percent of the yearly hours. There are about 100 different rains per year in the state, and the average total rain depth is about 1300 mm (50 inches). Most areas of the U.S. have substantially less rainfall and fewer individual rains. The average rain inter-event period is about three days in Alabama, but it can also, but rarely, be substantially longer. If pavement drainage requires more than three days, then pavement will remain saturated most of the time.

The pavement infiltration analyses discussed in Section 4 determined the rain depths capable of saturating typical Alabama highway pavement structures. Inter-event times for these specific rains (and greater) can be determined from the rainfall information in Appendix A for the complete state. These inter-event times are then compared to the drainage times needed for different pavement structures to determine the periods of saturation.

3.2 Rainfall Data Source and Description

The rainfall analyses involved gathering rain data, identifying a typical rainfall year, and statistically analyzing this typical data year for many locations throughout the region. The rainfall data analyzed was National Weather Service data stored on a CD-ROM database (EarthInfo Inc. of Boulder, Colorado). Ninety-two rainfall stations (locations shown in Figure 3.1) were selected from within Alabama and from adjacent states for these analyses, based on the duration and completeness of the rain record for each location.

The selection of a typical rain year for the state was based on annual rain totals and the monthly rainfall distributions for each location examined. All years from 1948 to 1992, for which data were recorded on the CD-ROM, were examined and ranked according to their closeness to the long-term average rainfall. Annual and monthly total rain depths for each weather station were obtained from the CD-ROM database and transferred to a Microsoft Excel file. An example rain file (for Birmingham) is shown in Table 3.2, and a summary table showing monthly totals for all 92 locations is shown in Table 3.3.

As shown on Table 3.2, average rain depths, standard deviation, and coefficient of variances were calculated for each month. The monthly average rain depths were summed to yield a yearly average rain depth. A standard score was then calculated for each rainfall year based on the deviation from the yearly average. The years were then sorted according to these standard scores, as shown on Table 3.2. The years with the best annual standard scores were selected as a representative group of typical rain years for each station. The groups of typical rain years from each station were then examined for monthly rainfall patterns. The typical rainfall year for the complete state was finally selected from these candidate rain years, based on the monthly patterns that best

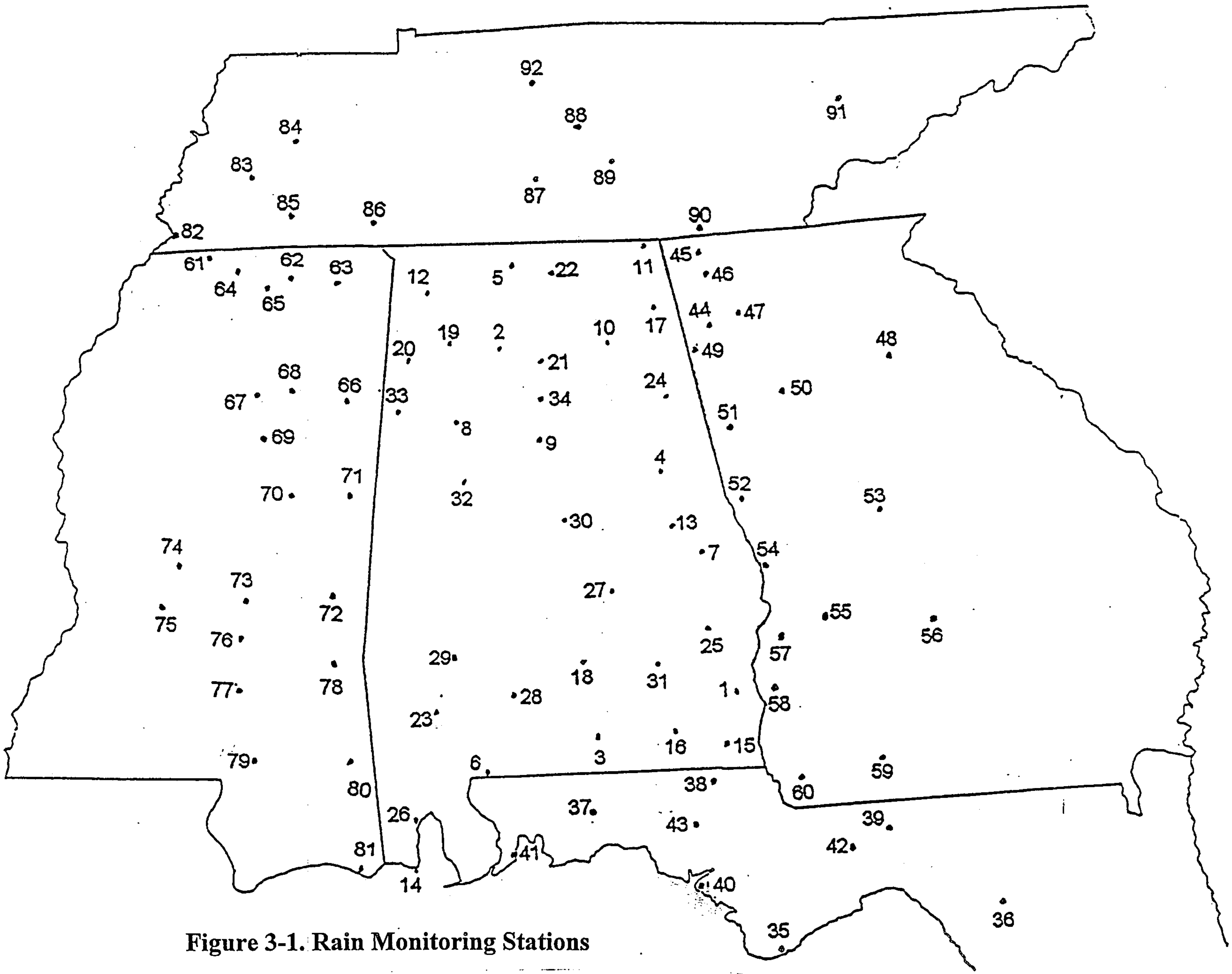


Figure 3-1. Rain Monitoring Stations

Table 3-2. Monthly Rain Depths for Birmingham, 1950-1990

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL	RESIDUAL	STANDARD SCORE
1950	4.51	4.28	5.47	1.97	4.55	4.97	13.70	5.49	2.50	2.41	1.69	3.92	55.46	-1.90	0.23
1951	3.68	4.36	11.42	6.19	1.15	7.31	4.10	2.98	9.74	2.77	1.98	8.71	64.39	-10.83	1.30
1952	4.35	3.05	5.44	1.38	3.31	4.15	1.11	8.18	2.69	2.03	2.50	4.96	43.15	10.41	1.25
1953	7.82	6.01	4.31	4.97	4.27	1.22	4.81	2.80	2.98	0.21	1.62	9.04	50.06	3.50	0.42
1954	6.35	3.37	3.51	3.20	3.23	1.93	3.58	2.74	1.24	1.77	3.96	5.78	40.66	12.90	1.55
1955	4.70	6.47	4.16	8.42	4.81	2.06	6.85	0.82	0.00	3.73	4.69	1.64	46.35	7.21	0.87
1956	1.85	8.98	6.09	5.47	2.38	1.69	7.97	7.12	2.26	3.49	2.18	4.16	53.64	-0.08	0.01
1957	6.00	3.73	6.08	5.41	2.96	7.70	2.62	4.19	9.59	1.81	5.67	4.01	59.77	-6.21	0.75
1958	3.42	5.14	3.03	3.51	2.33	3.10	6.79	1.98	5.74	2.31	3.38	1.29	42.02	11.54	1.39
1959	4.16	3.52	5.13	2.81	8.27	2.09	3.61	3.64	5.95	6.21	3.84	2.46	51.69	1.87	0.23
1960	5.05	3.36	6.31	2.24	2.28	2.74	2.06	4.09	2.73	3.45	3.24	3.21	40.76	12.80	1.54
1961	1.49	17.67	9.22	4.33	2.45	4.85	10.17	3.56	2.42	2.05	4.29	13.98	76.48	-22.92	2.76
1962	8.64	4.39	5.21	2.99	1.26	3.59	3.89	3.49	3.69	2.03	6.41	2.55	48.14	5.42	0.65
1963	7.32	3.25	6.31	6.70	3.72	8.44	6.54	1.53	1.21	0.11	4.00	5.94	55.07	-1.51	0.18
1964	5.16	4.11	9.44	9.90	3.20	4.08	4.34	2.78	3.26	2.95	3.24	5.09	57.55	-3.99	0.48
1965	3.21	6.22	6.10	2.54	1.37	8.17	4.87	3.55	2.60	0.67	2.80	2.06	44.16	9.40	1.13
1966	4.74	8.67	3.77	8.37	3.30	2.87	4.99	6.48	5.12	3.13	2.28	2.34	56.06	-2.50	0.30
1967	2.84	4.74	1.79	1.35	9.32	4.37	6.60	10.85	2.84	4.23	6.42	11.49	66.84	-13.28	1.60
1968	5.56	1.20	6.17	6.23	3.51	0.67	9.39	1.81	3.42	1.20	4.66	7.38	51.20	2.36	0.28
1969	7.78	3.17	5.19	5.32	11.10	3.75	2.91	1.76	6.85	2.51	2.66	6.07	59.07	-5.51	0.66
1970	2.47	2.35	11.36	5.56	2.27	3.55	3.37	7.01	1.05	7.04	2.31	3.32	51.66	1.90	0.23
1971	3.58	9.28	6.65	4.25	2.64	6.57	8.90	3.68	3.35	1.21	1.76	5.92	57.79	-4.23	0.51
1972	9.30	2.15	4.79	2.56	3.82	2.70	3.55	2.01	8.09	3.35	4.47	5.76	52.55	1.01	0.12
1973	6.85	2.33	9.71	5.33	8.29	3.74	8.36	5.41	2.64	0.96	4.91	7.58	66.11	-12.55	1.51
1974	6.85	4.94	2.43	5.43	5.43	1.42	4.69	8.28	4.94	1.49	4.13	5.97	56.00	-2.44	0.29
1975	7.23	4.96	7.57	3.19	4.15	2.44	7.33	3.33	3.69	3.74	4.15	3.49	55.27	-1.71	0.21
1976	4.12	1.80	14.15	1.99	9.00	2.75	4.92	3.34	4.91	1.59	2.23	4.35	55.15	-1.59	0.19
1977	5.08	3.89	8.70	6.73	3.51	0.96	6.24	0.87	10.43	7.52	4.10	2.01	60.04	-6.48	0.78
1978	4.54	1.31	3.07	2.64	8.51	5.04	5.09	2.09					N/A	N/A	N/A
1987										1.16	3.15	3.08	N/A	N/A	N/A
1988	5.55	2.52	3.18	3.18	1.22	0.79	2.95	3.43	8.57	3.41	6.33	2.84	43.97	9.59	1.15
1989	4.76	4.31	5.70	3.40	3.82	8.00	6.42	0.38	7.38	1.52	4.63	3.39	53.71	-0.15	0.02
1990	7.38	7.43	5.81	2.38	4.12	2.08				3.10	3.40	5.70	N/A	N/A	N/A
AVERAGE	5.20	4.78	6.16	4.31	4.24	3.74	5.57	3.86	4.40	2.66	3.66	4.98	53.83		
ST DEV	1.95	3.15	2.83	2.07	2.61	2.28	2.69	2.45	2.80	1.76	1.38	2.87	8.31		
COV	0.38	0.66	0.46	0.48	0.62	0.61	0.48	0.63	0.64	0.66	0.38	0.58	0.15		

SUM OF MONTHLY AVERAGES: 53.56

SORTED SCORES	YEAR
0.01	1956
0.02	1989
0.12	1972
0.18	1963
0.19	1976
0.21	1975
0.23	1959
0.23	1950
0.23	1970
0.28	1968
0.29	1974
0.30	1966
0.42	1953
0.48	1964
0.51	1971
0.65	1962
0.66	1969
0.75	1957
0.78	1977
0.87	1955
1.13	1965
1.15	1988
1.25	1952
1.30	1951
1.39	1958
1.51	1973
1.54	1960
1.55	1954
1.60	1967
2.76	1961
N/A	1978
N/A	1987
N/A	1990

STATION NAME: BIRMINGHAM
COUNTY: JEFFERSON

Table 3-3. Long-Term Monthly and Annual Rain for Regional Stations

STATE	STATION	NUMBER	BEG.DATE	END DATE	% COVER.	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEPT	OCT	NOV	DEC	YEARLY
ALABAMA	ABBEVIL	1	6/48	12/90	95.20	3.93	4.61	5.12	3.90	3.78	4.49	5.39	3.73	4.01	2.15	3.40	4.84	49.36
	ADDISON	2	7/48	12/90	88.70	5.34	4.67	5.59	4.69	4.11	3.41	4.84	3.20	3.46	2.31	4.07	4.74	50.41
	ANDALUS	3	4/80	12/90	88.60	4.09	5.24	7.14	3.28	4.69	4.33	5.46	4.05	4.47	2.03	4.19	4.60	53.57
	ASHLAND	4	7/48	12/90	85.70	4.81	5.28	6.17	5.00	4.04	3.99	5.08	3.56	3.98	3.18	3.58	4.94	53.61
	ATHENS	5	7/82	12/90	84.50	4.03	5.75	2.97	3.99	5.43	4.87	5.66	2.83	4.60	4.30	5.49	6.05	55.95
	ATMORE	6	3/65	12/82	85.50	4.92	5.34	6.17	3.46	5.26	4.98	6.80	6.43	5.37	2.89	4.01	5.44	61.06
	AUBURN	7	8/48	4/61	80.10	3.68	5.75	5.83	5.17	3.61	4.05	5.67	3.01	4.61	2.13	2.52	5.69	51.72
	BERRY	8	7/48	12/90	91.30	5.28	4.82	6.17	4.59	4.40	3.93	4.43	3.65	3.31	2.52	4.21	4.49	51.80
	BHAM	9	7/48	12/90	96.90	5.37	4.89	6.22	4.33	4.21	3.72	5.52	3.91	4.23	2.61	3.90	4.92	53.84
	BOAZ	10	7/48	12/90	89.70	5.23	4.58	5.94	4.84	4.24	3.65	4.29	2.75	3.86	2.56	3.49	4.84	50.28
	BRIDGPO	11	1/83	12/90	92.60	4.19	5.40	4.81	4.37	4.75	4.08	6.10	4.57	3.39	3.96	4.80	5.88	56.29
	COLBERT	12	8/82	12/90	81.60	4.19	5.40	4.81	4.37	4.75	4.08	6.10	4.51	3.39	3.96	4.80	5.88	56.23
	DADEVILL	13	8/48	12/90	92.50	4.68	4.94	5.99	5.22	3.37	3.59	5.37	3.45	3.62	2.58	3.72	4.83	51.36
	DAUPHIN	14	8/75	12/90	98.30	4.53	2.53	6.97	5.10	5.63	6.27	8.30	3.43	4.77	3.87	5.83	2.73	59.97
	DOTHAN	15	5/52	12/90	87.80	4.70	5.06	4.93	3.99	3.77	4.97	5.30	4.22	3.98	2.63	3.27	4.47	51.28
	ENTERP	16	6/71	12/90	81.60	5.13	5.25	6.97	3.42	5.54	4.57	5.36	3.26	3.33	2.45	3.60	4.65	53.71
	FORT PA	17	7/48	12/90	87.10	4.32	4.20	5.68	4.12	3.85	3.03	4.64	3.85	3.43	2.56	3.44	4.55	47.66
	GREENVIL	18	1/71	12/90	92.20	4.71	5.50	5.78	4.32	4.20	4.18	5.07	3.81	4.09	2.18	4.18	4.83	52.85
	HALEYVI	19	7/48	12/90	94.10	5.72	4.98	6.21	4.55	4.94	3.83	4.24	3.74	4.14	2.95	4.44	5.64	55.39
	HAMILTO	20	1/68	12/90	82.30	4.71	3.95	6.14	4.80	5.49	3.88	4.19	3.15	4.25	2.92	4.82	6.09	54.39
	HANCEV	21	5/73	12/90	93.70	4.69	4.52	6.07	5.08	5.52	3.61	4.99	3.20	5.18	3.82	4.63	4.63	55.93
	HUNTSVIL	22	1/59	12/90	99.30	5.18	4.76	6.61	4.83	5.04	4.09	4.75	3.53	4.08	3.32	4.76	5.64	56.58
	JACKSON	23	1/62	12/90	81.60	4.23	4.60	5.31	4.30	4.64	4.14	6.16	3.98	3.72	2.78	4.14	4.47	52.48
	JACKVILL	24	7/48	12/90	95.30	4.75	4.96	5.79	4.51	3.95	4.02	4.84	3.39	4.13	2.65	3.74	4.62	51.35
	MIDWAY	25	8/48	12/90	88.50	3.83	4.53	5.42	4.22	3.51	4.08	5.20	3.77	3.55	2.08	3.16	4.16	47.51
	MOBILE	26	9/48	12/90	95.50	4.66	5.36	6.62	4.92	5.54	5.44	7.72	6.76	6.14	2.99	3.95	5.20	65.30
	MONTGO	27	8/48	12/90	100.00	4.26	5.05	6.00	4.52	3.94	3.79	5.12	3.39	4.51	2.40	3.88	4.94	51.79
	PETERM	28	7/48	10/90	89.90	3.93	5.09	5.77	4.00	4.10	4.06	5.20	3.95	3.97	2.29	3.53	4.60	50.48
	THOMASV	29	1/68	12/90	90.30	6.68	6.34	7.66	4.72	5.04	3.82	4.78	4.12	2.12	1.46	4.82	3.94	55.50
	THORSBY	30	8/48	12/71	95.50	4.18	5.21	6.30	5.13	3.37	4.03	5.00	3.70	3.14	2.44	3.40	5.15	51.05
	TROY	31	7/48	12/90	92.50	3.96	4.55	6.02	4.23	3.86	3.71	5.71	3.52	3.72	2.35	3.31	4.35	49.30
	TUSCAL	32	1/58	3/87	92.00	2.37	4.03	5.55	3.81	4.04	4.77	8.4	1.28	5	3.64	3.93	1.01	47.83
	VERNON	33	7/48	12/90	87.60	5.82	4.85	5.43	4.57	3.61	3.46	4.58	2.77	3.83	2.72	4.20	5.06	50.92
	WARRIO	34	1/58	12/90	82.40	4.69	5.11	5.34	5.03	3.83	3.27	3.79	3.15	3.40	2.65	3.63	4.60	48.49
FLORIDA	APALICH	35	1/42	12/90	98.10	3.40	3.77	4.52	3.44	2.76	4.72	7.67	7.23	8.24	2.88	3.00	3.49	55.12
	BRANFO	36	9/44	12/90	95.10	2.54	2.97	3.56	3.42	3.02	5.48	7.07	6.06	5.32	3.22	2.03	2.58	47.28
	CRESTV	37	6/59	9/72	93.90	3.18	4.84	5.01	3.90	2.74	5.59	6.20	6.78	4.50	3.92	2.65	4.51	53.82
	GRACEV	38	1/42	12/90	93.90	4.37	5.03	6.11	4.24	3.88	4.48	5.43	5.35	4.29	2.29	3.11	4.49	53.06
	MONTICE	39	9/71	12/90	97.50	4.55	4.91	5.77	3.41	4.29	5.20	5.12	6.07	3.64	2.34	3.77	4.32	53.38
	PANAMA	40	1/42	12/71	90.30	3.13	3.97	5.03	3.41	2.47	4.31	7.12	6.01	5.36	2.76	3.29	3.87	50.74
	PENSAC	41	1/42	2/64	96.40	4.42	4.59	7.10	5.45	3.87	5.90	7.12	6.48	8.71	2.64	4.22	4.27	64.76
	TALLAHA	42	9/48	12/90	97.70	4.39	5.45	6.24	4.01	4.74	6.96	8.67	7.22	5.46	3.16	3.59	4.70	64.58
	VERNON	43	1/42	1/65	96.90	3.82	4.03	6.29	5.26	4.90	5.78	7.75	5.95	5.59	2.09	3.50	4.40	59.36
GEORGIA	ROME	44	7/48	12/90	95.90	4.42	4.56	5.68	4.33	3.43	3.83	4.14	3.17	3.65	2.45	3.93	4.47	48.05
	CHICKA	45	1/80	12/90	88.90	4.59	5.34	6.06	3.13	4.14	2.67	3.87	3.22	3.96	3.59	4.98	4.77	50.31
	LAFAYET	46	7/48	12/90	81.10	4.57	4.69	5.60	4.29	4.32	3.90	4.84	3.58	4.32	3.33	4.66	4.62	52.72
	ADAIRSV	47	7/48	9/86	87.40	4.36	4.11	5.91	4.38	3.77	3.67	4.37	3.40	3.65	2.66	3.74	4.38	48.40
	ATHENS	48	6/48	12/90	99.00	4.73	4.49	5.46	3.92	4.51	3.72	5.11	3.68	3.45	3.25	3.47	3.97	49.75
	CAVE SP	49	3/49	7/79	94.00	4.72	4.83	5.92	5.15	3.66	3.86	4.11	3.35	3.99	2.96	3.15	4.61	50.32
	ATLANTA	50	7/48	12/90	100.00	4.60	4.66	5.52	4.27	3.91	3.37	5.17	3.58	3.52	2.77	3.69	4.17	49.23
	CAROLT	51	7/48	12/90	92.20	4.74	4.67	5.83	4.26	4.32	3.72	4.96	3.13	3.45	2.62	3.89	4.38	49.97
	LAGRAN	52	7/51	12/90	94.80	4.16	4.56	5.48	4.63	3.44	3.54	4.90	3.23	3.57	2.50	3.57	4.36	47.93
	MACON	53	1/49	12/90	98.20	3.96	4.87	4.71	3.51	3.54	3.60	4.53	3.70	2.97	2.11	2.67	4.20	44.36
	COLUMB	54	8/48	12/90	100.00	4.20	4.72	5.74	4.39	4.16	3.98	5.62	3.76	3.28	2.05	3.41	4.84	50.15
	AMERIC	55	7/48	12/90	99.20	4.17	4.58	4.72	3.56	3.49	4.12	4.88	3.59	3.17	1.78	2.88	4.24	45.18
	ABBEVIL	56	7/48	12/90	93.20	3.44	4.07	4.55	3.15	2.85	3.65	5.06	3.92	3.04	2.13	2.39	3.19	41.43
	LUMPKIN	57	6/48	12/90	81.90	3.93	4.85	4.81	3.82	3.26	4.10	5.54	3.40	2.69	1.80	3.02	3.66	44.87
	EDISON	58	7/48	12/90	95.00	4.05	4.35	5.25	3.85	4.03	4.48	5.62	3.74	3.48	2.12	2.94	4.28	48.18
	COOLIDG	59	7/48	7/90	91.50	3.64	4.42	4.52	3.76	3.96	4.88	5.58	4.42	3.49	2.04	2.48	3.68	46.87
	BAINBRIG	60	3/49	12/77	93.10	3.77	4.14	4.40	3.70	3.35	3.96	6.12	3.82	4.08	2.34	2.13	3.65	45.47
MISS	BYHALIA	61	6/48	12/90	91.50	4.24	4.42	4.96	5.51	5.00	3.24	3.56	3.21	3.11	2.86	4.38	5.06	49.54
	RIPLEY	62	8/48	12/90	94.20	4.99	4.78	5.96	5.56	4.77	3.89	4.61	2.90	3.65	3.13	5.37	4.89	54.49
	BOONEV	63	3/61	12/90	92.50	4.48	4.41	5.85	5.45	5.81	3.45	4.10	2.61	3.68	3.30	4.99	5.24	53.38
	HOLLYSP	64	1/48	12/90	91.10	4.65	4.75	5.56	5.31	4.71	4.17	4.31	3.25	3.15	3.29	5.16	5.33	53.64
	HICK ORY	65	7/48	12/90	96.50	4.88	4.75	5.76	5.19	5.01	4.04	3.88	2.98	3.60	3.07	4.78	4.96	52.91
	ABERDE	66	9/51	2/90	95.80	4.57	4.67	5.51	5.22	4.47	3.74	3.55	2.62	3.17	3.20	4.13	5.10	49.94
	CALHOU	67	9/47	12/90	95.50	5.06	4.86	5.75	5.01	4.31	3.60	3.96	2.66	3.24	2.73	4.77	5.46	51.43
	HOUSTO	68	8/48	12/87	93.50	4.94	4.49	5.70	4.97	4.38	3.51	3.73	3.18	3.25	2.94	4.28	5.33	50.70
	EUPORA	69	5/51	12/90	85.60	4.86	4.43	5.83	5.26	4.42	3.33	4.07	2.52	3.04	3.40	4.62	5.62	51.41
	LOUSVIL	70	7/65	12/90	88.80	5.53	4.78	5.16	5.84	5.11	2.99	5.39	3.33	4.31	3.13	4.81	5.32	55.68
	MACON	71	7/48	9/72	96.80	5.42	5.57	5.59	5.94	3.89	3.46	5.62	3.91	3.29	2.80	3.88	5.53	54.88
	MERIDIA	72	7/48	12/90	98.80	4.88	5.17	6.52	5.14	4.37	3.71	5.27	3.53	3.70	2.72	4.21	5.62	54.84
	FOREST	73	6/48	12/90	91.30	5.39	5.02	5.54	4.87	4.74	3.73	5.29	3.89	3.66	3.03	4.27	5.02	54.45
	CANTON	74	1/48	12/90	94.50	5.16	4.63	5.70	4.97	5.20	3.12	3.96	3.21	2.77	2.35	4.28	5.47	50.82
	JACKSON	75	8/63	12/90	100.00	5.06	4.68	5.76	5.73	5.41	3.00	4.32	3.82	3.63	3.34	4.70	5.79	55.23
	COLLINS	76	7/48	12/90	95.10	4.39	5.01	6.01	5.06	5.36	3.79	5.36	4.17	3.94	2.64	4.01	5.09	54.83
	RALEIGH	77	7/64	12/90	81.20	5.42	5.37	6.57	5.98	4.70	3.24	4.98	4.25	3.51	3.06	4.37	5.63	57.07
	SHUBUTA	78	7/48	8/83	98.60	4.52	5.08	6.51	5.14	4.20	3.73	6.22	3.46	3.98	2.46	3.36	5.51	54.15
	PURVIS	79	7/48	12/90	89.90	4.85	5.74	5.91	5.32	5.37	4.49	6.14	5.12	4.12	3.09	4.29	5.33	59.77
	LEAKSVIL	80	3/57	12/90	93.10	4.58	5.55	6.73	4.90	5.30	4.59	6.53	5.21	4.94	3.16	4.10	5.40	60.99
	PASCAG	81	5/73	12/90	82.40	5.09	5.08	6.36	3.86	5.15	4.99	6.79	6.96	6.82	3.84	4.73	4.39	64.05
TENN	MEMPHIS	82	9/48	12/90	99.40	4.69	4.49	5.21	5.52	5.02	3.60	3.91	3.52	3.35	2.89	4.77	5.29	52.27
	BROWNS	83	9/48	12/90	93.60	4.02	3.96	4.52	5.19	4.72	3.54	3.51	2.85	3.16	2.67	3.86	4.72	46.72
	HUMBOL	84	9/48	12/87	95.20	4.34	3.93	5.01	4.97	5.27	3.86	3.62	2.78	3.84	2.85	4.15	4.64	49.26
	BOLIVAR	85	9/48	12/90	91.90	3.63	3.87	4.61	4.73	4.52	3.38	3.41	3.15	3.92	2.90	4.47	4.95	47.53
	SAVANA	86	9/48	9/80	100.00	5.24	4.56	5.93	4.92	4.64	3.84	3.92	2.88	3.13	2.76	4.62	5.23	51.68
	LEWISBU	87	9/48	12/90	96.30	4.97	4.17	5.79	4.26	4.61	3.48	4.61	2.98	3.82	3.33	4.43	5.03	51.48
	MURFRE	88	9/48	12/90	95.90	4.54	4.50	5.25	4.35	4.84	3.86	4.29	3.58	3.99	2.90	4.34	4.49	50.91
	SUMITVI	89	9/48	12/80	98.50	5.63	4.46	6.14	4.40	4.61	4.18	4.67	3.83	3.58	2.80	4.14	5.31	53.73
	CHATTA	90.	9/48	12/90	100.00	5.28	4.89	5.82	4.29	4.07	3.58	4.79	3.46	4.22	3.16	4.53	5.07	53.15
	KNOXVIL	91	9/48	12/90	100.00	4.54	4.25	4.95	3.67	3.91	3.82	4.40	3.09	2.94	2.76	3.84	4.47	46.65
	NASHVIL	92	9/48	12/90	100.00	4.38	4.13	4.97	4.20	4.61	3.77	3.74	3.29	3.32	2.60	3.95	4.59	47.55

represented the long-term average monthly conditions. The 1976 rain year was selected as the best typical rain year for Alabama because it had a reasonably close fit to the long-term conditions for all stations examined, and because the rain stations that reported data for that year were well distributed throughout the state. Twenty-two weather stations in Alabama were therefore selected for the additional analyses.

The second phase of the rainfall analysis involved evaluating hourly rain data for each of the 22 weather stations for 1976, identified during the typical rainfall year analyses. Probability analyses of rain depth, inter-event period, and peak hourly rain intensities were prepared and plotted. Contour plots of this data for specific probability values were also prepared for the state. These plots are shown in Appendix A.

The hourly rainfall information used in these more detailed analyses was also from the National Weather Service, obtained from the EarthInfo CD-ROM database. An hourly rainfall data file was created for each station and then translated into a SLAMM rainfall data file (Pitt 1986). An example hourly rain file is shown in Table 3.4 for Birmingham for the 1976 rain year. The SLAMM program was then used to calculate depth, duration, antecedent dry period, and average intensity for each rain at each weather station. SLAMM rain files containing rain date, depth, duration, previous antecedent dry period, average intensity, and peak hourly intensity were then created for each station. An example SLAMM rain file is shown in Table 3.5.

3.3 Rainfall Depth Analyses

Table 3.6 shows the occurrence probabilities of different rainfall depths, and other average conditions, for the 22 Alabama locations for 1976. These yearly average rain depths, and the geographical location for each weather station, were used to create contour plots of average yearly rainfall conditions throughout the region (using SYSTAT and SYGRAPH from SYSTAT Inc., Evanston, Ill.).

About 110 to 135 rains (of at least 0.25 mm, or 0.01 inch, in depth) occur per year at all stations throughout Alabama. About one-third of all of these rains are between 0.25 and 2.5 mm (0.01 and 0.1 inch) in depth. Therefore, up to 90 rains are greater than 2.5 mm (0.1 inch). The median rain depth is between 2.5 mm and 12.5 mm (0.1 and 0.5 inches), while 1 to 5 percent of the rains (about 1 to 7 per year) are greater than 65 mm (2.5 inches) in depth.

Figure 3.2 (average annual rain depths throughout the region) shows the increasing rainfall pattern as one goes from north to south across Alabama. The annual average rain depth is about 1200 mm (49 inches) in Huntsville, about 1360 mm (53.5 inches) in

Table 3-4. Hourly Rain Depths for Birmingham, 1976

HOURLY RAIN FILE

CITY: BIRMINGHAM, AL.

DATE	0100	0200	0300	0400	0500	0600	0700	0800	0900	1000	1100	1200	1300	1400	1500	1600	1700	1800	1900	2000	2100	2200	2300	2400	DAILY SUM
01/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
01/02/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.01
01/03/1976	0.02	0.10	0.03	0.02	0.06	0.21	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.45
01/07/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.17	0.15	0.04	0.01	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.58
01/11/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.06	0.07	0.08	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.25
01/13/1976	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.34	0.00	0.00	0.00	0.00	0.42
01/20/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.01	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05
01/24/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03
01/25/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.00	0.00	0.00	0.00	0.02	0.13	0.08	0.58	0.25	0.23	0.28	1.60
01/26/1976	0.26	0.39	0.04	0.02	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.73
02/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
02/05/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.03	0.03	0.07	0.09	0.30
02/06/1976	0.09	0.05	0.06	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.21
02/11/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
02/18/1976	0.00	0.00	0.00	0.24	0.05	0.17	0.11	0.01	0.00	0.00	0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.67
02/21/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.45	0.15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.61
03/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
03/05/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.12	0.02	0.00	0.05	0.06	0.00	0.00	0.01	0.30
03/06/1976	0.00	0.00	0.01	0.09	0.10	0.08	0.07	0.10	0.07	0.00	0.00	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.55
03/08/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.16	0.33	0.33	0.02	0.00	0.92
03/09/1976	0.00	0.00	0.00	0.01	0.07	0.10	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.19
03/12/1976	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.08	0.17	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.40	0.45	0.08	0.25	0.00	0.00	0.00	1.48
03/15/1976	0.00	0.00	0.00	0.05	0.08	0.25	0.29	0.29	0.22	0.16	0.11	0.12	0.23	0.22	0.12	0.02	0.02	0.31	0.02	0.00	0.00	0.00	0.00	0.00	2.51
03/16/1976	0.27	0.33	0.21	0.18	0.13	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.13
03/20/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.39	0.24	0.67
03/21/1976	0.12	0.26	0.12	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.51
03/24/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.00	0.00	0.00	0.01	0.04
03/26/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.22	0.12	0.00	0.02	0.02	0.05	0.02	0.00	0.01	0.00	0.00	0.02	0.53
03/27/1976	0.14	0.53	0.20	0.16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.03
03/29/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.81	0.06	0.42	0.70	0.05	0.01	0.00	0.00	2.06
03/30/1976	0.00	0.13	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.20	0.18	0.12	0.00	0.07	0.07	0.01	0.00	0.14	0.14	0.16	1.33
03/31/1976	0.36	0.12	0.38	0.03	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.90
04/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
04/11/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.06	0.06	0.02	0.02	0.21
04/13/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
04/14/1976	0.00	0.00	0.01	0.01	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04
04/24/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.38	0.12	0.00	0.00	0.10	0.70
04/25/1976	0.02	0.10	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.14
04/30/1976	0.02	0.03	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.20	0.02	0.01	0.00	0.47	0.03	0.00	0.89
05/01/1976	0.04	0.07	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.14
05/06/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.02	0.00	0.00	0.00	0.00	0.57	0.00	0.23	0.85
05/07/1976	0.43	0.22	0.08	0.02	0.10	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.89
05/08/1976	0.01	0.08	0.03	0.05	0.06	0.05	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.30
05/10/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.09	0.02	0.19
05/11/1976	0.04	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07
05/13/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.03	0.05	0.05	0.00	0.00	0.00	0.00	0.37	0.23	0.04	0.23	0.15	1.25
05/14/1976	0.05	0.02	0.04	0.07	0.05	0.04	0.05	0.05	0.06	0.10	0.14	0.15	0.18	0.11	0.85	0.41	0.06	0.02	0.05	0.05	0.03	0.00	0.00	0.00	2.58
05/15/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
05/16/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07
05/22/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.01
05/23/1976	0.00	0.00	0.07	0.07	0.00	0.71	0.41	0.18	0.11	0.25	0.14	0.03	0.00	0.04	0.02	0.05	0.01	0.14	0.06	0.02	0.01	0.00	0.00	0.00	2.32
05/26/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.01	0.00	0.02
05/27/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02
05/28/1976	0.00	0.00	0.00	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.24
05/29/1976	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04
06/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.28	0.08	0.02	0.01	0.04	0.04	0.00	0.00	0.00	0.48

Table 3-4. Hourly Rain Depths for Birmingham, 1976 (Continued)

06/18/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03
06/19/1976	0.00	0.00	0.00	0.00	0.00	0.02	0.43	0.87	0.06	0.01	0.00	0.00	0.00	0.02	0.01	0.00	0.05	0.09	0.00	0.00	0.00	0.00	0.00	0.02	1.58
06/20/1976	0.02	0.00	0.00	0.08	0.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.20
06/30/1976	0.00	0.00	0.00	0.00	0.00	0.06	0.02	0.38	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.46
07/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
07/04/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.59	0.30	0.03	0.02	0.02	0.00	0.00	0.00	0.00	0.04	0.04	0.01	0.04	0.00	0.00	0.00	1.17
07/13/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.26
07/16/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03
07/21/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.09
07/23/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.26	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.26
07/27/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.68	0.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	1.01
07/28/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.58	0.07	0.23	0.00	0.57	0.18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.63
07/29/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.15	0.01	0.00	0.00	0.00	0.00	0.00	0.17
07/30/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.11	0.05	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.23
07/31/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07
08/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
08/06/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.28	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.30
08/07/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.54	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.54
08/15/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.06
08/16/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.78	0.14	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.93
08/24/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.12	0.01	0.01	0.00	0.00	0.00	0.00	0.17	0.45	0.76
08/25/1976	0.08	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10
08/27/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.05	0.00	0.00	0.24	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.34
08/28/1976	0.00	0.00	0.01	0.00	0.00	0.10	0.00	0.00	0.00	0.00	0.00	0.00	0.16	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.28
08/29/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03
09/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.01	0.00	0.00	0.02	0.00	0.03	0.35	0.92	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.41
09/03/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.02	0.00	0.06	0.11	0.01	0.00	0.00	0.00	0.00	0.00	0.25
09/04/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.01	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05
09/05/1976	0.00	0.00	0.00	0.02	0.03	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.31	0.01	0.00	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.44
09/06/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.04
09/07/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.11
09/10/1976	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
09/21/1976	0.00	0.01	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06
09/26/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.12
09/27/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03
09/28/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.02	0.07	0.11
09/29/1976	0.40	0.01	0.03	0.45	0.79	0.36	0.17	0.00	0.00	0.00	0.00	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.28
10/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10/06/1976	0.00	0.00	0.00	0.00	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05
10/08/1976	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.09	0.01	0.01	0.01	0.00	0.00	0.00	0.16
10/16/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.02	0.00	0.00	0.01	0.05
10/20/1976	0.00	0.00	0.00	0.14	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.15
10/25/1976	0.00	0.00	0.01	0.04	0.10	0.10	0.04	0.20	0.05	0.00	0.00	0.00	0.00	0.01	0.04	0.05	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.64
10/30/1976	0.00	0.05	0.05	0.01	0.01	0.01	0.00	0.32	0.05	0.00	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.54
11/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
11/11/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.02	0.03	0.00	0.02	0.06	0.00	0.00	0.00	0.01	0.04	0.03	0.23
11/14/1976	0.00	0.00	0.00	0.00	0.00	0.10	0.10	0.03	0.10	0.10	0.17	0.10	0.03	0.02	0.01	0.01	0.01	0.02	0.05	0.01	0.03	0.01	0.00	0.01	0.91
11/20/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.07	0.01	0.00	0.02	0.02	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.22
11/26/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.01	0.00	0.00	0.00	0.00	0.03	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.12
11/27/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02
11/28/1976	0.00	0.03	0.00	0.01	0.14	0.05	0.01	0.01	0.00	0.02	0.03	0.10	0.07	0.07	0.02	0.01	0.04	0.04	0.01	0.03	0.02	0.01	0.01	0.00	0.73
12/01/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
12/06/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.00	0.02	0.06	0.03	0.04	0.05	0.05	0.03	0.01	0.00	0.06	0.05	0.47
12/07/1976	0.07	0.01	0.01	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.12
12/11/1976	0.00	0.00	0.00	0.00	0.01	0.04	0.02	0.02	0.04	0.02	0.04	0.02	0.01	0.00	0.00	0.00	0.01	0.02	0.04	0.12	0.08	0.03	0.01	0.01	0.54
12/12/1976	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.00	0.00	0.00	0.13	0.15	0.05	0.03	0.03	0.03	0.04	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.55
12/14/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.11	0.18
12/15/1976	0.02	0.04	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07
12/20/1976	0.03	0.03	0.04	0.00	0.05	0.35	0.18	0.05	0.14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.87
12/25/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.07	0.15	0.27	0.34	0.25	0.08	0.08	0.04	0.03	0.02	0.00	0.01	0.00	0.00	0.00	0.00	1.35
12/30/1976	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.14	0.18
12/31/1976	0.00	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02

Table 3-5. Birmingham Rain Durations and Intensities, 1976

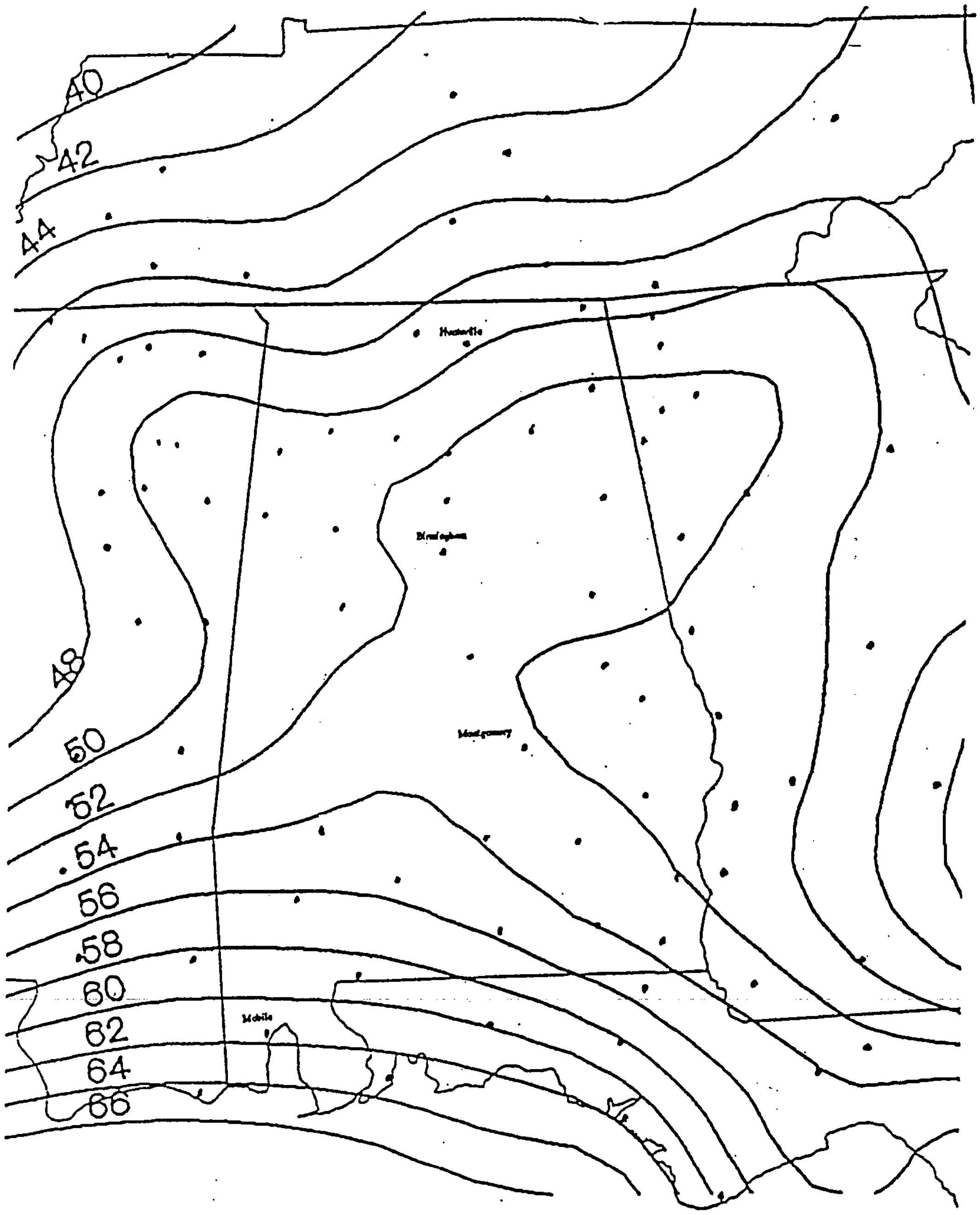
NUMBER	DATE	MONTH	DEPTH	DURATION	INTENSITY	ANTICED	PEAK_HR
1	01/02/	1	0.46	0.38	0.05	2.42	0.21
2	01/07/	1	0.58	0.38	0.06	4.00	0.20
3	01/11/	1	0.25	0.21	0.05	3.71	0.08
4	01/13/	1	0.04	0.04	0.04	1.79	0.01
5	01/13/	1	0.38	0.08	0.19	0.33	0.34
6	01/20/	1	0.05	0.21	0.01	6.46	0.02
7	01/24/	1	0.03	0.08	0.02	3.88	0.02
8	01/25/	1	2.33	0.83	0.12	1.00	0.58
9	02/05/	2	0.51	0.38	0.06	10.50	0.09
10	02/11/	2	0.01	0.04	0.01	5.21	0.01
11	02/18/	2	0.67	0.33	0.06	6.71	0.24
12	02/21/	2	0.61	0.13	0.20	3.08	0.45
13	03/05/	3	0.85	0.96	0.04	12.92	0.12
14	03/08/	3	1.11	0.71	0.07	2.21	0.33
15	03/12/	3	0.26	0.21	0.06	2.75	0.17
16	03/12/	3	1.20	0.17	0.30	0.29	0.45
17	03/15/	3	3.64	1.13	0.13	2.25	0.33
18	03/20/	3	0.04	0.08	0.02	4.17	0.03
19	03/20/	3	1.14	0.25	0.19	0.42	0.39
20	03/24/	3	0.04	0.25	0.01	3.58	0.02
21	03/26/	3	1.56	0.71	0.09	1.46	0.53
22	03/29/	3	2.20	0.50	0.18	2.46	0.81
23	03/30/	3	2.12	0.92	0.10	0.38	0.38
24	04/11/	4	0.21	0.21	0.04	11.38	0.06
25	04/13/	4	0.05	0.29	0.01	1.96	0.01
26	04/24/	4	0.84	0.38	0.09	10.50	0.38
27	04/29/	4	0.09	0.33	0.01	4.88	0.04
28	04/30/	4	0.76	0.46	0.07	0.33	0.47
29	05/06/	5	1.71	0.63	0.11	5.50	0.43
30	05/07/	5	0.03	0.08	0.02	0.38	0.02
31	05/07/	5	0.30	0.33	0.04	0.29	0.08
32	05/10/	5	0.06	0.08	0.03	2.21	0.05
33	05/10/	5	0.20	0.25	0.03	0.25	0.09
34	05/13/	5	3.83	1.42	0.11	2.33	0.65
35	05/15/	5	0.01	0.04	0.01	0.58	0.01
36	05/16/	5	0.07	0.08	0.04	0.79	0.05
37	05/22/	5	2.33	1.04	0.09	6.46	0.71
38	05/26/	5	0.02	0.17	0.00	2.92	0.01
39	05/27/	5	0.02	0.04	0.02	0.58	0.02
40	05/28/	5	0.23	0.33	0.03	0.54	0.19
41	05/28/	5	0.05	0.13	0.02	0.50	0.03
42	06/01/	6	0.48	0.42	0.05	3.38	0.28
43	06/18/	6	0.03	0.04	0.03	16.58	0.03
44	06/19/	6	1.78	1.00	0.07	0.71	0.87
45	06/30/	6	0.46	0.13	0.15	10.00	0.38
46	07/04/	7	1.17	0.58	0.08	3.96	0.59
47	07/13/	7	0.26	0.04	0.26	8.71	0.26
48	07/16/	7	0.03	0.04	0.03	3.00	0.03
49	07/21/	7	0.09	0.04	0.09	4.92	0.09
50	07/23/	7	0.26	0.04	0.26	2.00	0.26
51	07/27/	7	0.91	0.08	0.46	3.92	0.68
52	07/27/	7	0.01	0.04	0.01	0.29	0.10
53	07/28/	7	1.63	0.25	0.27	0.42	0.58
54	07/29/	7	0.17	0.13	0.06	1.00	0.15
55	07/30/	7	0.23	0.13	0.06	0.50	0.11
56	07/31/	7	0.07	0.04	0.07	1.08	0.07
57	08/06/	8	0.30	0.08	0.15	6.13	0.28
58	08/07/	8	0.54	0.04	0.54	0.79	0.54
59	08/15/	8	0.06	0.13	0.02	8.04	0.03
60	08/16/	8	0.93	0.13	0.31	0.79	0.78
61	08/24/	8	0.86	0.46	0.08	7.96	0.45
62	08/27/	8	0.34	0.25	0.06	2.42	0.24
63	08/28/	8	0.11	0.17	0.03	0.33	0.01
64	08/28/	8	0.17	0.08	0.09	0.25	0.16
65	08/29/	8	0.03	0.04	0.03	1.08	0.03
66	09/01/	9	1.41	0.42	0.14	2.58	0.92
67	09/03/	9	0.25	0.29	0.04	1.79	0.11
68	09/04/	9	0.05	0.29	0.01	0.46	0.03
69	09/05/	9	0.44	0.38	0.05	0.58	0.31
70	09/06/	9	0.04	0.04	0.04	1.21	0.04
71	09/07/	9	0.11	0.08	0.06	0.75	0.09
72	09/09/	9	0.01	0.04	0.01	2.42	0.01
73	09/21/	9	0.06	0.08	0.03	11.00	0.05
74	09/26/	9	0.12	0.08	0.06	5.38	0.02
75	09/27/	9	0.03	0.04	0.03	0.67	0.03
76	09/28/	9	2.38	0.67	0.15	1.54	0.79
77	10/06/	10	0.05	0.08	0.03	6.67	0.03
78	10/06/	10	0.16	0.04	0.16	0.38	0.01
79	10/08/	10	0.01	0.04	0.01	1.46	0.01
80	10/08/	10	0.15	0.21	0.03	0.50	0.09
81	10/16/	10	0.05	0.25	0.01	7.88	0.02
82	10/20/	10	0.15	0.08	0.08	3.13	0.14
83	10/25/	10	0.64	0.58	0.05	4.88	0.20
84	10/30/	10	0.54	0.46	0.05	4.38	0.32
85	11/11/	11	0.23	0.54	0.02	11.96	0.06
86	11/14/	11	0 91	0.79	0.05	2.21	0.17
87	11/20/	11	0.22	0.29	0.03	5.33	0.07
88	11/26/	11	0.12	0.38	0.01	5.71	0.04
89	11/27/	11	0.02	0.08	0.01	0.79	0.01
90	11/28/	11	0.73	0.92	0.03	0.46	0.14
91	12/06/	12	0.46	0.79	0.02	7.50	0.07
92	12/11/	12	0.17	1.58	0.00	3.92	0.15
93	12/14/	12	0.92	0.21	0.18	2.17	0.11
94	12/19/	12	0.87	0.38	0.10	4.88	0.35
95	12/25/	12	0.29	0.54	0.02	4.92	0.34
96	12/30/	12	0.54	0.04	0.54	4.71	0.01
97	12/30/	12	0.86	0.29	0.12	0.29	0.14

Table 3-6. Regional Rain Probabilities, 1976

NAME	X	Y	AVG ANTICEDENT	PK INTENSITY	AVG INTENSITY	.01inDEP	.1inDEP	.25inDEP	.5inDEP	.75inDEP	1.0inDEP	1.25inDEP	1.5inDEP	1.75inDEP	2.0inDEP	2.25inDEP
ABBEVIL	20.1	8.9	1.9	0.11	0.1	100	66	47	30	23	20	13	11	8	6	4.8
BERRY	10.4	18.9	3.2	0.21	0.1	100	71	53	44	30	23	19	14	11	8	5.5
BHAM	13.6	18.2	1.5	0.11	0.051	100	68	52	39	27	18	13	10	8.5	8	7
BOAZ	15.9	21.5	2.3	0.1	0.1	100	64	42	30	23	17	13	10	9	7	5.5
DADEVIL	18.1	14.7	3	0.2	0.1	100	71	44	32	23	18	13	10	7.5	5	2.5
DOTHAN	19.7	7	2.15	0.2	0.1	100	62	40	28	17	16	14	11	7.5	6	4.5
GREENV	14.8	10	2.25	0.2	0.1	100	71	46	32	22	15	14	12	9	7	6.5
HALYVIL	10.3	21.6	1.6	0.12	0.05	100	66	50	42	34	19	15	11	4.5	3.2	2.4
HAMILT	8.8	21	1.85	0.2	0.1	100	61	48	38	30	23	11	8	4	3.5	3
HANCEV	13.8	20.8	2.4	0.2	0.1	100	64	41	32	23	14	10	7.5	3.4	3	2.7
HUNTSVIL	14.2	24.1	2.3	0.13	0.052	100	68	58	40	22	16	15	12	10	9	7
LAGRNG	20.8	15.9	2.4	0.1	0.1	100	62	41	30	22	17	14	9	7	4.5	4
JACKSON	9.4	8.3	2	0.1	0.1	100	65	43	35	25	14	10	9	7	6	5
JAKVIL	18	19.6	3.15	0.21	0.1	100	70	48	34	26	19	14	11.5	11	10	6
MRIDIAN	5.9	12.5	2.15	0.13	0.07	100	68	52	37	26	18	14	13	7	4.5	4
MOBILE	8.5	4.5	1.7	0.12	0.041	100	54	46	34	22	13	11	9	8	7	5.5
MNTGMRY	15.8	12.3	2.3	0.12	0.042	100	74	62	49	38	18	14	13	10	7	6.5
PTRMAN	12.2	8.9	2.2	0.21	0.1	100	62	42	34	25	17	13	10	8	6.5	5
TOMVIL	10	10.3	2.35	0.1	0.1	100	61	39	31	24	19	15	11	9	7	5.5
TROY	17.3	9.8	2	0.1	0.1	100	68	43	33	22	17	10	7.5	6.5	5	4
TUSCAL	10.7	16.6	2.55	0.17	0.86	100	70	54	38	28	24	20	17	10	8	6
VERNON	8.5	18.9	2.8	0.21	0.1	100	70	54	44	33	21	12	9.5	5	4	3.4

NAME	2.5inDEP	2.75inDEP	3.0inDEP	3.25inDEP	3.5inDEP	3.75inDEP	4.0inDEP	4.25inDEP	4.5inDEP	5.0inDEP
ABBEVIL	4.5	4.2	4	3.7	3.5	2.7	2	0	0	0
BERRY	4.5	2.8	2	1.3	1.1	0.8	0	0	0	0
BHAM	3.9	3.8	3.6	3.5	3.4	3	2.9	2.5	2.1	0
BOAZ	4	3	2	1	0.9	0	0	0	0	0
DADEVIL	1.9	1.4	1	0	0	0	0	0	0	0
DOTHAN	4	3.7	3.5	3.3	3	2.7	2	1	0	0
GREENV	6	3.5	2.5	1.2	0	0	0	0	0	0
HALYVIL	1	0	0	0	0	0	0	0	0	0
HAMILT	1.3	0	0	0	0	0	0	0	0	0
HANCEV	2.3	2	1.8	1.4	1.2	0.8	0	0	0	0
HUNTSVIL	6	5	4.5	3.5	2.5	1.8	1.4	1.3	1.2	1
LAGRNG	3.5	2.2	1.8	1.6	1.4	1.2	1	0	0	0
JACKSON	2.5	1.6	1.1	0	0	0	0	0	0	0
JAKVIL	5	2.7	2.3	2	1.8	1.6	1.4	1.2	0	0
MRIDIAN	3.7	3.3	3	2.8	2.6	2.4	2	1.1	1	0
MOBILE	4.5	4.2	4	3	2.7	2.6	2.5	2.4	2.3	2
MNTGMRY	6	5	4.6	4.3	4	2	0	0	0	0
PTRMAN	4	3.5	3	2.8	2.6	2.5	2.3	1.8	0.8	0
TOMVIL	5	4.5	4	3.5	2.5	1.2	0	0	0	0
TROY	3.3	3	2.75	2.5	2	1.8	1.5	1	0	0
TUSCAL	5	4	3	2.8	2.7	2.5	2	1.8	1.6	1.5
VERNON	2.8	1.3	0	0	0	0	0	0	0	0

Figure 3-2. Contour Map of Annual Average Rain Depths



Birmingham and Montgomery, and about 1600 mm (62 inches) in Mobile. This rain depth is shown to be relatively constant in the northern two-thirds of the state, but increases much more rapidly between Montgomery and the Gulf coast.

The probability plots in Appendix A indicate the probability that a rain will occur of at least the indicated depth. As an example, the Mobile plot shows that the median (50 percentile) rain depth was about 4 mm (0.15 inch) and that 10 percent of all rains were greater than 30 mm (1.2 inches). The largest rain monitored in Mobile in 1976 was about 180 mm (7 inches).

The rain depth contour plots show the probabilities of occurrence for the rains indicated, or greater. As an example, about 50 percent of all rains in the central part of the state are at least 6.4 mm (0.25 inches) in depth. However, very few rains greater than 100 mm (4 inches) occur in the northern part of Alabama, but account for about three percent of all rains along the Gulf coast.

3.4 Inter-Event Period Analyses

As noted previously, the inter-event periods between rains is a very significant factor in determining the lowest pavement moisture levels that will be obtained. Table 3.6 shows that typical state-wide inter-event periods between all rains varied between 1.5 and 3.2 days, and averaged about 2.3 days. The Appendix A inter-event probability plots indicate the frequencies of occurrence for different inter-event periods for the 22 Alabama locations for the 1976 typical rain year, and the contour plots indicate how inter-event periods vary throughout the state. As an example, the whole state has a 30 to 40 percent probability of having at least a 3-day inter-event period. This decreases to about 10 to 20 percent for 7-day, or longer, inter-event periods. It is very unusual to have inter-event periods of 15 days, or longer, anywhere in the state, but especially along the Gulf coast.

Inter-event periods can be easily calculated for any specific rain depth and location. For example, 20 percent of all rains are at least 25 mm (1 inch) in Birmingham. Since there are 110 rains in Birmingham per year, 22 rains would be 25 mm (1 inch), or larger. The inter-event period for these rains would therefore be about 365 days/22 events = 17 days. The actual value may be about 85 percent of this value (14 days) because it rains about 15 percent of the time.

3.5 Rainfall Intensity Analyses

According to classical Hortonian infiltration theory, rainfall intensity affects the amount of water that can infiltrate. If the rainfall intensity is greater than the infiltration rate for the surface, then the maximum infiltration rate into the surface will occur and the excess rainfall will contribute to runoff. If the rainfall intensity is less than the infiltration rate capacity of the surface, then the actual infiltration will be limited to the rainfall intensity. Pavement infiltration rate tests were conducted in the laboratory and in the field, as discussed in Section 4. As seen in that section, pavement infiltration rates were observed to typically be greater than 25 mm/hr (1 inch/hr), but only for short periods of time (10 minutes). Long-term (throughout a rain) pavement infiltration rates are expected to be much less.

These analyses were therefore conducted to determine typical rain intensities for Alabama rains for comparison to the observed pavement infiltration rates. Table 3.6 shows that the average rain intensities varied from about 1 mm to 2.5 mm/hr (0.04 to 0.1 inch/hr), while the peak hourly rain intensities averaged for all rains varied from about 2.5 to 5.3 mm/hr (0.1 to 0.21 inch/hr).

Appendix A contains probability and contour plots of average and peak hourly rain intensities. As an example, average rain intensities in Huntsville were greater than 0.25 mm/hr (0.01 inch/hr) 90 percent of the time and greater than 5 mm/hr (0.2 inch/hr) 10 percent of the time. The contour plot for peak hourly rain intensity shows a possible increasing trend from north to south, as might be expected.

4.0 INFILTRATION AND PERCOLATION IN HIGHWAY PAVEMENTS

4.1 Overview

Surface water runoff infiltrating pavement during rains is thought to be the major source of water affecting pavement moisture levels. Other sources of water that may be important in specific situations include groundwater entering the pavement structure from beneath the roadbed and other surface runoff flooding the roadway from springs or other intermittent or continuous nearby sources. Because of the site specific nature and rarity of these other potential pavement water sources, this research project only examined the effects of rain water infiltration on pavement moisture levels. This section describes a series of laboratory and field experiments that directly measured the infiltration rates of surface waters into highway pavements.

4.2 Laboratory Pavement Infiltration Tests

Laboratory infiltration tests allowed long-term observations of how infiltration rates varied with time. Eight pavement test samples were obtained from the highway pavement moisture test sites, as noted on Table 4.1. The pavement test samples were removed during the moisture sensor installations. Sections of pavement were cut from the test locations using large diameter pavement saws. Three locations were cut at each test site to accommodate the moisture sensors. Each pavement section was about 1 meter (40 in) long by 100 mm (4 in) in width and was full depth to the base layer. All pavement sections were about 0.3 m deep, except for the test section on I-459 at Grants Mill Rd. (poor drainage location) where the pavement was only half as thick. The moisture sensors were located near the bottom of the base layer. The laboratory infiltration test specimens were obtained from these larger sections by cutting with a tungsten-carbide tipped concrete specimen saw.

The vertical sides of the eight small sample sections were coated with an impermeable, non-penetrating sealant (a commercial silicon seal) to prevent water from seeping out the sides of the samples. The top wearing surface of the samples and the bottom base layer surfaces of the samples were not coated, enabling free water flow vertically through the samples. A 50 mm (2 in) diameter clear standpipe was placed on the exposed top wearing surface of each sample and sealed to the specimen with silicone sealant.

The test samples with their vertical standpipes were clamped onto a stand for stabilization. A control sample (a standpipe sealed to an impervious surface) was also simultaneously monitored to measure any evaporation, or other problems. The standpipes were filled with pink colored water to improve visibility to an initial depth of 150 mm (6 in) above the pavement surface. The standpipe tops were covered to eliminate water evaporation and were then monitored for two weeks. Upon completion of the tests, incremental infiltration rates were calculated, as shown on Table 4.2.

These tests found very little infiltration of water through the pavement sections (less than 0.1 mm/hr, or 0.005 in/hr). Because of the large diameter of the standpipes, the test sensitivity was limited. The largest infiltration rate observed only resulted in about a 40 mm (1.5 in) drop in water level over the two week period of observation. However, these tests did show that highway pavement infiltration rates, even though initially high (as indicated in the field tests) do not account for much water volume over long periods of time. The test specimens were also in good condition, having no cracks or seams in their small surface areas.

Other reasons for the small observed infiltration rates in these test specimens, compared to pavement test results reported in the literature, as summarized in Section 2.3.3, is their large thickness and lack of full-thickness micro-scale cracks that can form flow channels. Relatively thin (75 to 100 mm, 3 or 4 in, thick) and less dense pavements typical of city streets can easily have continuous cracks that penetrate the complete thickness of the pavement, enabling significant water movement. More significantly, the thinner city pavements also have many more large cracks and pavement seams that pass water relatively easily, compared to the thicker and much more dense highway pavements.

4.3 Field Pavement Infiltration Tests

Numerous field tests of in-situ infiltration rates were conducted in order to supplement the above laboratory tests. These tests allowed pavement observations of infiltration rates for a variety of highway conditions, specifically the effects of small and large cracks.

TABLE 4.1

Laboratory Infiltration Test Sample Descriptions

Sample Description	Cross Section Description	Test #
Highway 79 / Poor Drainage	3.0" x 3.75"	1
Highway 79 / Poor Drainage	3.0" x 3.75"	2
Highway 79 / Good Drainage	2.75" x 3.25"	3
Highway 79 / Good Drainage	2.75" x 3.25"	4
I - 459 / Poor Drainage	2.75" x 4.25"	5
I - 459 / Poor Drainage	3.0" x 4.25"	6
I - 459 / Good Drainage	2.75" x 2.75"	7
I - 459 / Good Drainage	2.75" x 2.75"	8

The test apparatus was constructed based on a similar unit developed by the Dept. of Main Roads, New South Wales (Gerke 1982). This device is also very similar to that described by Bauer (1966) for the direct measurement of water pressure head and in-situ hydraulic conductivity in soils. It consisted of a hollowed out 50 mm (2 in) thick circular aluminum plate, 0.3 m (12 in) in diameter, with a clear standpipe connected at the center. The standpipe was much smaller in diameter than the inside diameter of the plate, giving a 200 times amplification of drawdown rates. A 100 mm drawdown easily observed in the clear standpipe therefore corresponded to an actual infiltration of 0.5 mm, which would have been impossible to observe. A valve was also located near the center of the plate to bleed air from inside of the apparatus when filling with colored water at the beginning of each test.

Sealing of the aluminum plate directly to the asphalt pavement was not practical because the pavement surface was very irregular and the only sealant that was found to form a watertight seal when subjected to the water pressure during the test required several hours for curing. Therefore, a 3 mm thick Plexiglas plate (containing a hole in the center, which corresponded to the inside diameter of the aluminum plate) was first sealed to the asphalt. The aluminum plate was then sealed to the Plexiglas plate before each test. Six Plexiglas plates were sealed to the asphalt in the test areas at a time using silicone sealant, and at least three hours were needed to sufficiently cure the sealant so they wouldn't leak when subjected to the maximum 0.3 m water head during the test.

Table 4-2. Laboratory Infiltration Test Results

HIGHWAY 78
POOR DRAINAGE LOCATION

TEST 1: standpipe diameter: 2" sample description: 3" x 3.75"

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
6.00	N/A	N/A	N/A	N/A	N/A
6.00	0.00	0.00	24.00	24.00	0.00
6.00	0.00	0.00	47.50	23.50	0.00
6.00	0.00	0.00	72.50	25.00	0.00
6.00	0.00	0.00	96.33	23.83	0.00
6.00	0.00	0.00	171.41	75.08	0.00
6.00	0.00	0.00	193.08	21.67	0.00
6.00	0.00	0.00	243.66	50.58	0.00
6.00	0.00	0.00	335.50	91.83	0.00

TEST 2: standpipe diameter: 2" sample description: 3" x 3.75"

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
6.00	N/A	N/A	N/A	N/A	N/A
5.99	0.01	0.0075	24.00	24.00	0.0000673
5.98	0.02	0.0075	47.50	23.50	0.0000691
5.96	0.02	0.0075	72.50	25.00	0.0000638
5.93	0.03	0.0075	96.33	23.83	0.0000679
5.89	0.04	0.0075	171.41	75.08	0.0000279
5.84	0.05	0.0075	193.08	21.67	0.0000966
5.79	0.05	0.0075	243.66	50.58	0.0000414
5.73	0.06	0.0075	335.50	91.83	0.0000228

HIGHWAY 78
GOOD DRAINAGE LOCATION

TEST 1: standpipe diameter: 2" sample description: 2.75" x 3.25"

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
6.00	N/A	N/A	N/A	N/A	N/A
5.85	0.15	0.15	24.00	24.00	0.002196916
5.80	0.20	0.05	47.50	23.50	0.000747686
5.75	0.25	0.05	72.50	25.00	0.000703013
5.55	0.45	0.20	96.33	23.83	0.002950118
5.35	0.65	0.20	171.41	75.08	0.000936352
5.25	0.75	0.10	193.08	21.67	0.001622068
5.15	0.85	0.10	243.66	50.58	0.000694952
4.90	1.10	0.25	335.50	91.83	0.000956949

TEST 2: standpipe diameter: 2" sample description: 2.75" x 3.25"

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
6.00	N/A	N/A	N/A	N/A	N/A
5.94	0.06	0.06	24.00	24.00	0.000836196
5.88	0.12	0.06	47.50	23.50	0.000853965
5.82	0.18	0.06	72.50	25.00	0.000802749
5.76	0.24	0.06	96.33	23.83	0.000842162
5.70	0.30	0.06	171.41	75.08	0.000267296
5.60	0.40	0.10	193.08	21.67	0.001543035
5.50	0.50	0.10	243.66	50.58	0.000661286
4.80	1.20	0.70	335.50	91.83	0.002549657

INTERSTATE 459
POOR DRAINAGE LOCATION

TEST 1: standpipe diameter: 2" sample description: 2.75" x 4.25"

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
6.00	N/A	N/A	N/A	N/A	N/A
5.90	0.10	0.10	24.00	24.00	0.001119996
5.85	0.15	0.05	47.50	23.50	0.000571913
5.80	0.20	0.05	72.50	25.00	0.000537598
5.79	0.21	0.01	96.33	23.83	0.000112799
5.70	0.30	0.09	171.41	75.08	0.000322215
5.65	0.35	0.05	193.08	21.67	0.000619985
5.60	0.40	0.05	243.66	50.58	0.000265717
5.30	0.70	0.30	335.50	91.83	0.000878142

TEST 2: standpipe diameter: 2" sample description: 3" x 4.25"

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
6.00	N/A	N/A	N/A	N/A	N/A
5.60	0.40	0.40	24.00	24.00	0.004106654
5.05	0.95	0.55	47.50	23.50	0.005765434
4.50	1.50	0.55	72.50	25.00	0.005420783
4.15	1.85	0.35	96.33	23.83	0.003618956
3.20	2.80	0.95	171.41	75.08	0.003117731
2.95	3.05	0.25	193.08	21.67	0.002842631
2.40	3.60	0.55	243.66	50.58	0.002679311
1.75	4.25	0.65	335.50	91.83	0.001744067

INTERSTATE 459
GOOD DRAINAGE LOCATION

TEST 1: standpipe diameter: 2" sample description: 2.75" x 2.75"

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
6.00	N/A	N/A	N/A	N/A	N/A
5.99	0.0113	0.0113	24.00	24.00	0.000195592
5.98	0.0226	0.0113	47.50	23.50	0.000199754
5.97	0.0339	0.0113	72.50	25.00	0.000187768
5.95	0.0452	0.0113	96.33	23.83	0.000196967
5.94	0.0565	0.0113	171.41	75.08	0.000062522
5.93	0.0678	0.0113	193.08	21.67	0.000216623
5.92	0.0791	0.0113	243.66	50.58	0.000092807
5.91	0.0904	0.0113	335.50	91.83	0.000051119

TEST 2: standpipe diameter: 2" sample description: 2.75" x 2.75"

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
6.00	N/A	N/A	N/A	N/A	N/A
5.99	0.00625	0.00625	24.00	24.00	0.000106181
5.99	0.01251	0.00625	47.50	23.50	0.000110483
5.98	0.01876	0.00625	72.50	25.00	0.000103854
5.97	0.02501	0.00625	96.33	23.83	0.000108953
5.97	0.03126	0.00625	171.41	75.08	0.000034581
5.96	0.03751	0.00625	193.08	21.67	0.000119813
5.96	0.04376	0.00625	243.66	50.58	0.000051331
5.95	0.05001	0.00625	335.50	91.83	0.000028273

This procedure allowed more tests to be taken because all the Plexiglas plates could be sealed to the pavement early in the day, allowing all of them to cure at the same time. Once all of the Plexiglas plates' sealants were cured (usually about three hours later), the testing procedure began.

Once the Plexiglas plates were sealed to the pavement, the aluminum plate test apparatus was sealed to each smooth Plexiglas plate using a quick-setting waterproof putty. Once the test apparatus was finally installed, the air bleed valve was opened and colored water was poured into the standpipe using a funnel. All of the air was removed from inside the apparatus when water began to pour out of the bleed valve, which was then closed. The water level in the standpipe was 0.3 m at the beginning of each test. When the water level reached the starting depth, a stopwatch was started, and times were recorded for specific water elevation changes. These falling head values were then used to calculate incremental infiltration rates. These incremental rates were then plotted and Horton infiltration coefficients were calculated using a non-linear curve fitting program (SYSTAT).

The tables in Appendix B show all of the data for the field in-situ infiltration tests conducted. These data all show decreasing infiltration rates with time, as shown on Figures 4.1 through 4.4. The I-459 plot shows two apparent curves. The lower curve (lower rates) is for test locations having good pavement surfaces, while the upper curve (higher rates) is associated with data from test sites having cracks. Table 4.3 summarizes the Horton equation parameters, and their statistical significance. The Horton equation is as follows:

$$F = F_c + (F_o - F_c)\, e^{-kt}$$

where F is the infiltration rate occurring at any time t, F_c is the final constant infiltration rate, and F_o is the high, initial infiltration rate and k is a rate constant. Except for the two unusual conditions, these data are very close and indicate high initial infiltration rates, ranging from 25 to 125 mm (1 to 5 in/hr), but very low final, constant rates of basically zero.

TABLE 4.3
Horton Infiltration Equation Coefficients

test site	F_c (in/hr)	F_o (in/hr)	k (1/hr)	corrected$\bar{R}^2$
I-459 no cracks (good with drains)	0.031	1.077	1.59	0.95
I-459 with cracks (good with drains))	1.82	27.1	20.9	0.90
I-459 (poor with drains)	-0.053	5.02	0.89	0.74
H-79 newly paved (good without drains)	5.03	48.1	13.0	0.63
H-79 (poor without drains)	0.083	0.92	1.67	0.85

The I-459 site having cracks had extremely high initial infiltration rates 690 mm/hr (27 in/hr) and high final rates 46 mm/hr (1.8 in/hr). The cracks (about 6 mm, 0.25 in, wide and 3 mm, 0.1 in, deep) were able to sustain a very high flow of water. The effects of the cracks on pavement moisture levels are therefore dependent on the amount of pavement cracking. Sites with substantial cracking would respond quickly to rainfall and have rapid increases in pavement moisture levels. They may, or may not, drain rapidly, depending on how well the cracks carry the water away after the rain ends.

The newly paved H-79 site that didn't have the wear surface installed yet also showed extremely high infiltration rates (initial rates of 1200 mm/hr, 48 in/hr, and final rates of 125 mm/hr, 5 in/hr). This pavement surface was truly a "porous" pavement. Because these tests were only affecting a very small area (0.3 m, 12 in, diameter), they were not affected by the percolation characteristics of the pavement material. In actual rains, all of the pavement surface will be contributing water to the pavement and may saturate the pavement much sooner than these extremely high tests indicate, effectively reducing the F_c and k values. When the wear surface is in place, it is expected that the

Figure 4-1. Horton Infiltration Rates (I-459/Good Drainage)

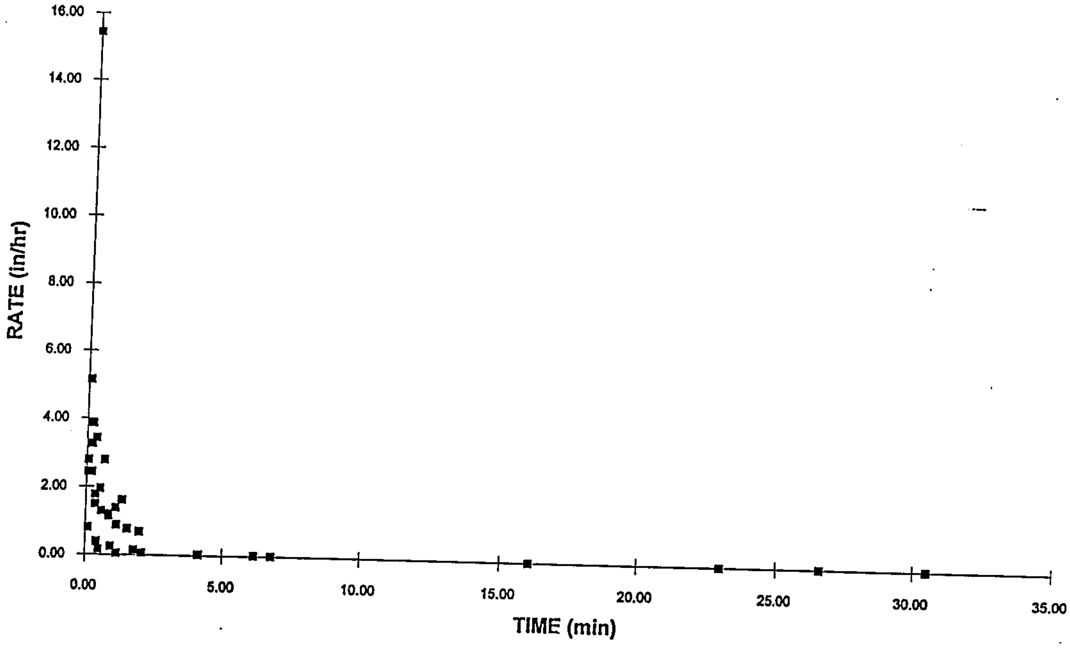


Figure 4-2. Horton Infiltration Rates (I-459/Poor Drainage)

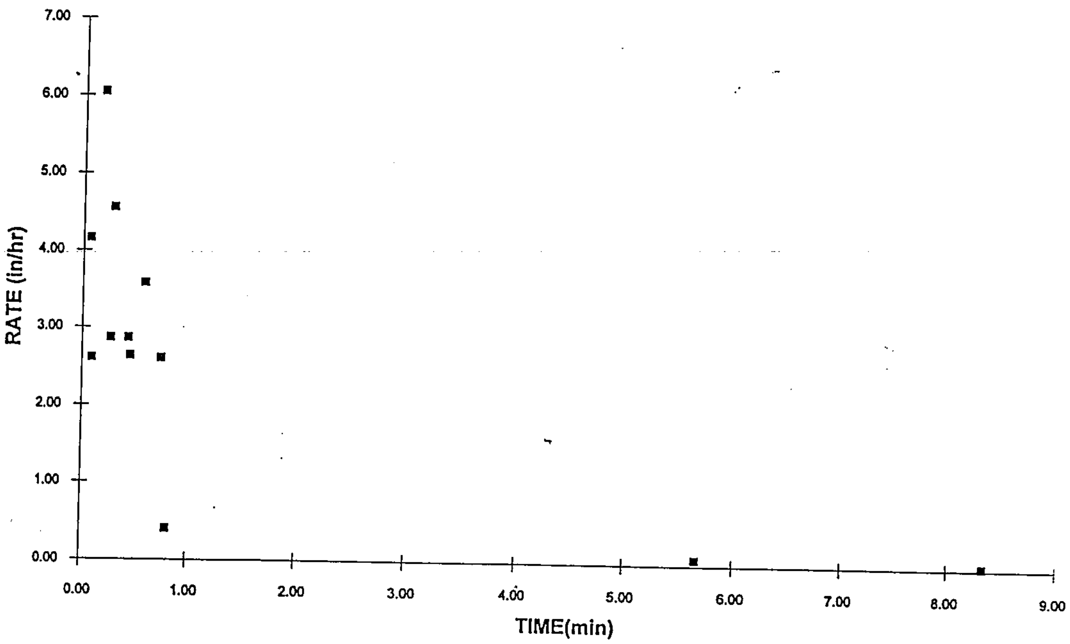


Figure 4-3. Horton Infiltration Rates (H-79/Good Drainage)

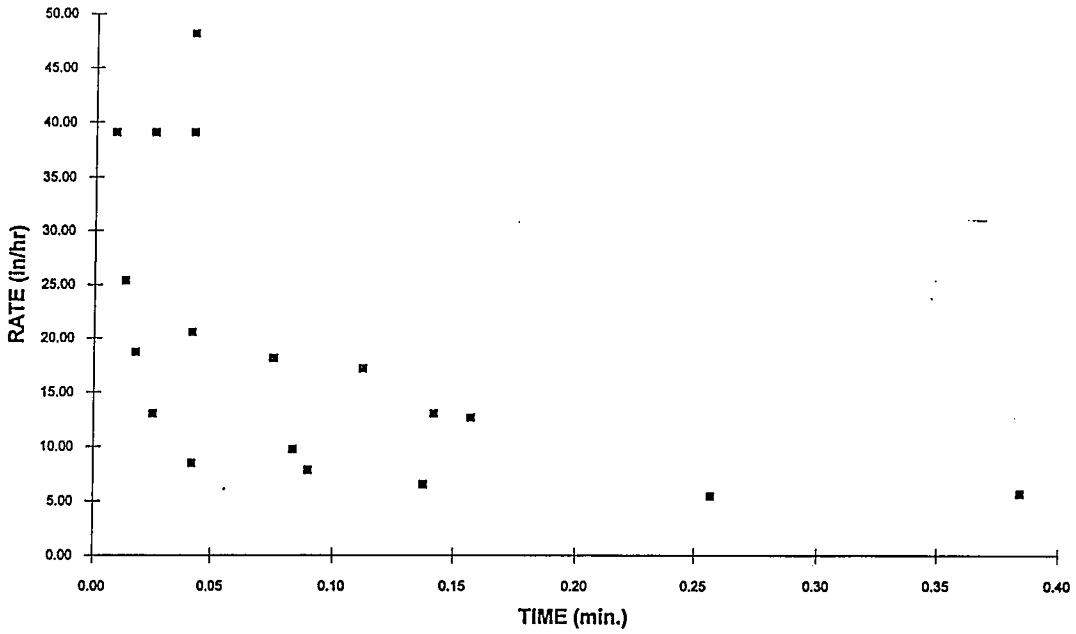
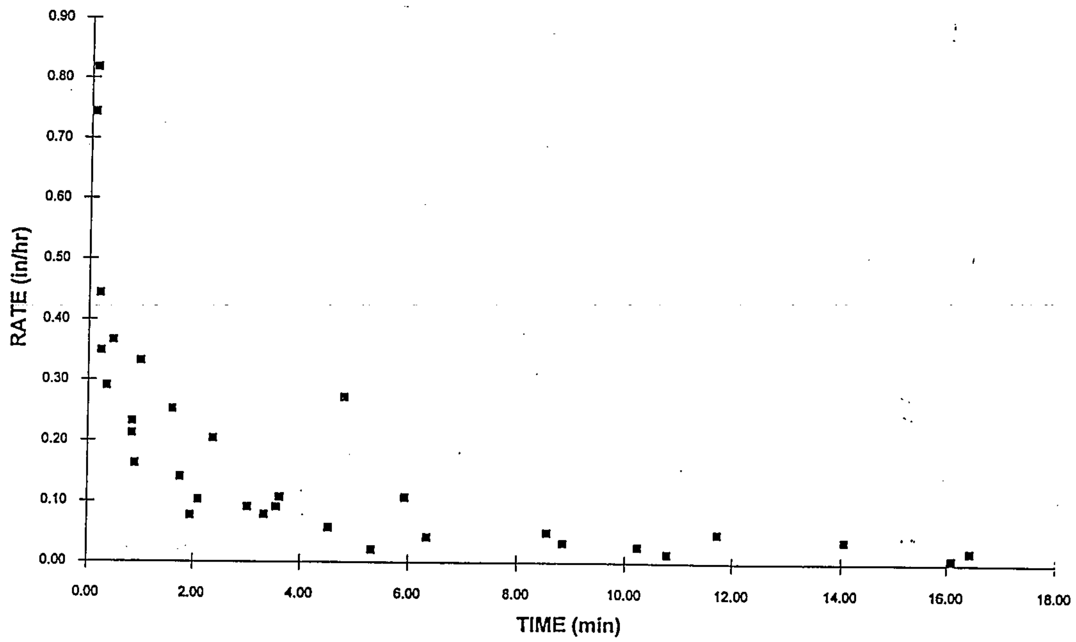


Figure 4-4. Horton Infiltration Rates (H-79/Poor Drainage)



infiltration rates would substantially decrease to close to the typical rates observed at the "normal" sites.

The total amount of water that could infiltrate the normal pavements before they dramatically decreased in their infiltration rates (local saturation, limited by percolation of pavement near the surface) may be about 25 mm (1 inch), based on these tests. The actual value would be somewhat less than this because of lateral flow that probably occurred.

5.0 MOVEMENT OF MOISTURE IN PAVEMENT STRUCTURES

5.1 Overview

The previous two sections of this report have focused on the collection of data and on the physical effects related to the amount of water that will enter a pavement structure. It is the purpose of the present section to discuss mathematical modeling that has been performed with respect to the prediction of the movement and fate of that water once it has entered the aggregate and soil sublayers in the pavement structure. The present section also discusses efforts that were performed to instrument and monitor rainfall and base course saturation conditions at several locations in the Birmingham metropolitan area.

Some discrepancies have been noted between the saturation levels predicted by the mathematical models and those which were observed in the data collection effort. That these discrepancies exist has been able to be observed for the simple reason that this project involved both mathematical modeling and actual data collection. Most previous studies have involved only modeling or only data collection. The discrepancies indicate that there is a need for further improvements to models which are intended to describe the physical processes taking place in highway pavement structures.

The following subsection provides an overview of the physical principles involved in the analysis of flow through porous media, and is followed by a number of additional subsections which describe implementations of the theory through the use of numerical solution codes. Both 1- and 2-dimensional representations of the pavement drainage problem are employed, as are both event-based and continuous simulations. The difference between the 1-D and 2-D representations is that, in the latter, water can move both vertically and laterally in the sublayers. It is assumed in the 1-D case that water can move only laterally (i.e., parallel to the roadway surface). The 2-D formulation is much more difficult to set up and solve, but has the potential of providing much more reliable results. It is also capable of showing how the moisture is distributed throughout the pavement section. The terms event-based and continuous, as used here, have the same meanings as they do in rainfall-runoff modeling applications. In the context of this pavement drainage report, an event-based simulation simply considers the drainage of a pavement section during a rainfall inter-event period under the assumption that the sublayers are initially saturated. That is, the event-based formulations examine no more than the time that it takes a pavement to drain from saturation. The continuous simulations, on the other hand, consider time on a continuous basis and are not constrained to only the inter-event periods. Infiltration into the sublayers can occur during the simulation, and hence one can examine the mechanisms of wetting as well as drying of the pavement structure.

Following the presentations of the mathematical modeling efforts and results, an additional subsection is included to describe the data collection efforts which were performed. Section 6.1.3 of this report remarks on the differences between the actual data and numerical results, and presents some avenues which should be explored.

5.2 Physics of Water Movement in Porous Media

At the most basic level, there are two issues that must be addressed when considering the flow of a liquid and/or gas through a porous medium. The first of these issues relates to the ability of the medium to transmit the fluid(s), and the second relates to the ability of the medium to store or retain the fluid(s). The ability of a saturated porous medium to transmit a fluid is usually expressed in terms of its hydraulic conductivity (Darcy, 1856) or its intrinsic permeability (Nutting, 1930). The ability of a porous medium to store water is related primarily to its overall porosity, though the actual sizes of the pores have a substantial influence on how much of the stored water may be withdrawn. Clays, for instance, have high porosities, and hence are capable of storing large quantities of water, but are not capable of transmitting water easily, nor can water be easily withdrawn from them.

In the case of unsaturated flow, the transmission and storage characteristics of a porous medium become relatively complicated and are dependent on both the degree of saturation of the medium (or, equivalently, its volumetric moisture content) and on the previous wetting and drying history. That is, strictly speaking, the characteristics depend not only on the properties of the porous medium and the fluid, but they depend also on time. For practical purposes, the time effects are usually ignored, and the hydraulic properties of the medium are expressed in terms of saturation-dependent relationships for pressure head and relative permeability. The methods of Brooks and Corey (1964) and van Genuchten (1980) are those most commonly applied for this purpose. Further discussions of these methods are presented in the sequel.

Modeling and prediction of the movement and fate of a fluid in a porous medium involves the application of fundamental physical laws expressing both conservative and constitutive relationships. In applications where moisture movement only is of concern, and where heat or other effects are unimportant, these relationships take the form of a continuity equation expressing the principle of conservation of mass, an integrated and simplified form of the Bernoulli equation expressing the principle of conservation of energy, and a modified form of Darcy's law (Darcy, 1856) expressing a constitutive relationship between the macroscopic fluid velocity (the Darcy velocity) and the total energy gradient. In environments where heat effects may not be negligible, additional equations are required in order to express the principles of conservation of (heat) energy, and to quantify the coupling that occurs between heat and fluid mass transport. The following paragraphs describe these issues in more detail.

5.2.1 Conservation of Mass. In fluid mechanics, as in other subjects addressing continua as opposed to finite and discrete objects, fundamental physical laws must be written with respect to a control region which is usually fixed in space. This control region can correspond to any physical entity, such as a block of soil, and has a capacity to be filled with the fluid of interest (in our case liquid water). The fluid can also, as a consequence of it's motion, cross the boundaries of the control region. In the case of a porous medium containing an incompressible fluid whose movement can take place in one direction only (1-dimensional flow), the principle of conservation of mass can be written as

$$\frac{\partial \theta}{\partial t} + \frac{\partial v_x}{\partial x} = 0 \qquad (5-1)$$

where θ is the volumetric moisture content of the control region (the volume of water in the region divided by the total volume of the region), t is time, v_x is the Darcy velocity of the fluid, and x is the direction in which the fluid motion takes place. In the case of 2-dimensional flow, the corresponding equation is

$$\frac{\partial \theta}{\partial t} + \frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} = 0 \qquad (5\text{-}2)$$

where v_y is the Darcy velocity in the y direction. Both of the equations (5-1) and (5-2) are based on the assumption that liquid water only is of concern. In cases where water vapor transport may also be significant, additional equations may be written to express the necessary conservative relationships.

5.2.2 Conservation of Energy. The integrated form of Bernoulli's equation (Prasuhn, 1980, pp. 134-137) is one of the most widely used laws in applied fluid mechanics. This law states that the total energy Φ per unit weight of fluid at any cross section in the flow is equal to the sum of the potential energy z due to elevation (the elevation head), the potential energy p/γ due to pressure (the pressure head), and the kinetic energy $v^2/2g$ due to the fluid motion (the velocity head). Since the velocity head in subsurface flow is usually negligible in comparison to the pressure and elevation head terms, the Bernoulli equation can be written in the form

$$\Phi = \frac{p}{\gamma} + z \qquad (5\text{-}3)$$

The pressure in a fluid may be either positive or negative when a gage, as opposed to absolute, pressure scale is employed. Positive fluid pressures are normally associated with fully saturated regions in the porous medium, whereas negative fluid pressures are associated with regions of unsaturated flow. Pressures of zero occur on the boundaries between saturated and unsaturated flow. Because of the phenomenon of capillarity, there do exist regions known as capillary fringes, where the porous medium is fully saturated but where the fluid pressure is also negative.

When negative fluid pressures are encountered in porous media, it is common practice to write the Bernoulli equation as

$$\Phi = \psi + z \qquad (5\text{-}4)$$

where z is the elevation head at any point in the flow region with respect to an arbitrary datum, and where $\psi = -p/\gamma$ is the pressure, or suction, head at that point.

In the case of an unsaturated porous medium, there is a relationship that exists between the saturation S (equal to the volume of fluid divided by the volume of pores), or volumetric moisture content θ, of the medium and the suction head ψ. Figure 5.1 is an illustration of a typical soil water retention curve which depicts this relationship. It may be noted there that the relationship is not one-to-one, i.e., that it exhibits hysteresis, as the relationship is different if the soil is drying than if it is wetting. It may also be noted that there is a value of the suction head (see point A in the figure) below which the soil remains completely, or nearly so, saturated. This value of suction head is known as the air entry, or bubbling, pressure head ψ_b, and depends on the nature of the porous medium. The absolute value of the bubbling pressure head tends to be greater for fine-grained materials and clays than for coarse-grained materials. This is manifested by the common observation that capillary forces cause a fluid to rise to greater heights in fine-grained media. A final point which may be noted from Figure 5.1 is that

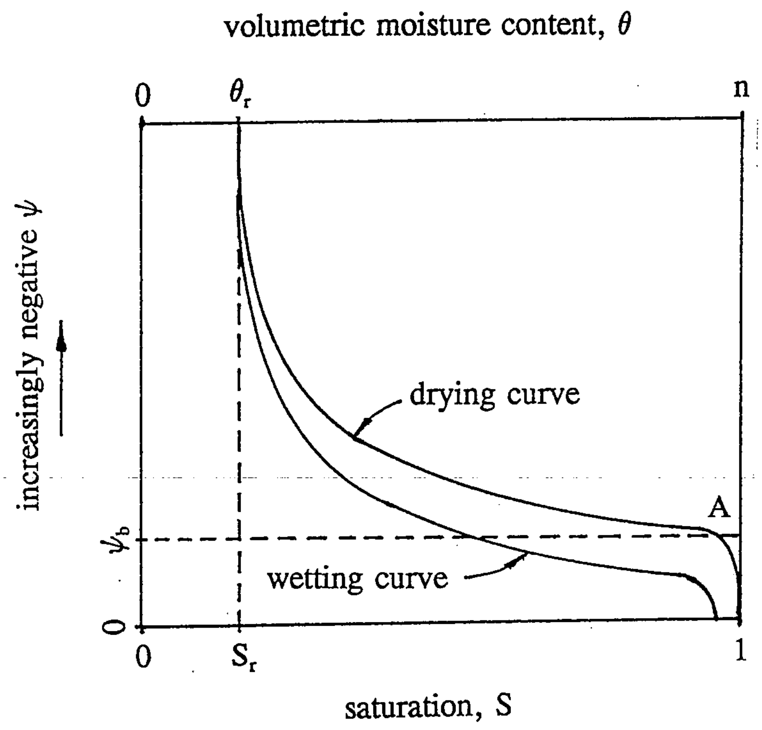


FIGURE 5.1
Typical Soil Water Retention Curve

there is a lower limit to the degree of saturation that can be attained, even after prolonged periods of time. This is known as the residual saturation and is denoted by S_r. The only ways in which the actual saturation S of a porous medium can drop below this value are through evaporation and oven drying. Vegetation can also extract water from soils to result in saturation levels below S_r, but this is unlikely to be of relevance in the context of pavement drainage. The magnitude of S_r, like that of the bubbling pressure head, is dependent on the soil type, and tends to be larger for fine-grained materials than for coarse-grained ones.

Despite the fact that soil water retention curves are hysteretic in nature, it is generally assumed in applications (for the lack of anything better) that the relationship is one-to-one and does not depend on the wetting/drying history of the medium. Mathematical equations that have been proposed to represent soil water retention curves have been given by Brooks and Corey (1964) and van Genuchten (1980). The Brooks and Corey equation is expressed in terms of the saturation S and can be stated as

$$S = \begin{cases} 1; & \psi \le \psi_b \\ S_r + (1-S_r)\left(\dfrac{\psi_b}{\psi}\right)^{\lambda}; & \psi \ge \psi_b \end{cases} \tag{5-5}$$

where λ is an empirical parameter known as the pore size distribution index. The equation suggested by Van Genuchten is cast in terms of the volumetric moisture content θ instead of the saturation S and is given as

$$\theta = \theta_r + \frac{n-\theta_r}{[1+|\alpha\psi|^{a}]^{b}}; \qquad \theta_r \le \theta \le n \tag{5-6}$$

where n is the porosity of the medium, θ_r is the moisture content corresponding to a saturation of S_r, and α, a, and b are empirical parameters.

5.2.3 Darcy's Law. Darcy's law (Darcy, 1856) for flow through porous media, much like the Manning equation for flow in open channels, is a constitutive relationship between the velocity of flow and the total energy gradient. Based on empirical evidence gathered from laboratory experiments, Darcy concluded that this relationship was a linear one and could be expressed as

$$v_x = -K_x \frac{\partial \Phi}{\partial x} \tag{5-7}$$

where v_x is the Darcy velocity in the x direction, Φ is the total energy, and K_x, known as the hydraulic conductivity, is a constant of proportionality. The hydraulic conductivity can be seen to have the same units as velocity (length per time), and is dependent on the properties of the fluid as well as on the porous medium itself. Civil engineers, who are usually concerned with water at a fairly constant temperature, typically treat K_x as a function of the medium only.

Even though Darcy developed Eqn. (5-7) based on empirical information, that same relationship may also be derived by application of Newton's laws (see McWhorter and Sunada, 1977, pp. 65-71). This latter approach provides a more fundamental insight into the nature of flow through porous media, and also shows that the hydraulic conductivity can be expressed as

$$K = \frac{k \rho g}{\mu} \tag{5-8}$$

where g is the acceleration of gravity, ρ is the fluid's density, and μ is the fluid's dynamic viscosity. The term k is called the intrinsic permeability of the porous medium, and has units of length squared. The intrinsic permeability depends only on the porous medium itself, whereas the hydraulic conductivity, because of the influence of ρ and μ, depends on both the fluid and the porous medium.

As an empirical observation, it is found in the case of unsaturated flow through porous media that the hydraulic conductivity, or intrinsic permeability, depends on the volumetric moisture content θ as well. This is a logical conclusion as the presence of air in the voids takes up space in them and reduces the cross-sectional area that is effective in transmitting flows. In order to represent the effects of saturation level on the permeability of a porous medium it is common to use a definition of relative permeability k_r, which is the ratio of the actual permeability $k(\theta)$ at a given saturation, or moisture content, level to the permeability k_s which exists at saturation. That is,

$$k_r = \frac{k(\theta)}{k_s}. \tag{5-9}$$

The general behavior of a typical relative permeability relationship is shown in Figure 5.2. It may be seen from that figure that the permeability of a porous medium to a fluid such as water is zero if the saturation level drops to the residual saturation S_r. In other words, fluid movement, and hence drainage of a pavement structure, halts completely when the level of saturation of the sublayers is equal to S_r. An additional observation which may be drawn from Figure 5.2 is that the curve of relative permeability is frequently quite steep in the region close to saturation. Because of this, even small amounts of air in the voids can have a very significant effect on the actual permeability of the porous medium, which in turn is related to how quickly a pavement structure will drain. This observation tends to emphasize the fallacy of using the saturated hydraulic conductivity as a decisive parameter in pavement drainage.

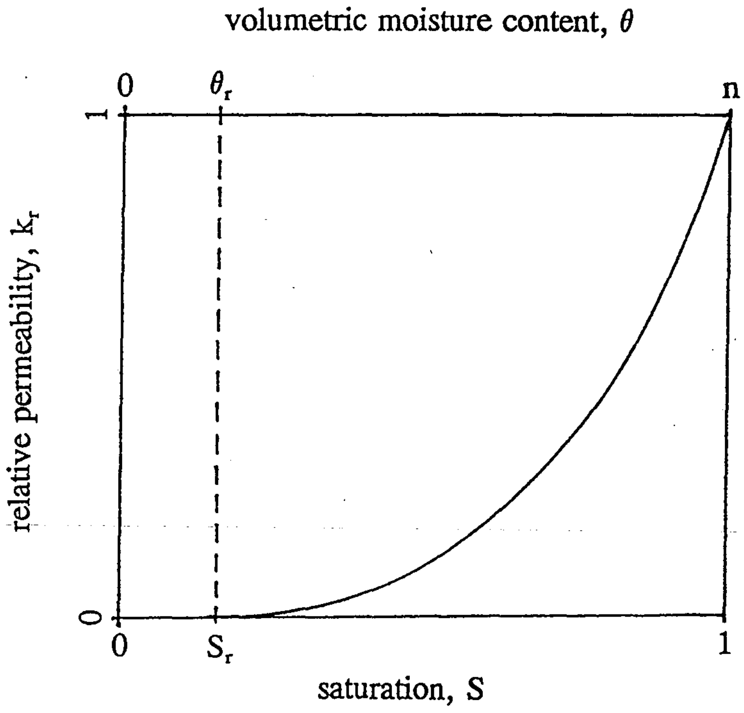


FIGURE 5.2
Typical Relative Permeability Relationship

It should be noted that relative permeability curves, like soil water retention curves, exhibit the phenomenon of hysteresis. Again, however, this is usually ignored and one-to-one mathematical relationships are employed. Brooks and Corey (1964), based on some previous work by Burdine (1953), have suggested the following relationship for relative permeability:

$$k_r = \begin{cases} 1; & \psi \leq \psi_b \\ \left(\dfrac{\psi}{\psi_b}\right)^{2+3\lambda}; & \psi \geq \psi_b \end{cases} \tag{5-10}$$

Note that this relationship gives k_r as a function of ψ rather than of S or θ, but this is valid because ψ itself depends on S or θ. van Genuchten (1980), employing results of Mualem (1976), has suggested that the relative permeability relationship be given as

$$k_r = \left(\frac{\theta - \theta_r}{n - \theta_r}\right)^{1/2} \left\{ 1 - \left[1 - \left(\frac{\theta - \theta_r}{n - \theta_r}\right)^{1/b} \right]^b \right\}^2; \qquad \theta_r \leq \theta \leq n \tag{5-11}$$

The only significant difference between the Brooks and Corey and van Genuchten models is that the van Genuchten model is continuously differentiable at all points. The Brooks and Corey equation exhibits a discontinuity in the first derivative at the value of ψ_b, which can sometimes lead to convergence difficulties when it is employed in numerical modeling codes.

5.2.4 Complete Models. In the case of a 1-dimensional model where flow takes place in a homogeneous material and in a direction x whose positive axis is inclined at an angle α above the horizontal, a substitution of Eqn. (5-4) into Eqn. (5-7), and a substitution of that result into Eqn. (5-1) yields an equation of the form

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial x}\left[D(\theta)\frac{\partial \theta}{\partial x} + K(\theta)\sin\alpha\right] \tag{5-12}$$

where $D(\theta)$ is the soil water diffusivity defined as

$$D(\theta) = K(\theta)\frac{\partial \psi}{\partial \theta} \tag{5-13}$$

A similar model for the two dimensional case, where x is the horizontal direction and y is the vertical direction (positive upwards), is derived by using Eqn. (5-2) instead of (5-1). In this case one obtains for a homogeneous and isotropic material the relationship

$$\frac{\partial\theta}{\partial t} = \frac{\partial}{\partial x}\left(D(\theta)\,\frac{\partial\theta}{\partial x}\right) + \frac{\partial}{\partial y}\left(D(\theta)\,\frac{\partial\theta}{\partial y} + K(\theta)\right) \tag{5-14}$$

Equations (5-12) and (5-14) are alternative forms of Richards' equation (Richards, 1931), and are the equations that have been solved in this study to evaluate the movement of water in pavement structures. Except in some simple and unrealistic cases, analytical solutions of this equation are not known. One is forced, therefore, to resort to numerical methods. Discussions of computer programs which have been employed to solve the equations are provided in the following section.

5.3 Computer Codes

For the purposes of the present study, three different numerical modeling codes have been employed to solve the Equations (5-12) and (5-14) which were derived in the previous section. Two different codes were used in the case of 1-dimensional flow, and one code was used in the case of 2-dimensional flow. All three of the codes employed had been previously developed by other investigators, but some modifications were made to one of the 1-D codes in order to handle the data types that were available in the present effort.

The 1-D codes that were employed in this project consisted of SUBDRAIN and SUBDRAIN-C, both of which were developed by McEnroe and Zou (1993) at the University of Kansas as a part of a KDOT/KTRANS pavement drainage project. SUBDRAIN implements an implicit finite difference scheme for the solution of Eqn. (5-12), and employs the Brooks and Corey (1964) relationships for the unsaturated hydraulic properties. SUBDRAIN simulates the drainage of a pavement base course material, with or without edge drains, from an initially fully saturated condition, and produces as its output both the time to 85 percent saturation and the time to 50 percent drainage. SUBDRAIN-C, which is a generalization of SUBDRAIN, implements a continuous rather than event-based simulation. A primary input to SUBDRAIN-C, as it was modified for the present project, consists of a time series of hourly precipitation amounts. Outputs from SUBDRAIN-C are the probability distributions of average saturation levels of the base course in each month of the year.

The 2-D modeling code that was used in this project consisted of SUTRA, and was developed by the U.S. Geological Survey (Voss, 1984). SUTRA implements a finite element scheme for the solution of an equation of the same general type as that of Eqn. (5-14), and provides a certain amount of flexibility to the user in terms of specification of boundary conditions and unsaturated flow relationships. SUTRA is also capable of simulating either contaminant or heat transport as well as fluid mass transport. It was found in the present study, however, that the solution algorithms would not converge when fluid mass and heat transport were both considered. It was also determined that rather simple piecewise linear approximations to the unsaturated flow properties had to be used in order to obtain solutions. Because of the difficulties that were experienced with the SUTRA code, it was used only for event-based simulations to study the drainage of an initially saturated base course layer.

5.4 1-Dimensional, Event-Based Simulations

As noted in Section 5.3, the computer code entitled SUBDRAIN (McEnroe and Zou, 1993) was employed in this study for 1-dimensional analyses of the drainage of an initially fully saturated base course layer. An illustration of the pavement and base course geometry that is inherent to the SUBDRAIN modeling code is shown in Figure 5.3. It is assumed for simulation purposes that the phreatic surface is initially coincident with the plane of the interface between the top of the base course and the bottom of the pavement, and the simulation of the drainage of the layer proceeds through time until further drainage is not possible. It is assumed by the program that both the pavement and the subgrade underlying the base course are impermeable.

Results provided by the SUBDRAIN program include the minimum degree of saturation which is attainable in the cross section, the time to 50 percent drainage, and the time to 85 percent of saturation. These values are based on spatial averaging of the moisture levels throughout the pavement section. In reality, it is true that some portions of the base course layer do not drain at all, i.e. they remain completely saturated, while other portions drain relatively well. As discussed in Section 5.5, the 2-dimensional modeling analyses which were performed in this project show that the portions of the base course underlying the outer lanes and shoulder

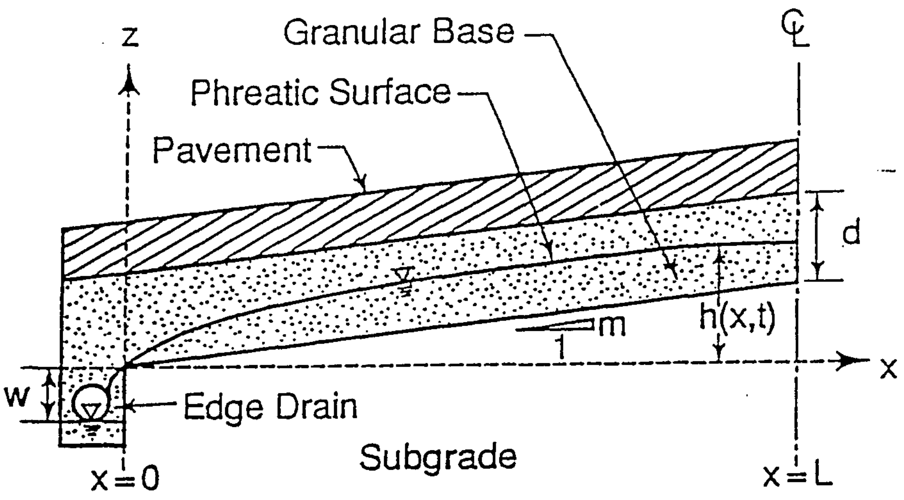


FIGURE 5.3
Pavement Cross Section as Used in Program SUBDRAIN
(Source: McEnroe and Zou, 1993)

areas of pavements are generally poorly drained, if they are drained at all, while the portions underlying the inner lanes near the pavement centerline tend to be better drained.

Geometric variables describing the pavement geometry which must be input to the SUBDRAIN program are, referring to Figure 5.3, the pavement half-width L, the pavement cross slope m $\approx$ sin α, and the base course thickness d. The depth w below the bottom of the base course to the invert of an edge drain must also be specified to the program. If edge drains do not exist, the value of w should be set equal to zero.

Additional variables which must be provided to the SUBDRAIN program relate to the hydraulic properties of the base course material. These properties include the saturated hydraulic conductivity K, the porosity n, the residual saturation S_r, the air-entry, or bubbling pressure head, ψ_b, and the pore size distribution index λ.

Figure 5.4 illustrates a typical input file for the SUBDRAIN program. Auxiliary programs have been developed along with SUBDRAIN and assist the user in the preparation of data files. Values shown in Figure 5.4 for the various input parameters are thought to be

```
 1.  Name of project                              = UAB-UA project
 2.  Hydr. conduct. of base matl., ft/day         = 60.0
 3.  Porosity of base material                    = 0.25
 4.  Resid. water cont. of base material          = 0.05
 5.  Pore-size distrib. index of base matl.       = 4.0
 6.  Air-entry head for base material, in.        = 4.0
 7.  Thickness of granular base, in.              = 12.0
 8.  Max. drainage dist., trans. dir., ft.        = 24.0
 9.  Slope of pavement, trans. dir., %            = 2.00
10.  Depth of depression of edge drain, in.       = 0.00
```

FIGURE 5.4
Typical Input File for Program SUBDRAIN

reasonably representative of a wide range of pavements in Alabama, though there are some uncertainties as to the exact values of some of the parameters, notably those of the air-entry head (or bubbling pressure head) and the pore size distribution index. There will also, of course, be variations in the parameters from one site to another throughout the state.

Even though there are some uncertainties in some of the parameter values that are shown in Figure 5.4, we are not convinced that it is necessary or even desirable to try to improve those estimates, at least not for the purposes of this study. Reasons for this are that these parameters can be both time-consuming and expensive to obtain based on laboratory experimentation, and there is always the question as to whether lab-determined values are representative of those which actually occur in the field. The mathematical models of the physical processes taking place during the drainage and wetting cycles of a pavement base course material are also quite simplified representations of what is actually going on, in that they cannot account for the hysteresis in the unsaturated flow relationships, nor can they account for the phenomenon of air trapping, which certainly occurs during wetting cycles. The models used in this study are also not capable of representing heat effects on the flow behavior, which may be significant in some cases (see Section 6.1).

In view of the simplifications which have necessarily been made in the mathematical modeling efforts, it is not clear that any benefits to this study would be realized by attempting to better ascertain the parameter values. We believe instead that a greater insight into the relative effects of the various factors inherent in the pavement drainage problem may be gained by simply using the models to conduct sensitivity analyses. By systematically adjusting various input parameters to the modeling code while holding all others at a constant value, one can then perform a series of modeling simulations to study how the drainage times, etc., depend on the various input parameters. These results can then provide guidance to pavement designers as to which design parameters have the greatest influences on the pavement drainage characteristics.

With this objective in mind, the following subsections present detailed results that were obtained by using the SUBDRAIN program to study the effects of the various parameters on subdrainage characteristics. First discussed is a baseline, or benchmark, case in which there are no edge drains present. The second subsection then considers the effects of edge drains, and the third subsection presents the detailed results of the sensitivity analyses.

5.4.1 Baseline Parameters. As a benchmark case against which other simulations can be compared, the SUBDRAIN program was executed using the parameter values shown in Figure 5.4. As already stated, these parameter values are believed to be reasonably representative of many pavement designs that are used in Alabama. Results obtained for the baseline case are as follows:

Minimum saturation after prolonged gravity drainage:	36.8 percent
Elapsed time to reach 85 percent saturation:	4.4 hours
Elapsed time to attain 50 percent drainage:	15.2 hours

When 50 percent drainage has occurred, the degree of saturation is equal to 68.4 percent, i.e. it is the average of the minimum saturation level of 36.8 percent and the full saturation level of 100 percent.

Based on a comparison of the result for the time to 50 percent drainage to the water removal times given in Table 1.2, one would be inclined to classify the quality of drainage of this baseline case as being "good". These results do not provide enough information, however, to assign a drainage coefficient m_i based on Table 1.1. It should also be recalled that these results are obtained in the SUBDRAIN program by spatially averaging the saturation levels within the base course layer. In actuality, the region of the base course layer near the pavement edge will have a minimum saturation level greater than the spatial average, while the region near near the pavement centerline will have a minimum saturation level less than the spatial average.

5.4.2 Edge Drain Effects. To see how the presence of edge drains affects the base course drainage characteristics, SUBDRAIN was again executed with the same parameter values as shown in Figure 5.4, except that the edge drain depression distance w (see Figure 5.3) was taken to be 12 inches. Results that were obtained in this case are:

Minimum saturation after prolonged gravity drainage:	20.2 percent
Elapsed time to reach 85 percent saturation:	3.1 hours
Elapsed time to attain 50 percent drainage:	18.0 hours

The degree of saturation after 50 percent drainage is equal to 60.1 percent.

It is interesting to compare the results for this simulation with those which were obtained for the baseline case discussed in Subsection 5.4.1. It can be seen that the provision of edge drains reduces the spatially averaged minimum saturation level and that it reduces the elapsed time to reach a saturation level of 85 percent. The elapsed time to attain 50 percent drainage has increased, however, by the provision of edge drains. This conclusion does make sense, however, and is not an indication that the modeling code is faulty. Note in the baseline case that the saturation level after 50 percent drainage is 68.4 percent while that in the present case with edge drains is 60.1 percent. More water has been drained from the base course layer in the edge drain case, which naturally takes more time.

Based on Table 1.2, and on the interpretation that the times given there relate to the time to 50 percent drainage, one would conclude based on the simulation results that the provision of edge drains actually reduces the quality of drainage of a pavement section rather than improves the quality of drainage. This, of course, is nonsense, but it does suggest that there should be some re-thinking of the AASHTO criteria for determining suitable values of the drainage coefficients to be used in pavement design procedures. McEnroe (1994) has noted this as well and has suggested that the time to 85 percent of saturation is more meaningful than is the time to 50 percent drainage as an indicator of the quality of drainage of a pavement section. It is not clear, however, that the m_i coefficients given in Table 1.1 should be the same if this alternative interpretation of water removal times were to be used.

5.4.3 Sensitivity Studies. One benefit which may be realized by the construction of a mathematical simulation model is that it can be employed to study the behavior of a physical system over a wide range of conditions. Because of this trait, one can employ the model to gain insight into the relative effects of various parameters and conditions to facilitate decision-making as to which are most important and have the greatest influence on the system behavior. This insight can usually be gained much more efficiently and economically using models than by performing large numbers of field or laboratory experiments. Of course, real data provided by experimental results are necessary for the calibration and validation of numerical models. Also, the degree to which models can be reliably used when used to predict conditions for which they were not calibrated (i.e., for extrapolation purposes) depends on the quality of the model in terms of its accuracy of representation of the various physical processes involved. Fortunately, most models tend to be reasonably robust in terms of their abilities to delineate general trends, even though the values which are output do not agree exactly with experimental data. This is disturbing to some, who have developed strong distrusts of models because they do not always agree with real data. Clearly, these individuals are expecting too much from a model, and do not fully understand either the limitations of models or the purposes for which models are built.

The following paragraphs, each of which is devoted to a particular parameter which is relevant to the pavement drainage problem, provide insight into the effects of that parameter on pavement drainage. One's perspective in reviewing the figures presented should focus on general trends. Exact values of the response parameters shown in the figures (the times and minimum degree of saturation) should be ignored.

Hydraulic Conductivity

Figures 5.5 and 5.6, respectively, illustrate the effect of changing the hydraulic conductivity of the base course material. Figure 5.5 shows that changes to the hydraulic conductivity alone has no effect on the minimum degree of saturation which may be attained, and tends to emphasize the fact that hydraulic conductivity is only one hydraulic property of the base course that must be considered. Of course, there is some degree of correlation that exists between hydraulic conductivity and other hydraulic parameters of interest, but the effects of this

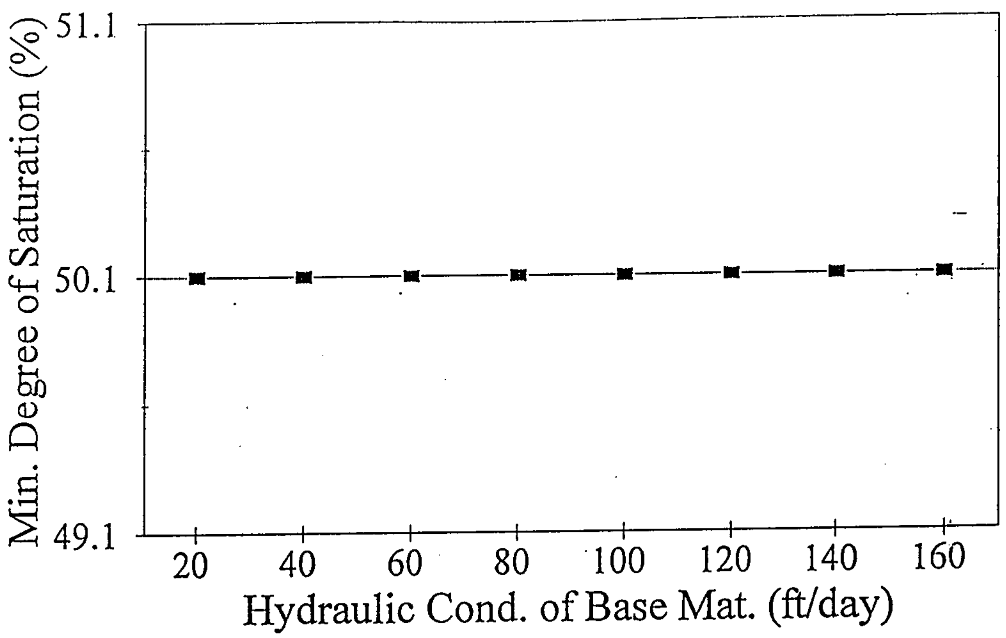


FIGURE 5.5
Effect of Hydraulic Conductivity on Minimum Degree of Saturation

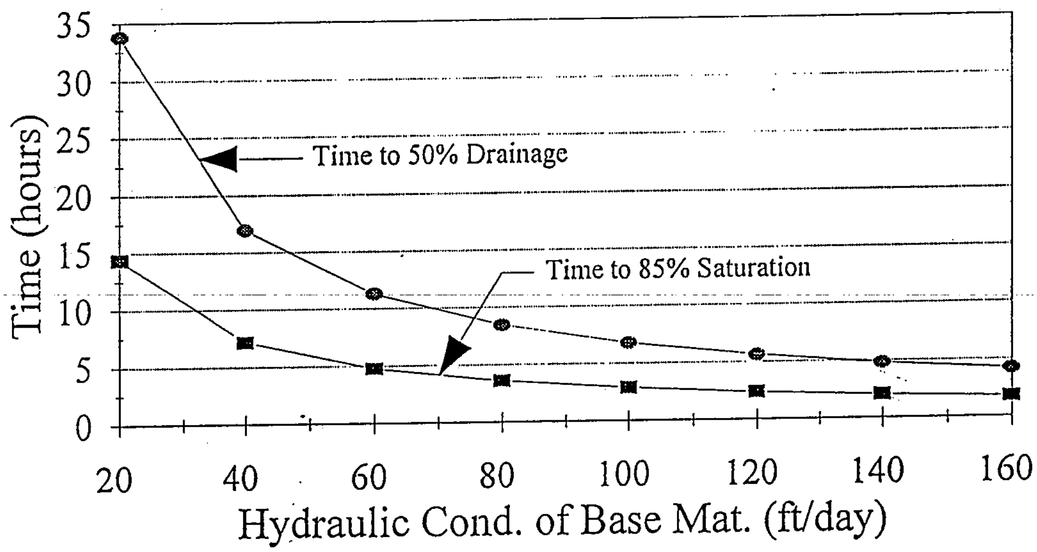


FIGURE 5.6
Effect of Hydraulic Conductivity on Drainage Time

correlation are not indicated in Figure 5.5. The effect of this correlation would probably be manifested by a decreasing minimum degree of saturation with increasing hydraulic conductivity. Figure 5.6 shows clearly that the times to 50 percent drainage and 85 percent saturation both decrease when the hydraulic conductivity is increased; this conclusion is obvious.

Porosity

An increase in the porosity of the base course material, as illustrated in Figure 5.7, will be accompanied by a decrease in the minimum degree of saturation. This can be rationalized on the basis of pore sizes, which in granular materials, tend to be larger when the porosity is larger. When pores, and hence the porosity, of the material are small, capillary forces are strong and may completely prevent drainage of the pores. The opposite is true for large pores.

Figure 5.8 shows that increasing the porosity tends to increase the drainage times, though the time to 50 percent drainage increases must faster than does the time to 85 percent saturation. In fact, the latter is almost independent of porosity. Rationalization of these results may be made on the basis that when the porosity is increased, there is more water in the pavement section which must be drained. This naturally takes more time. Not depicted in Figure 5.8 is the effect of the correlation which likely exists between porosity and hydraulic conductivity. As porosity increases, so should hydraulic conductivity, and this effect would modify the results presented, though in an unknown way.

Residual Water Content

Since the residual water content is the water content of a material after prolonged gravity drainage, the illustration in Figure 5.9 that the minimum degree of saturation increases with the residual water content should be obvious. The behavior shown in Figure 5.10 that the time to 50 percent drainage should decrease with increases in the residual water content can be explained on the basis of less water actually being drained. That is, an increase in the residual water content means that less water actually drains from the pavement section. Naturally, the amount of time for drainage should therefore also decrease. An opposite effect is noted in the case of

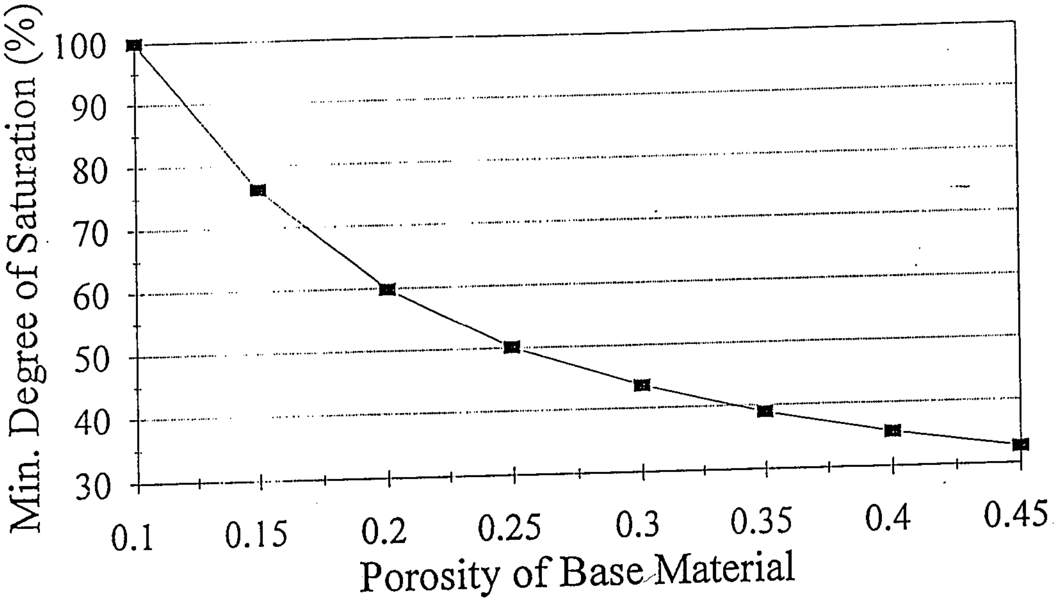


FIGURE 5.7
Effect of Porosity on Minimum Degree of Saturation

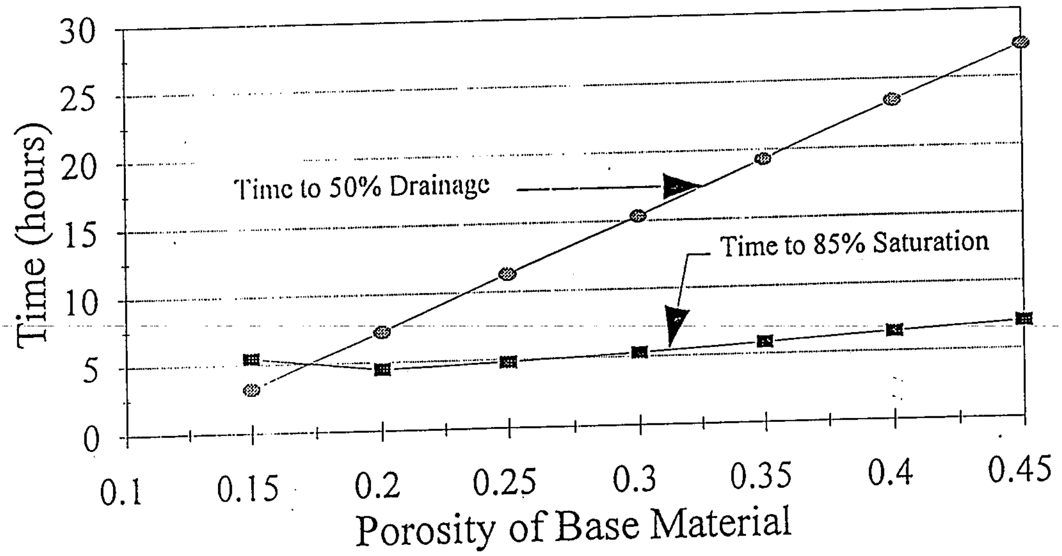


FIGURE 5.8
Effect of Porosity on Drainage Time

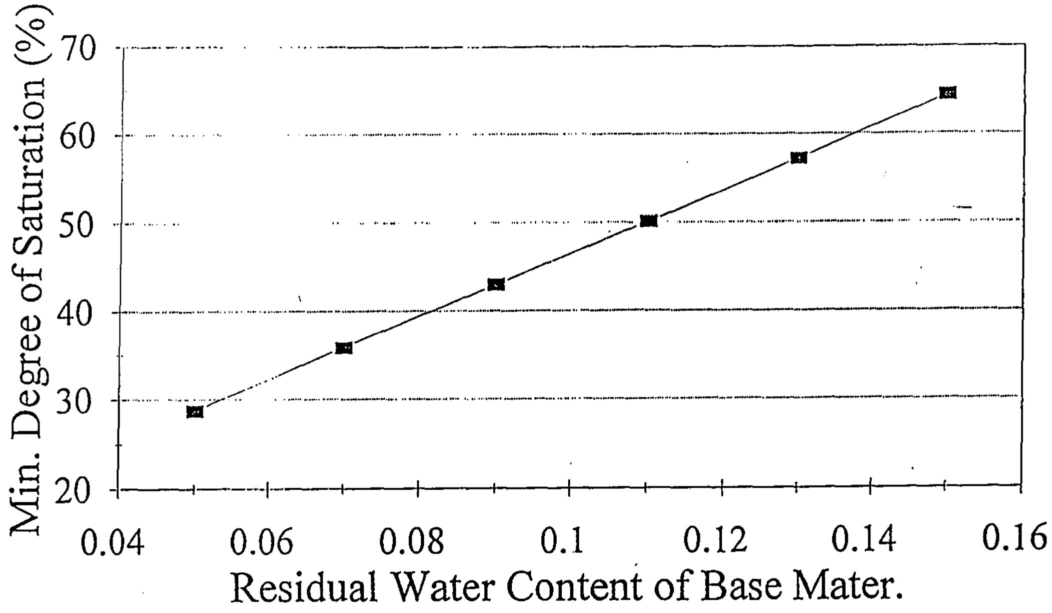


FIGURE 5.9
Effect of Residual Water Content on Minimum Degree of Saturation

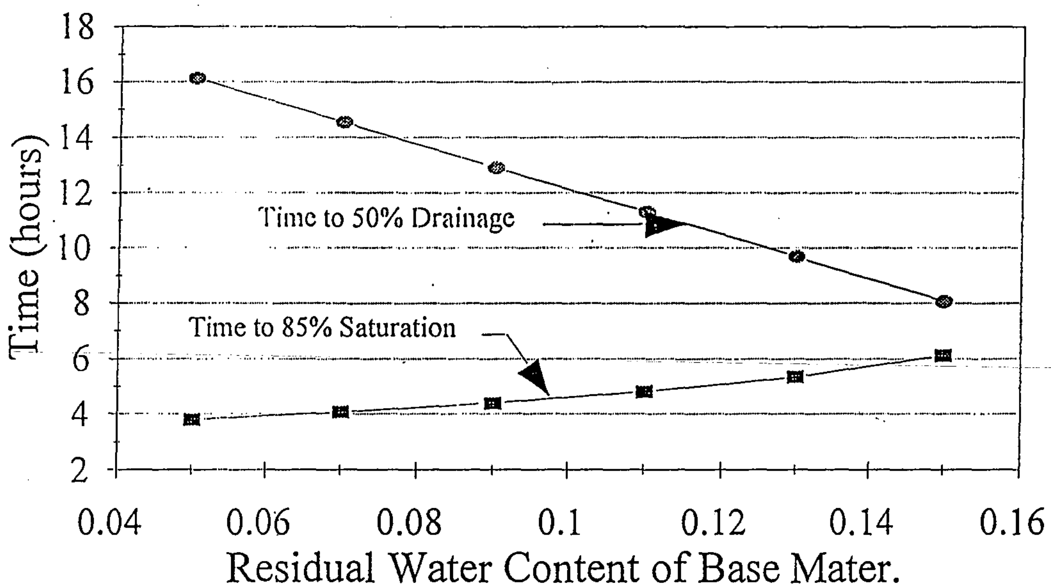


FIGURE 5.10
Effect of Residual Water Content on Drainage Time

the time to 85 percent saturation and, while this may seem at first to be hard to explain, it can be rationalized based on the recognition that if the residual water content were to be very high, drainage to a level of 85 percent saturation might not even be possible. Indeed, for higher and higher residual water contents, the drainage time to any particular degree of saturation should approach infinity.

Pore Size Distribution Index

Figure 5.11 shows that increasing the pore size distribution index should result in a decreased minimum degree of saturation. Poorly graded soils, in which the grain sizes are fairly uniform, tend to be characterized by high values of the pore size distribution index. Similarly, well graded soils tend to be characterized by small values of the pore size distribution index. Where soils are well graded, and hence have a significant number of very small pores which can not be drained by gravity, it is logical that the minimum degree of saturation should be higher.

Figure 5.12 indicates that drainage times are relatively insensitive to the pore size distribution index. This suggests that this parameter is of primary importance in the area of water retention characteristics, and has almost no bearing of water transmission characteristics.

Air Entry Pressure Head

The air entry pressure head can be thought of as a measure of the thickness of the capillary fringe in a porous medium. The capillary fringe is a region which remains saturated even though it lies above the phreatic surface. Thus, in a relatively confined region such as the layer of base course material in a pavement section, the thicker is the capillary fringe, the smaller will be the amount of water that actually drains from the layer. This is depicted clearly in Figure 5.13, which shows that an increase in the air entry pressure head is accompanied by an increase in the minimum degree of saturation.

Figure 5.14 shows that the time to 50 percent drainage decreases as the air entry pressure head is increased, and that the time to 85 percent saturation tends to increase with increasing air entry pressure head. In an extreme case where the air entry pressure head is very large, and

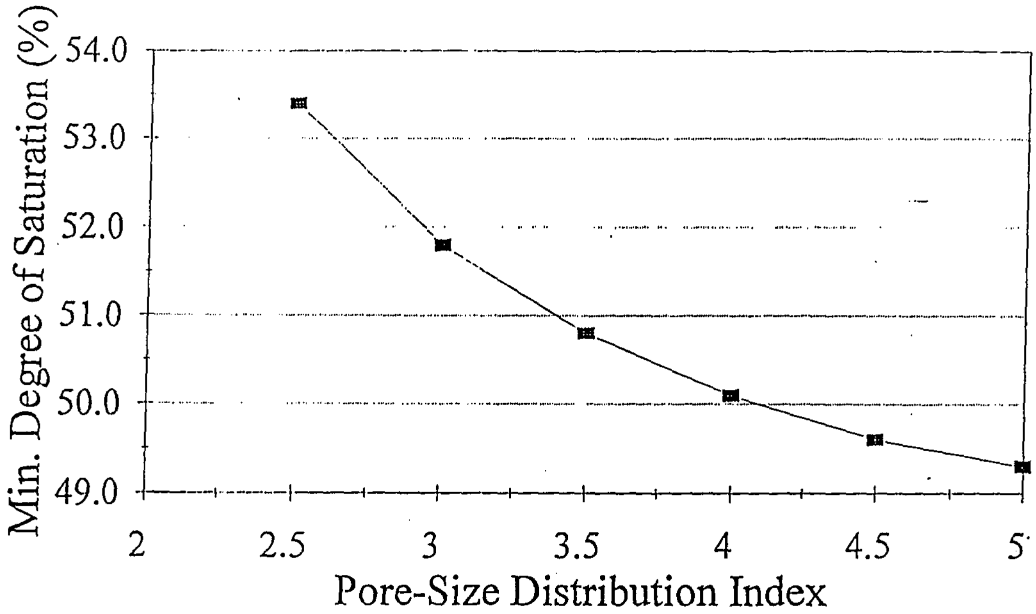


FIGURE 5.11
Effect of Pore Size Distribution Index on Minimum Degree of Saturation

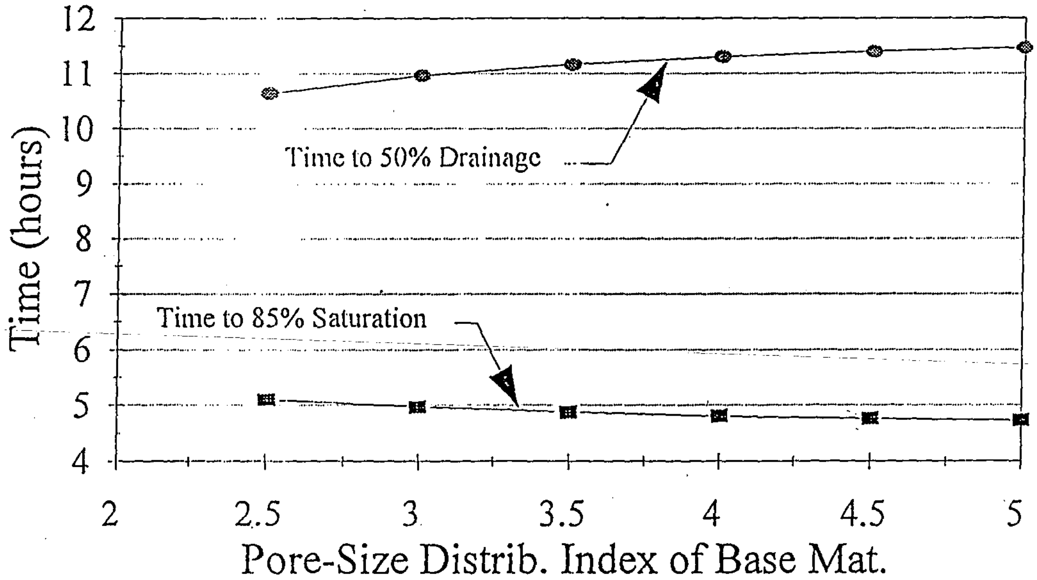


FIGURE 5.12
Effect of Pore Size Distribution Index on Drainage Time

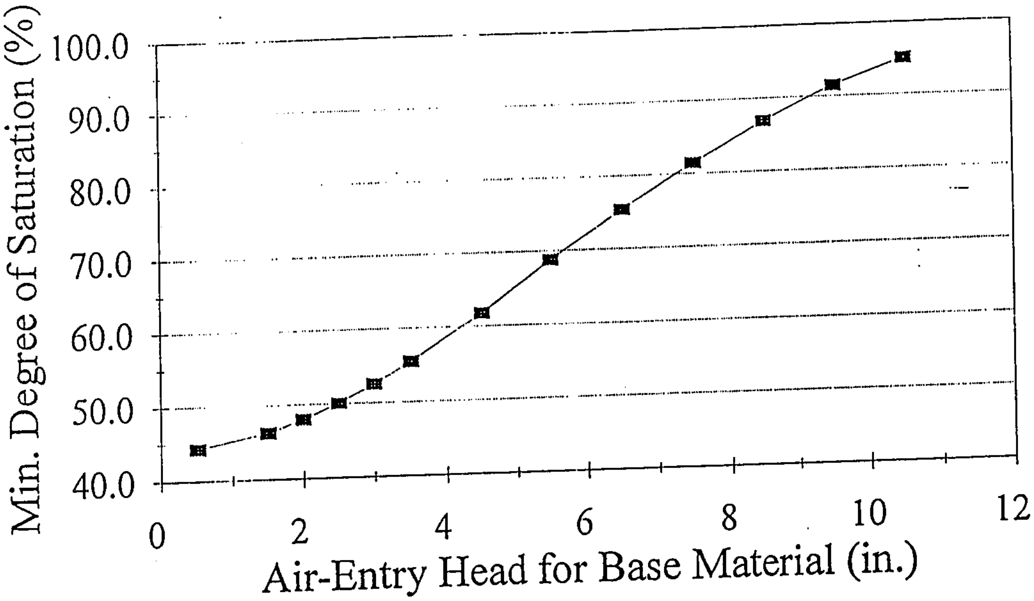


FIGURE 5.13
Effect of Air Entry Pressure Head on Minimum Degree of Saturation

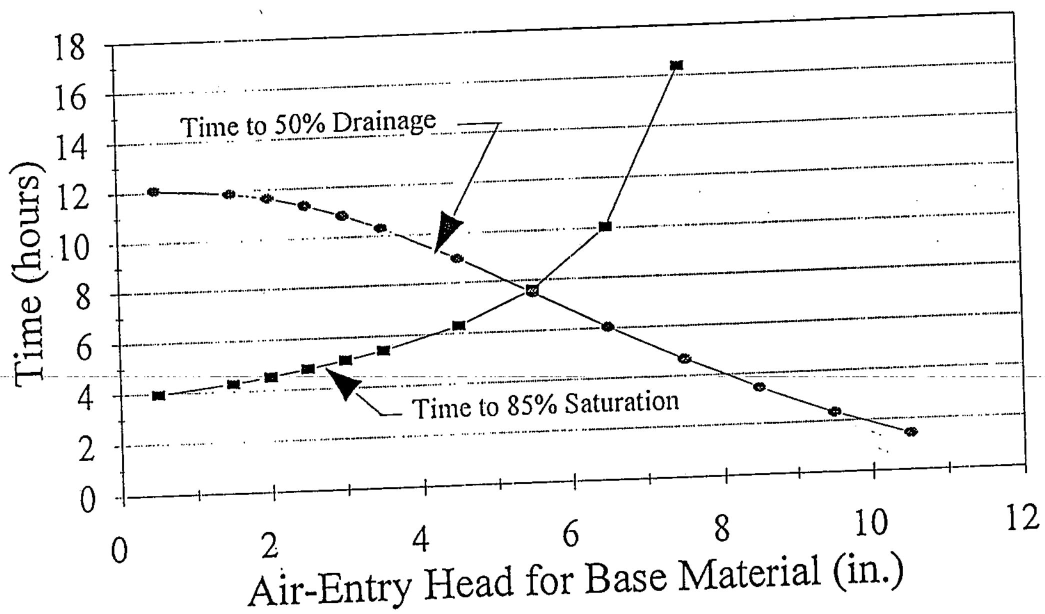


FIGURE 5.14
Effect of Air Entry Pressure Head on Drainage Time

hence the capillary fringe is very thick, the pavement would not drain at all and the time to 85 percent saturation would be infinite. Since no water would drain at all, it would take zero time (the time to 50 percent drainage) for one-half of no water to drain.

Base Course Thickness

Assuming that the air entry pressure head, i.e. the capillary fringe thickness, is a constant, an increase in the thickness of the base course layer will also result in an increase in the fraction of the base course layer that can drain to moisture contents which are less than full saturation. When the degree of saturation is then spatially averaged over the entire base course layer, the increased layer thickness will result in a decreased average minimum degree of saturation. This is clearly evident in Figure 5.15.

Figure 5.16 shows that both the time to 50 percent drainage and the time to 85 percent saturation decrease when the base course layer thickness is increased. This can be explained on the basis of the fact that when there is a greater saturated thickness (at the beginning of the simulation), the total head gradient in Darcy's law will be greater and the flow velocity will thus increase. The greater flow velocity obviously causes drainage to occur over a shorter time period.

Pavement Width

The width of a highway pavement, which is clearly related to the distance over which subsurface water must travel laterally to reach either the shoulder or an edge drain, has a strong influence on the amount of time that it takes for drainage to occur. Figure 5.18 shows this behavior clearly.

Figure 5.17 shows that the spatially averaged minimum degree of saturation tends to decrease as the pavement width (or flow path length) is increased. Assuming that the subgrade underlying the base course is impermeable, as has been done in this modeling effort, then the phreatic surface at the completion of drainage will coincide with the bottom of the base course layer at the shoulder. All portions of the base course layer lying below a horizontal line drawn

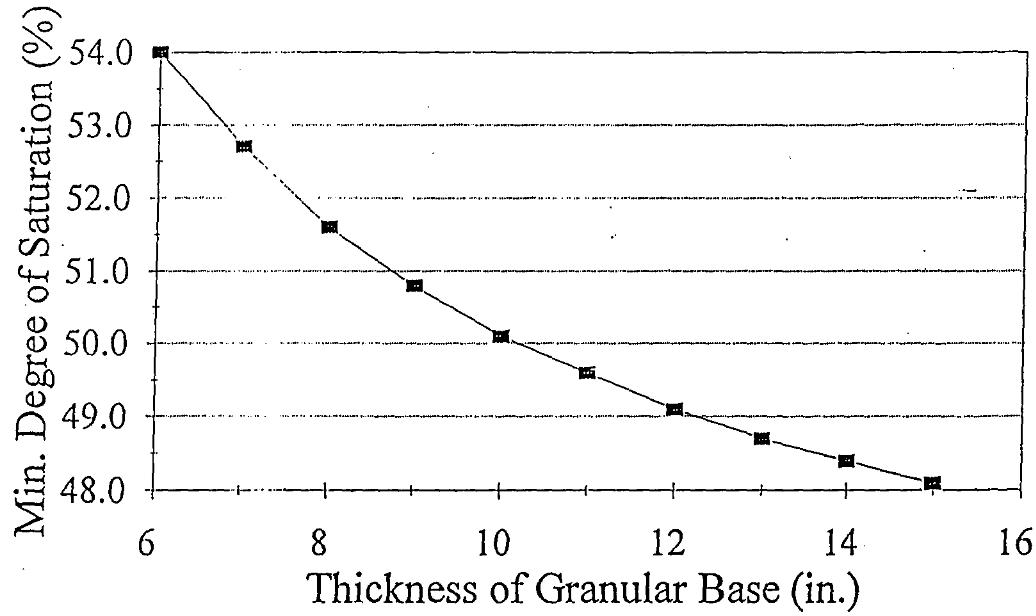


FIGURE 5.15
Effect of Base Course Thickness on Minimum Degree of Saturation

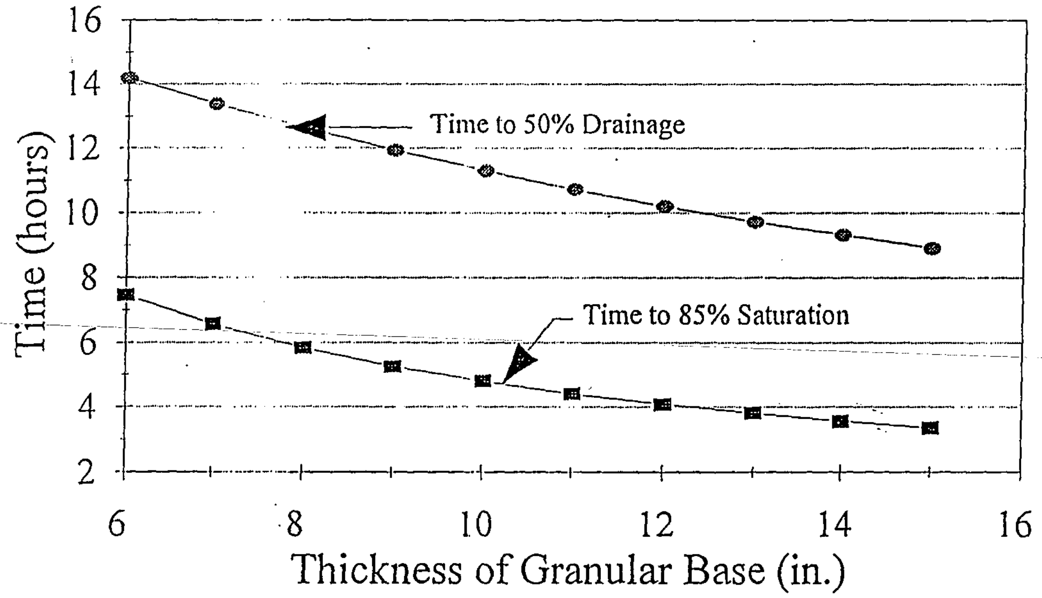


FIGURE 5.16
Effect of Base Course Thickness on Drainage Time

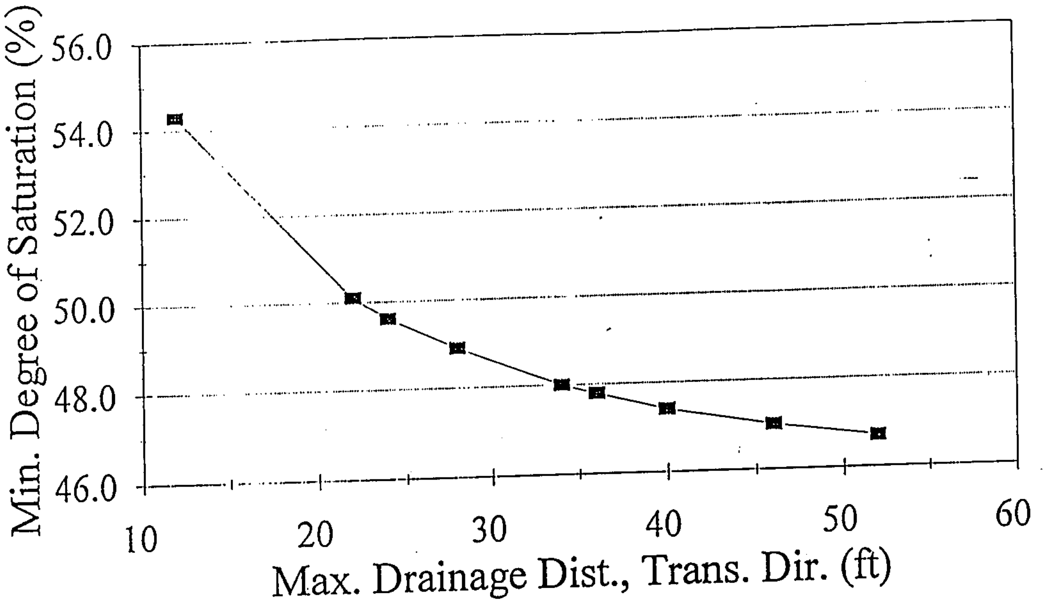


FIGURE 5.17
Effect of Pavement Width on Minimum Degree of Saturation

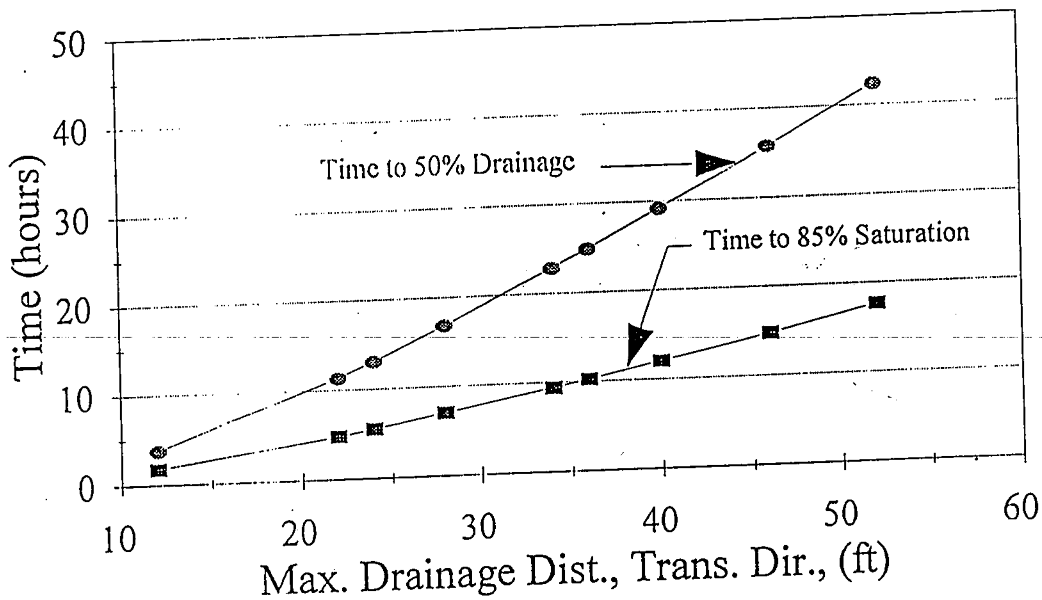


FIGURE 5.18
Effect of Pavement Width on Drainage Time

ψ_b units above that point will be saturated, and portions of the base course layer above that line will be drained, though to varying degrees. Thus, for a given pavement transverse slope, increasing the pavement width will increase the fraction of the total base course area which lies above the horizontal line. After spatial averaging, one will then conclude that the minimum degree of saturation tends to decrease as the pavement width is increased.

Pavement Transverse Slope

The same reasoning that was just described for the case of pavement width can be used to rationalize the behavior illustrated in Figure 5.19, which shows that the minimum degree of saturation decreases as the pavement transverse slope is increased. Increasing the slope tends to result in an increased fraction of the total base course area which lies above the horizontal line referred to earlier, and hence results in a decrease in the spatially averaged value of the minimum degree of saturation.

Figure 5.20 shows that both the time to 50 percent drainage and the time to 85 percent saturation tend to decrease as the pavement transverse slope is increased. Much like the case where the base course thickness is increased (as described earlier), this results in an increase in the total head gradient in Darcy's law. The increased flow velocities therefore give rise to shorter drainage times.

5.5 2-Dimensional, Event-Based Simulations

SUTRA, which is an acronym for Saturated-Unsaturated TRAnsport, is a finite element code that has been developed by the U.S. Geological Survey (Voss, 1984) for simulation of saturated and/or unsaturated flow in subsurface systems. The code can also be used to simultaneously simulate either the transport of a contaminant or the transport of thermal energy in the groundwater, but coupling per se of the various processes is not accounted for. That is, the modeling code does not have the ability to represent heat- or concentration-driven flows.

For the purposes of this project, where unsaturated flow is of primary interest, but where heat effects are of some interest as well, it was attempted to use the SUTRA code to compute both saturation and temperature levels throughout the base course layer. It was assumed that

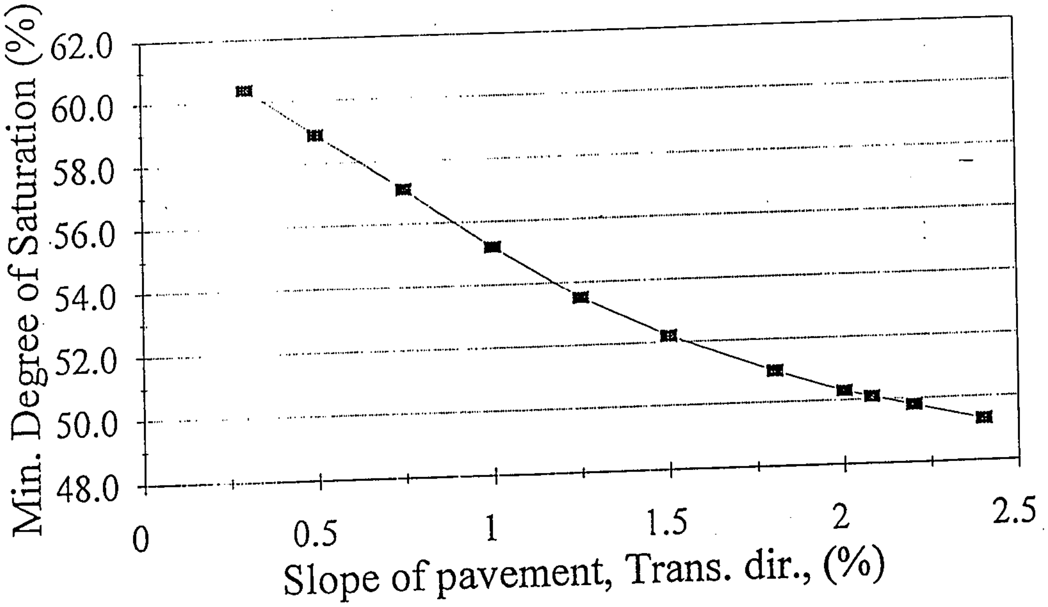


FIGURE 5.19
Effect of Pavement Transverse Slope on Minimum Degree of Saturation

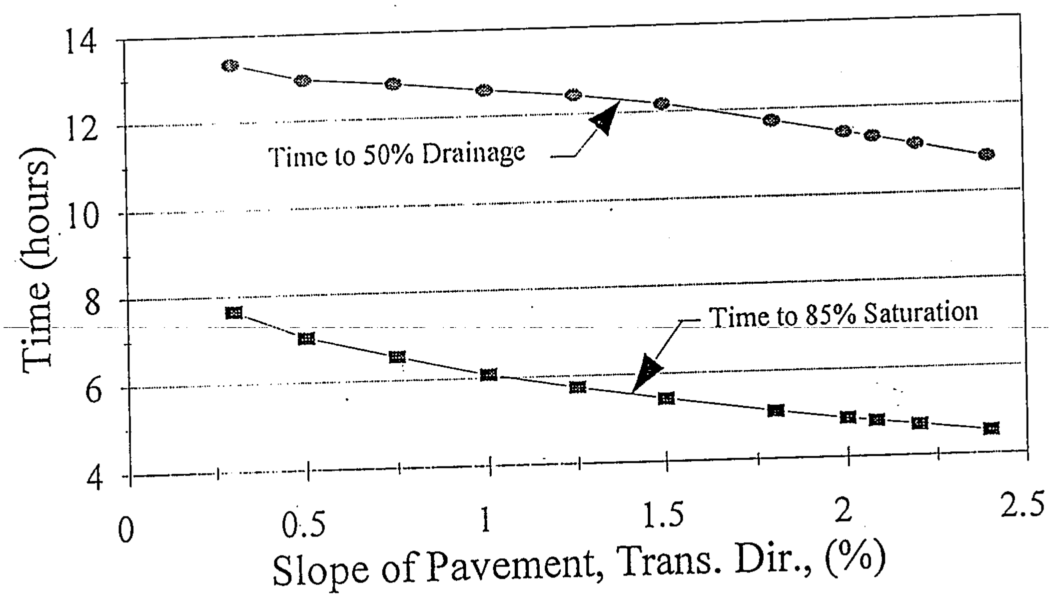


FIGURE 5.20
Effect of Pavement Transverse Slope on Drainage Time

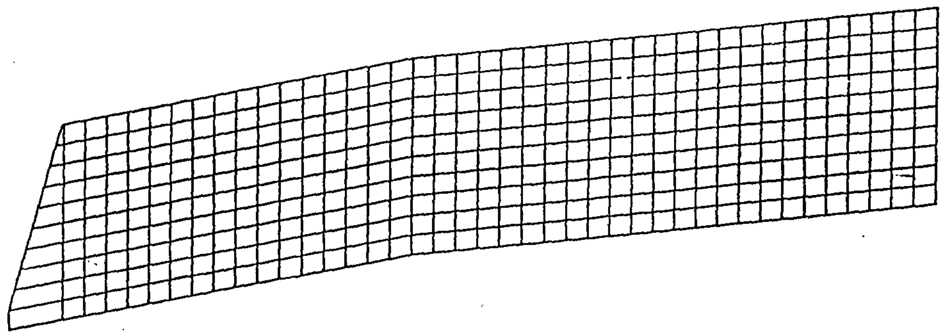


FIGURE 5.21
Finite Element Mesh Used for SUTRA Simulations

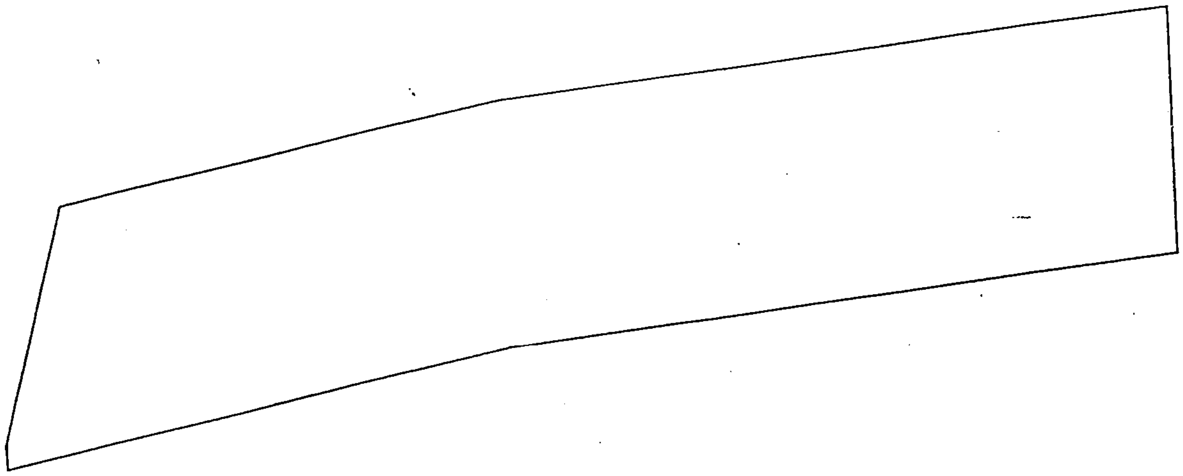
both the pavement overlying the base course and the subgrade materials were impervious, as the code permits unsaturated hydraulic properties to be defined for one material type only. Since SUTRA, as originally developed, implemented the Van Genuchten hydraulic relationships (see Equations 5-6 and 5-11), it was also attempted to use those relationships. The finite element mesh, consisting of quadrilateral finite elements, which was used to represent a roadway base course layer is shown in Figure 5.21. The vertical scale of that figure has been distorted to better illustrate the computational geometry.

A number of difficulties were experienced with the SUTRA modeling code which were computational in nature. It seems that the computational techniques employed in the code are simply not powerful or robust enough to adequatedly handle the difficulties encountered when trying to simulate both heat transport and unsaturated flow. Indeed, when attempts were made to simulate both of these processes, the iterative calculations necessitated by the implicit nature of the code would simply not converge, even when a time step size as small as 1 second was tried. To overcome this difficulty, it was decided to neglect heat effects entirely. Since the code was not of a coupled type, the information lost by neglecting this effect would be minimal anyway.

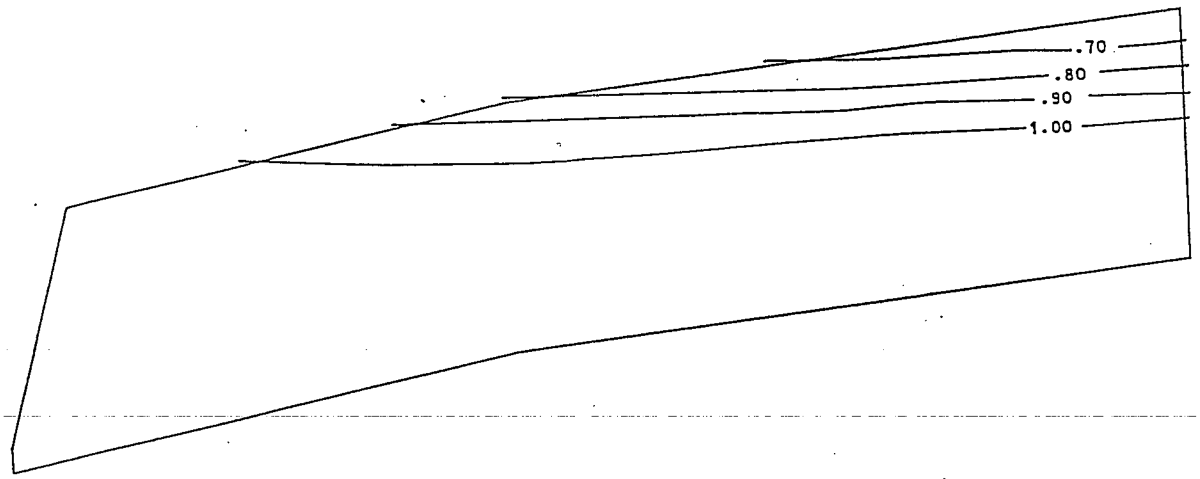
Even when heat effects were removed from consideration, it was found that the code still experienced severe difficulties with convergence. Discussions with Clifford Voss, the original developer of the code, indicated that the problem was due to the extreme nonlinearity of the Van Genuchten hydraulic relationships, and that a solution to the problem was to use simple piecewise linear approximations to the curves shown in Figures 5.1 and 5.2. Modifications were made to the computer code to accomplish this, and even then a non-iterative, as opposed to iterative, solution scheme had to be employed. Since the use of a non-iterative solution technique is accompanied by a potential for inaccurate and/or unstable solutions, a number of simulation runs were made with various time step sizes in order to determine the largest time step size that could be used without introducing significant changes in the solution.

Results of two simulations that were accomplished using the SUTRA code are shown in Figures 5.22 and 5.23. The only difference in these two simulations was in the value used for the saturated intrinsic permeability k_s (see Equations 5-8 and 5-9) of the base course layer. Contours shown in Figures 5.22 and 5.23 are curves of constant saturation level (the 0.70 contour, for example, is the locus of points where the degree of saturation is 70 percent). It was assumed in the simulations that the layer of base course material was initially completely saturated (S = 1.00 everywhere), and the simulations were ran until drainage was complete.

Several conclusions may be drawn from an inspection and comparison of Figures 5.22 and 5.23. First, the initial drainage rate (say between the times of zero and 1 hour) is more rapid when the permeability is greater, but the difference in the rates seems to diminish as time progresses. At time t = 3 hours, both simulations indicate that drainage is nearly complete. A second conclusion which is evident is that, at the completion of drainage, the distribution of moisture is essentially the same regardless of the permeability. That is, the permeability affects the rate at which a layer will drain, but says nothing about how well (in terms of the ultimate degree of saturation) the layer will drain. A final conclusion which has been drawn from these simulation results is that while the region of a pavement structure near the centerline of the roadway may drain to relatively low saturation values, the regions near the pavement edges will tend to remain at much higher levels of saturation. This conclusion is based on the assumption that the subgrade material is impervious however, which is likely a reasonable assumption for much of Alabama because of the clayey soils. It is expected that this conclusion may not be valid where the subgrade material is quite permeable, but the modeling code limitations did not permit this to be explored.

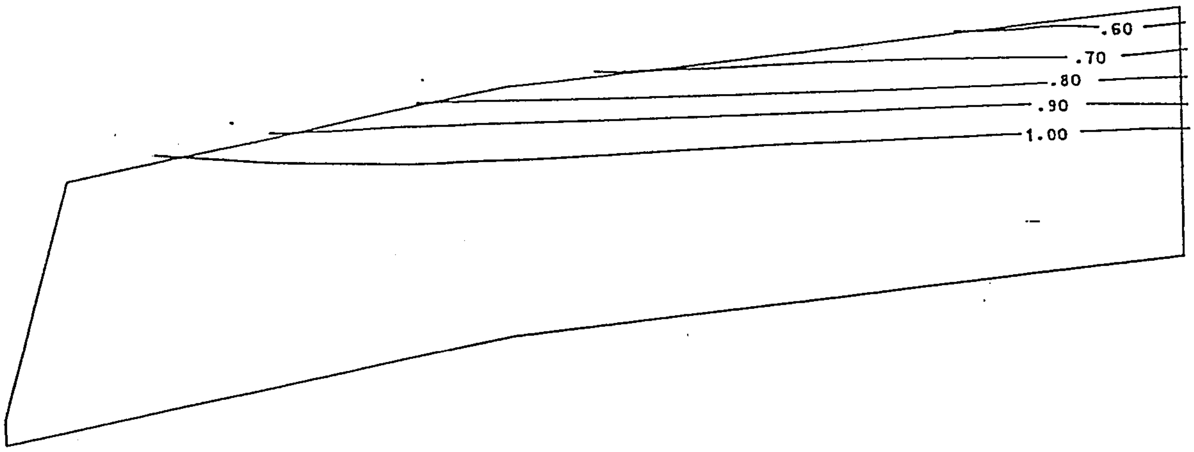


t = 0 hours

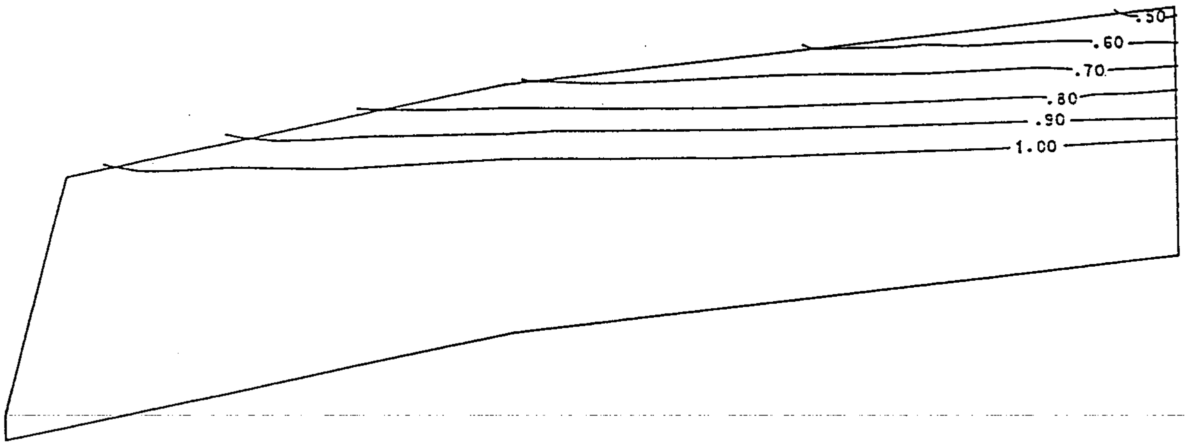


t = 1 hour

FIGURE 5.22
Simulation Results for Base Course Layer Saturation (k_s = 2×10^{-8} m^2)

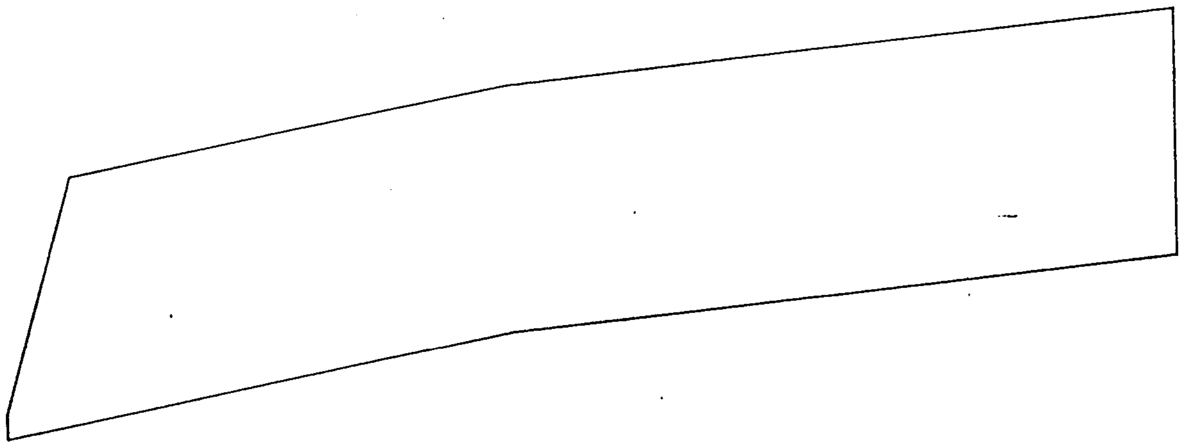


t = 2 hours

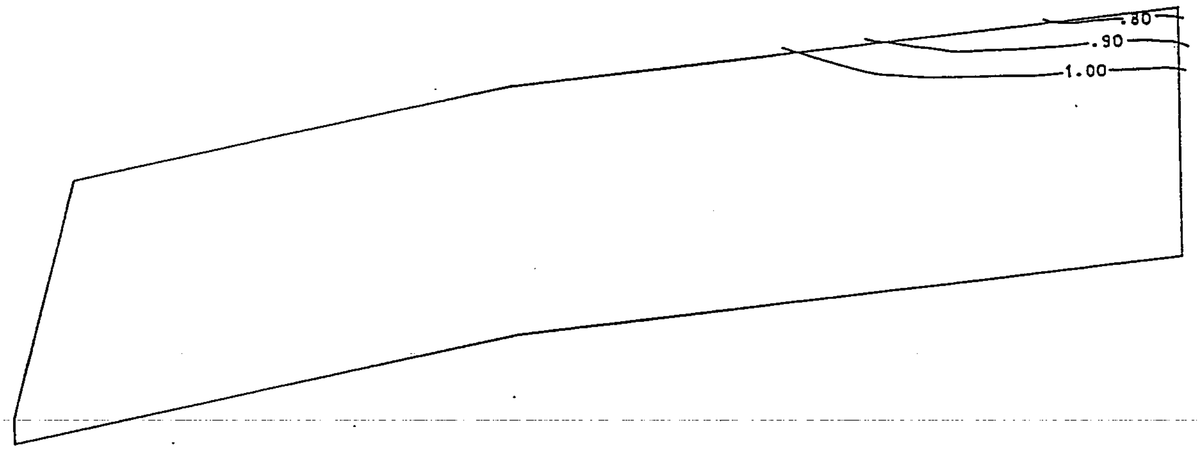


t = 3 hours

FIGURE 5.22 (Continued)

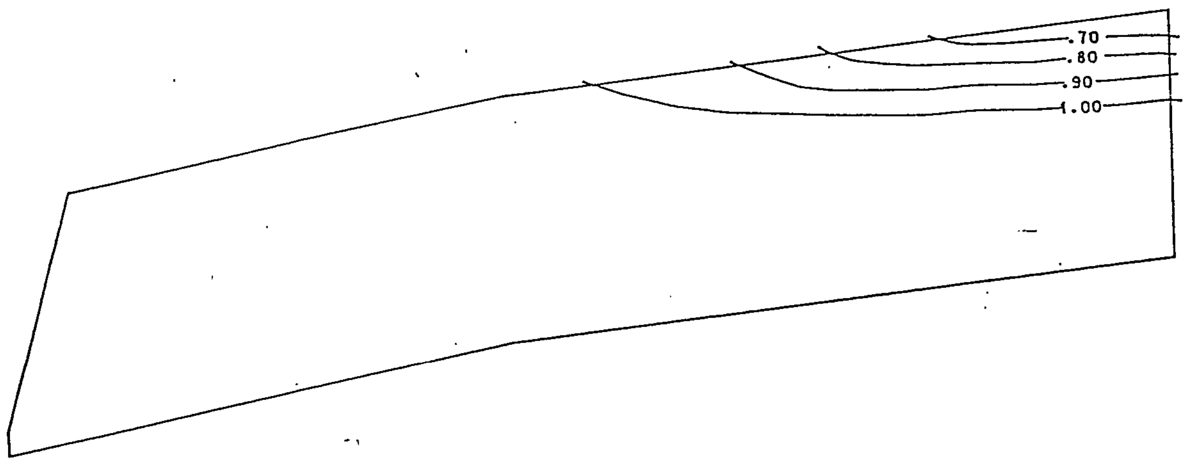


t = 0 hours

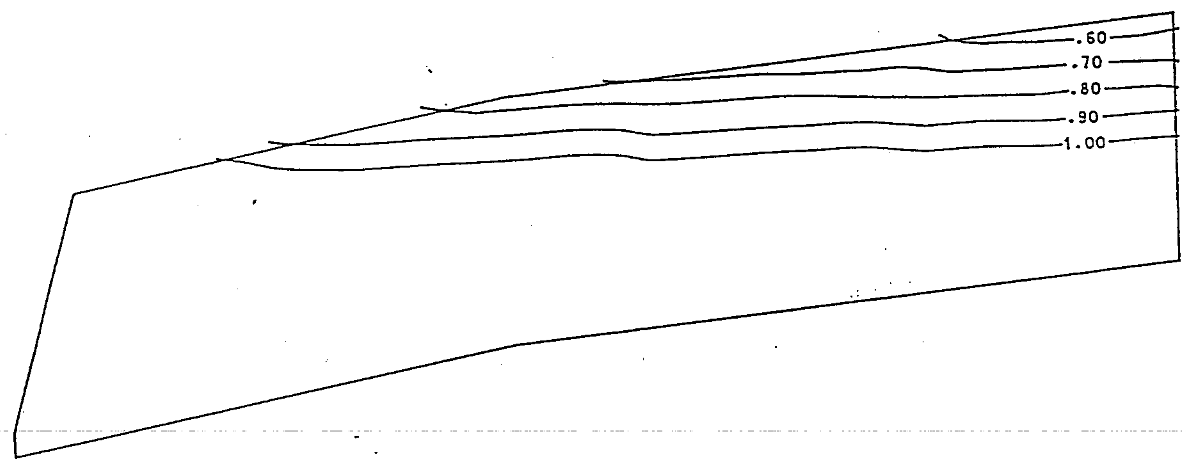


t = 1 hour

FIGURE 5.23
Simulation Results for Base Course Layer Saturation (k_s = 2×10^{-11} m^2)



t = 2 hours



t = 3 hours

FIGURE 5.23 (Continued)

5.6 1-Dimensional, Continuous Simulations

As noted earlier in Section 5.1, the continuous simulations that are described in this section differ from the event-based simulations described in the previous two sections in that time is considered on a continuous basis. Whereas the previous sections considered only the drainage of a base course layer during a rainfall inter-event period from an intially fully saturated state, the present section addresses the more general and difficult problem of modeling the base course saturation behavior during both wetting and drying cycles.

A computer code entitled SUBDRAIN-C, which was developed by McEnroe and Zou (1993) at the University of Kansas as a part of a KDOT/KTRANS pavement drainage project, was used in this project for 1-D continuous modeling purposes. As already noted, this computer modeling code is a generalization of the SUBDRAIN code described in Section 5.4. The 2-D SUTRA code described in Section 5.5 could have been used for continuous simulations as well; however, considering the difficulties that were experienced with it in the much simpler event-based simulations, it was eliminated from consideration. A consequence of this is that spatial variations in saturation level throughout the base course layer have not been able to be determined. The SUBDRAIN-C code, like the SUBDRAIN code on which it is based, simply reports spatially averaged saturation levels.

The SUBDRAIN-C code requires that two input files be prepared to provide data to the modeling code. The first file, whose preparation is facilitated by use of an additional, auxiliary program that is provided with SUBDRAIN-C, contains information on the base course layer geometry and hydraulic properties. This data file is much like the one illustrated in Figure 5.4 for the SUBDRAIN code, except that it contains three additional pieces of information. The additional information required consists of the asphalt layer thickness, the ratio of infiltrated water volume to precipitation volume, and the maximum rate of infiltration into the base course layer. An example data file is illustrated in Figure 5.24. A detailed inspection of the code reveals that the asphalt layer thickness is not actually used by the code (even though it must be provided). We have concluded that the original developers of the code have probably made plans for future enhancements to the code in which this information would be required. The infiltration parameters which must be specified in the data file are used to simulate the amount of precipitation that infiltrates into the pavement. The algorithm used in the code for this is

```
 1. Name of project                              = UAB-UA Project
 2. Hydr. cond. of base material, ft/day         = 60.0
 3. Porosity of base material                    = 0.25
 4. Resid. water cont. of base material          = 0.05
 5. Pore-size distribution index                 = 4.0
 6. Air-entry head, in.                          = 4.0
 7. Thickness of granular base, in.              = 12.0
 8. Thickness of pavement, in.                   = 10.0
 9. Max. drainage dist., trans. dir., ft         = 24.0
10. Slope of pavement, trans. dir., %            = 2.00
11. Depth of depression of edge drain, in.       = 0.00
12. Inflow / rainfall, %                         = 50.0
13. Maximum rate of inflow to base, in./hr       = 0.40
```

FIGURE 5.24
Example Pavement Data File for Program SUBDRAIN-C

quite simple, and not really a very realistic representation of the actual infiltration process, but is likely adequate in view of the other simplifications that have been made in the modeling process. The specified ratio of infiltration volume to rainfall volume is used during rainfall periods to determine how much of the rainfall depth during a period is infiltrated. The maximum rate of infiltration parameter is simply used as a threshold which can not be exceeded. If, for example, 2 inches of precipitation occurred during a 1-hour interval, and if the ratio specified were 0.5, then the model would tentatively use an infiltration depth of 1 inch of rainfall (translating into an infiltration rate of 1 in/hr). If the maximum rate of infiltration were specified as 0.5 in/hr, then this limiting threshold rate would be taken as the actual infiltration rate during the time period.

The second data file which must be provided for the SUBDRAIN-C code consists of precipitation data. A portion of this data file, which contains rainfall amounts recorded in Birmingham during 1976, is illustrated in Figure 5.25. The first three lines of this data file are self-explanatory. The fourth line indicates that the file contains only one year of rainfall data (any number of years can be used, and the number of years actually simulated by the model can be less than the number of years of data provided), and the fifth line is simply a header. The

```
24-hour rainfall data
Birmingham, Alabama
1976
  1 year
Year,Jdate,Period,Depth
1976    02    23    0.01
1976    03    01    0.02
1976    03    02    0.10
1976    03    03    0.03
1976    03    04    0.02
1976    03    05    0.06
1976    03    06    0.21
1976    03    07    0.01
1976    07    08    0.20
```

FIGURE 5.25
Rainfall Data File for Program SUBDRAIN-C

actual rainfall data follows on subsequent lines of the file. Information provided on each of these lines consists of the year, the Julian date as measured with respect to the start of the current year, the hourly period on that date during which precipitation occurred, and the depth of precipitation that fell during that period. To illustrate, the first rainfall data line indicates that 0.01 inch of precipitation fell during the 23rd hour (10:00 - 11:00 P.M.) of the second day (January 2) in 1976. Note that only the rainfall periods must be specified in the data file; any time periods not specified are taken by the code to be those in which no precipitation occurred. It is noted in passing that the code had to be modified and recompiled to handle the hourly data which was available in this project. In Kansas, where the code was originally developed, rainfall data was available at 15-minute intervals. The use of hourly data makes the code run faster because fewer time steps are taken. It is also adequate to use hourly data as the saturation levels in the base course do not fluctuate much over short time intervals. That is to say, the response time of the system is much longer than one hour.

The primary output of the SUBDRAIN-C code, as it was originally written, consists of probability distributions depicting the percentages of time that simulated saturations in the base course layer exceeded various levels. These distributions are given for each month of the year, as well as on an annual basis. Figure 5.26 shows the distributions for an example run of the

Frequency Distribution for Water Content

Tabulated value is percentage of time
that indicated water content is exceeded

	Water content, in percent of saturation									
Month	>20	>40	>50	>60	>70	>75	>80	>85	>90	>95
Jan	100.0	100.0	100.0	15.5	1.9	.7	.0	.0	.0	.0
Feb	100.0	100.0	100.0	7.1	.0	.0	.0	.0	.0	.0
Mar	100.0	100.0	100.0	48.3	9.3	4.8	1.6	.0	.0	.0
Apr	100.0	100.0	100.0	9.6	.0	.0	.0	.0	.0	.0
May	100.0	100.0	100.0	32.0	7.3	1.9	.8	.0	.0	.0
Jun	100.0	100.0	100.0	10.3	.0	.0	.0	.0	.0	.0
Jul	100.0	100.0	100.0	17.6	.7	.0	.0	.0	.0	.0
Aug	100.0	100.0	100.0	12.6	.0	.0	.0	.0	.0	.0
Sep	100.0	100.0	100.0	13.8	1.1	.0	.0	.0	.0	.0
Oct	100.0	100.0	100.0	6.9	.0	.0	.0	.0	.0	.0
Nov	100.0	100.0	100.0	11.5	.0	.0	.0	.0	.0	.0
Dec	100.0	100.0	100.0	23.3	.7	.0	.0	.0	.0	.0
Ann	100.0	100.0	100.0	17.7	1.8	.6	.2	.0	.0	.0

FIGURE 5.26
Example Output File Generated by Program SUBDRAIN-C

program, and is self-explanatory. For the purposes of this project, additional modifications were made to SUBDRAIN-C so that it would also create a second output file. This second file is comprised of rows, one for each hour of the total simulation period, containing the rainfall amount during that time period, and the spatially averaged saturation level in the base course during that time period. A plot of the information contained in that additional output file is shown in Figure 5.27. For purposes of plotting, the data values have been aggregated over 6 hour periods rather than presenting all of the 1-hour data. The simulation results for only the first 3 months of 1976 are shown in the figure.

It is clear from an examination of Figure 5.27 that spatially averaged base course saturation levels, as simulated by the modeling code, respond fairly quickly to precipitation excitations. Whenever there is a precipitation event shown in the lower part of the figure, there is a corresponding and quick rise in the saturation level. A return of the saturation level to that

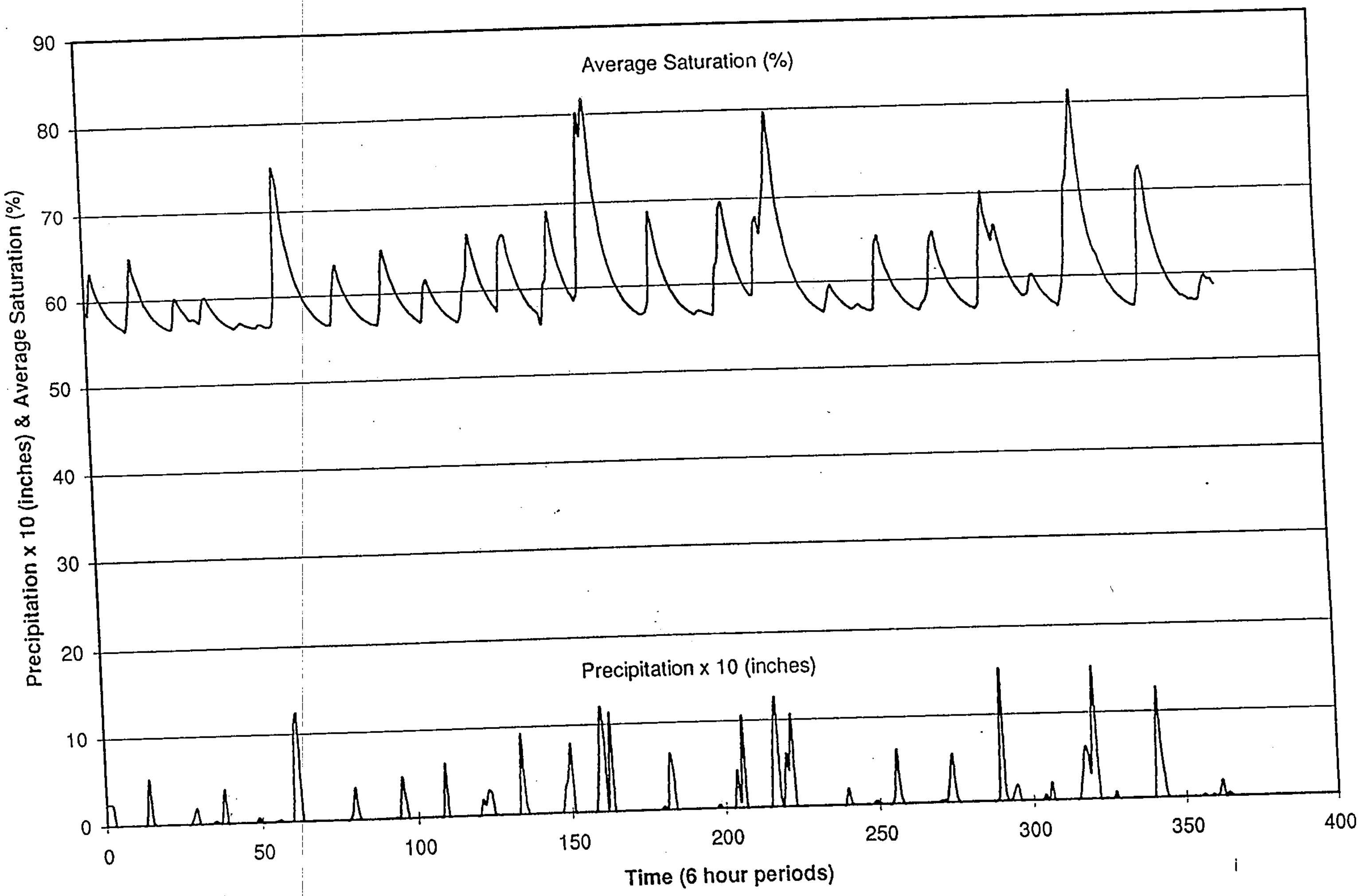


FIGURE 5.27
Birmingham Precipitation and Simulated Base Course Saturation Levels for First Three Months of 1976

existing in the base course prior to the precipitation event occurs on average about 100 hours (4 days) after the rainfall event. It is also evident that the spatially averaged saturation level never drops below a lower threshold, which is governed by the base course geometry and hydraulic properties. Discussions of how the various base course geometric and hydraulic parameters are related to this threshold are provided in Section 5.4.3.

The resemblance of Figure 5.27 to precipitation hyetographs and streamflow hydrographs as they are commonly presented for rainfall-runoff modeling studies is unmistakable. It suggests that if one is interested only in spatially averaged saturation levels, and does not care about how moisture is distributed in the pavement section, that ideas similar to the unit hydrograph can be employed to predict the time variation of saturation level for a given sequence of rainfall pulses. It is noted, however, that this would be a black-box type of modeling approach, and would have little, if any, physical basis. Parameters in such a model would also be difficult, if not impossible, to relate to physically measurable quantities in any meaningful way. A consequence of this is that such a model would likely yield very poor results if it were to be used to make predictions for cases differing from those for which it was developed and calibrated. Of course, the benefit of such a simplified model would lie in its ease of use.

The mathematical form of a black-box model such as that referred to in the previous paragraph would be identical to the convolution procedure used in unit hydrograph-based rainfall-runoff modeling. In the present context, the spatially averaged saturation level S could be expressed as a function of time as

$$S(t) = S_{\min} + \int_{-\infty}^{t} i_e(\tau)\, h(t-\tau)\, d\tau \qquad (5\text{-}15)$$

where S_{min} is the minimum degree of saturation attainable, $i_e(\tau)$ is the "effective" rainfall intensity as a function of time, and $h(t-\tau)$ is an impulse response function. S_{min} is analogous to base flow in a stream, and $h(t-\tau)$ is analogous to an instantaneous unit hydrograph. A discrete analogue of Eqn. (5-15), requiring a summation instead of an integration, could also be written. Procedures for accomplishing this are detailed by Chow et al. (1988).

5.7 Field Moisture Measurements

The drainage of water from pavement structures is an important factor in the design of freeways. Knowledge of the factors that affect the infiltration of water into pavements and the drainage of this water from pavements is needed to provide adequate long-term performance of roadways. This proposal describes a method to determine the saturation of a variety of pavements for Alabama rain and soil conditions. These saturation conditions can then be used to directly predict pavement drainage quality needed for pavement structural design.

5.7.1 Laboratory Measurements of Highway Base (Drainage Layer) Material

UAB's soil testing laboratory was used to analyze permeability of some typical construction materials. However, the in-situ determinations will be the most accurate. Cedergren (1974) has found that permeabilities of typical pavement structures vary over a broader range than most any other engineering parameter, and are usually over-estimated. This supports the reliance on actual field measurements as much as possible in this proposal. Various soil tests (sieve analyses, porosity, residual moisture content, and permeability) were conducted on the limestone aggregate drainage layer materials to determine values of these parameters which affect subsurface drainage.

Sieve analyses (Figure 5-28) were performed to determine the particle size distribution of material in the drainage layer. A sample of material was measured using an Ohaus 700 Series Triple Beam Balance. The sample was then placed in a stack of pre-weighed Soiltest sieves with openings of 25, 12.5, 9.5, 3.35, 2.36, 1.18, 0.30, 0.18, 0.10, and 0.075 mm. The sieve stack was then shaken for ten minutes in a Soiltest Model CL - 305A Sieveshaker. Upon completion of the sieve shaking, the sieves, still holding soil, were disassembled and weighed. The amount of soil in each particle range was then calculated as the difference between the weights of the sieve and the sieve plus soil. From the amounts caught on the sieves, the percent finer was calculated as the percentage of material passing each sieve. The particle size distribution curve was then plotted on semi-log paper as percent finer vs. sieve opening. The bulk density of the crushed limestone base material was found to be about 10,100 kg/m^3 (130 lb/ft^3).

Porosity tests were conducted on the drainage layer material for use in the modeling tasks. The porosity (25 percent) was determined by measuring the volume of water necessary to saturate a specific volume of soil. This was done by placing a sample of soil in a container of constant volume, and slowly adding water while vibrating air bubbles from the container. Upon saturation, no more air bubbles exist in the container and the sample will absorb no more water. Once saturation is achieved, the porosity may

Figure 5-28. Crushed Limestone Base Material Particle Size Distribution

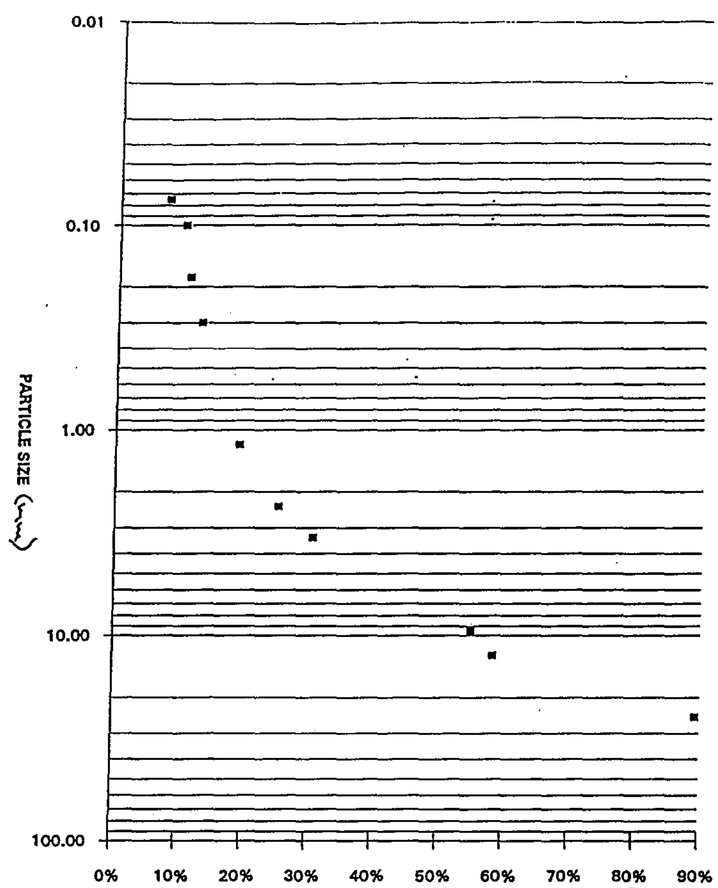
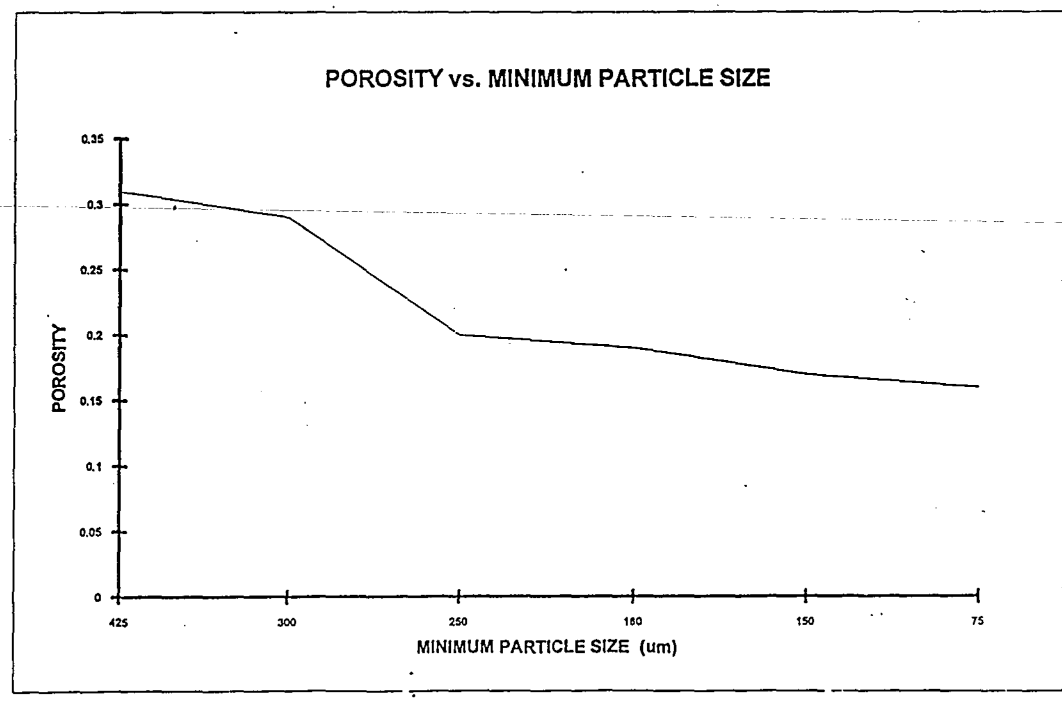


Figure 5-29. Porosity Changes when Removing Fine Particles



be calculated by dividing the volume of water added by the total volume of the saturated sample. In order to examine the relative effect of removing fines from the base material in order to improve pavement drainage, special tests were conducted on the sieve fractions after specific small size fractions were removed. As an example, Figure 5-29 shows the porosity of the crushed limestone base material after different small particle sizes are removed. This figure shows that the porosity doubled if all material smaller than 300 μm was removed, compared to removing all material smaller than 75 μm.

The residual moisture content of the crushed limestone drainage layer material was also evaluated for use in the modeling procedure. It was determined to be 0.05 for the crushed limestone base material. The residual moisture content was determined by saturating a specific volume of dry soil, and then measuring the amount of water that will not gravity-drain from the sample. This was done by placing a sample of soil in a container with a porous bottom and then restricting drainage while the sample is saturated. Upon saturation, the container was allowed to gravity-drain. After gravity-draining, the sample was weighed and the volume of water residing in the sample was calculated. The residual moisture content was calculated by dividing the volume of residual water by the total volume of voids. Figure 5-30 shows the effects of removing fines from the base material. As expected, the material having more fines had a greater moisture retaining capacity. Removing the fines reduced the capillary forces that could hold the water, with about a 25 percent improvement in gravity drainage.

Constant head permeability tests were conducted to determine the permeability constant of the drainage layer material for use in the modeling procedure. The permeability was found to be 18.2 m/day (60.1 ft./day) at 20°C. These permeability tests were conducted many times using a variety of heads. After these tests were completed, fine particles were systematically removed from the samples, and permeability tests were again conducted, as shown on Figures 5-31 and 5-32. The permeability testing procedure began by placing a compacted sample of drainage layer material in a Soiltest CN - 405 Permeameter and attaching it to a large Soiltest CL - 278E Reservoir. The valve between the reservoir and the permeometer was then opened, allowing water to flow into the permeometer. A bleed valve was opened to allow saturation and escape of air from the sample. Upon saturation, the bleed valve was closed, and one minute was allowed for steady state flow to be established through the sample. Once steady state was achieved, several measurements of water exiting the permeometer were taken and the average flow rate was calculated. Once the average flow rate was known, the permeability constant for the material was calculated by dividing the average flow rate by the cross-sectional area of the soil sample.

Figure 5-30. Residual Moisture Changes when Removing Fine Particles

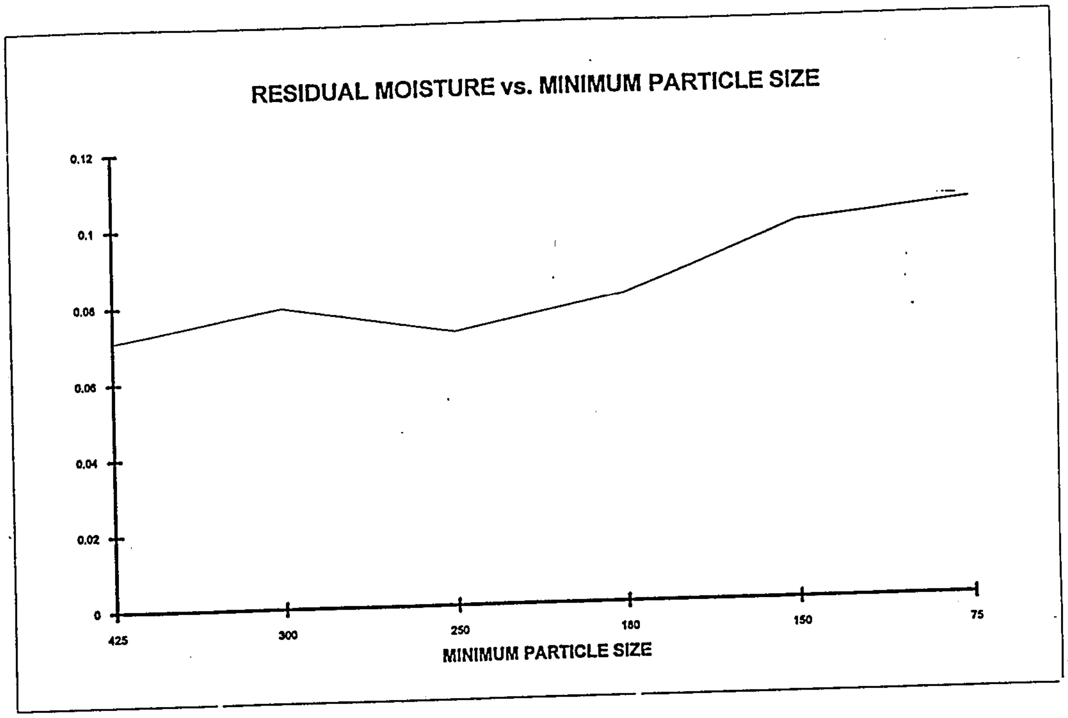


Figure 5-31. Permeability Changes when Removing Fine Particles

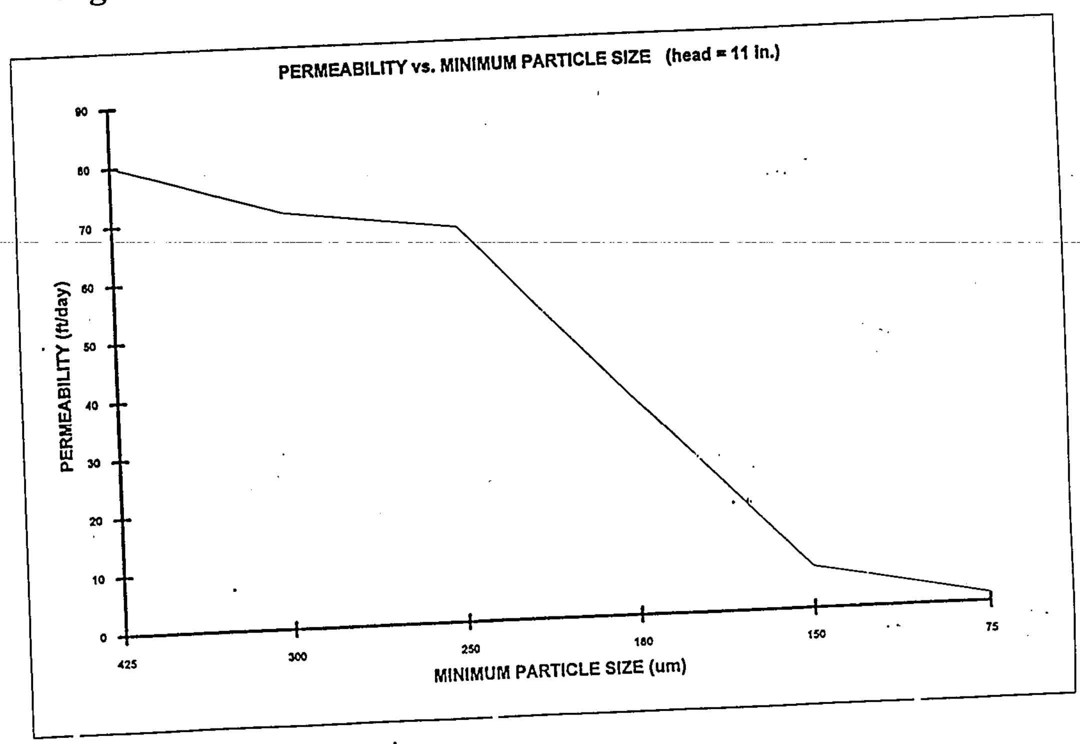


Figure 5-33. Moisture Calibration Curve for Base Material

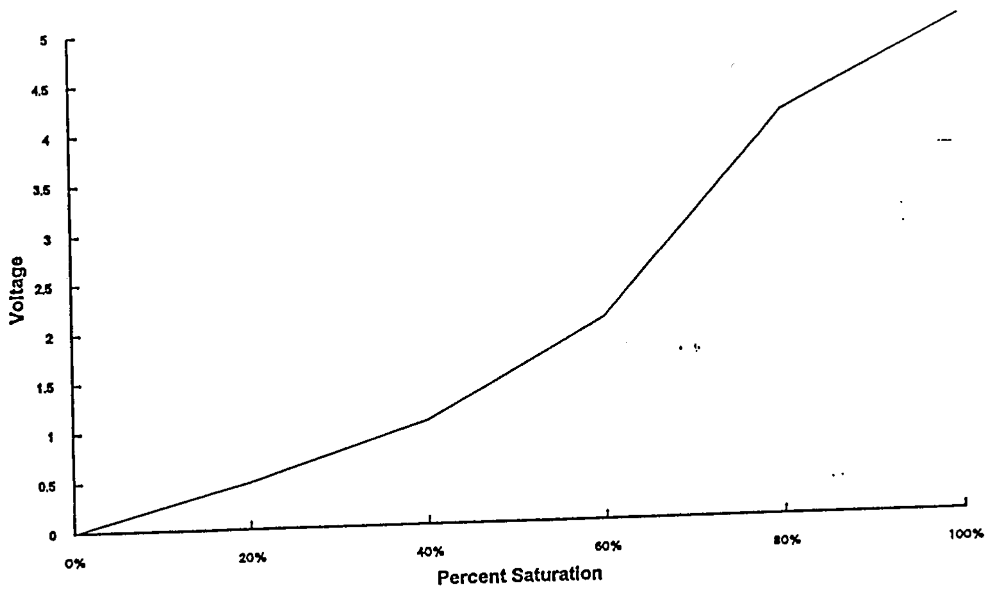
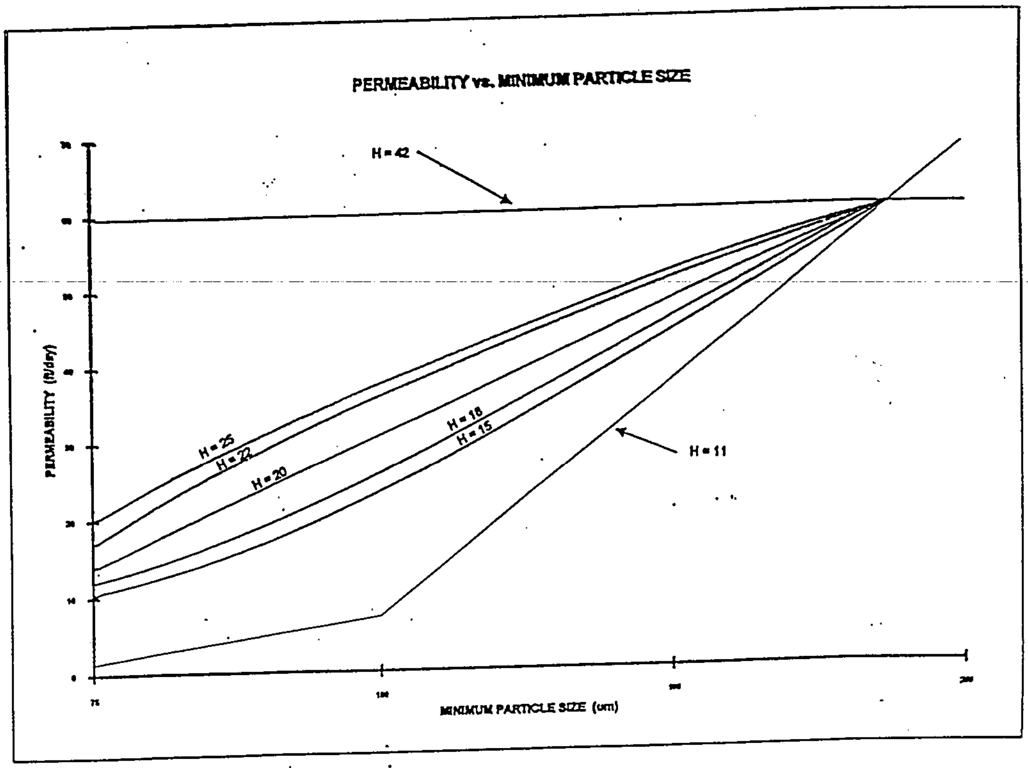


Figure 5-32. Permeability Changes by Pressure Head



After conducting constant head permeability tests on the unaltered samples, additional tests were conducted on four modified samples that had more and more fines removed. These samples were generated by sieving the original sample, and removing fine particles beginning with particles less than 0.075 mm. for the first sample, 0.15 mm. for the second sample, 0.18 mm. for the third sample, and 0.25 mm. for the fourth sample. Each sample was again tested to establish whether the permeability significantly increases with removal of a certain particle size under both 0.28 and 1.1 m (11 and 42 inches) of head, as shown in Figure 5-32. The different heads had very little effect on the permeabilities of the unaltered sample, or the sample with all of the fines greater than 425 μm were removed. The different heads greatly affected the permeabilities of the unaltered sample and with few of the fines removed. The greatest head test had little effect on the permeabilities of the altered samples. Typical roadways would experience relatively low heads on the base material during rains. Removing fines from the base material is therefore expected to have significant beneficial effects on their permeabilities, and therefore drainage characteristics.

Laboratory calibrations of the moisture sensors were conducted before the field in-situ moisture tests were conducted. Figure 5-33 indicates the moisture response (in volts) compared to percent saturation. Special lined troughs were constructed to hold the moisture sensors and crushed limestone base material. A series of different moisture levels (that were conventionally measured) were tested. The electronic moisture sensors exhibited almost a constant relationship between voltage (maximum of 5 volts) and moisture (maximum of 100 percent).

5.7.2 Continuous Moisture Measurements at Test Sites

This work element directly examined pavement drainage times and percentage moisture levels for typical types of Alabama highway pavement conditions. The continuous moisture measurements were made at four locations representing areas having edge drains and areas not having edge drains, and having and not having known drainage problems. All of these tests were conducted on flexible pavement sites. These tests represented a 2^3 complete factorial experimental design (Box, et al. 1978) and were analyzed using Design-Ease (Stat-Ease, Minneapolis, Mn):

Site	edge drains	drainage condition
I-459 at Horse Track Interchange	yes	good
H-79 at Center Point Rd.	no	good
I-459 at Grants Mill Rd.	yes	poor
H-79 at Thompson Tractor	no	poor

These four sites cover most of all possible conditions expected in Alabama. They were located in the Birmingham area which has rain conditions similar to much of the state (with the exception of the greater rains found along the Gulf coast), the drainage layer (pavement base) material was crushed limestone that is used in most of the state, two sites had edge drains and two did not, and the "good" and "poor" drainage conditions represented deep and shallow groundwater conditions, respectively. These were all of the significant factors reported in the literature and expected to affect drainage conditions and pavement design based on moisture.

The drainage condition factor used n selecting these sites was based on areas known to have drainage problems and those that do not. The known presence of standing water near the roadway (typically within 0.5 m of the road surface) indicated poor drainage conditions, while generally dry shoulder and roadside ditch areas indicated good drainage conditions. The "poor" drainage sites were also selected based on known pavement deterioration problems.

The goal of this proposal task was to develop a simple procedure to determine reasonable drainage times of Alabama highway pavement structures, using readily available information. Factors affecting pavement structure drainage times were expected to include pavement and base material permeability, porosity, slope, thickness, and drainage distance. Obviously, construction procedures and special waterproofing will also affect drainage times. These factors were all investigated as part of this task and the earlier reported modeling efforts.

Drainage time of the pavement structure and percentage of time that pavement is saturated is the currently recommended method for selecting the m_i drainage coefficient for flexible pavements (AASHTO 1993). This coefficient is selected based on the time required to drain the base layer to 50% saturation, and the percentage of time that the pavement structure is exposed to moisture levels approaching saturation.

This task has several elements to better examine the wide variety of conditions that affect pavement drainage. The best option available was to install soil moisture probes at each test location. Soil moisture probes were therefore permanently installed at two roadway and one shoulder location for long-term drainage measurements. The probes selected for use were of a new design that overcame many limitations of older styles of gypsum block moisture sensors. The gypsum sensors slowly dissolve and loose contact with surrounding soil. Because these locations were permanently sealed under roadway asphalt patching material, probe maintenance was not possible. The electronic sensors used also enabled us to continuously record pavement structure moisture levels on a data logger and not rely on infrequent manual readings after rains. This enabled us to observe and measure many unique moisture patterns at the test sites.

Each site was monitored using a tipping bucket rain gauge (WeatherMeasure model 6011), three electronic moisture sensors (AquaTel 29 from Global Water) and a continuously recording data logger (UL 16 from Global Water). Base material moisture, subgrade moisture, and rain were recorded every five minutes. Two of the moisture sensors were located in the outside lanes of the test areas and were placed beneath the pavement and about 50 mm above the bottom of the crushed limestone base layer, or about 0.5 m (18 inches) below the pavement surface. Two sensors were used to obtain redundant data in order to measure the consistency of the moisture levels and to have a back-up in case one of the sensors was damaged (as happened at the H-79 good site). After the moisture sensors were placed, new base material was carefully placed to the bottom of the pavement layer and pavement patching material was packed to the pavement surface. Three different surface patching methods were used because of problems in permanent sealing of the disturbed pavement surfaces. Loop sealant was initially used to seal any cut seams and the surface of the asphalt patch. However, this did not hold up for more than a few weeks. An epoxy sealant was then used for the top 10 mm of patch which worked very well, especially after being overcoated with a pavement adhesive tape along the long cracks.

The third moisture sensor was placed on the roadway shoulder at a depth equal to the bottom of the base material. This sensor was placed in subgrade material and was used to indicate the presence of shallow local groundwater that may infiltrate upwards into the base material.

The data loggers periodically shorted out due to high atmospheric moisture levels. Therefore, all sites were visited at least twice a week during the later portions of the field study to reset the loggers in order to minimize missing data. Even though some observation periods were missed, the use of the continuous moisture sensors and the tipping bucket rain gage enabled us to acquire a great deal of data. Appendix C contains the complete logs for the rain gauges and moisture sensors. The following table shows the start and end periods (all in 1994) for the field monitoring activities:

Site	Start	End
I-459 at Horse Track Interchange (good with drains)	January 4	June 13
H-79 at Center Point Rd. (good without drains)	February 6	May 30
I-459 at Grants Mill Rd. (poor with drains)	March 29	June 9
H-79 at Thompson Tractor (poor without drains)	March 21	May 29

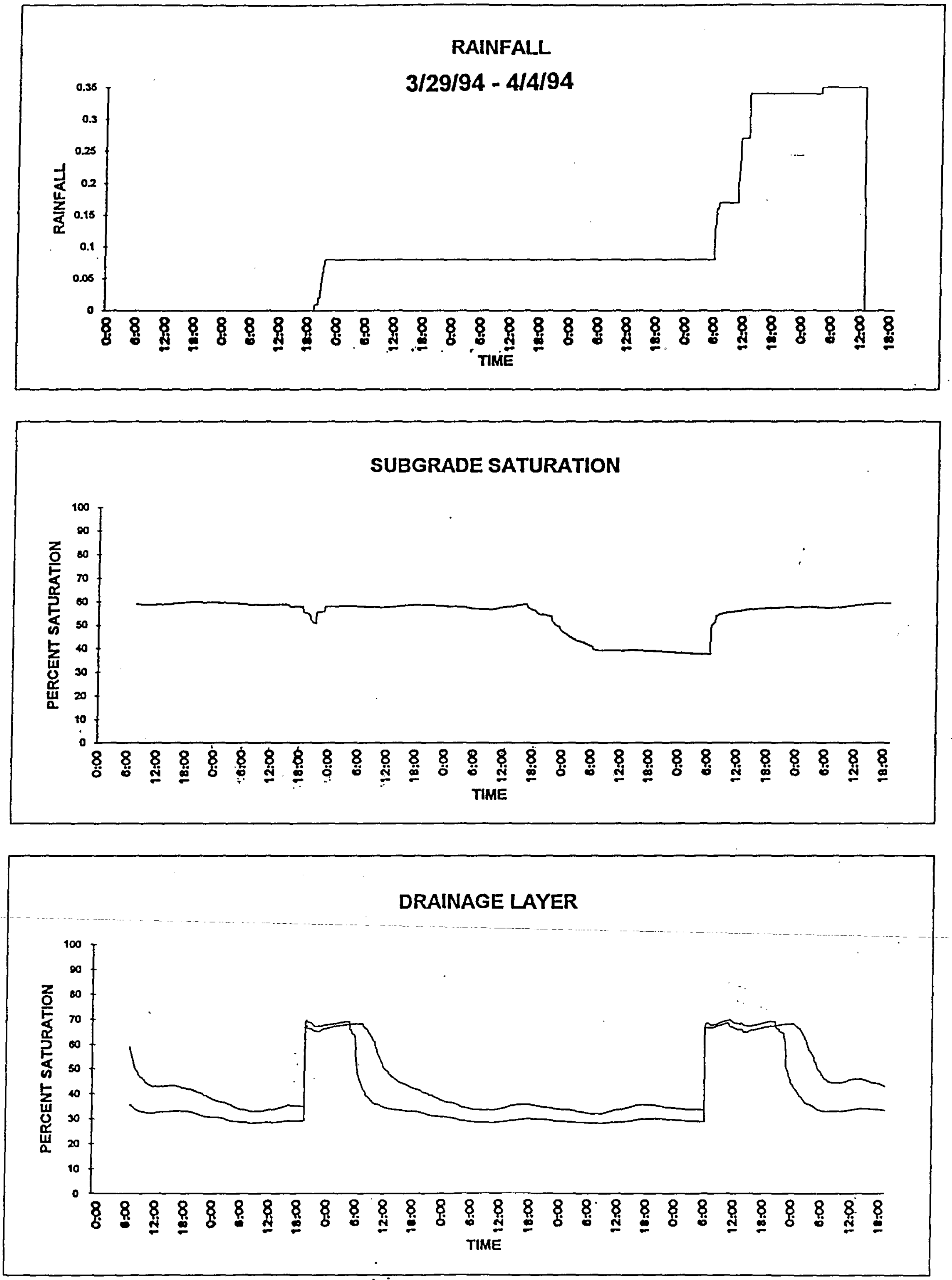
The initial I-459 site was installed first because of the ease of access (the road temporarily had no traffic because of bridge construction). The other sites were installed later after various problems in sealing the sites were solved, as noted above.

Several very interesting moisture patterns were observed at these sites. Figure 5-34 shows how quickly the poor drainage (with edge drains) site responded to rains. The rapid moisture rise (starting at the very beginning of the rain) was followed by a similarly rapid drop in moisture to levels approaching pre-storm conditions within about 6 hours after the rain stopped. In contrast, Figure 5-35 shows the good drainage location (with edge drains) located several miles north of the site shown previously. Even though this site was indicated to be well drained, it experiences very little moisture fluctuations during or after rains. During installation, it was obvious that the I-459 good site had 0.3 m of well compacted asphalt in good condition over the base material, while the I-459 poor site had much thinner pavement that was much more fragile. Even though the good drainage site was less affected by high local groundwater and rain water infiltration was very small, it maintained a relatively high moisture level throughout the study period.

Another interesting phenomenon noted was the effect that air temperature had on base material moisture levels. Figure 5-36 is an example of the H-79 poor drainage site that experienced dramatic diurnal moisture fluctuations. The lowest moisture levels occurred during midday hours and the highest during nighttime. This fluctuation started in early April and continued until the moisture levels were quite low, and apparently more stable. An extreme example of the effects of temperature on pavement moisture levels is shown on Figure 5-37. The H-79 good drainage site was repaved during the week of April 25. The dramatic and rapid decline in moisture occurred when the hot asphalt overlay was placed. The very low moisture levels remained low until a major rain about a week later.

5-40

Figure ~~5-34~~. Moisture Changes at I-459 (Poor Drainage Conditions)



5-41

Figure ~~5-35~~. Moisture Changes at I-459 (Good Drainage Conditions)

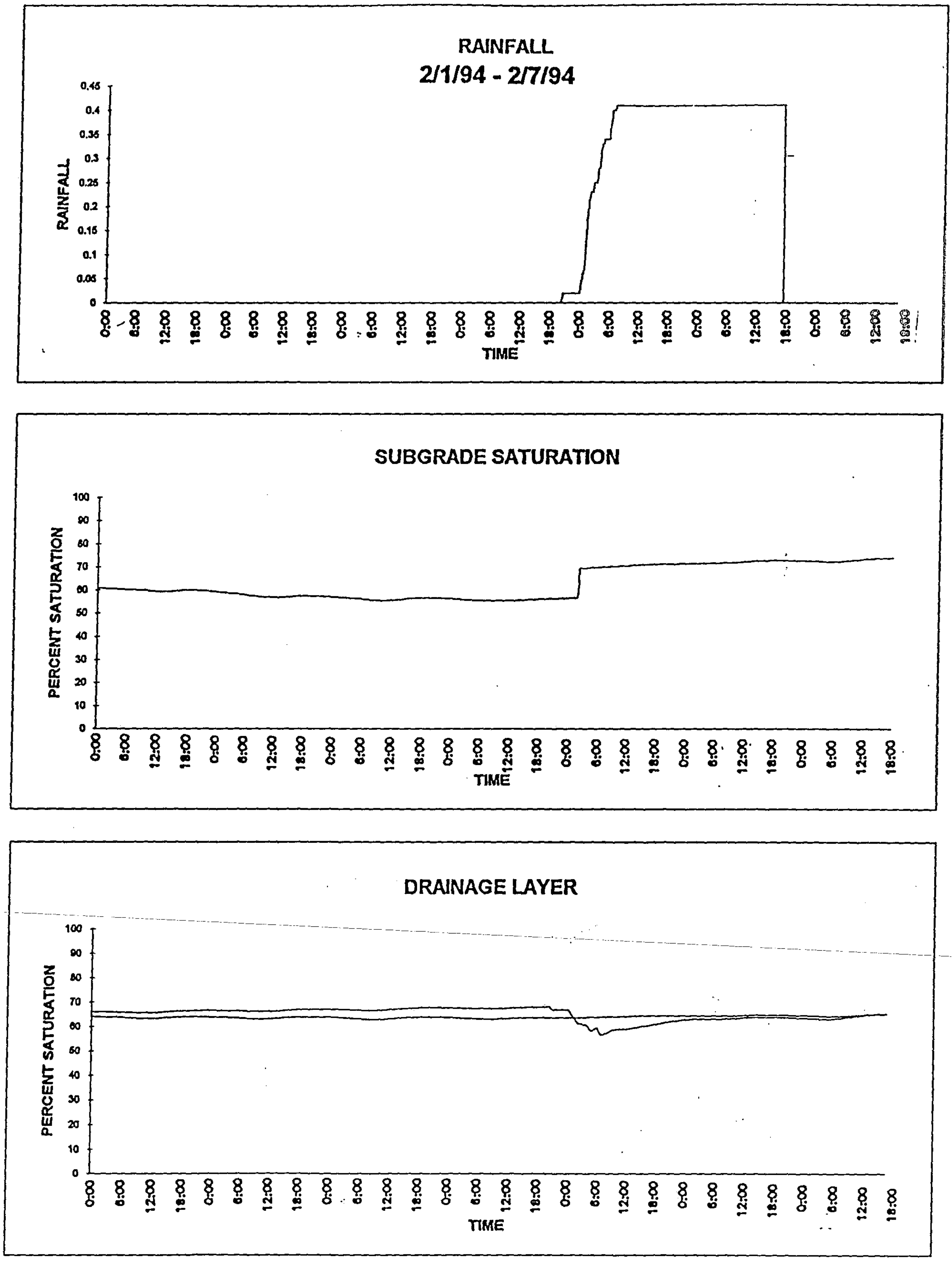
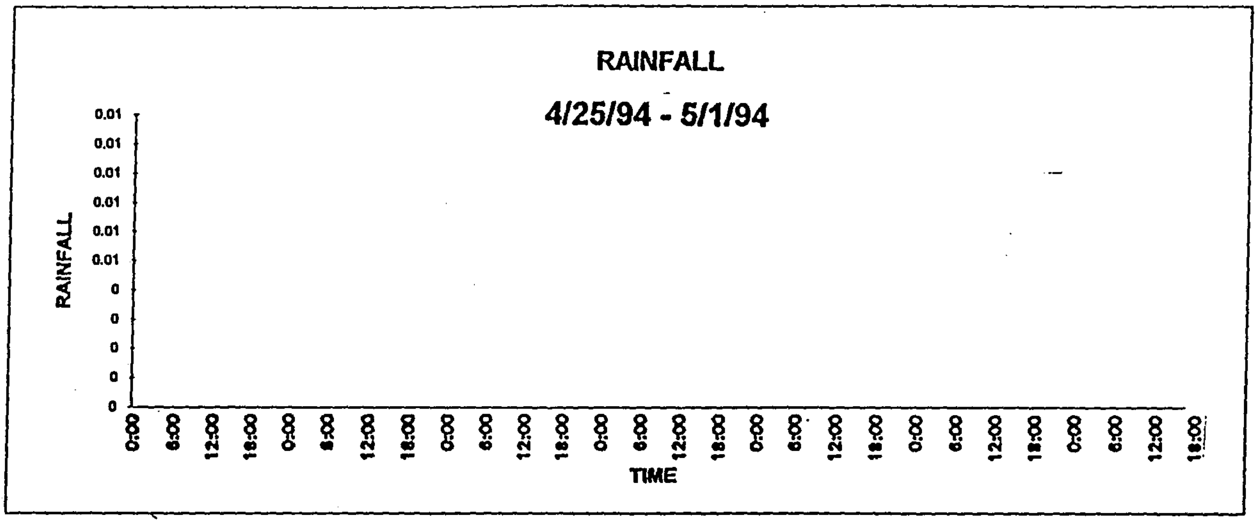
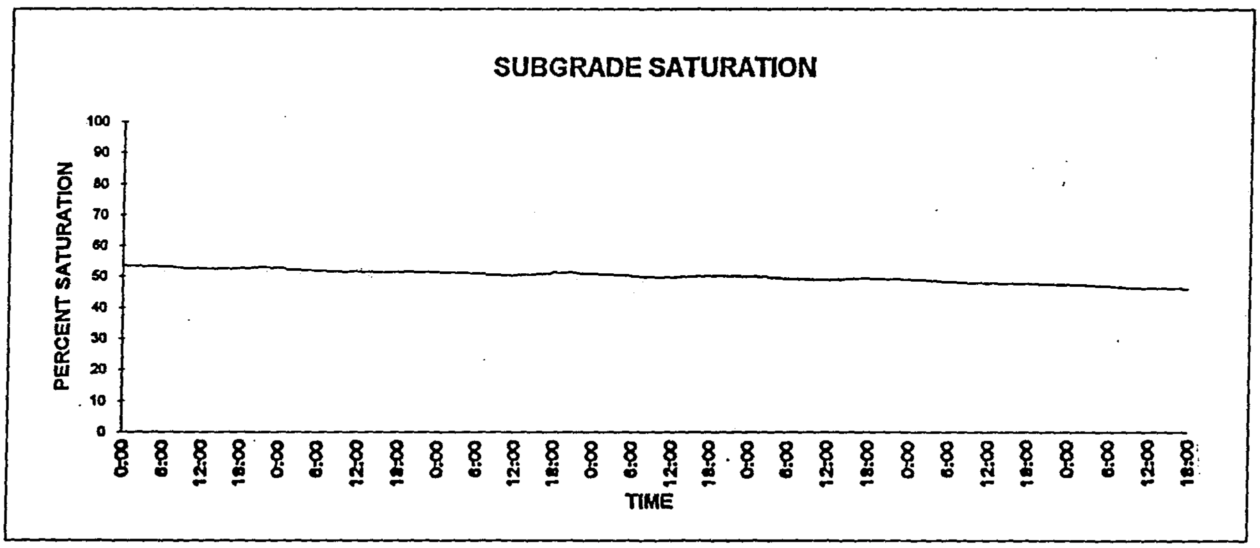


Figure ~~5-36~~ 5-92. Moisture Changes at H-79 (Poor Drainage Conditions)





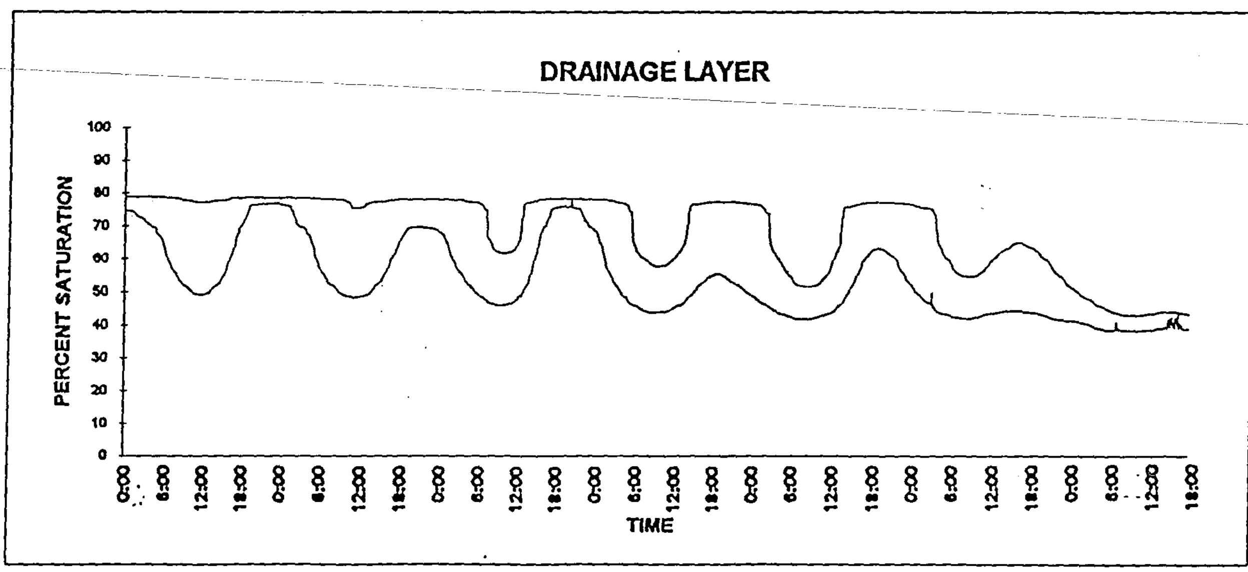
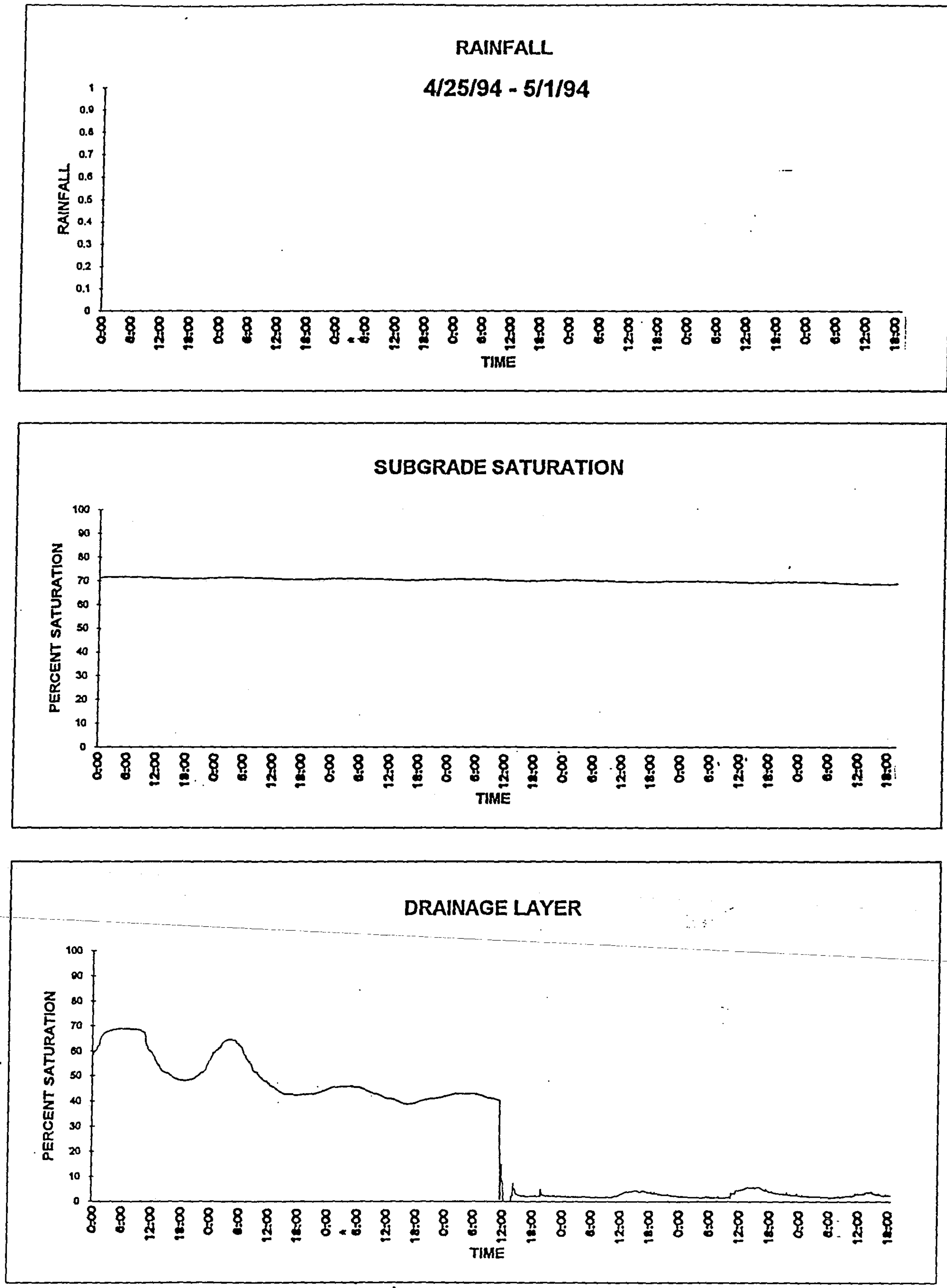


Figure ~~5-37~~ 5-43. Moisture Changes at H-79 (Good Drainage Conditions)



owever, the diurnal fluctuations again became apparent, generally holding the moisture levels down.

Figures 5-38 through 5-41 are probability plots of monthly observed moisture levels at the four sites. As noted previously, the I-459 good site had moderate moisture levels throughout the study, while the H-79 good site experienced extreme moisture levels due to the effects of the paving.

5.7.3 Significant Results and Implementation of Findings

Interstate 459 at the Birmingham Turf Club interchange is the "good drainage conditions, with edge drains" location. Continuous monitoring began on January 5. About 20 weeks of continuous data have been collected at this site, representing 26 major storms. This site is characterized with very little moisture changes. Equilibrium moisture levels have remained high, at about 60 to 80%. Infiltration tests were conducted at locations that had small cracks and at locations that had no cracks. The rates for the sites with cracks were much greater than the sites without cracks, as noted in Section 4. It is not likely that the high rates found at the sites with cracks would be sustained for long periods during actual rains. Rain water flowing through the cracks also did not contribute to base material moisture increases.

Interstate 459 at Grants Mill Rd. is the "poor drainage conditions, with edge drains" location. Continuous monitoring of quality data began on March 30. About 8 weeks of continuous data have been collected at this site, representing 13 storms. This site is characterized with relatively rapid moisture increases near the beginning of rains. It also drains relatively rapidly and has shown two sets of equilibrium moisture levels: about 20 to 30% or 50 to 60%. These relative rapid moisture changes may be due to the pavement structure at this location. When the pavement was cut to install the moisture sensors, it was noted that the pavement was only about 6 inches thick at the test location. The infiltration test results found high initial rates that only slightly decreased. However, two of the six infiltration test sites at this location had much lower rates. As noted above, the thin and poor condition pavement allowed water flowing in the cracks to dramatically affect base material moisture levels.

Highway 79 at Grants Mill Rd. is the "good drainage condition, without edge drain" location. Continuous monitoring began on February 7 and about 14 weeks of continuous data have been collected at this site. About 9 storms have been recorded at this site during this time. This site is characterized with relatively rapid moisture increases near the beginning of rains, but has slow moisture decreases after the rains end.

5-44

Figure ~~5-38~~. Probability of Moisture Levels at I-459 (Good Drainage Conditions)

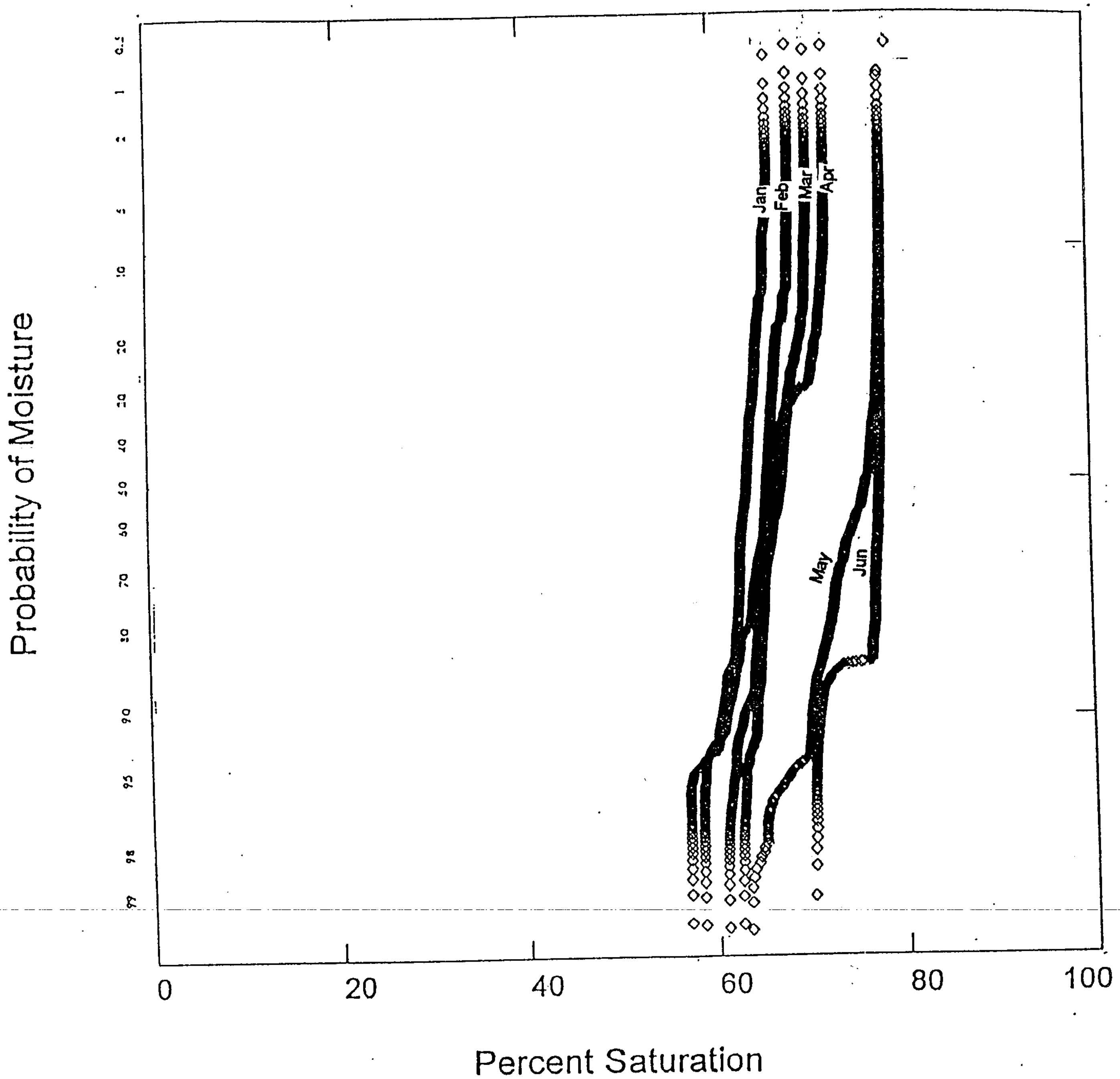


Figure 5-45 ~~5-39~~. Probability of Moisture Levels at I-459 (Poor Drainage Conditions)

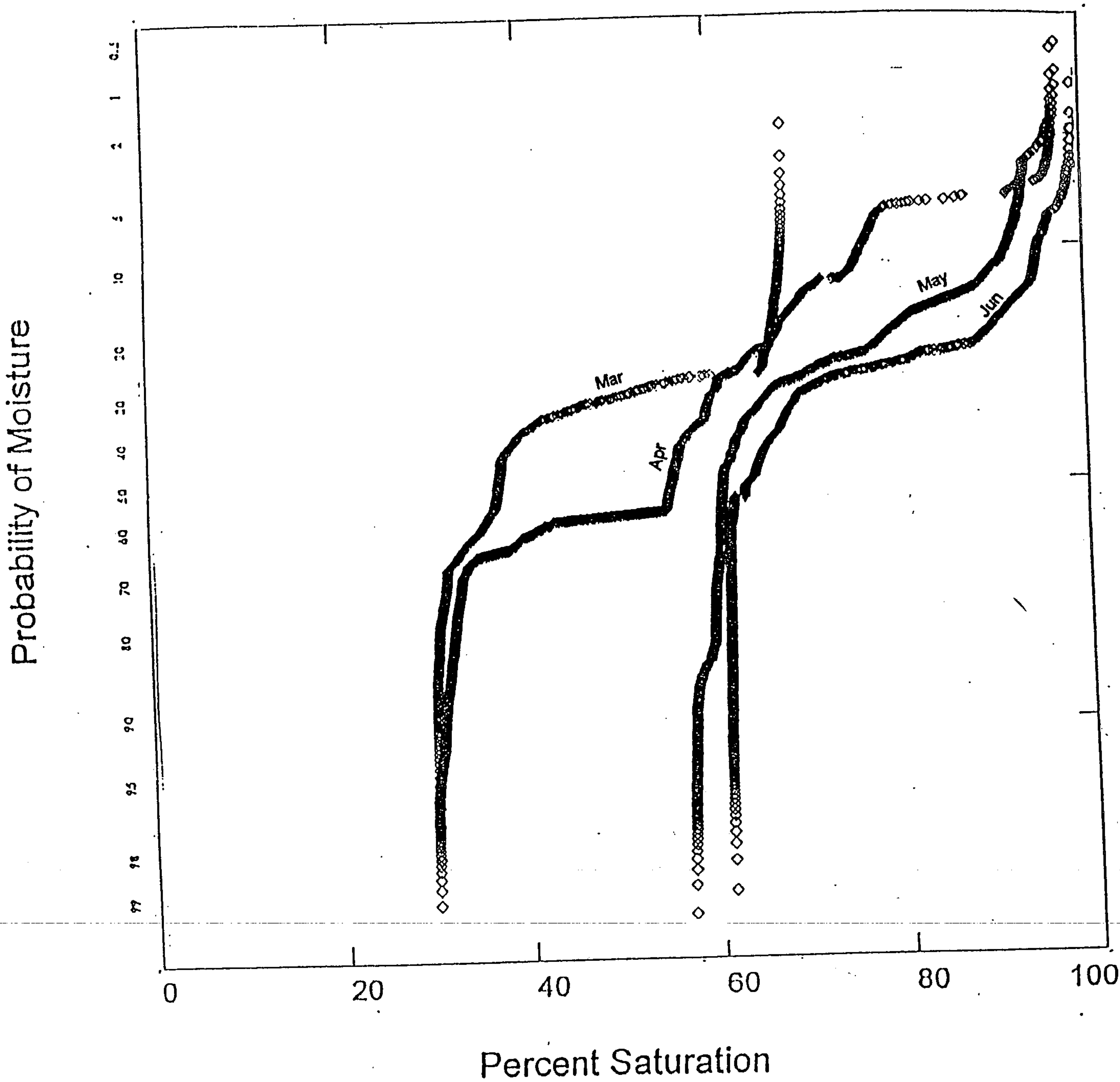


Figure ~~5-40~~ 5-96. Probability of Moisture Levels at H-79 (Good Drainage Conditions)

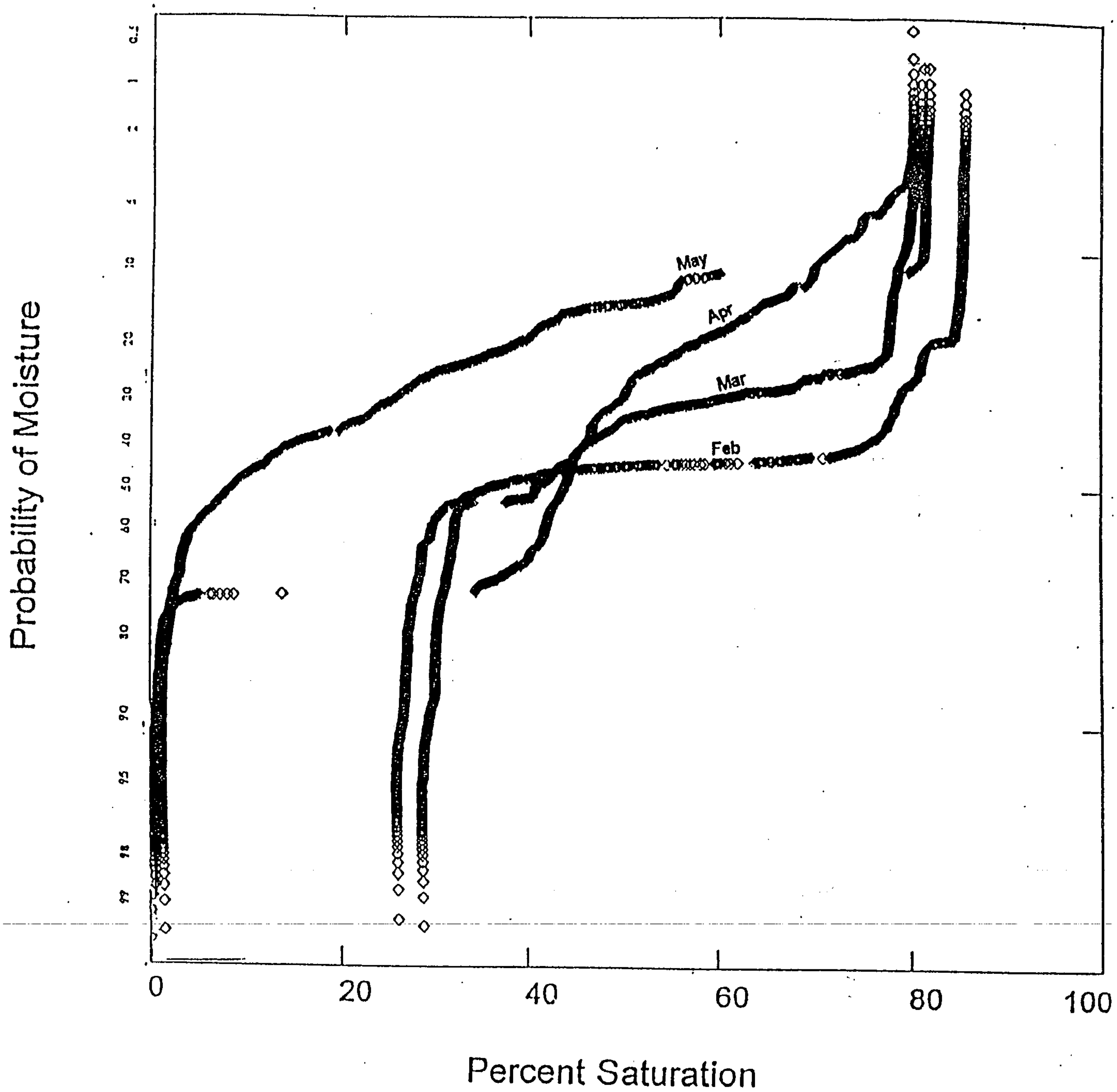
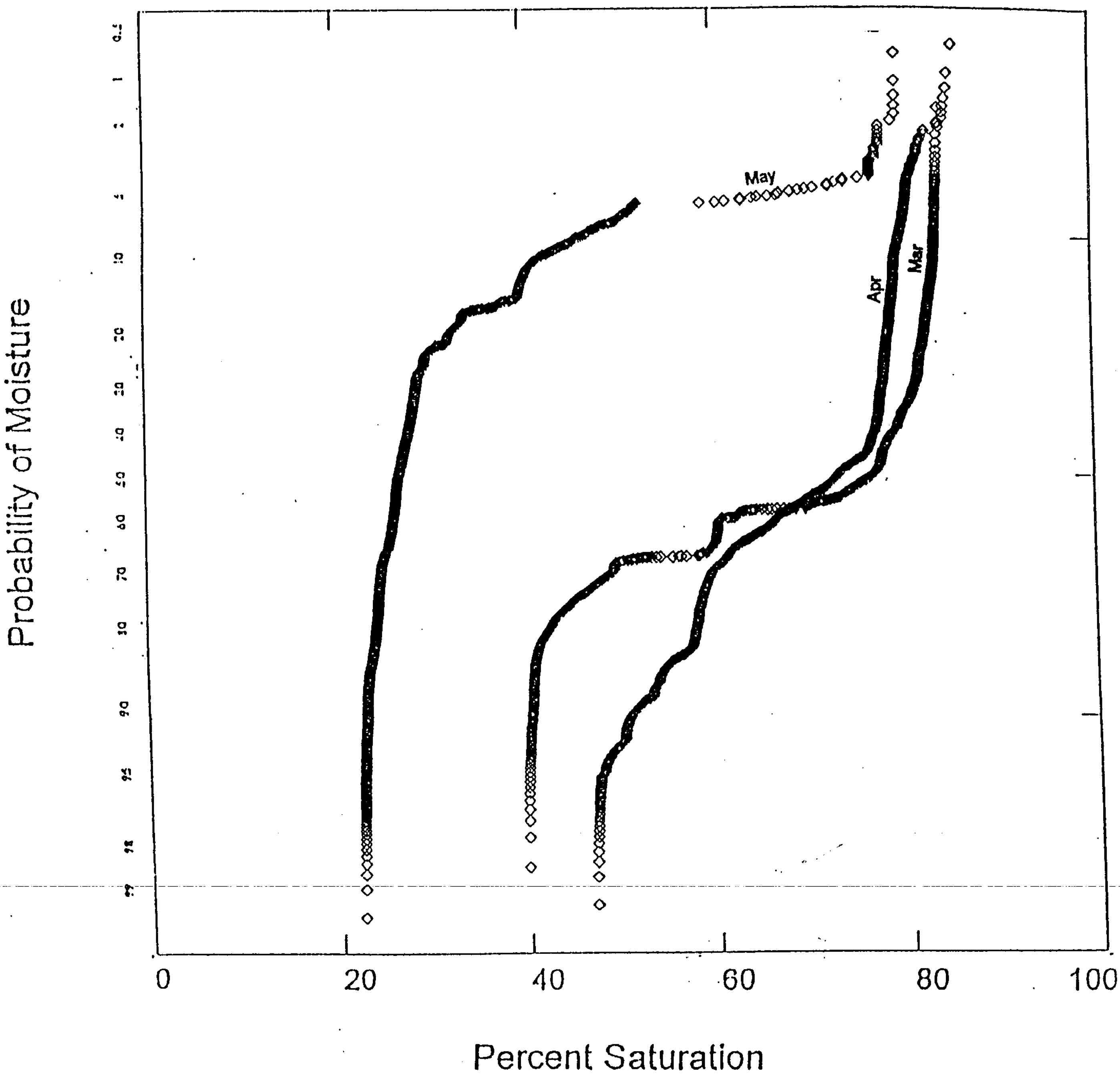


Figure 5-41. Probability of Moisture Levels at H-79 (Poor Drainage Conditions)

5-47



Equilibrium moisture levels remained at about 50 to 80% until the beginning of April, when significant diurnal moisture variations were noted. The moisture levels were lowest at midday (as low as 30%), but increased at night (to as high as 70%). Typical diurnal moisture changes of about 10% were noted in early April and increased to about 25% by the end of April. As noted above, this site was repaved on the morning of April 25 which resulted in almost complete removal of pavement moisture. A rainfall of about 25 mm (1 in) occurred during the morning hours of May 3 which increased the moisture level back up to approximately pre-repaving conditions. Obviously, temperature is playing a major role in determining pavement moisture levels for some local roadways. The new pavement surface placed at this site had a very high rainfall infiltration rate. This very high rate is not likely to occur during an actual rain because of lateral flow constrictions, as found during the modeling analyses.

Highway 79 at Thompson Tractor is the "poor drainage conditions, without edge drains" location. Continuous monitoring of quality data began on March 30 and about 8 weeks of continuous data have been collected at this site. About 7 storms have been recorded at this site during this time. This site is characterized with mixed (usually minimal) moisture increases near the beginning of rains, and can have rapid moisture decreases after the rains end, possibly affected by moisture levels in the subgrade and drainage ditch. Equilibrium moisture levels remained at about 70 to 80% until the beginning of April, when diurnal moisture variations were also noted at this location. Ten to 25% diurnal moisture changes were also seen at this location. After the moisture levels were reduced to about 30%, the diurnal changes ceased. The infiltration test results at this site showed high infiltration rates initially (0.5 to 0.8 inches per hour) and decreased to a steady rate of about 0.03 inches per hour after about 10 minutes.

These field tests have confirmed that the pore retention of the water in the drainage layer is probably overwhelming the material's ability to drain. However, the temperature is likely having a significant affect on the pavement moisture level. Other site specific conditions, such as subgrade moisture level (shallow groundwater depth) may also affect pavement drainage. Initial rainfall infiltration rates for the pavement can be quite high, but only for a very short period of time (usually less than 10 minutes).

The pavement structure moisture and drainage time information collected during this research, as summarized on Tables 5-1 through 5-6, allow the AASHTO m_i pavement design coefficient to be determined. The following table shows the quality of drainage and the percentage of time that the pavement structure is exposed to moisture levels approaching saturation (assumed to be 80%):

Table 5-1. Moisture Summary for Good Drainage Sites

HIGHWAY 79 / GOOD DRAINAGE LOCATION

FEBRUARY										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	23%	43%	45%	45%	50%	89%	100%	100%
MARCH										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	17%	28%	32%	38%	46%	100%	100%	100%
APRIL										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	1%	8%	15%	38%	90%	92%	93%	93%
MAY										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	0%	0%	0%	3%	7%	10%	20%	34%
Weighted Averages	0%	0%	11%	22%	26%	36%	56%	84%	88%	90%

INTERSTATE 459 / GOOD DRAINAGE LOCATION

JANUARY										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	0%	0%	99%	100%	100%	100%	100%	100%
FEBRUARY										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	0%	0%	100%	100%	100%	100%	100%	100%
MARCH										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	0%	40%	100%	100%	100%	100%	100%	100%
APRIL										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	0%	66%	100%	100%	100%	100%	100%	100%
MAY										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	19%	100%	100%	100%	100%	100%	100%	100%
JUNE										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	0%	100%	100%	100%	100%	100%	100%	100%
Weighted Averages	0%	0%	4%	47%	100%	100%	100%	100%	100%	100%

Table 5-2. Moisture Summary for Poor Drainage Sites

HIGHWAY 79 / POOR DRAINAGE LOCATION

MARCH										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of Time Greater Than Indicated	0%	0%	42%	67%	68%	77%	97%	100%	100%	100%
APRIL										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	17%	94%	97%	99%	100%	100%	100%	100%
MAY										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	0%	4%	8%	13%	14%	41%	100%	100%
Weighted Average	0%	0%	12%	46%	49%	54%	57%	71%	100%	100%

INTERSTATE 459 / POOR DRAINAGE LOCATION

MARCH										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	0%	0%	0%	30%	34%	66%	100%	100%	100%
APRIL										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	1%	7%	13%	38%	68%	79%	98%	100%	100%
MAY										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	0%	12%	18%	19%	85%	100%	100%	100%	100%	100%
JUNE										
Percent Saturation	100	90	80	70	60	50	40	30	20	10
Percentage of time Greater Than Indicated	11%	17%	37%	100%	100%	100%	100%	100%	100%	100%
Weighted Average	2%	6%	13%	26%	58%	77%	86%	99%	100%	100%

Table 5-3. Drainage Time Summary for I-459 (Good Drainage Conditions)

JANUARY

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
1/6/94	63%	63%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
1/14/94	63%	63%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
1/25/94	65%	65%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				

FEBRUARY

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
2/4/94	65%	65%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
2/8/94	65%	65%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
2/9/94	65%	65%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
2/10/94	65%	65%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
2/11/94	65%	65%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
2/12/94	65%	65%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
2/21/94	65%	65%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
2/22/94	65%	65%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				

MARCH

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
3/9/94	69%	69%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
3/19/94	68%	68%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
3/25/94	68%	68%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				

APRIL

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
4/4/94	66%	66%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
4/12/94	68%	68%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
4/15/94	67%	67%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				

MAY

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
5/2/94	69%	69%	N/A	N/A	N/A	=======>	NO RESPONSE TO RAIN EVENT				
5/3/94	71%	71%	N/A	N/A	=============>		NO RESPONSE TO RAIN EVENT				
5/7/94	75%	75%	N/A	N/A	=============>		NO RESPONSE TO RAIN EVENT				
5/9/94	75%	75%	N/A	N/A	=============>		NO RESPONSE TO RAIN EVENT				
5/12/94	75%	75%	N/A	N/A	=============>		NO RESPONSE TO RAIN EVENT				
5/14/94	75%	75%	N/A	N/A	=============>		NO RESPONSE TO RAIN EVENT				
5/26/94	78%	78%	N/A	N/A	=============>		NO RESPONSE TO RAIN EVENT				

JUNE

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
6/3/94	78%	78%	N/A	N/A	=============>		NO RESPONSE TO RAIN EVENT				
6/7/94	78%	78%	N/A	N/A	=============>		NO RESPONSE TO RAIN EVENT				

Table 5-4. Drainage Time Summary for I-459 (Poor Drainage Conditions)

MARCH

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
3/30/94	34%	69%	N/A	N/A	N/A	0.6	0.7	1	N/A	N/A	N/A

APRIL

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
4/4/94	32%	69%	N/A	N/A	N/A	0.9	1	1.5	N/A	N/A	N/A
4/5/94	35%	69%	N/A	N/A	N/A	1.4	1.7	2	N/A	N/A	N/A
4/12/94	32%	73%	N/A	N/A	0.5	1.2	1.3	2.3	N/A	N/A	N/A
4/15/94	36%	79%	N/A	N/A	1.5	2.5	N/A	N/A	N/A	N/A	N/A
4/27/94	57%	100%	0.2	0.3	0.35	1.5	N/A	N/A	N/A	N/A	N/A

MAY

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
5/3/94	58%	98%	0.2	0.35	======>???????????????=====>						
					(Upon maintainance, sensor returned to 58%)						
5/8/94	58%	97%	1	1.2	1.3	1.7	N/A	N/A	N/A	N/A	N/A
5/12/94	58%	100%	0.2	0.3	0.35	======>???????????????=====>					
					(Upon maintainance, sensor returned to 60%)						

JUNE

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
6/6/94	60%	99%	0.15	0.2	0.25	N/A	N/A	N/A	N/A	N/A	N/A
6/7/94	63%	92%	0.25	0.5	0.65	N/A	N/A	N/A	N/A	N/A	N/A
6/9/94	64%	95%	0.15	0.2	0.35	N/A	N/A	N/A	N/A	N/A	N/A
6/9/94	65%	90%	N/A	0.2	0.25	N/A	N/A	N/A	N/A	N/A	N/A

Table 5-5. Drainage Time Summary for H-79 (Good Drainage Conditions)

FEBRUARY

Date of	% Saturation Immediately:		Drainage Time to Indicated Percent Saturation Following Rain (days)								
Rain	Before	After	90%	80%	70%	60%	50%	40%	30%	20%	10%
2/7/94	32%	79%	N/A	N/A	1.7	N/A	N/A	N/A	N/A	N/A	N/A
2/9/94	63%	78%	N/A	N/A	2.2	2.3	2.4	2.9	4	N/A	N/A
2/21/94	30%	85%	N/A	2.5	======>????????????????=====>						
					(Upon maintainance, sensor returned to 32%)						

MARCH

Date of	% Saturation Immediately:		Drainage Time to Indicated Percent Saturation Following Rain (days)								
Rain	Before	After	90%	80%	70%	60%	50%	40%	30%	20%	10%
3/1/94	32%	82%	N/A	1.2	2.7	3	4	7.3	N/A	N/A	N/A
3/8/94	39%	81%	N/A	0.1	======>????????????????=====>						
					(Upon maintainance, sensor returned to 32%)						
3/30/94	32%	82%	N/A	======>????????????????=====>							
					(Upon maintainance, sensor returned to 42%)						

APRIL

Date of	% Saturation Immediately:		Drainage Time to Indicated Percent Saturation Following Rain (days)								
Rain	Before	After	90%	80%	70%	60%	50%	40%	30%	20%	10%
4/15/94	54%	70%	N/A	N/A	N/A	1.6	N/A	N/A	N/A	N/A	N/A

4/28/94 =====> SURFACE IS PAVED =====> GOES TO 0% SATURATION

MAY

Date of	% Saturation Immediately:		Drainage Time to Indicated Percent Saturation Following Rain (days)								
Rain	Before	After	90%	80%	70%	60%	50%	40%	30%	20%	10%
5/6/94	1%	57%	N/A	N/A	N/A	N/A	0.9	1.4	1.6	4	6
5/14/94	6%	36%	N/A	N/A	N/A	N/A	N/A	N/A	0.3	2	2.2

Table 5-6. Drainage Time Summary for H-79 (Poor Drainage Conditions)

MARCH

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
3/23/94	45%	82%	N/A	3.3	======>???????????????=====>						

(Upon maintainance, sensor returned to 50%)

APRIL

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
4/5/94	72%	72%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4/15/94	76%	76%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4/27/94	79%	79%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

4/18/94 ===> DIURNAL OSCILLATIONS BEGIN ===> DOMINATE RESPONSE

MAY

Date of Rain	% Saturation Immediately: Before	After	Drainage Time to Indicated Percent Saturation Following Rain (days) 90%	80%	70%	60%	50%	40%	30%	20%	10%
5/3/94	32%	32%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
5/14/94	29%	29%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
5/26/94	25%	62%	N/A	N/A	N/A	0.05	0.15	0.2	0.25	N/A	N/A

	Quality of Drainage (time to remove water)	Percent of Time Near Saturation	AASHTO coefficient (m_i)
I-459 at Horse Track Interchange (good with drains)	very poor (no response)	4%	0.95-0.75
H-79 at Center Point Rd. (good without drains)	fair (2 days to week)	11%	1.00-0.80
I-459 at Grants Mill Rd. (poor with drains)	good to fair (6 hrs to several days)	13%	1.15-0.80
H-79 at Thompson Tractor (poor without drains)	fair to poor (week, or longer)	12%	1.00-0.60
Overall Range:	good to very poor	4 to 13%	1.15-0.60

When examining these findings, it is very difficult to recognize any pattern in the observed m_i design coefficients:

good sites:	0.95-0.75 and 1.00-0.80 (1.00-0.75)
poor sites:	1.15-0.80 and 1.00-0.60 (1.15-0.60)
with drains:	0.95-0.75 and 1.15-0.80 (1.15-0.75)
without drains:	1.00-0.80 and 1.00-0.60 (1.00-0.60)

Statistical analyses using nonparametric ranking tests indicate that the "poor" sites actually had slightly better drainage conditions, while the presence of edge drains had no effect at all. The authors of this report aren't willing to accept these statistical conclusions alone. Obviously, other factors observed during the tests were probably much more important than the factors listed above. The major factors, from our observations, probably related more to air temperature effects and pavement condition (thickness and presence of deep cracks).

The conditions observed at these locations relate well to the broad range of conditions likely to be found in Alabama and represent the features that make Alabama significantly different from other areas where pavement moisture research has been conducted. These significant features include high levels of rain and hot summer temperatures. The amount of rain that occurs is capable of commonly saturating pavement. If the pavement is in poor condition (thin, deep cracks, and/or especially porous) then this rain can penetrate into the pavement base. It is expected that most highway pavements in Alabama are not in this poor classification, and would not respond very much or quickly to rainfall. The I-459 "good" site therefore likely represents the majority of Alabama highway conditions (having m_i design coefficients of 0.95-0.75). Even though this site had many shallow surface cracks, the pavement was thick and very dense and solid, preventing rainfall moisture from penetrating the pavement. The dense asphalt also severely restricted drainage, especially in conjunction with the crushed limestone base material which was very impermeable, irrespective of the presence of edge drains.

Unusually poor pavement conditions, such as observed at the I-459 "poor" site, may actually have better m_i design coefficients (1.15-0.80) because of the ease of drainage from the thinner and more highly (deeply) cracked pavement. Of course, pavement in this condition is not likely to be a design objective.

The H-79 sites are also unusual for new pavements because of the way they behaved during periods of high temperatures. It is not known why the I-459 sites experienced much smaller diurnal moisture fluctuations. Again, these temperature effects were much more important than the rainfall in determining moisture levels.

The conclusion is that most Alabama pavements should be classified as having poor to very poor quality of drainage, according to the AASHTO Guide. The laboratory and modeling tests of the pavement drainage layer material indicates that pavement under typical Alabama conditions would take a month, or longer, to drain to the 50 percent level from near saturation conditions. The time near saturation, however, is probably quite low and may be in the 5 to 10 percent range, or lower. The corresponding AASHTO flexible pavement m_i coefficient for much of Alabama conditions for modern pavements may therefore be in the relatively broad range of 1.05 to 0.4. Preliminary investigations of the benefits of removing the fines from the base material were also conducted during this research and found that the coefficient can likely be significantly improved.

6.0 SUMMARY AND RECOMMENDATIONS

6.1 Project Summary

This report has described the work performed and results obtained in an investigation whose focus has been on the development of a methodology to assist Alabama highway pavement designers in the selection of appropriate drainage coefficients for flexible pavements. Tabulated values of the necessary coefficients had been previously published in the AASHTO Guide for the Design of Pavement Structures (AASHTO, 1993), but it was left to users of the AASHTO Guide to determine the apppropriate values that should be used in the locale of their interest. These tables have been included in this report as Tables 1.1 and 1.2 (see page 1-3), and are repeated in the present section as Tables 6.1 and 6.2.

Development of the methodology that has been presented in this report has consisted of the performance of several tasks, and has involved both laboratory testing and field analyses as well as numerical modeling. Specific tasks which are described in this summary include:

(1) rainfall analyses for the State of Alabama;

(2) infiltration and percolation tests performed on flexible pavements; and

(3) instrumentation, data collection, and modeling analyses of subsurface moisture levels.

Our principal conclusions, as well as what we believe to be the most significant results, obtained by the completion of each of these tasks are described in the following subsections. Section 6.2 summarizes our recommendations pertaining to drainage coefficient selection.

6.1.1 Rainfall Analyses

Rainfall analyses that were performed in this project involved the determination of average rainfall depths, antecedent dry periods (inter-event periods), average rainfall intensities, and peak rainfall intensities at recording stations across the State of Alabama, as well as in neighboring states where necessary to define conditions near the state boundaries. Charts and graphs have been presented to show the probability distributions of these various descriptors at various sites throughout the state, and contour maps have been presented to provide information at other locations for which records do not exist.

One significant finding in the rainfall analyses was the recognition of how yearly rainfall averages vary between the northern and southern regions of Alabama. In northern Alabama, yearly rainfall averages do not vary widely from one location to another. In southern Alabama, however, the yearly rainfall averages increase significantly as one moves southward towards the Gulf of Mexico.

An additional conclusion is that the dry inter-event period between rainfall events in Alabama is relatively short. This means that there is not much time for a pavement base course layer to drain from a saturated state before it is subjected to additional moisture inflow by the next rainfall occurrence. There is only about a 20 to 35 percent probability in Alabama that at least 5 days elapse between rainfall events. This probability decreases to a range of about 1 to 5 percent for inter-event periods of at least 15 days.

In passing, we caution that the figures and probability distributions that have been presented in this report to summarize Alabama's rainfall characteristics should not be used to supplant those presented by the U.S. Weather Bureau and the National Weather Service when design storm information is required. As noted previously in Section 2.2 of this report, the focus in this project has been on average rainfall conditions as opposed to rainfall extremes. In other words, if one requires information on the T-year rainfall event, say for the design of a culvert or other hydraulic structure, the Weather Bureau and Weather Service publications should be consulted. Where one is interested in average rainfall patterns, however, the figures presented here should be sought.

6.1.2 Infiltration and Percolation Tests

Infiltration and percolation tests that were performed in this project were accomplished using a custom machined infiltrometer as described in Section 4.0. At each of four field investigation sites in the Birmingham metropolitan area, the infiltrometer was sealed to the pavement surface at several locations and the time variations in the rate of infiltration into the asphalt pavement were measured. Repetitions of the experiments were made at each of the four sites not only to validate and study the variability of the inferred infiltration rates, but also to compare the infiltration rates for areas where cracking of the pavement was present to the rates in which there was no pavement cracking.

Infiltration tests performed on Interstate 459 at the Birmingham Turf Club interchange showed that, as expected, infiltration rates for cracked locations are typically much greater than for uncracked locations. At this site, infiltration rates at cracked locations were as high as 15 inches per hour intially, and decreased to about 0.7 inches per hour as the tests progressed. Locations with no cracking displayed infiltration rates from zero to 0.2 inches per hour.

Four repetitions of infiltration tests performed on Interstate 459 at Grant's Mill Road showed initial infiltration rates between 2 and 6 inches per hour that decreased only slightly as the tests progressed. Two additional repetitions, however, showed much lower infiltration rates of about 0.03 inches per hour. It is noted that the asphalt thickness at this location was only about 6 inches. This could explain the consistently high infiltration rates observed in most of the repetitions.

Tests performed on Highway 79 near Thompson Tractor showed initial infiltration rates of about 0.5 to 0.8 inches per hour which decreased to a steady rate of about 0.03 inches per hour over a 10-minute period.

Additional tests performed on Highway 79 at a different location which had just received an asphalt overlay showed very high infiltration rates of about 5 inches per hour. It is believed that the asphalt overlay placed at this location may be an intermediate layer and that an additional final layer with a lower porosity will also be placed at a later date.

In no cases were infiltration and percolation tests performed to study inflow rates through pavement joints at the roadway shoulder. Work by other investigators, such as Cedergren (1974), has shown that joint infiltration rates are typically much higher than those for regular pavement areas, and that joint sealing is usually not very effective. Given that tests in the present project were accomplished only for unjointed areas, one can conclude that the already high infiltration rates observed should be increased somewhat to account for joint infiltration if one is interested in the total rate of infiltration to a pavement structure.

An additional conclusion which may be reached as a result of the infiltration tests is that pavement infiltration rates are usually much greater than is typically acknowledged, especially by rainfall-runoff modelers. In some cases, infiltration rates were observed that were as high or higher than one would expect for an exposed soil surface. However, the rate of decrease of the infiltration rates over time tends to be quite rapid. This rapid decrease in the infiltration rate, when considered along with the often very high initial rates, suggests that the asphalt layer in pavement sections may actually be more permeable than the underlying base course material. In effect, the base course material may be acting almost as a barrier to water movement, and the infiltration rate into the pavement decreases rapidly as the asphalt layer becomes saturated.

6.1.3 Subsurface Moisture Monitoring and Modeling

Subsurface moisture monitoring was accomplished in this project at the same four locations in the Birmingham area as were noted in the previous subsection. These sites were chosen in consultation with ADOT personnel, whose knowledge and experience were employed to select sites at which edge drains were known to either exist or not exist, and at which experience had suggested that drainage conditions were either good or poor. A factorial design was employed to select the four sites based on these characteristics, with sites being classified as to whether they had edge drains and whether their drainage conditions were thought to be poor or good.

Actual subsurface moisture monitoring was accomplished by the installation of three calibrated soil moisture sensors at each field site. Two sensors were placed in the base course layer under the outside lane of the highway pavement, and an additional sensor was placed in the shoulder area. All sensors were connected to a data logger mounted on a pole to the side of the pavement, and the logger was programmed to collect and store data every five minutes. A tipping bucket rain gauge was also mounted on the pole, and rainfall data was collected by the data logger as well.

Interstate 459 at the Birmingham Turf Club interchange represents the field site with "good drainage conditions and edge drains". Data collected at this site show that it is characterized by fairly constant moisture levels which tend to increase slowly near the beginning of rainfall occurrences. Saturation levels at this site have remained high through the testing period, and range from about 60 to 80 percent.

Interstate 459 at Grant's Mill Road is the "poor drainage conditions with edge drains" location. It has been observed that moisture levels at this site tend to increase relatively rapidly near the beginning of rainfall events. Drainage at this site is also relatively rapid. Two sets of equilibrium moisture levels have been observed at this site: one ranges from about 20 to 30 percent saturation, while the other is from about 50 to 60 percent saturation. The rapid moisture changes at this site may be related to the pavement structure; when the pavement was cut to install the moisture sensors, it was noted that it was only about 6 inches thick.

Highway 79 is the "good drainage conditions without edge drains" location. This site is characterized by relatively rapid moisture increases near the beginning of rains, but has slow moisture decreases in the inter-event periods between rains. Equilibrium saturation levels at this site remained in the 60 to 80 percent range until early April, 1994, at which time significant diurnal fluctuations became apparent. The moisture levels were the lowest at mid-day (as low as 30 percent), and were the highest at night (as high as 70 percent). Typical diurnal changes in the saturation level were about 10 percent in early April, and had increased to about 25 percent by the end of April. This site was also overlaid with a new layer of asphalt on the morning of April 25, which resulted in an almost complete removal of the moisture in the pavement section. The moisture levels returned to normal during the morning hours of May 3 when a rainfall depth of about 1 inch fell at the site. It is clear from these observations that heat effects are playing a significant role in determining pavement structure moisture levels in some locations.

Highway 79 near Thompson Tractor is the "poor drainage conditions without edge drains" location that was studied in this project. This site is characterized by mixed (but usually minimal) moisture increases near the beginning of rainfall events, and can have rapid moisture decreases during rainfall inter-event periods. This behavior may be affected by moisture levels in the subgrade and nearby drainage ditch. Equilibrium moisture levels at this site remained in the 70 to 80 percent range until early April, 1994, at which time diurnal fluctuations became apparent at this location also. The magnitudes of the diurnal fluctuations were essentially the same as those observed at the other Highway 79 site (10 percent in early April to 25 percent in late April). However, after the moisture levels were reduced to about 30 percent, the diurnal fluctuations ceased.

It may be observed from these discussions that the drainage conditions revealed by the field studies are opposite to what was initially expected based on observed roadside moisture levels and pavement repair problems at the sites. Reasons for this can not be currently explained, but may be related to site-specific groundwater and/or temperature conditions.

Numerical modeling that was performed in this project has provided results that are qualitatively, but not quantitively, similar to those obtained as a result of the field and laboratory tests. Two types of event-based numerical simulations have been accomplished (1-dimensional and 2-dimensional) to study base course layer drainage times, and 1-dimensional continuous simulations have also been performed to study the behavior of subsurface moisture on a continuous time basis as well.

The saturation results of the 1-D continuous simulations, while not very consistent with the actual field measurements of moisture levels, do display the same qualitative behavior. Namely, there tends to be a rise in moisture level when rainfall occurs, and the moisture level then recedes during inter-event periods. The moisture level as modeled never drops below a certain minimum degree of saturation, whose value depends on the pavement geometry and the hydraulic properties of the base course material. A comparison of the model results shown in Figure 5.27 with the actual rainfall and moisture data collected for the field monitoring sites shows that the model tends to show a much more certain and predictable rise in moisture at the beginning of rainfall events. The recession of moisture as reflected by the model is also much smoother and consistent than the frequently "bumpy" nature displayed by the field data. These differences are likely due to the fact that while the model assumes that the base course layer is nice and homogeneous, there is in reality an existence of heterogeneity and preferential pathways in the layer. As noted earlier, there are also heat effects which should be considered (at least in some cases), and there is almost certainly some air-trapping that occurs when rainfall events provide the water for infiltration. These effects can not be accounted for by the present model.

The 2-D event-based modeling results have shown that spatial variations in the degree of saturation from one location to another in a pavement structure can be quite significant, even in the idealized case of simple drainage of a perfectly saturated and homogeneous layer. The spatial variability in actual pavement structures should be expected to be considerably larger than that predicted by the model. The general trend in the distribution of moisture should be approximately the same, however. The model results show that the region of the base course near the center or inside lanes of a highway, which is near or at the pavement crown, tends to be better drained than the region near the pavement edge. This behavior may be explained, as was done in Sections 5.4 and 5.5, on the basis of capillary retention forces. This behavior was not able to be observed in the field testing program, however, as all of the moisture sensors installed were in the outside lanes. It would be advantageous if additional field monitoring, if any is accomplished at a future date, were to involve moisture sensors near the center or crown of the pavement also. It is interesting to note that the observation that the outside lanes of a pavement tend to stay wetter, and hence structurally weaker, than the inside lanes is consistent with the common observation when driving that the inside lanes are usually in much better shape (smoother, fewer potholes, etc.). Of course, there are other factors, such as frequency of traffic loading, that would play a role in this as well.

One conclusion that was reached as a result of the 1-D event-based modeling effort, and which was discussed in Sections 5.4.1 and 5.4.2, is that the time to 85 percent saturation is likely a more meaningful parameter in the context of pavement drainage than is the widely used time to 50 percent drainage. This same conclusion has also been reached by McEnroe (1994), and it suggests that there should be some re-thinking of the AASHTO criteria for determining suitable values of the drainage coefficients for flexible pavement design.

Results of the sensitivity studies performed using the 1-D event-based model should be able to be used rather effectively by pavement designers. To illustrate this, consider the case of making a design decision related to pavement width. Figure 5.17 shows that increasing the width will result in a reduction of the minimum degree of saturation of the pavement section. Figure 5.18, however, shows that the pavement drainage time will be increased if the pavement width is increased. Since one would desire both a low minimum degree of saturation and a short drainage time, it can be seen for this case that not both can be simultaneously attained. In effect, there is a trade-off involved which should be considered. A contrasting case is represented in Figures 5.15 and 5.16, which show the effects of changing the base course layer thickness. It is seen here that increases in the thickness will reduce both the minimum degree of saturation and the drainage time.

In general, the quantitative discrepancies that may be observed between the modeling results and the actual field-measured saturation levels indicate that the models are not really powerful enough to accurately represent the physical processes at work in pavement structures. Future modeling efforts which might be undertaken, either by ourselves or others, should concentrate on trying to use better representations of the physical processes. Based on our observations, new models should be designed at a minimum to be able to handle multiple layers of materials, each with different unsaturated hydraulic properties. The observation that heat effects are sometimes significant also indicates that new models should couple the heat equation with the governing groundwater flow equations so as to permit the representation of heat-driven flows. The air phase should also be modeled so as to permit a better representation of the effect of air-trapping. Most unsaturated flow models treat the air phase passively and consider only the liquid water in the soil pores.

6.2 Recommendations

There are a number of recommendations which may be made based on the completion of the research effort described in this report, and several of these have already been alluded to in the body of the report. Of course, the basis for these recommendations is the work that has been completed in this project, and the experience gained and data obtained may be viewed as rather limited. The recommendations made should therefore be viewed with the recognition in mind that our experience may not completely agree, and may even contradict, that of others. In any case, it is believed that our statements are warranted in view of the observations that have been made.

One of the more elusive issues that has been encountered in this project relates to the vagueness of the headings in Tables 6.1 and 6.2. It is simply not clear what is meant by "conditions approaching saturation", nor is it entirely clear what is meant by water "removal" times. We have concluded that the time to 85 percent saturation is probably more meaningful measure of water "removal" time than is the more commonly used time to 50 percent drainage, and while this in itself may represent some progress in this direction, it is still not enough to truly provide a basis for drainage coefficient selection. It is believed that this insight can be employed to help narrow the range of possible drainage coefficient values which might be appropriate for Alabama conditions, but there is still a significant uncertainty as to which column of numbers in Table 6.1 is the most appropriate. It seems that additional work relating moisture levels and pavement strength is required to resolve this issue. There is some discussion of this in Appendix DD of the AASHTO Pavement Design Guide, but there is still not enough data given to come to any conclusions. It is recommended that the ADOT consider the performance of a project of this nature, which would involve both falling weight deflectometer testing in conjunction with further moisture monitoring of the type that has been described in this project.

An additional issue deserving of further investigation is that of manipulation of the hydraulic properties of the commonly used limestone base course material in Alabama through the removal of fines and/or the addition of asphaltic binders. With the relatively large fraction of fines that currently exists in Alabama roadway bases, it is unlikely that very good drainage ever occurs. Water that gets into the base course layer tends to be held there by capillary retention forces, which increase in magnitude as the pore sizes in the base material decrease. Of course, removal of the fines can create construction difficulties, but this may be abe to be overcome, at least in part, by the use of asphaltic binders. The use of binders might also tend to retard the rate at which the limestone tends to be pulverized by repeated traffic loadings. There are at least two issues that should be investigated here: the first relates to the strength characteristics of the material when fines are removed and/or binders are used, and the second relates to economics. Where economics are evaluated, they should be based on life-cycle estimates of costs. That is, one should consider not only the additional expense involved in removing the fines, and possibly adding binders, but also the increase in pavement life which should result as a consequence of the improved drainage characteristics.

An additional recommendation made is that more work should be done related to the performance of highway edge drains. It is not believed that edge drains and pavement base course layers act in the ways which seem to be commonly accepted. Jeffcoat et al. (1992) have concluded that moisture entering edge drains probably derives not from percolation through the base course material itself, but rather from moisture moving through solution channels that have likely developed in the base course layer. This was evidenced not only by the quickness of the response of edge drain outflow to rainfall events, but also by the fact that tracers injected into the base course layer usually could not be detected in the edge drain outflow. The findings of the work reported here tend to support the conclusion reached by Jeffcoat et al. Based on these indications that the hydraulic behavior of highway bases and edge drains are likely different than what is commonly believed, it is questionable as to whether the expenses involved with the design and installation of edge drain systems are justifiable.

A final point made is that there appears to be a significant amount of work that should be done to improve numerical modeling codes intended for simulation of moisture movement in pavement structures. Currently available modeling codes are too simplified to represent what is actually going on, and in some cases can not even solve the simplified expressions. Future modeling efforts should concentrate on making the numerical solution strategies more robust (in terms of their ability to find a solution) and on improving the representations of the physical processes at work. At a minimum, new models should be able to handle multiple layers of materials, each of which may have different unsaturated hydraulic properties. The observation that heat effects are frequently significant in pavement structures also indicates that new models should couple the heat equation with the governing groundwater flow equations so as to permit the representation of heat-driven flows. The air phase in the soil pores should also be modeled explicitly, as opposed to employing passive representations, so as to better account for the phenomenon of air-trapping during infiltration events.

6.3 Project Implementation

Tables 6.1 and 6.2, which are a repetition of Tables 1.1 and 1.2 presented earlier in this report, summarize the recommended drainage coefficient values for flexible pavement design as set forth by the AASHTO (1993). As can be seen, and as already noted, selection of a drainage coefficient value must be based on an estimate of the quality of drainage of a pavement structure, as well as on an estimate of the percentage of time that the pavement structure is exposed to moisture levels approaching saturation.

The laboratory, field, and modeling analyses that have been performed in this project have certainly made some progress towards an understanding of these pavement drainage measures, but have been confounded by a large amount of unexplained variability (not related to the measured test data) in their results. The existence of this variability implies that the geometric, hydraulic, and environmental factors that were initially thought to be of primary importance in the context of pavement drainage are inadequate by themselves to yield reliable predictors. In effect, this means that there are other unaccounted for factors whose relative degrees of importance remain to be established. Indeed, even the four sites monitored in the Birmingham area displayed drainage characteristics that were in complete contradiction with what was expected based on our reviews of the published literature and the experience of ADOT

TABLE 6.1

Recommended m_i Values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements
(Source: AASHTO, 1993)

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation: Less Than 1%	1–5%	5–25%	Greater Than 25%
Excellent	1.40–1.35	1.35–1.30	1.30–1.20	1.20
Good	1.35–1.25	1.25–1.15	1.15–1.00	1.00
Fair	1.25–1.15	1.15–1.05	1.00–0.80	0.80
Poor	1.15–1.05	1.05–0.80	0.80–0.60	0.60
Very poor	1.05–0.95	0.95–0.75	0.75–0.40	0.40

TABLE 6.2

Relationship Between Quality of Drainage and Water Removal Times
(Source: AASHTO, 1993)

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very poor	(water will not drain)

personnel relating to pavement repair problems. To illustrate, the four sites examined in this project were selected using a factorial design intended to cover the range of conditions pertaining to edge drains and apparent wetness that were felt to be of the greatest importance for Alabama highways. The conclusions reached from our data collection and analysis efforts, however, suggest that neither of these factors appears to be very important. It appears based on our results that the presence of edge drains has no effect whatsoever, and the sites which were intially thought to be poorly drained now appear to be the better drained ones.

Data which has been collected in this project, though a step in the right direction towards reducing the uncertainty and hence improving understanding of the pavement drainage problem, was not adequate to develop reliable predictors of pavement drainage conditions within the narrow ranges of interest for Alabama conditions. Only through a relatively long-term and well designed data collection and experimental program will the apparent variability problems be able to be overcome.

In spite of the observed variability, it can be said of our project results that the quality of drainage of the four Birmingham sites that have been examined in some detail is probably somewhere in the range of good to fair. It is evident in some cases, however, that the quality of drainage may be rather poor. Because of thermal effects, changing amounts of infiltration from one storm to the next, and the extreme spatial variability of hydraulic properties within a base course layer, the quality of drainage even at a single site will often appear to be different from one rainfall event to the next. In view of this variability, and in the interest of providing a recommendation that should lead to conservative designs, it would appear that the "fair" quality of drainage classification may be appropriate for the Birmingham sites. Given that the limestone base course material which was present at all of the monitored sites is also widely used at other locations throughout the state, and that rainfall amounts are not too different across the state (except where they increase sharply near the Gulf coast), this classification would appear to be reasonable for much of the rest of the state as well. Caution should be exercised in areas near the Gulf coast where environmental conditions are significantly different from other portions of the state, and in areas where base course materials have been obtained from sources other than the limestone quarry in the Birmingham area. It should also be expected that variations will occur where there are differences in groundwater levels. Highways in regions with high groundwater tables should be expected to be more poorly drained than regions with low groundwater tables. The data and results for the Birmingham sites did not show this to be the case, however.

With respect to the issue of the percentage of time at which moisture levels are approaching saturation, it has not been possible to come to any definitive conclusions. The vagueness of the term "approaching saturation" has been the main culprit in this regard. In effect, the question is what is "approaching saturation"? Is it 80 percent, or 90 percent, or is it some other figure? Because of this uncertainty, it is again possible only to provide a range on a recommended design value, and little can be said in a predictive sense as to when the upper limit of the range should be applied and when the lower limit should be applied. Given the observations made in the Birmingham area that Alabama pavement moisture levels are usually rather high (because of the frequent rains and the strong capillary retention forces which tend to prevent the base course layers from draining well), it is tentatively recommended that the 5 to 25 percent column in Table 6.1 be employed for highways in Alabama. Combining this with the uncertainty in drainage quality, which may range from good to poor, it is seen from Table 6.1 that the range of drainage coefficients used should be from about 0.60 to 1.15. This recommendation is again made for the Birmingham sites, which are probably reasonably representative of conditions at most other locations throughout Alabama. Differences may be noted in areas where different base course materials are used, or in the southern parts of the state where environmental conditions are much different. Of course, this range from 0.60 to 1.15 is still a rather wide one, but it is considerably smaller than the 0.40 to 1.40 range that is spanned by Table 6.1. Further reductions in the recommended range of drainage coefficient values will be possible only through additional data collection, as noted earlier, as well as through work directed to increasing the understanding of the influential factors affecting the pavement structure moisture levels, and how the moisture diminishes the integrity and strength of highway pavements.

7.0 LITERATURE CITED

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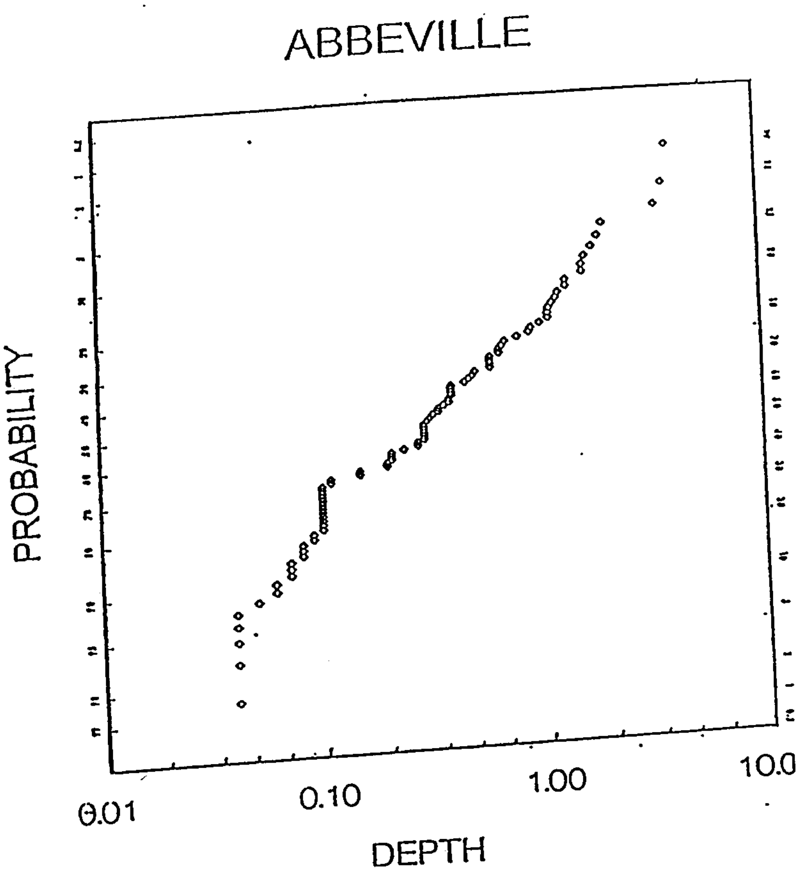
APPENDIX A

REGIONAL RAIN ANALYSES



ABBEVILLE
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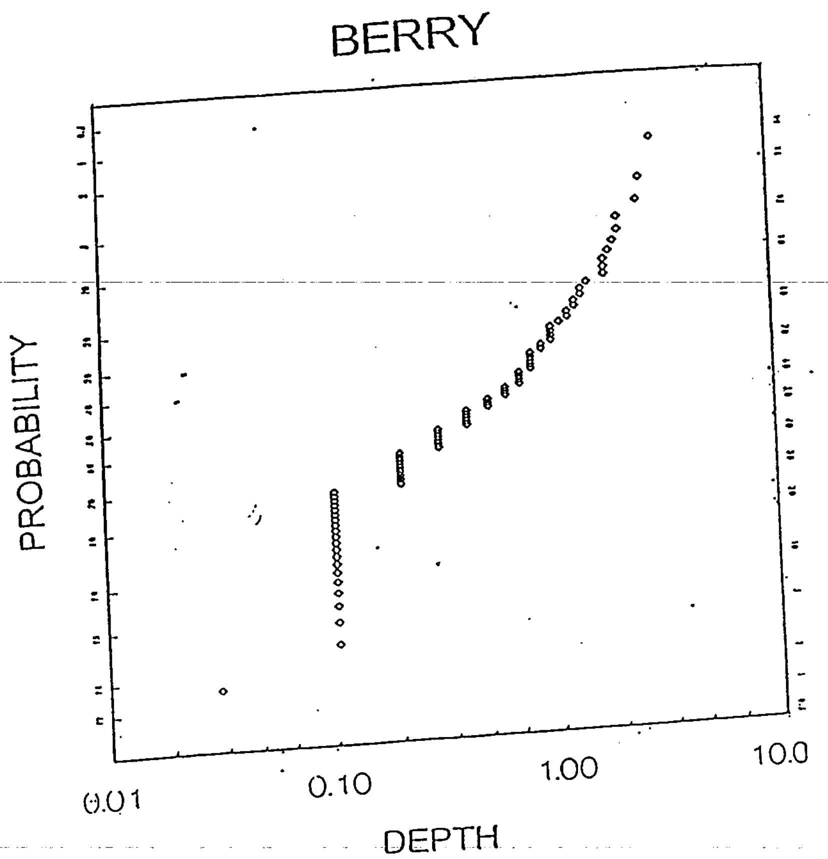
DEPTH



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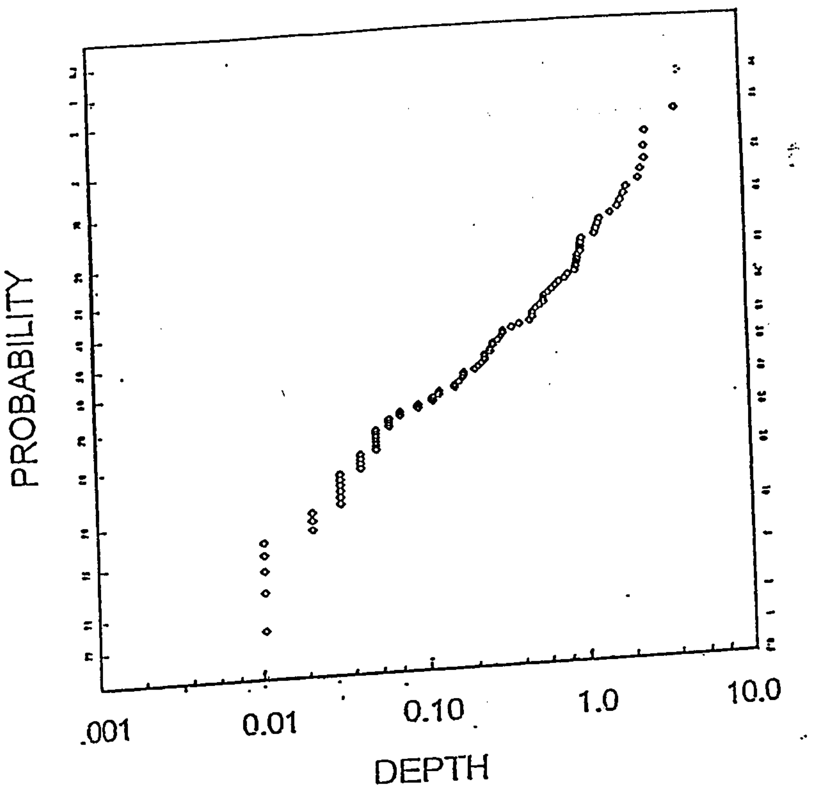
DEPTH

BIRMINGHAM



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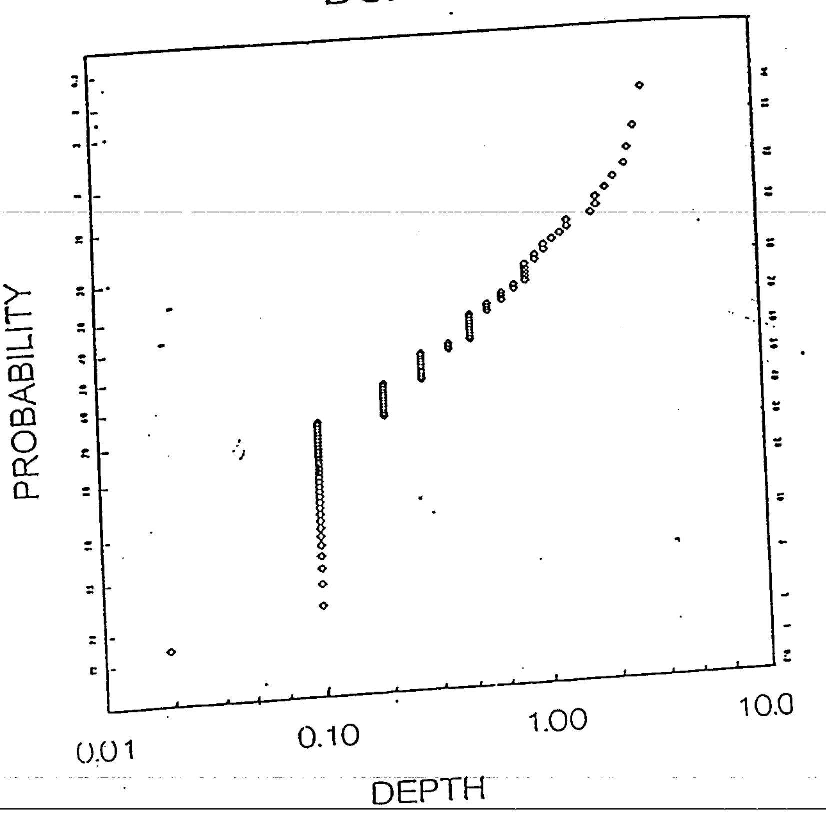

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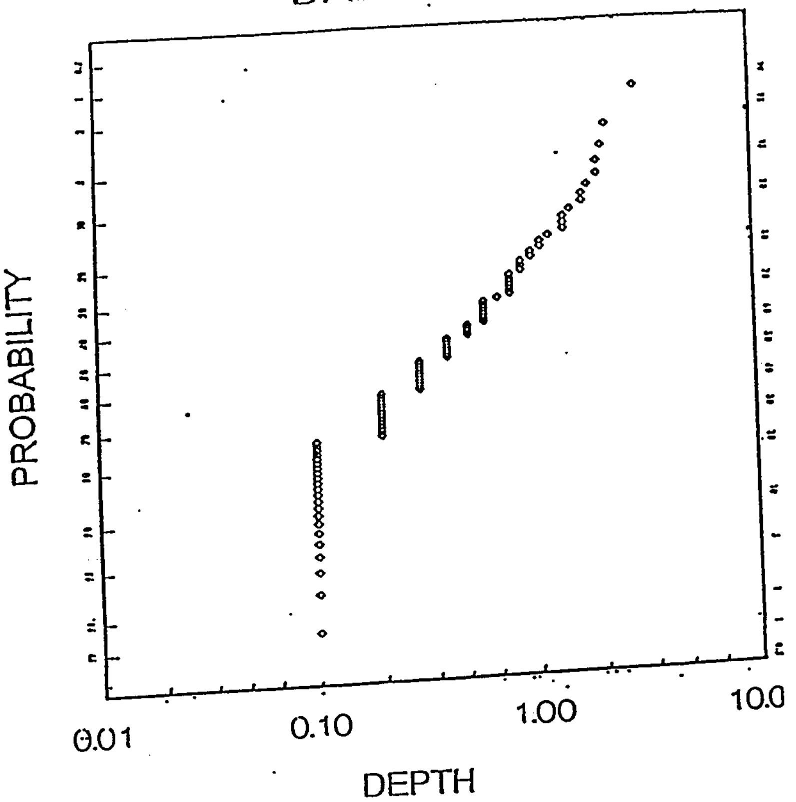

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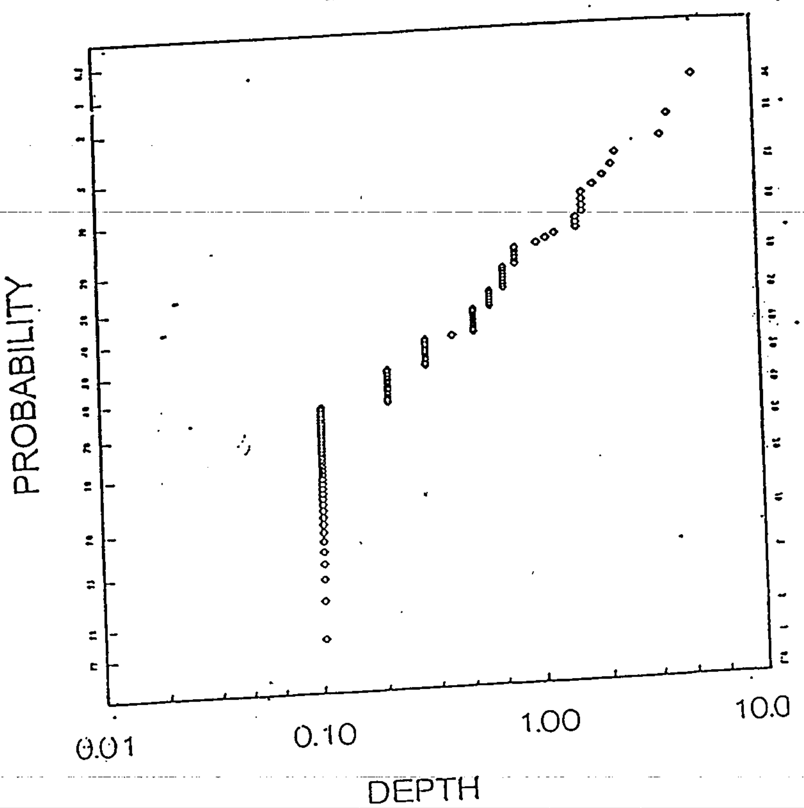

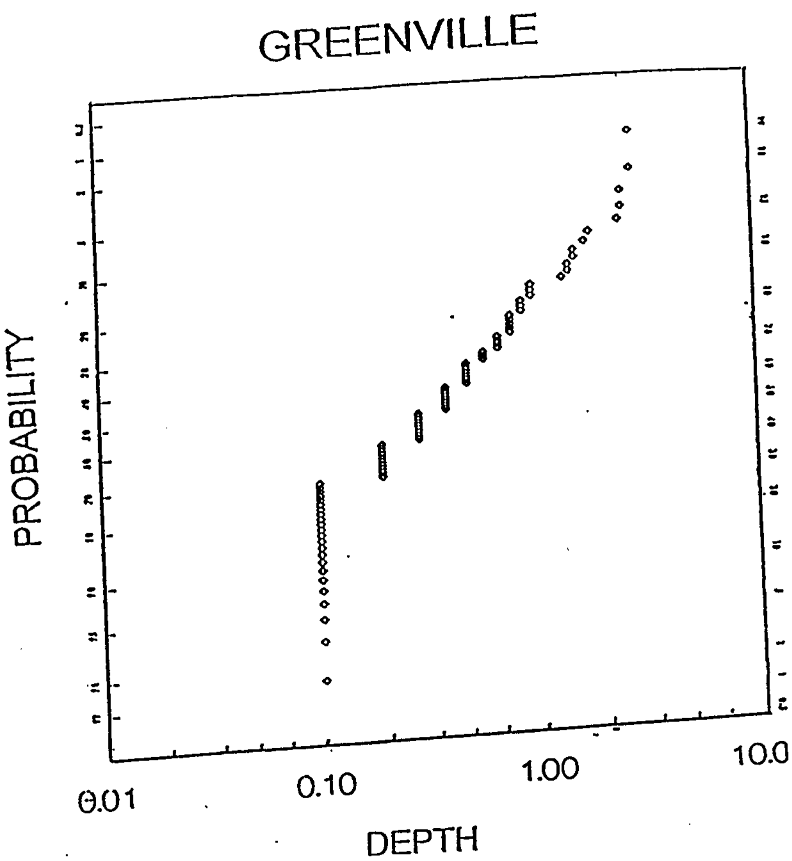
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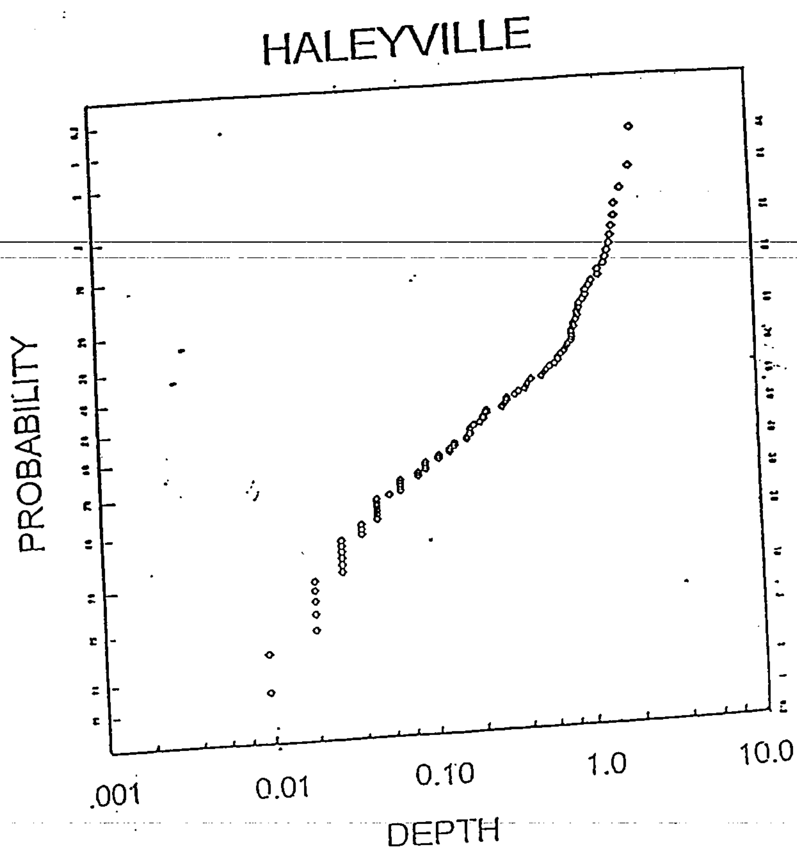
DEPTH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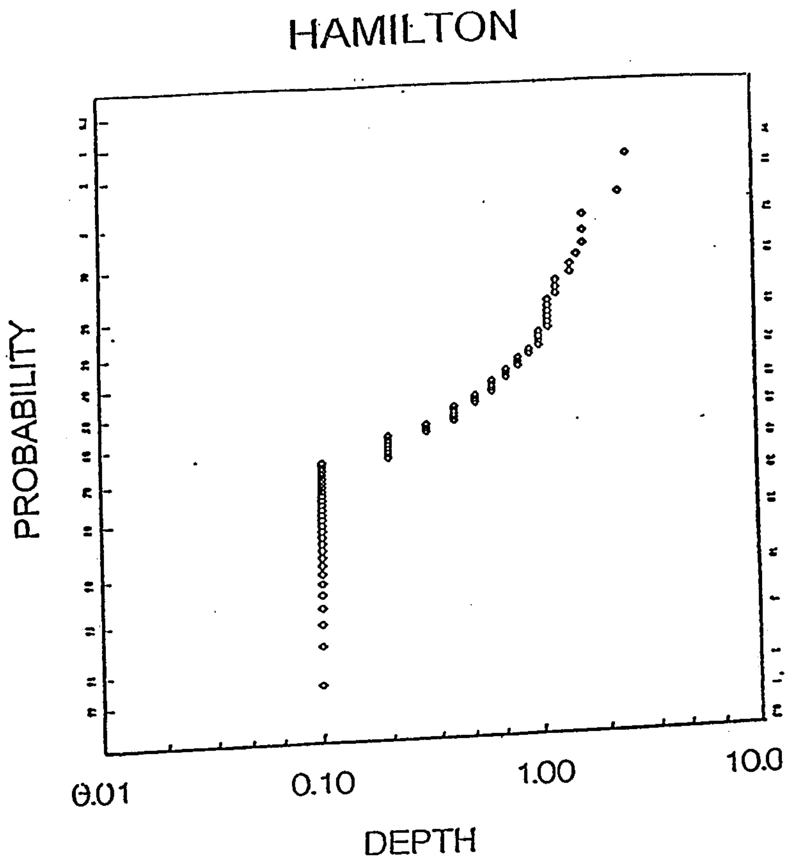
10.0
DEPTH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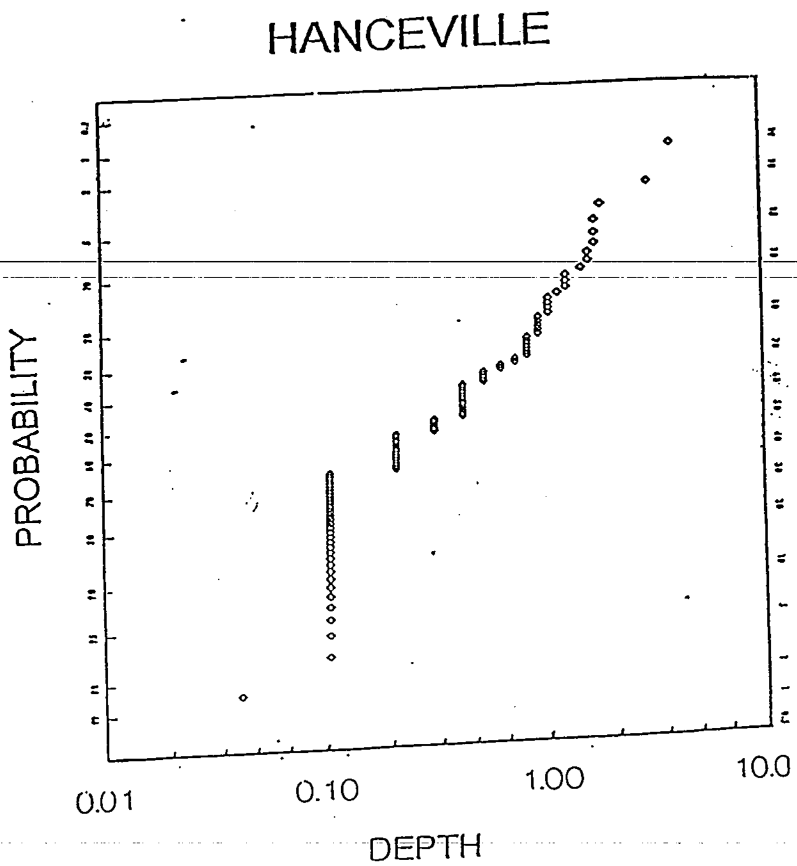
DEPTH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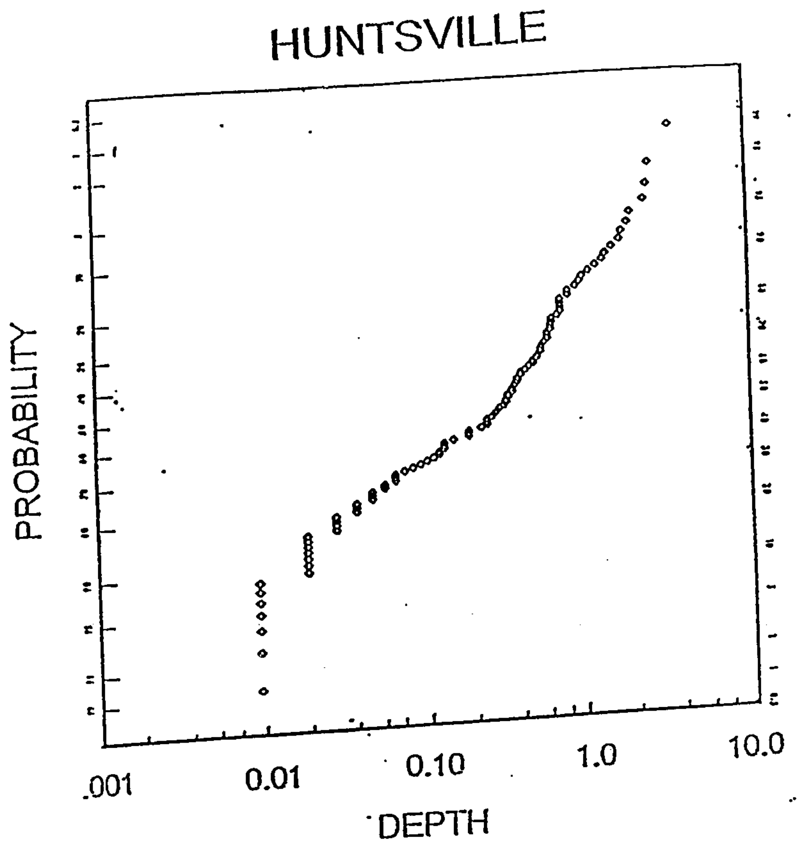
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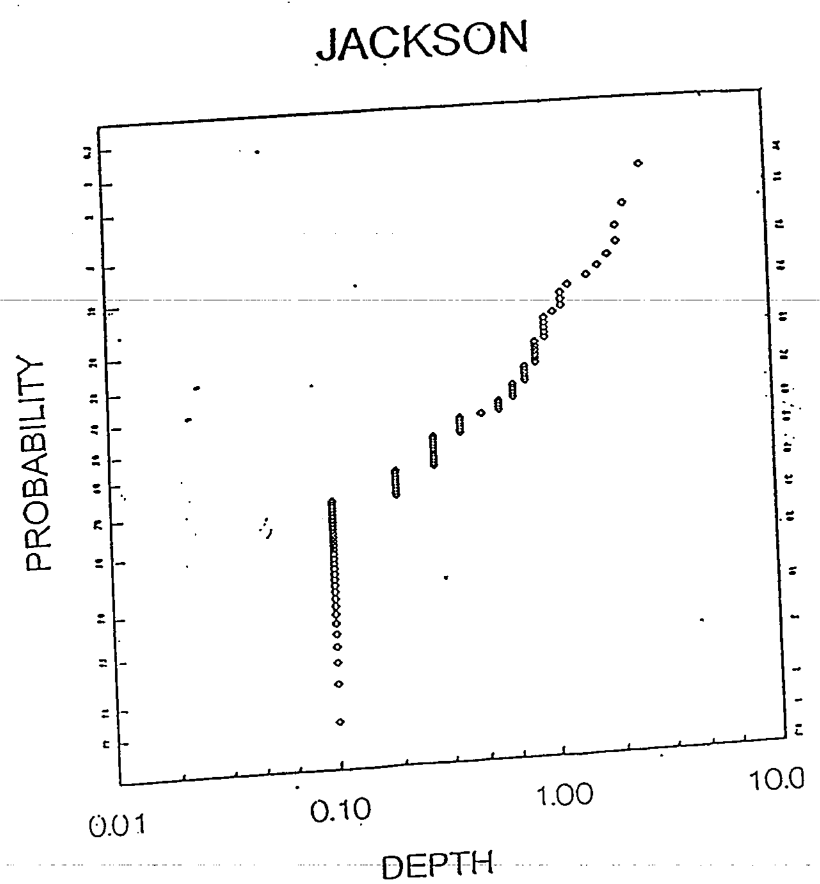
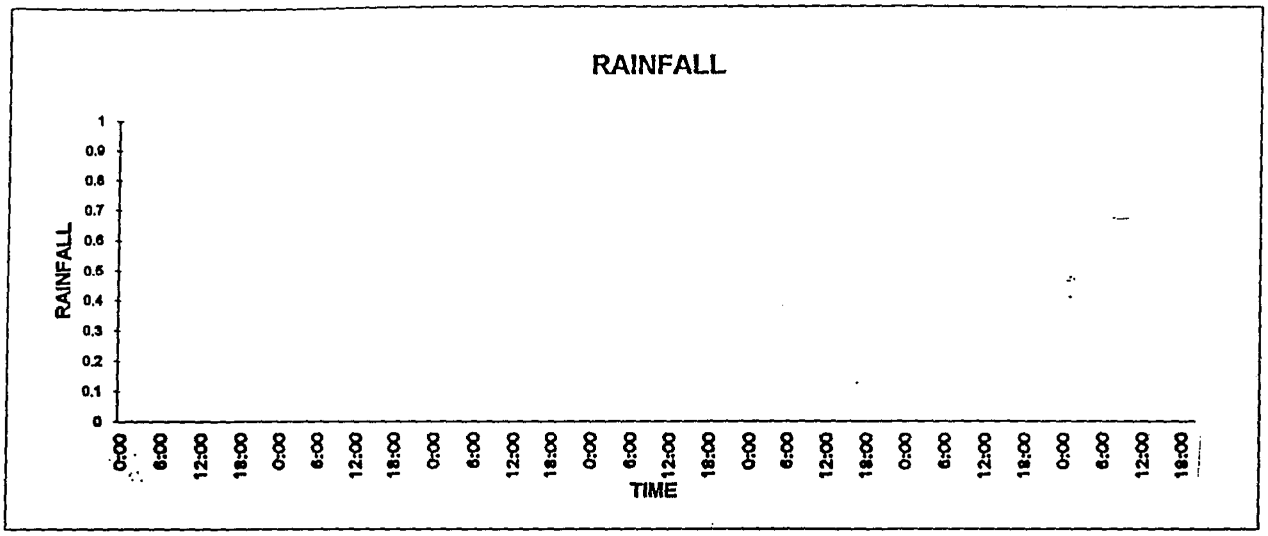
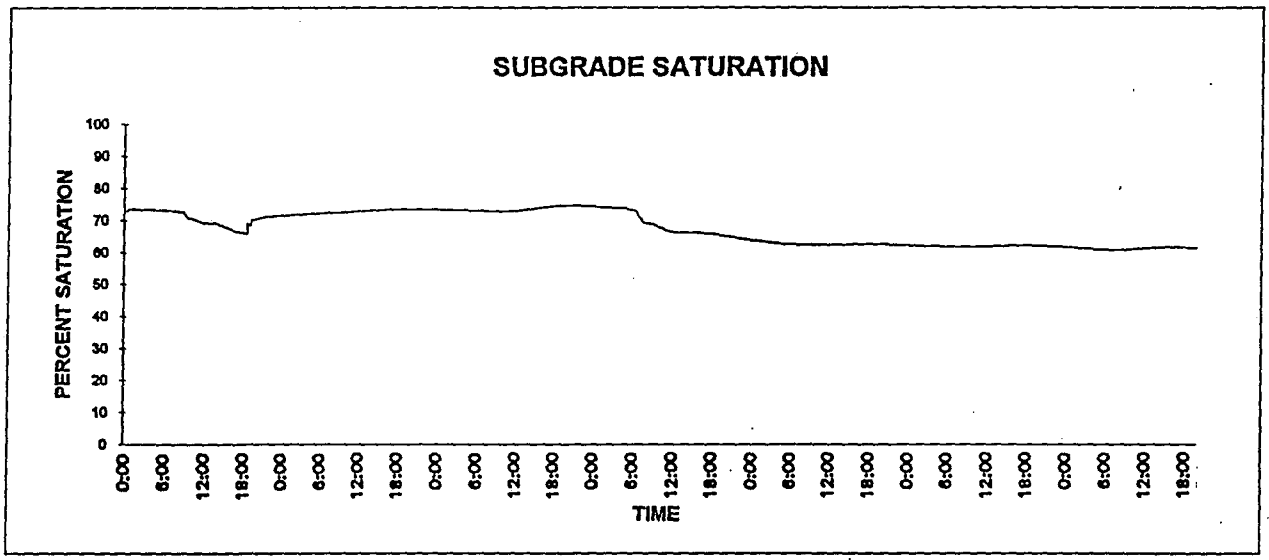
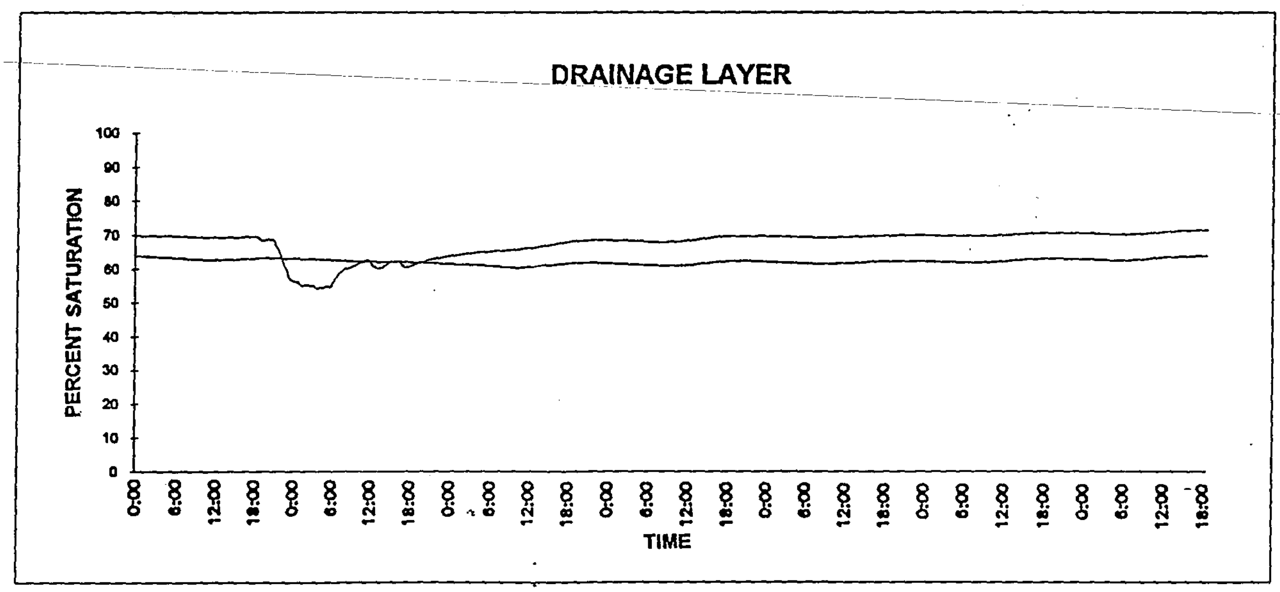
DEPTH


JACKSON
PROBABILITY
0.01
0.10
1.00
10.0
DEPTH

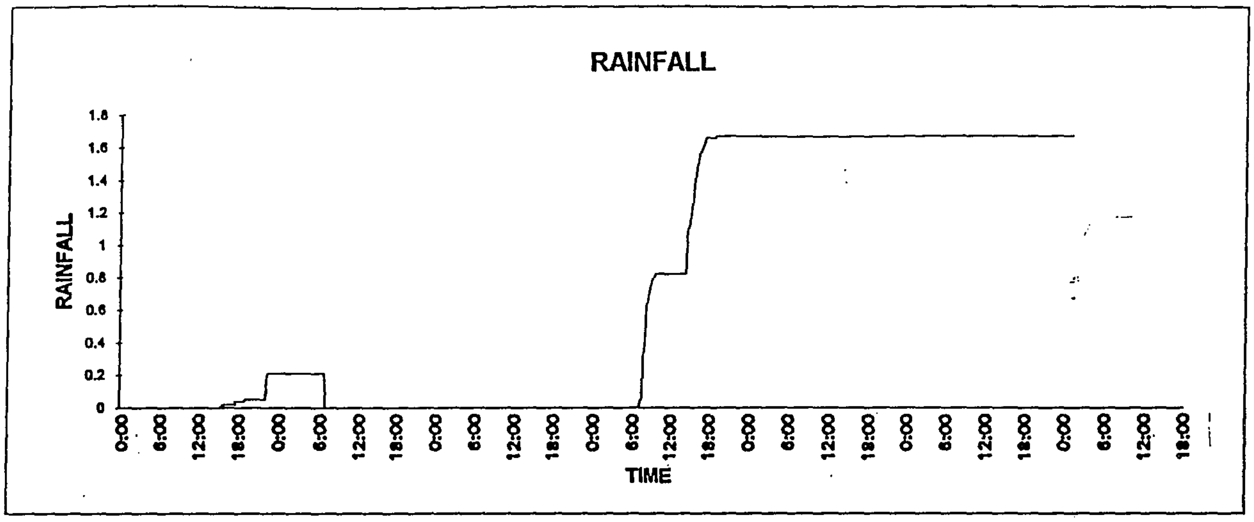
Interstate 459
Good Drainage Location
4/5/94 - 4/11/94

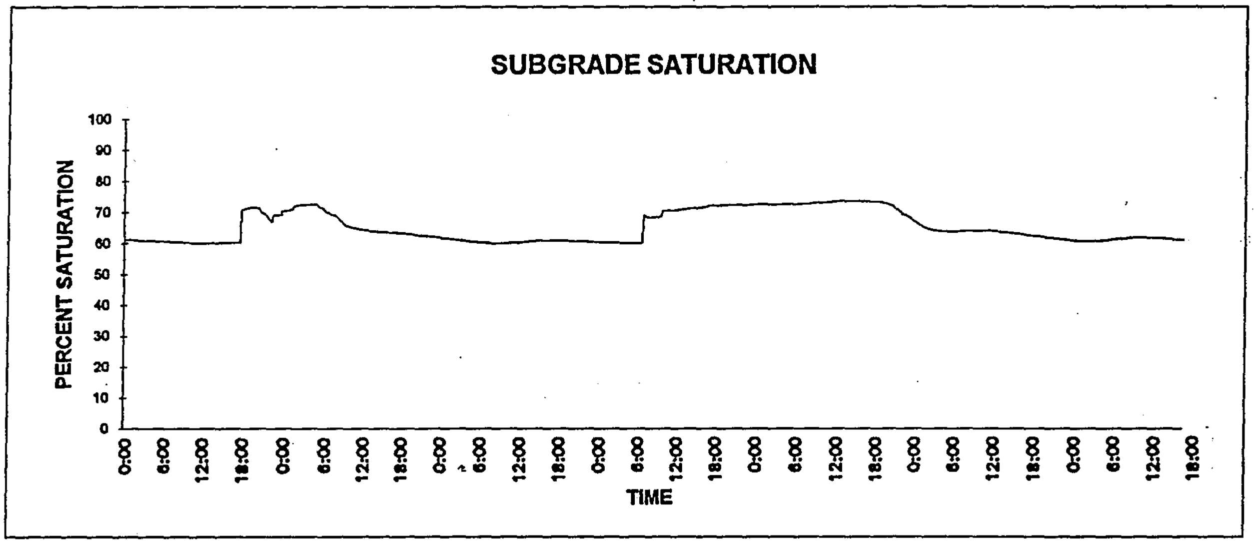


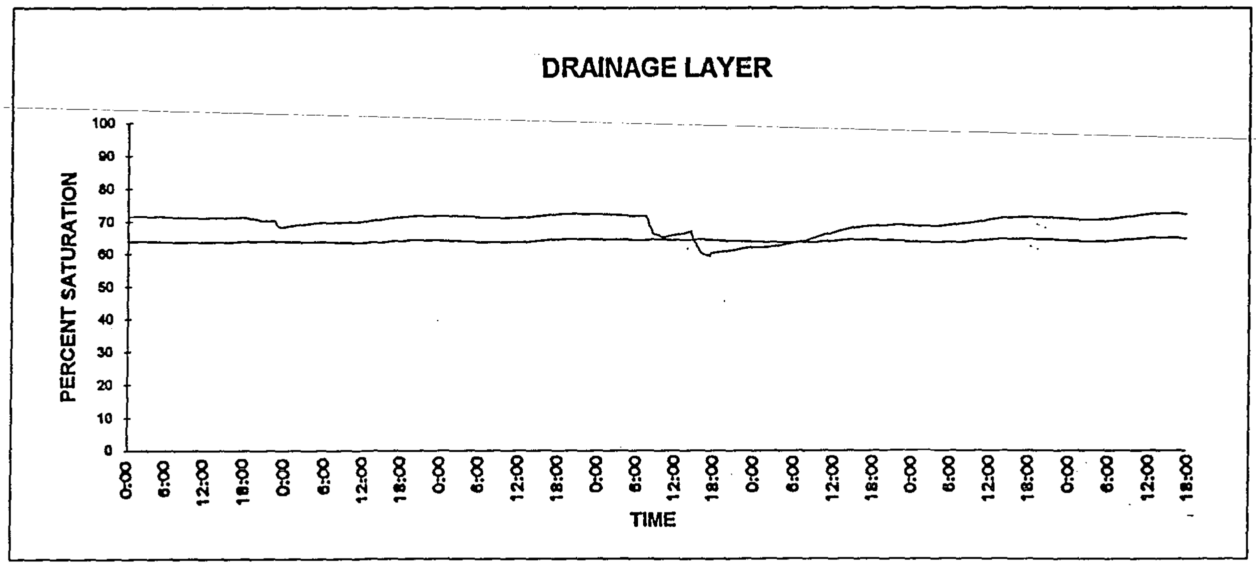




Interstate 459
Good Drainage Location
4/12/94 - 4/18/94







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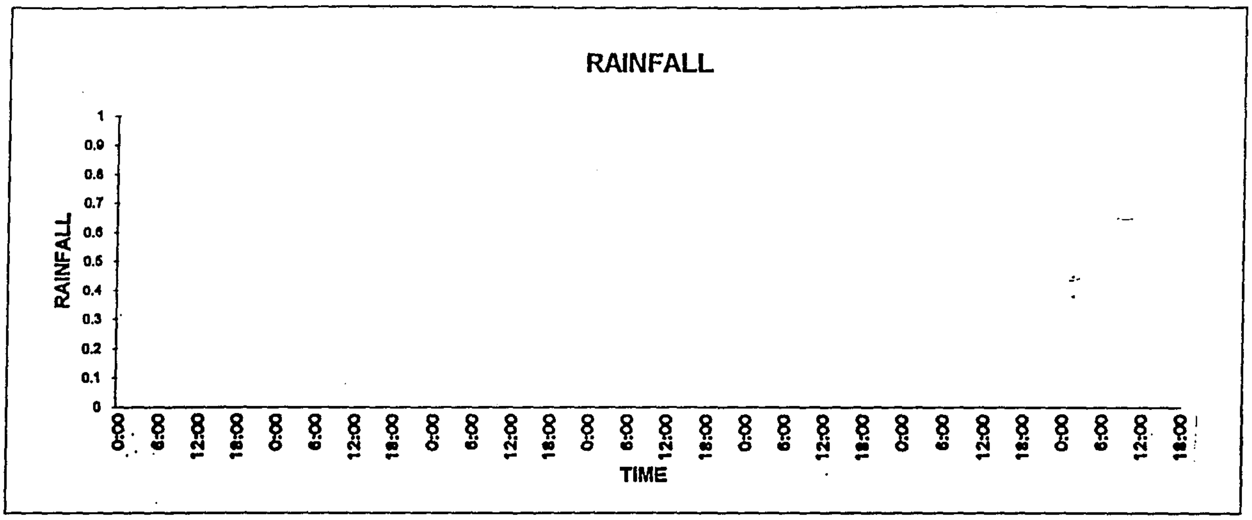
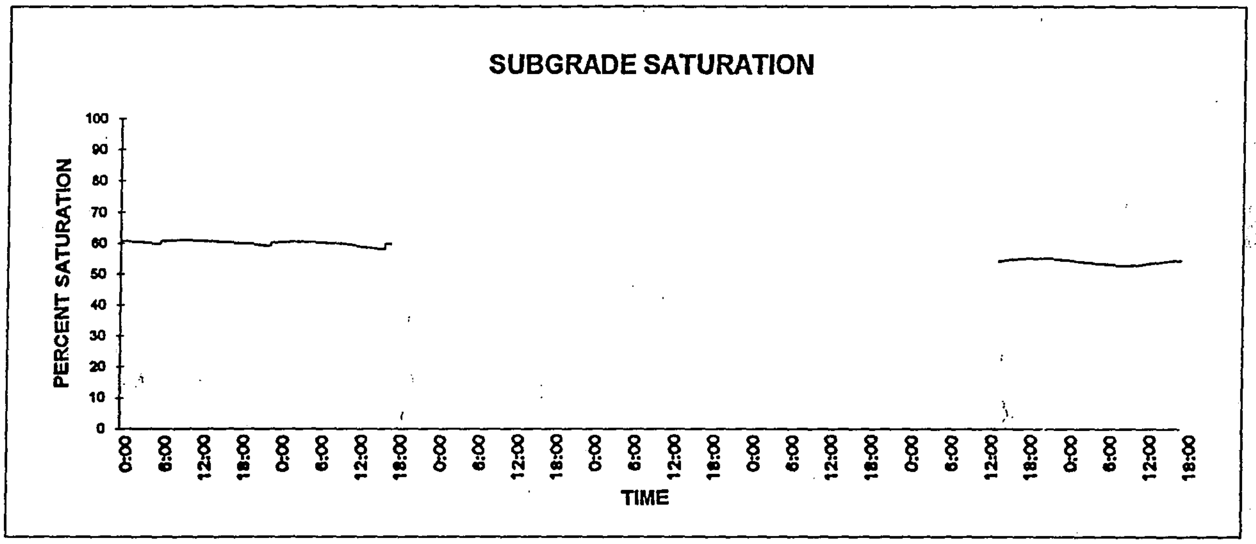
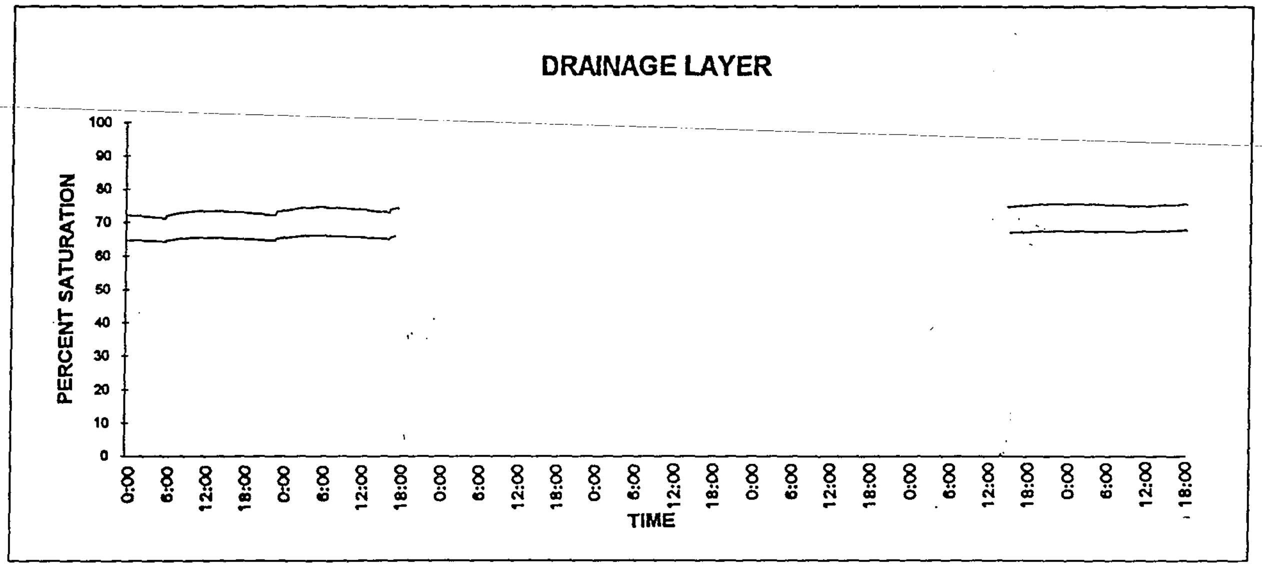
Good Drainage Location
4/19/94 - 4/25/94







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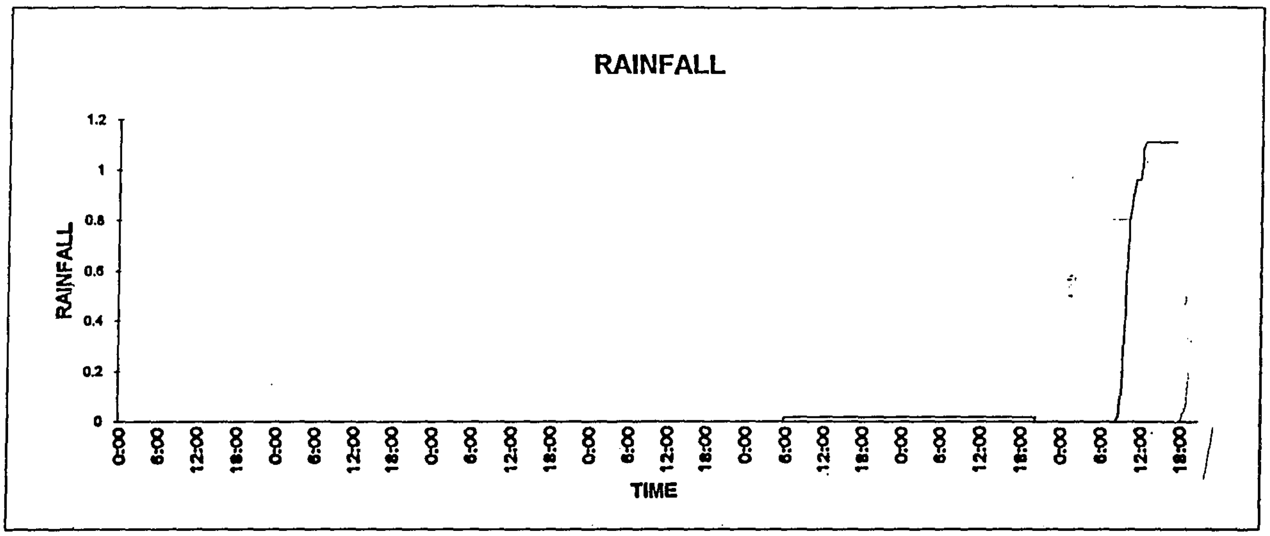
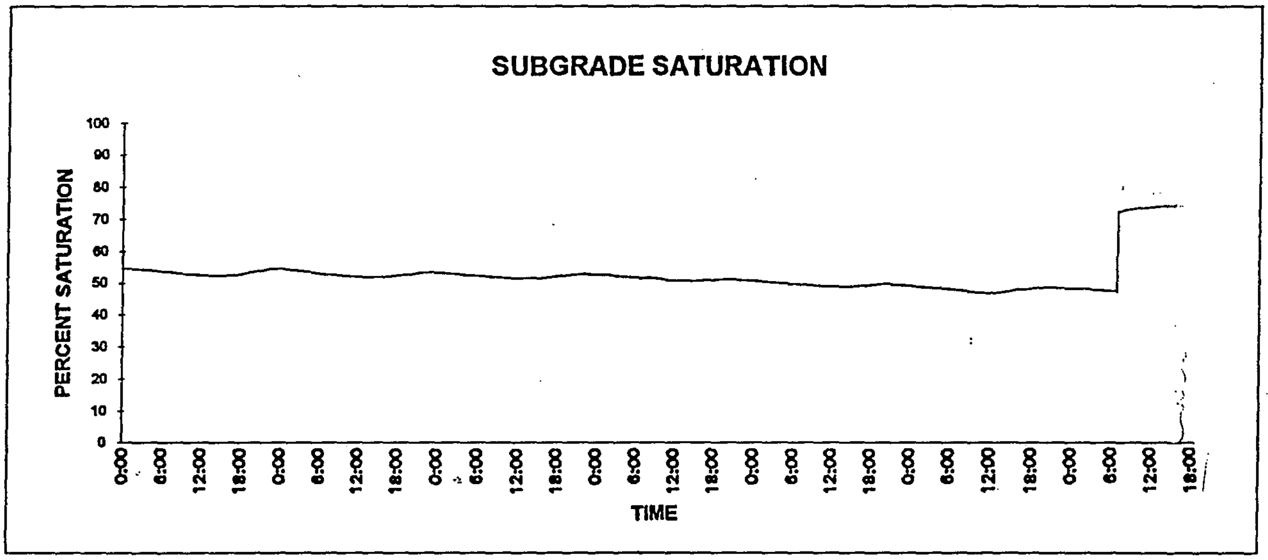
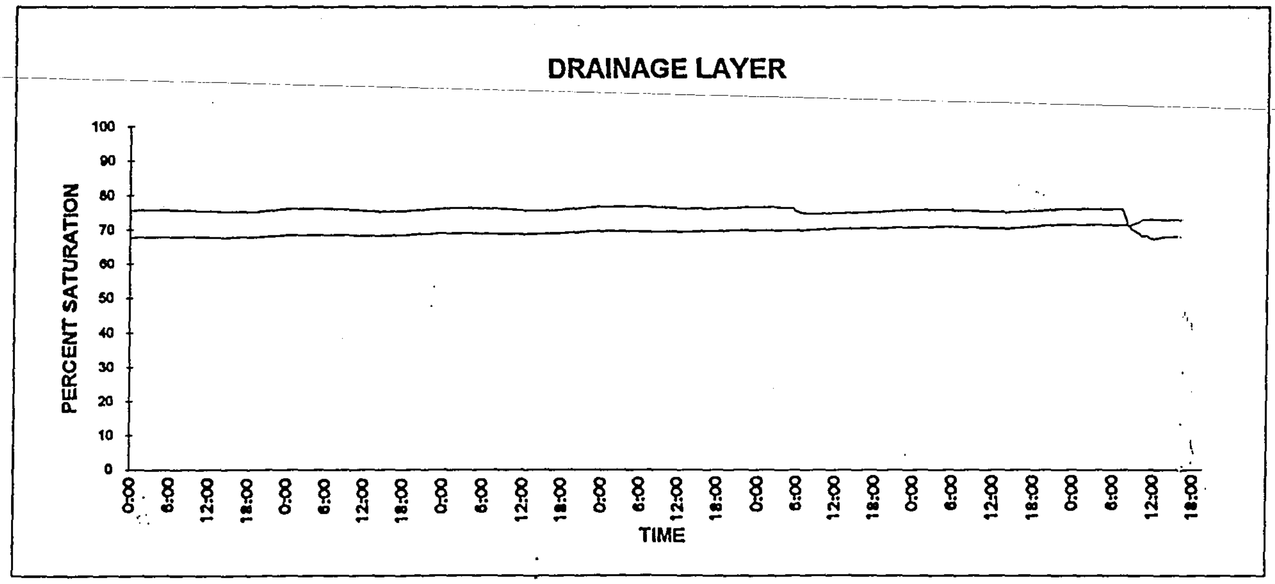
Good Drainage Location
4/26/94 - 5/2/94







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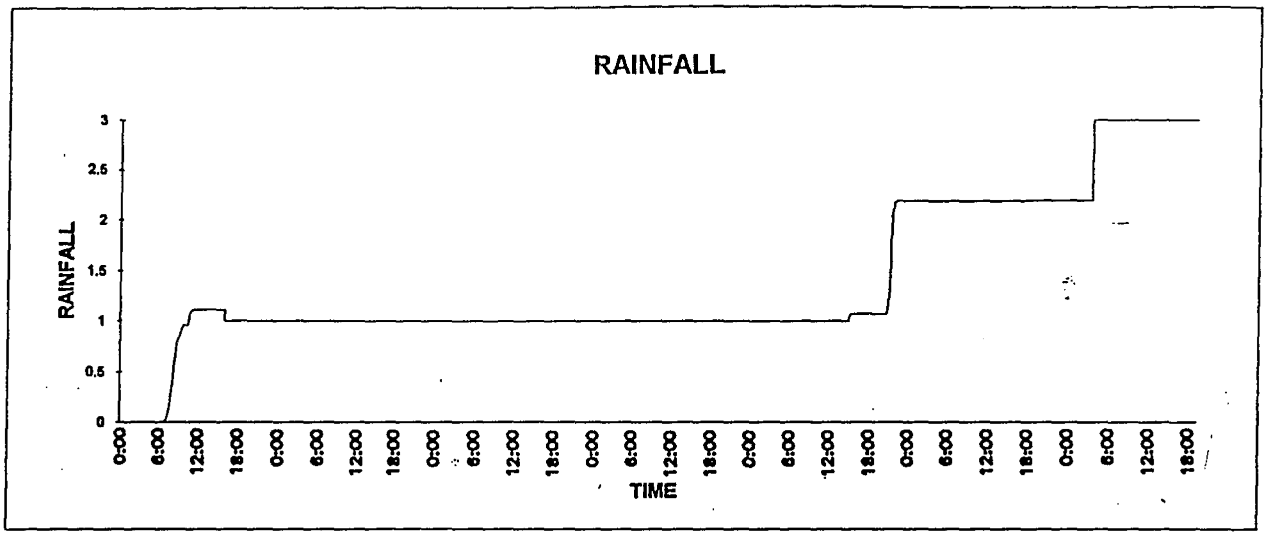
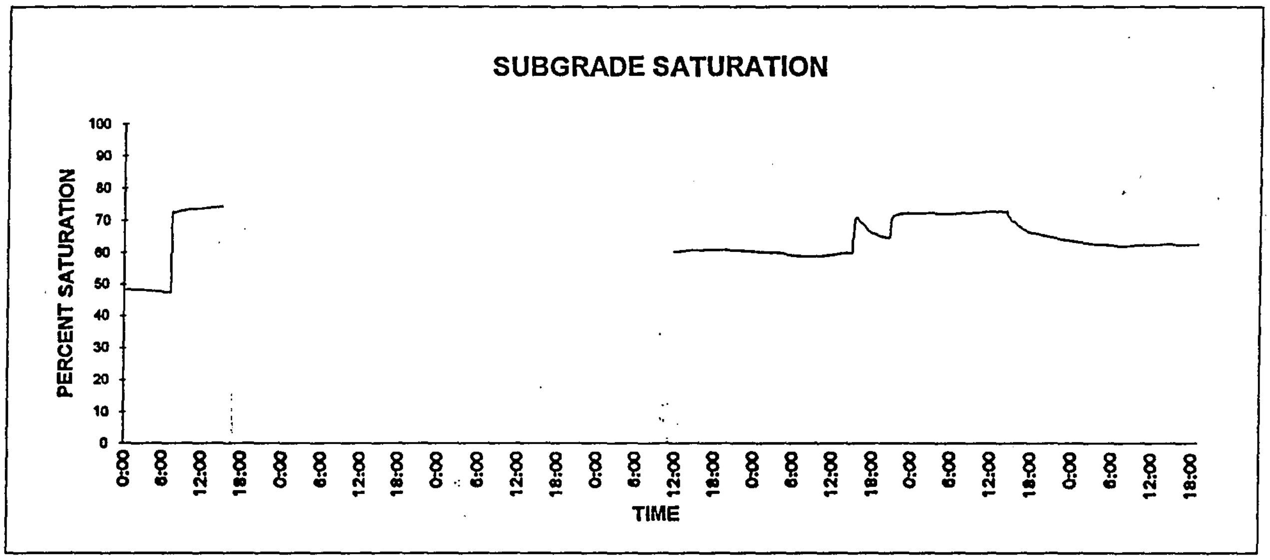
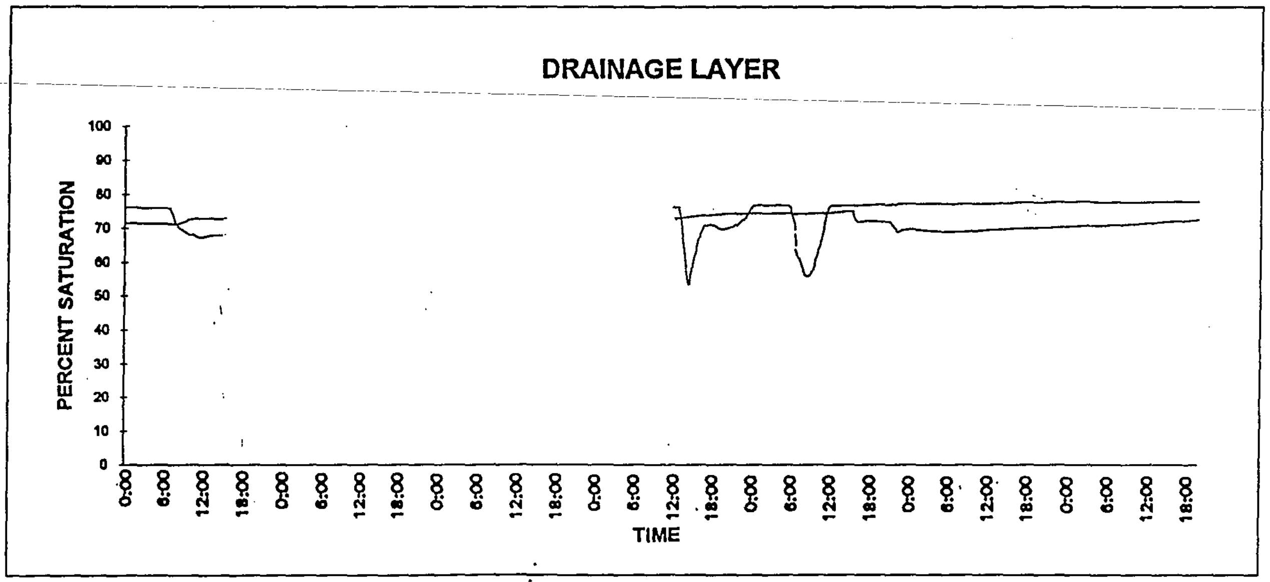
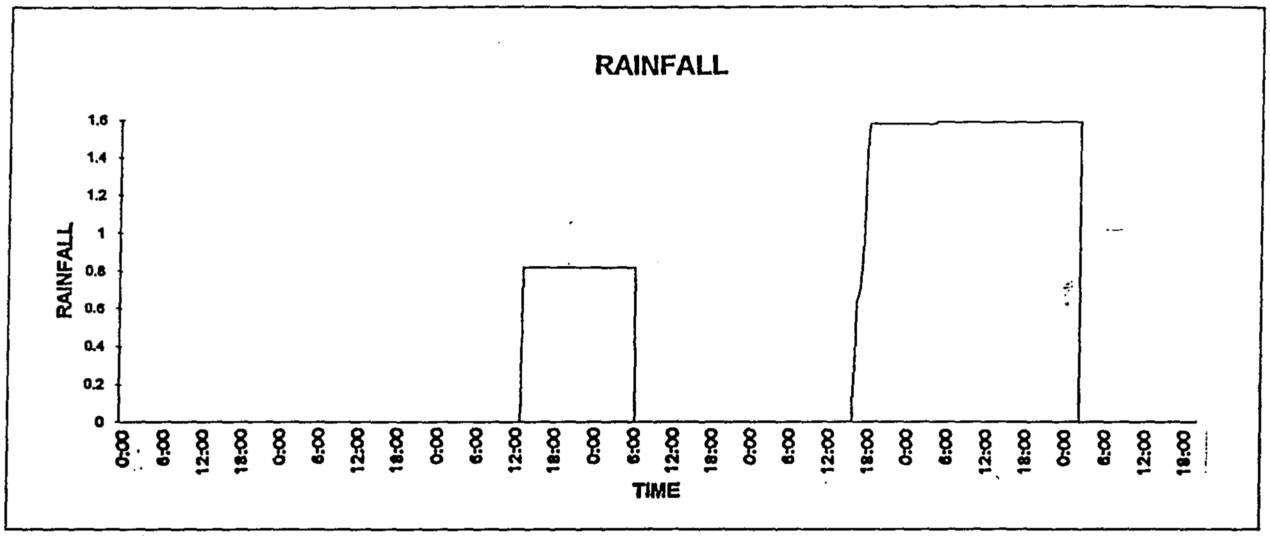
Good Drainage Location
5/3/94 - 5/9/94

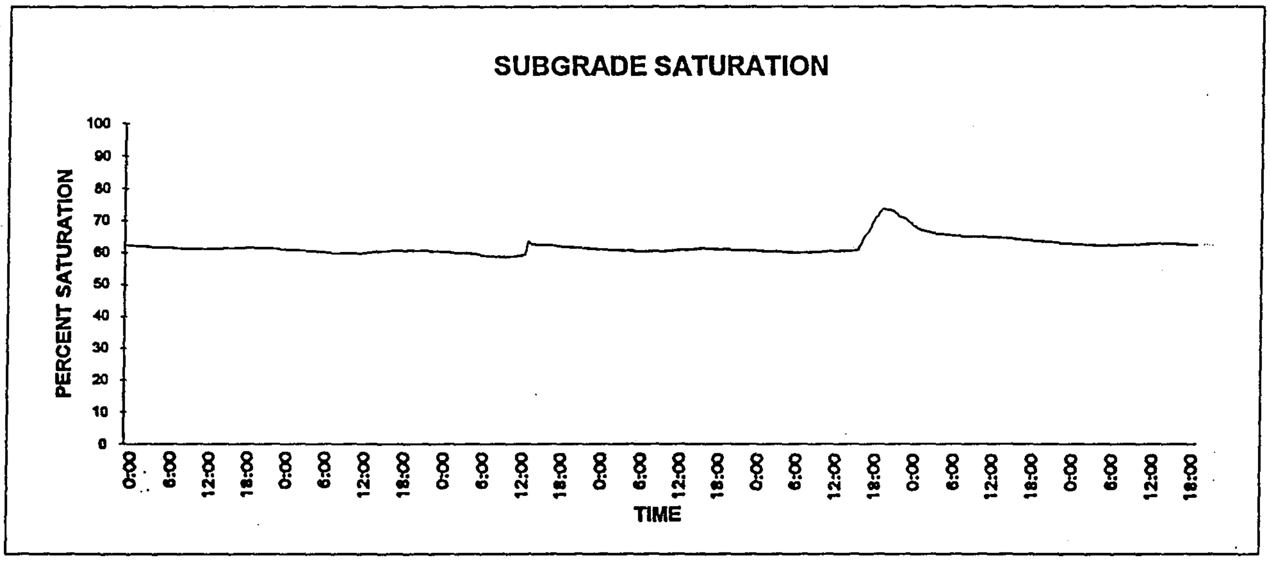


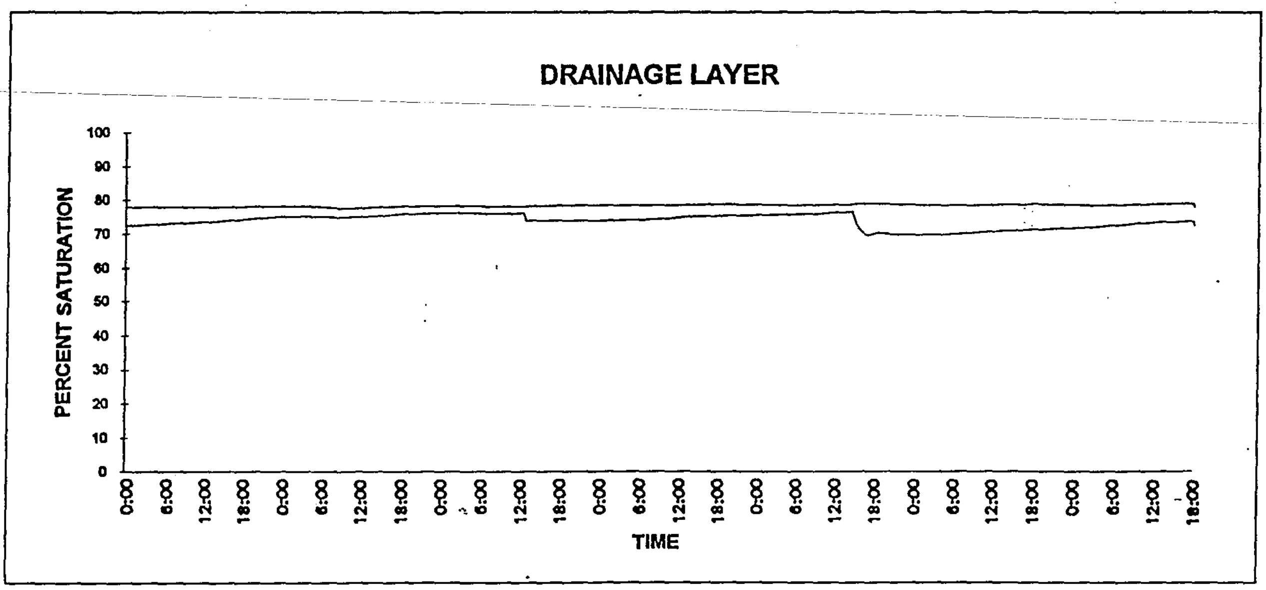




Interstate 459
Good Drainage Location
5/10/94 - 5/16/94







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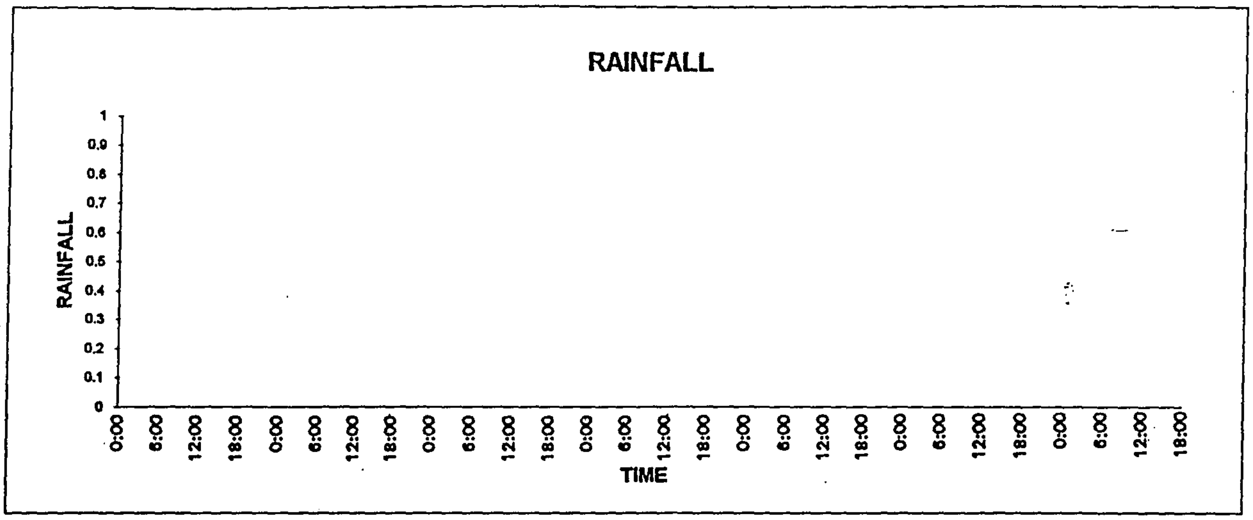
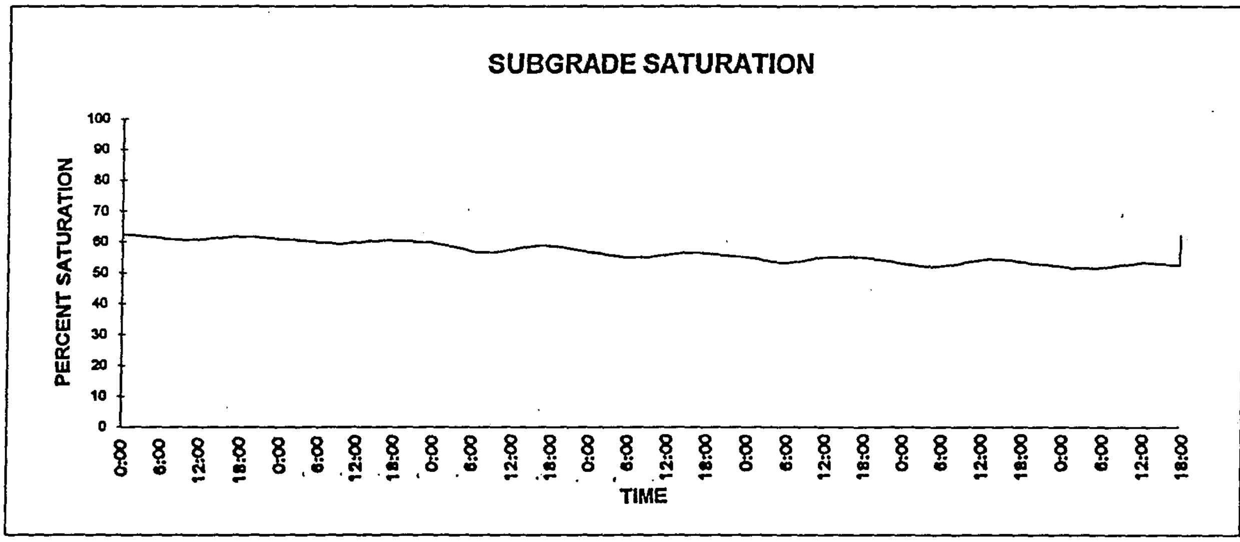
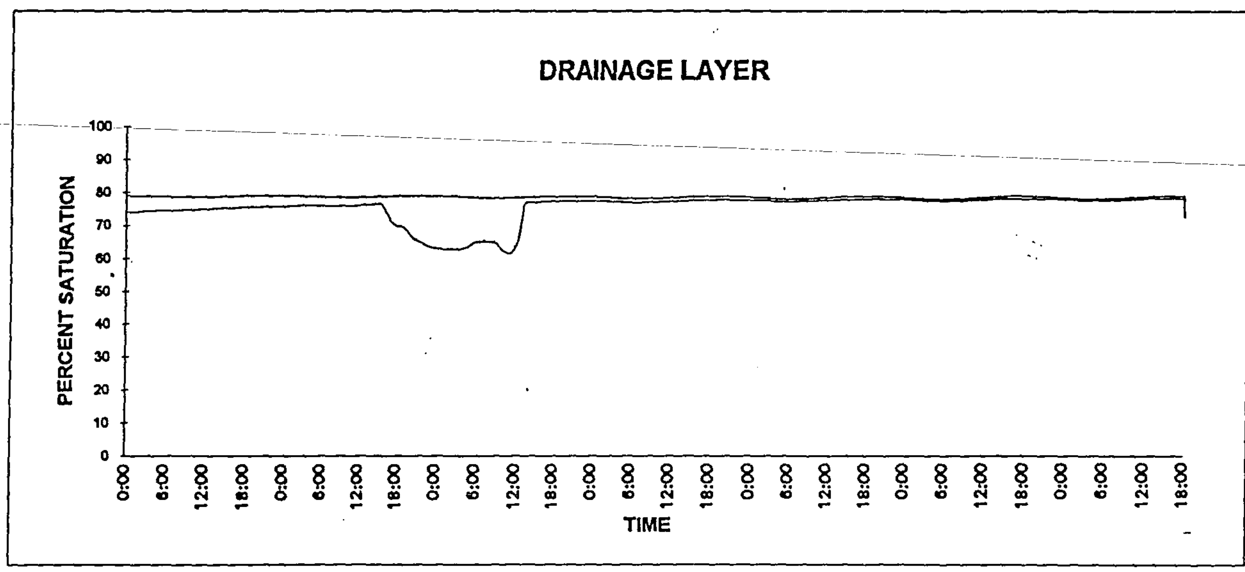
Good Drainage Location
5/17/94 - 5/23/94







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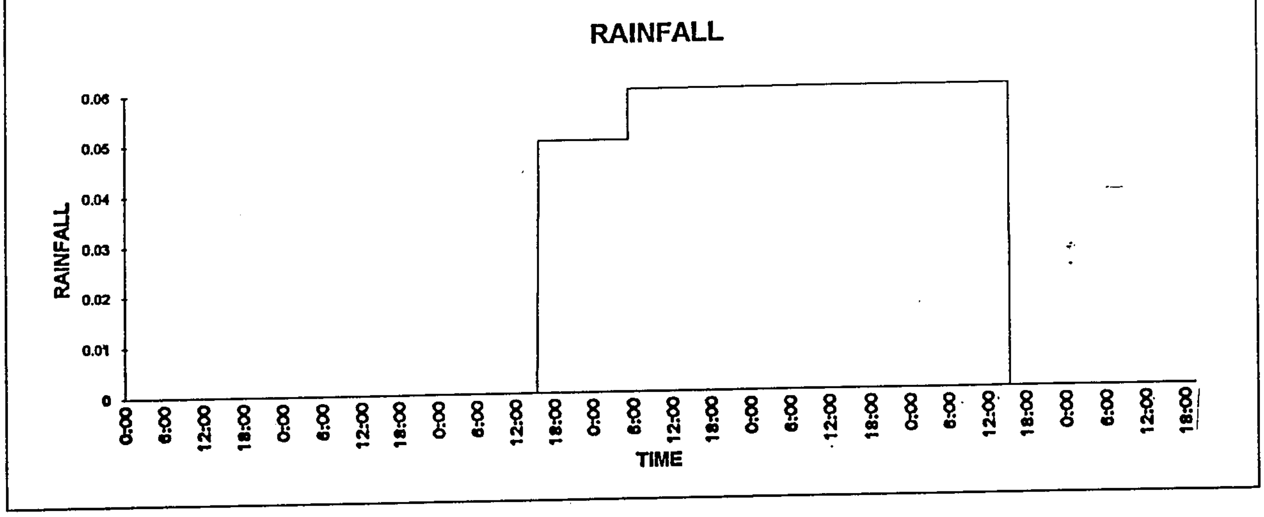
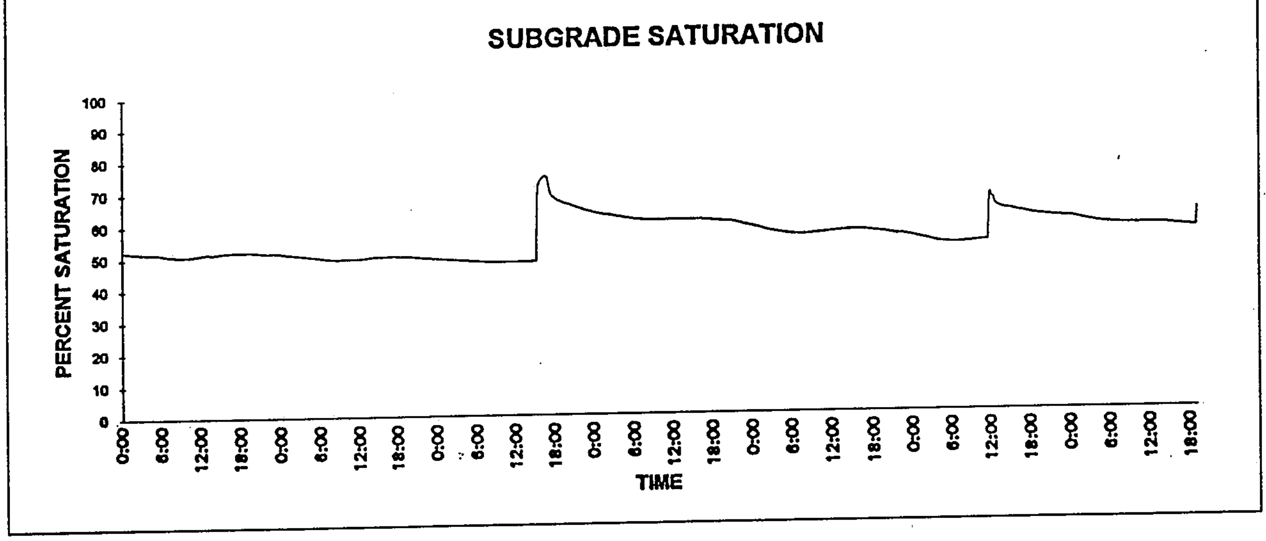
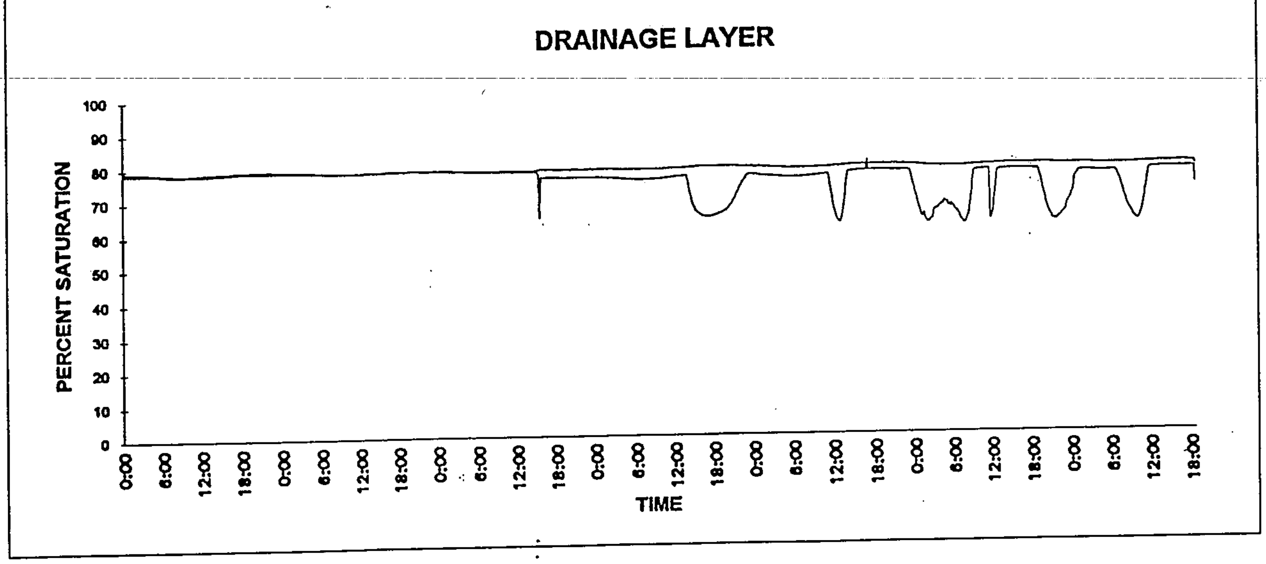
5/24/94 - 5/30/94







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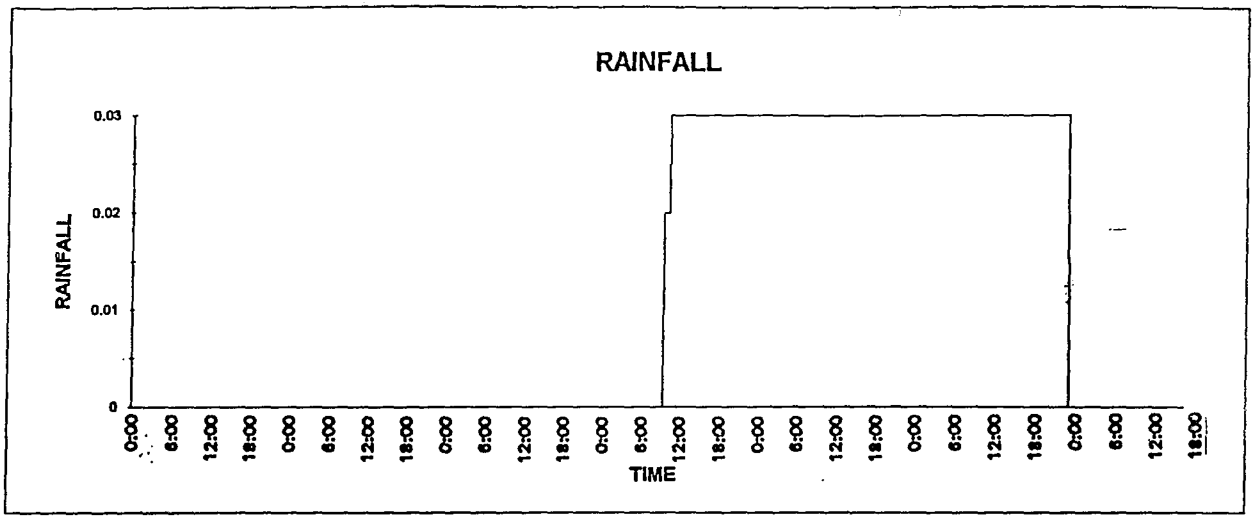

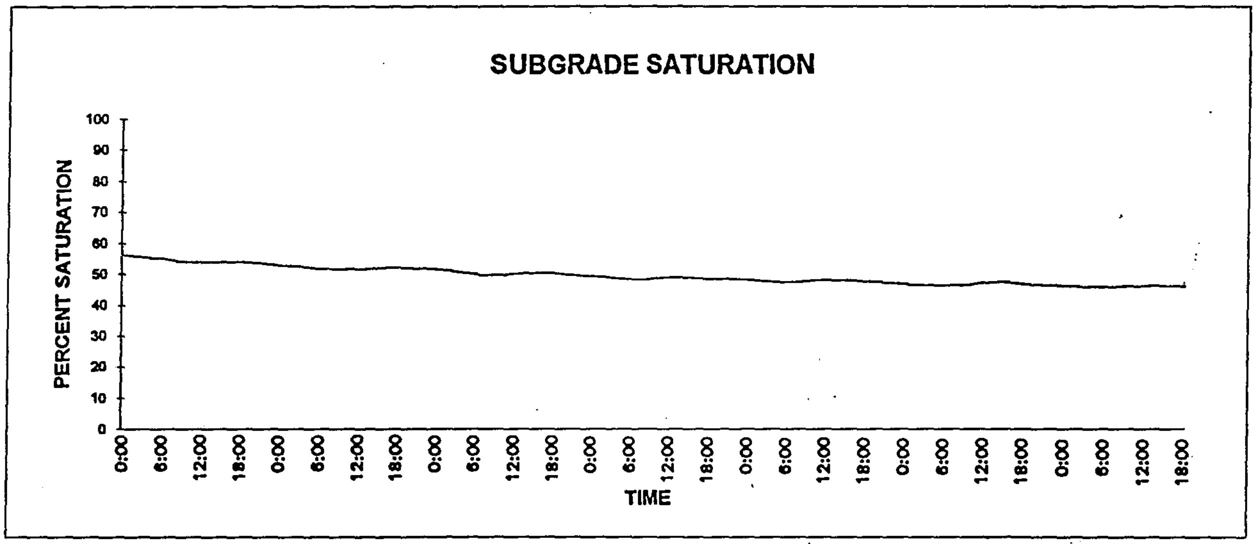

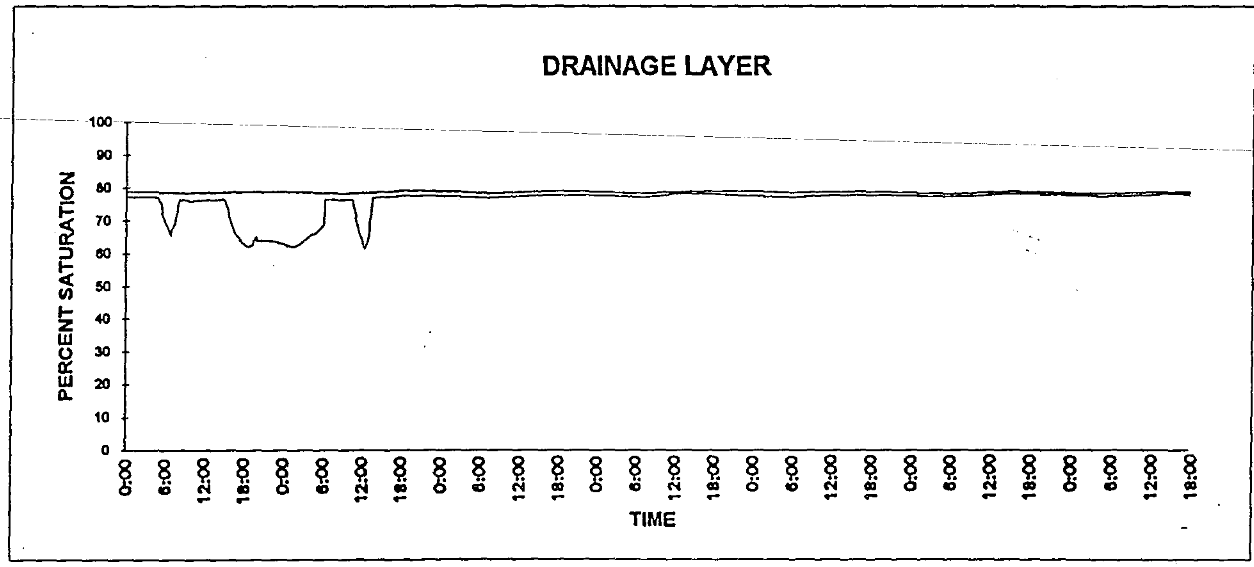

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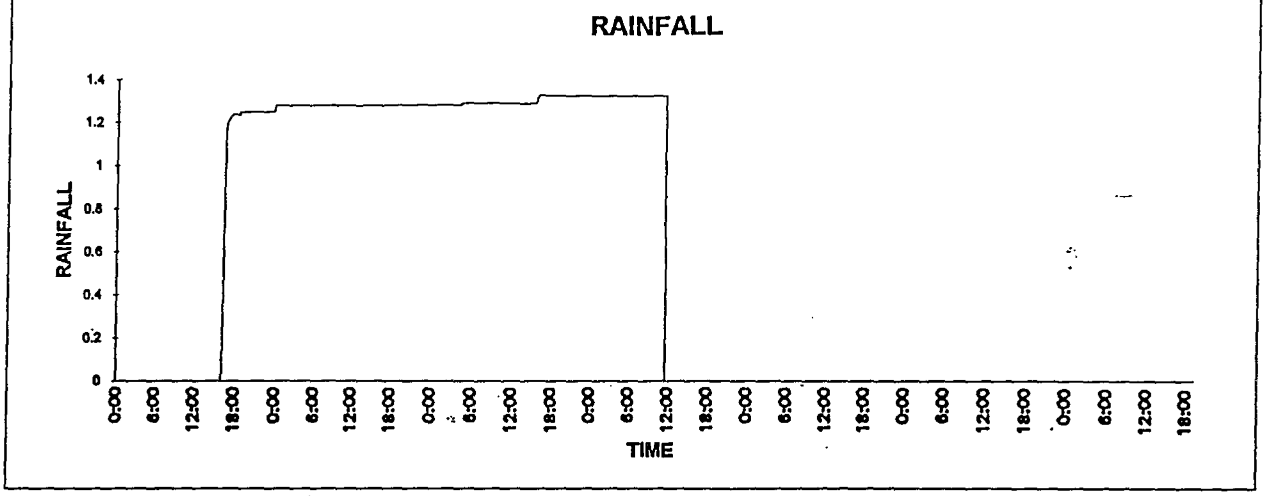
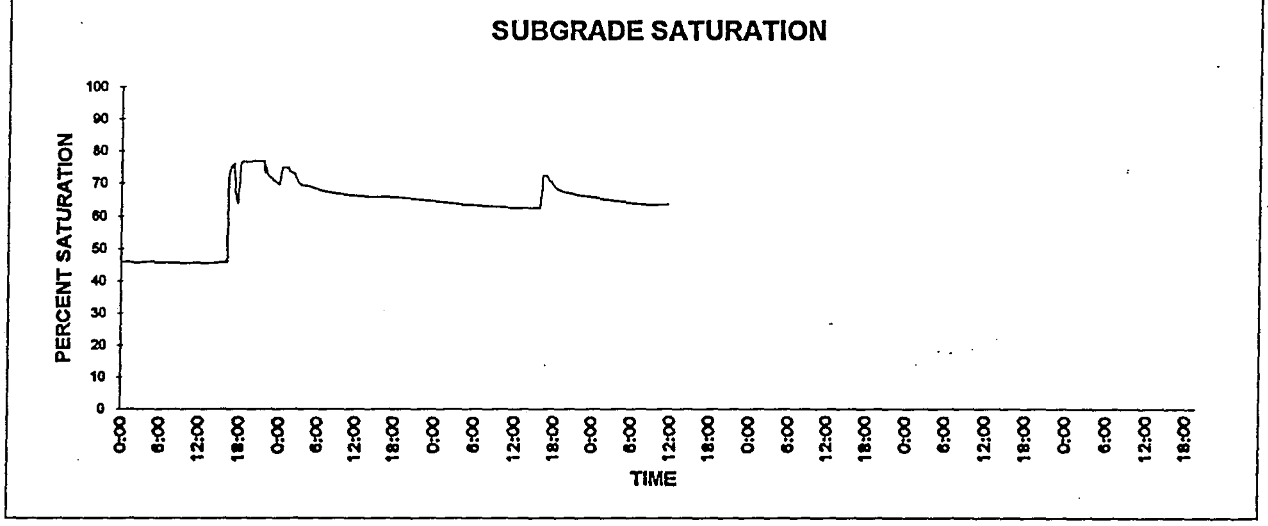
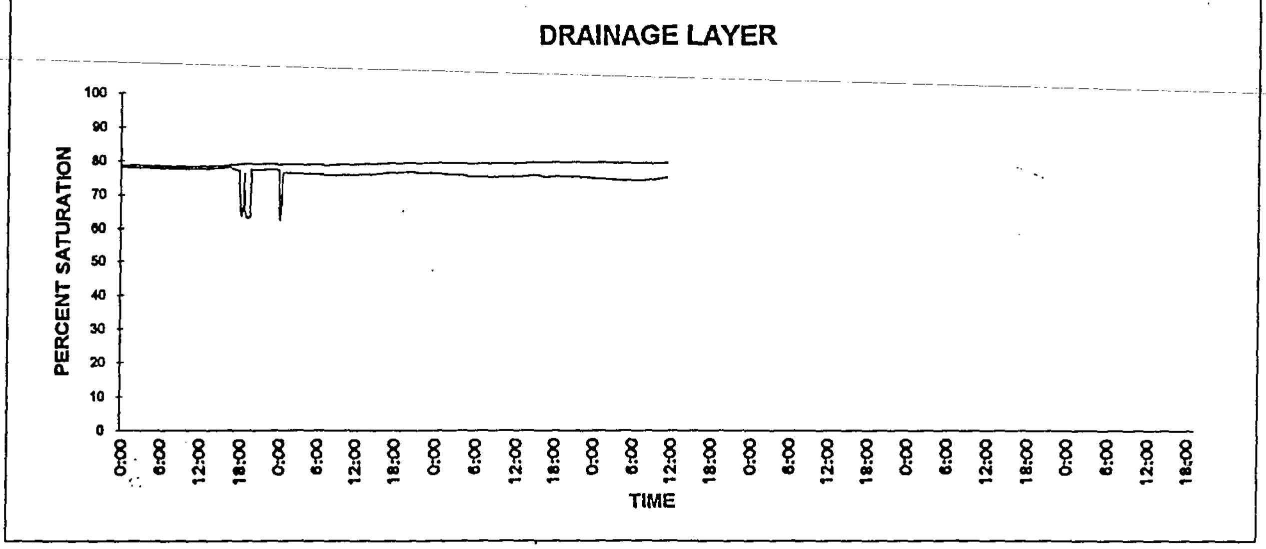
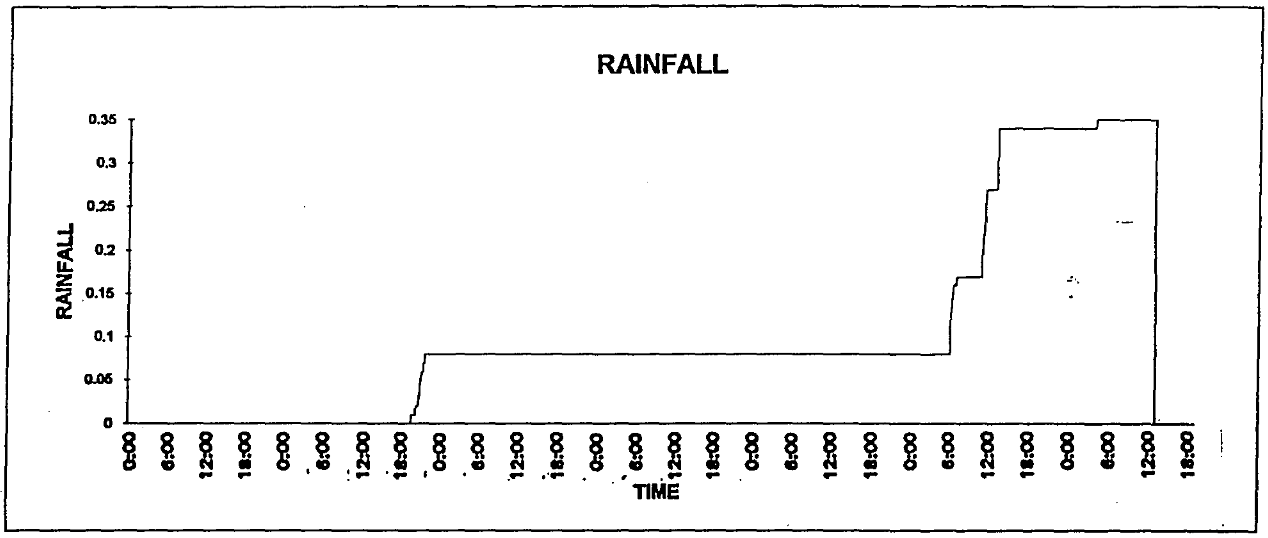
6/7/94 - 6/13/94

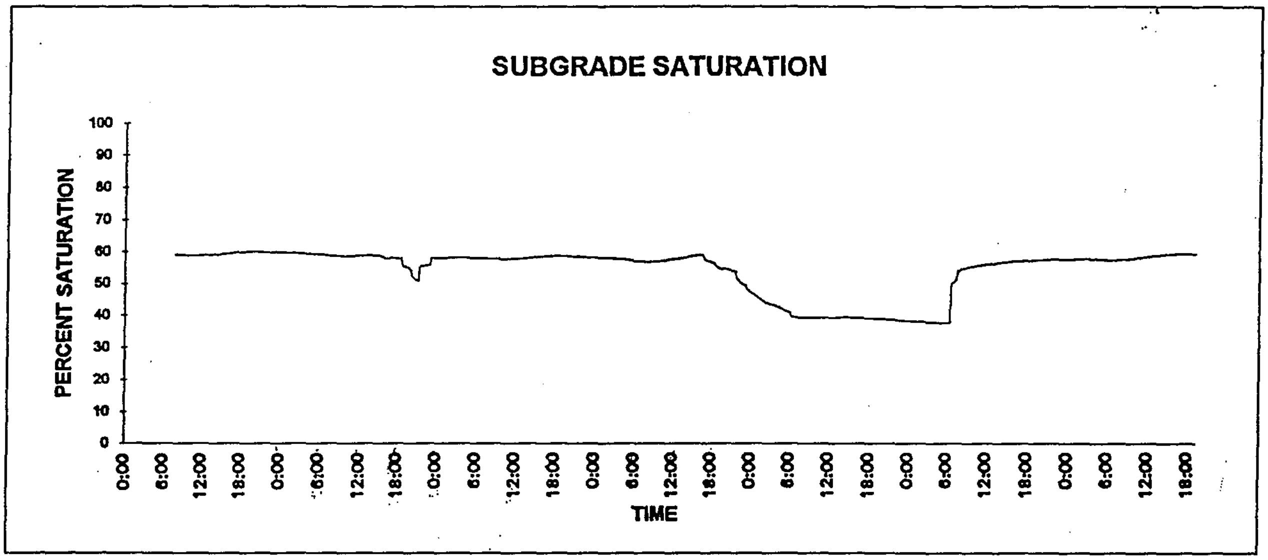


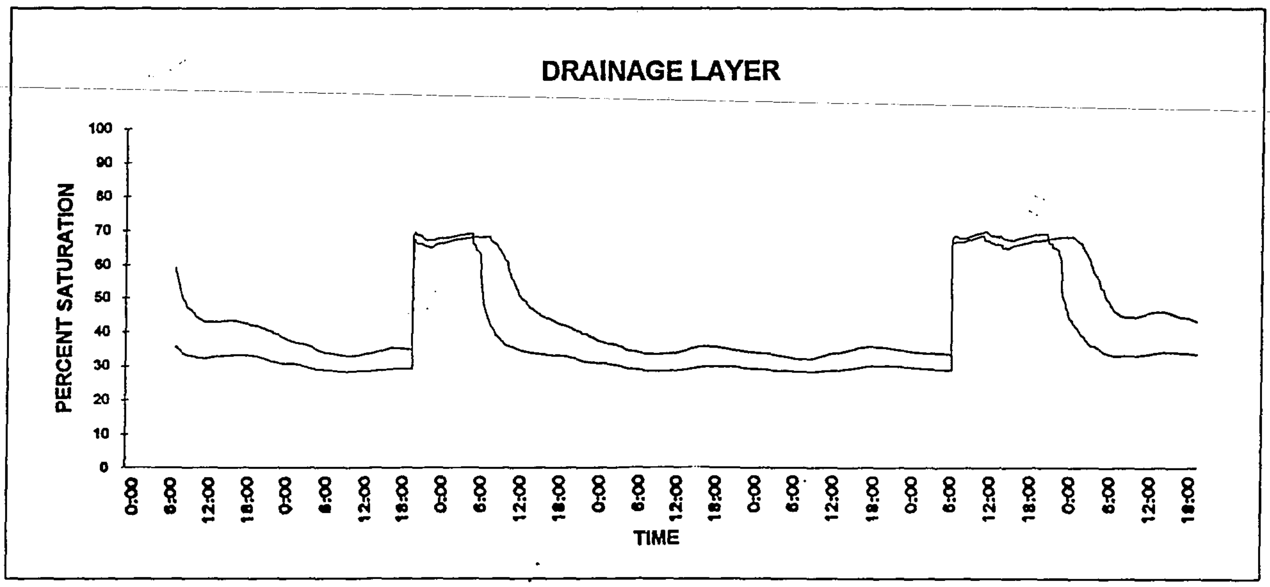




Interstate 459
Poor Drainage Location
3/29/94 - 4/4/94







Interstate 459
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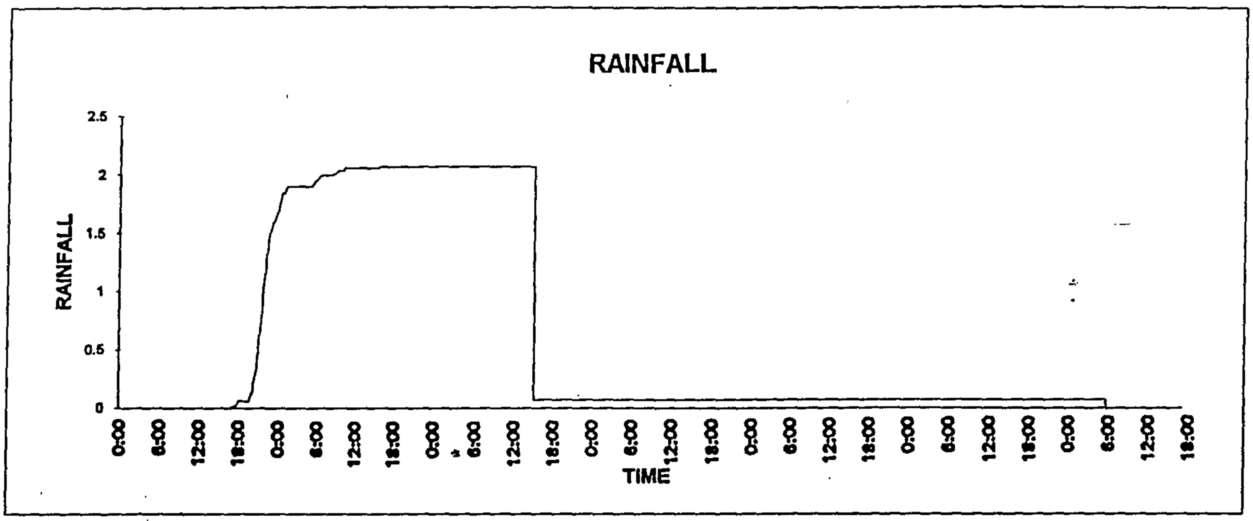
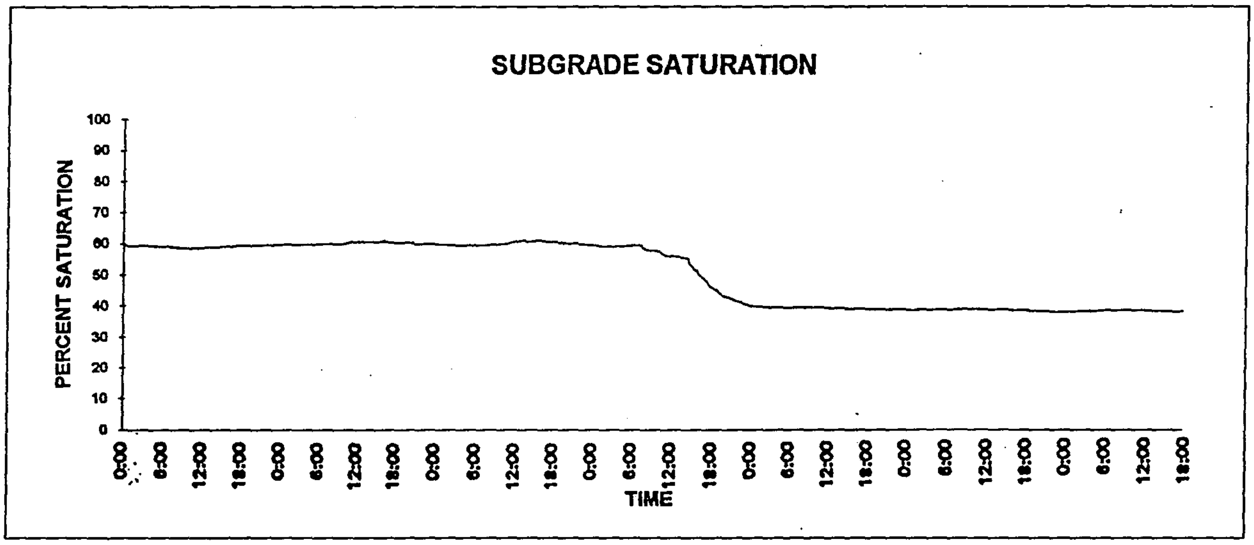
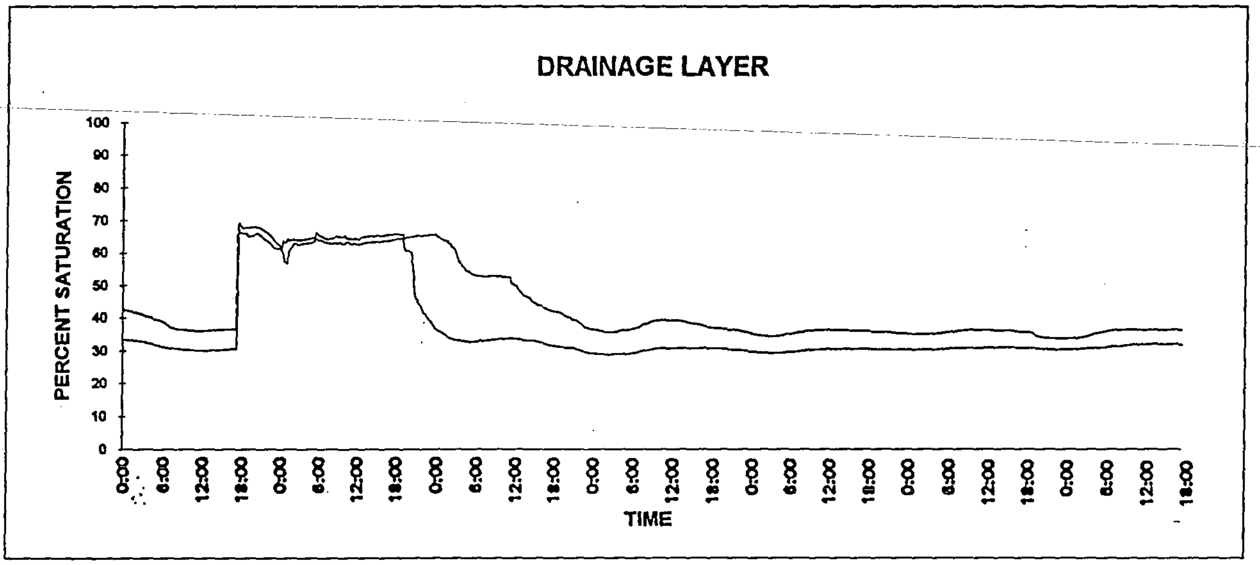
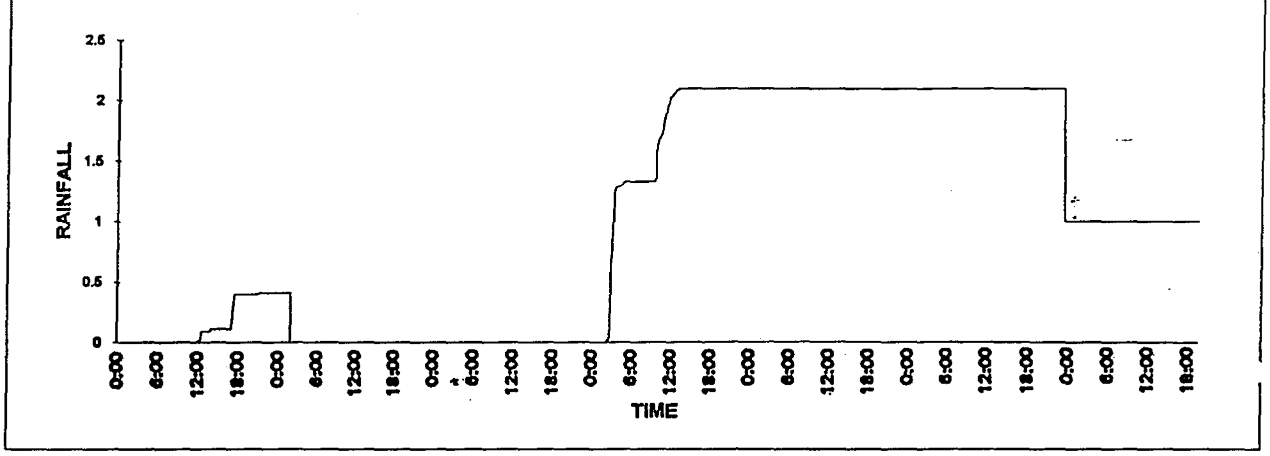
4/5/94 - 4/11/94

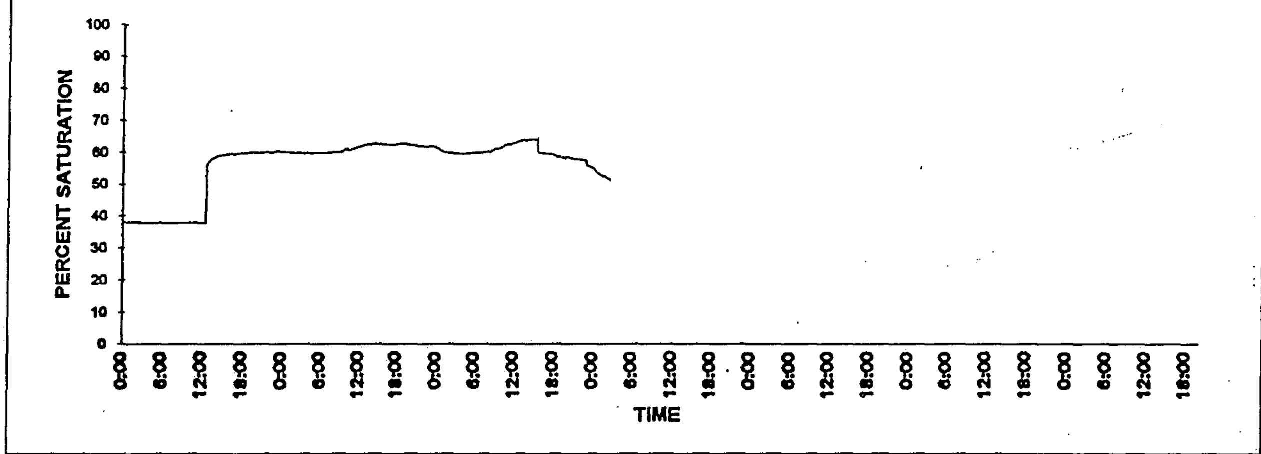


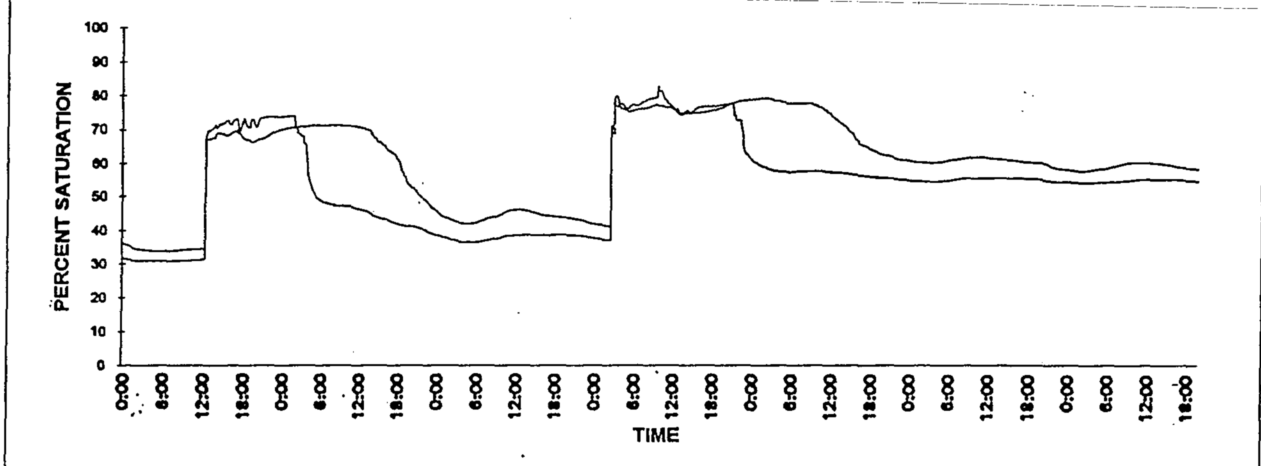




Interstate 459
Poor Drainage Location
4/12/94 - 4/18/94







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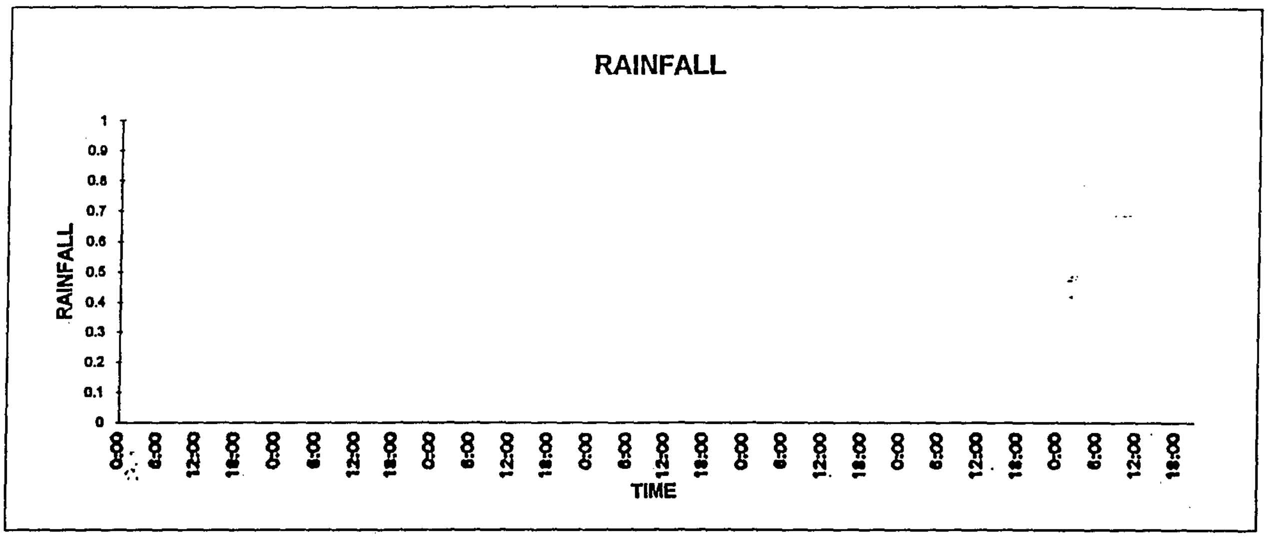
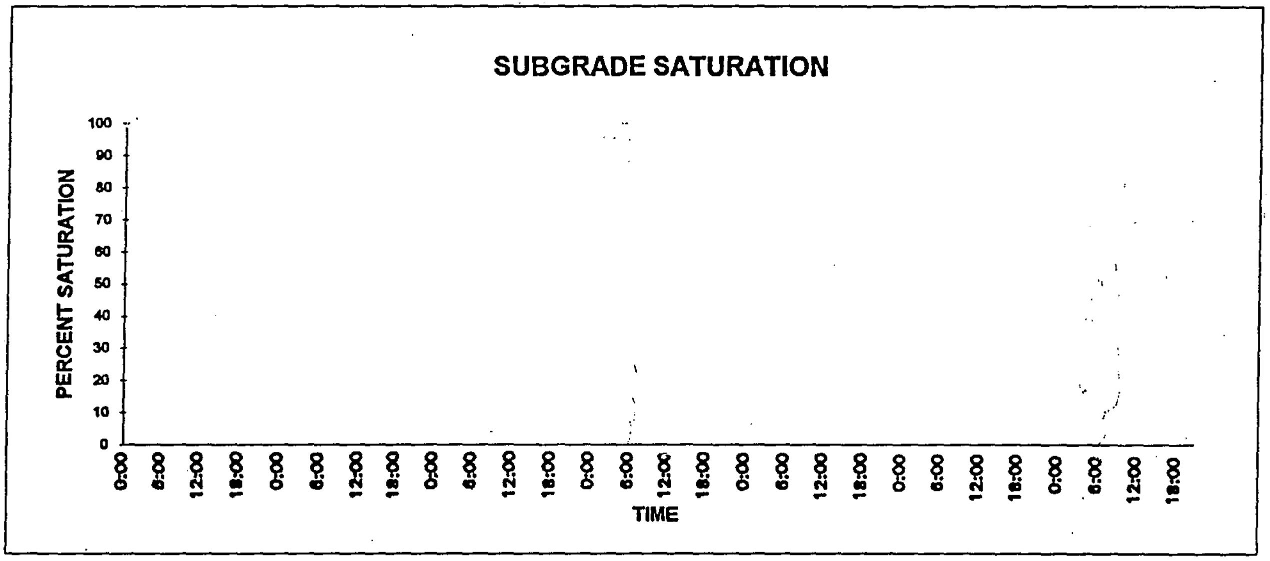
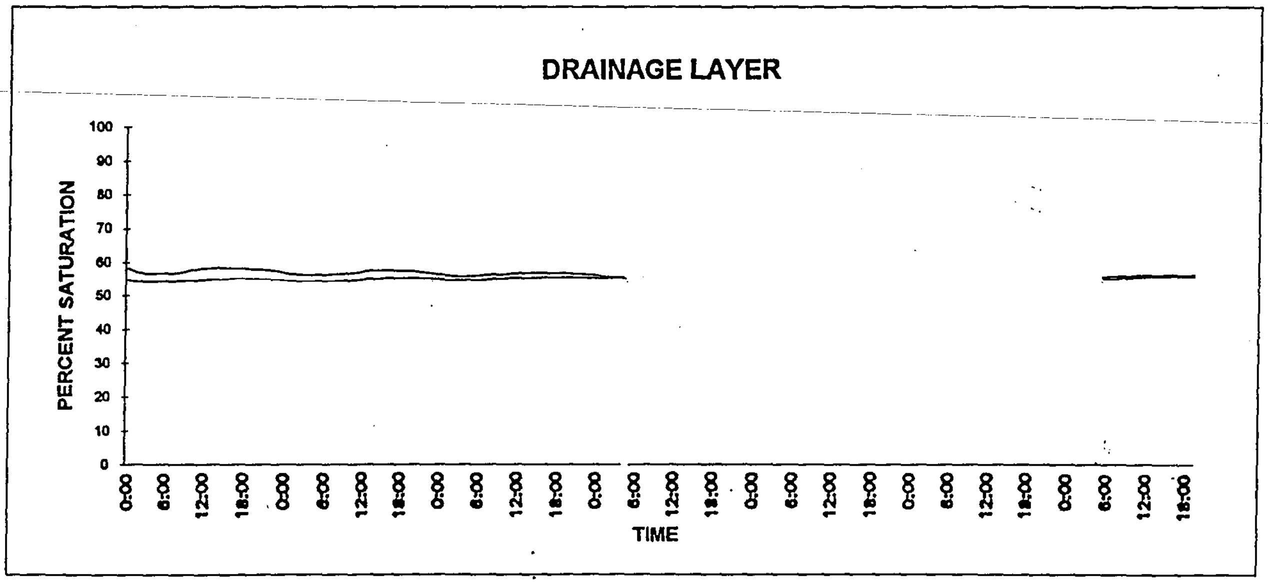
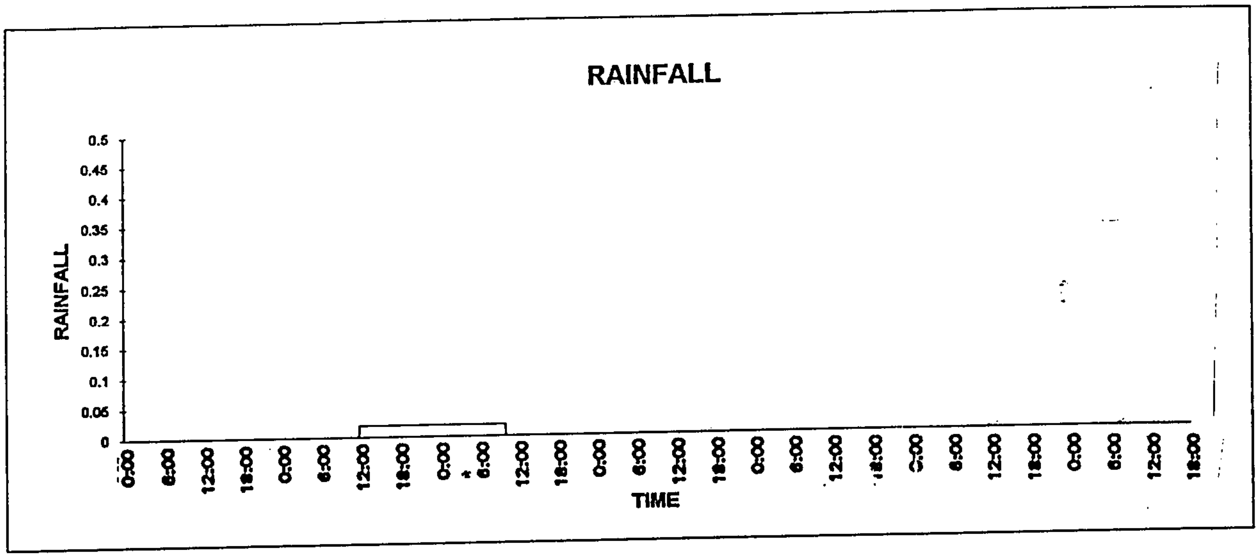
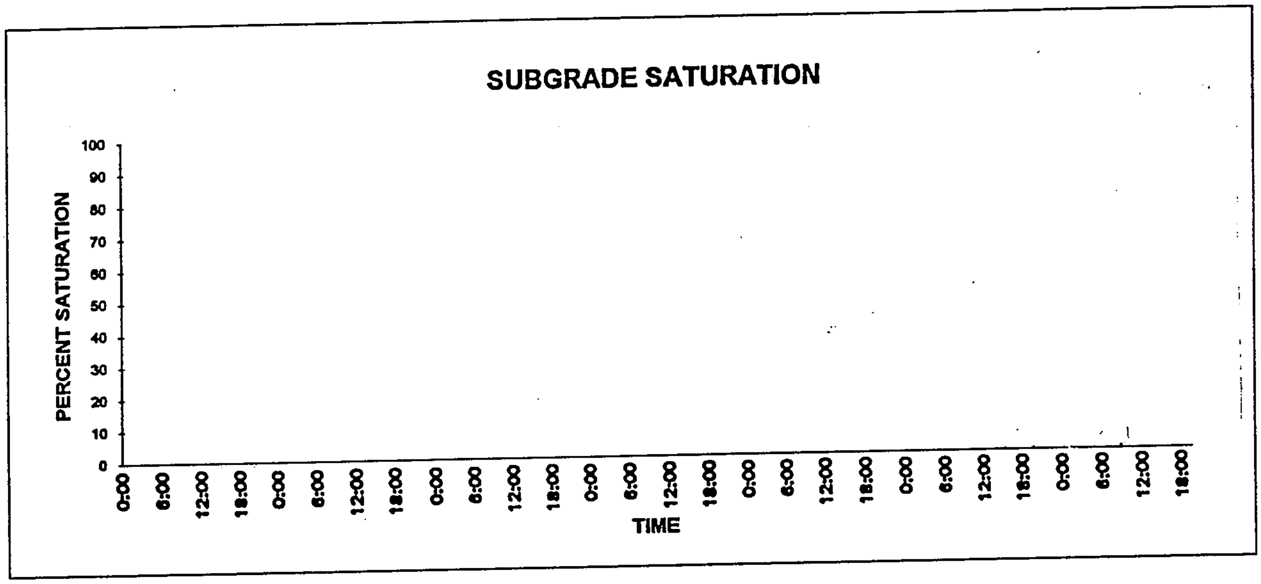


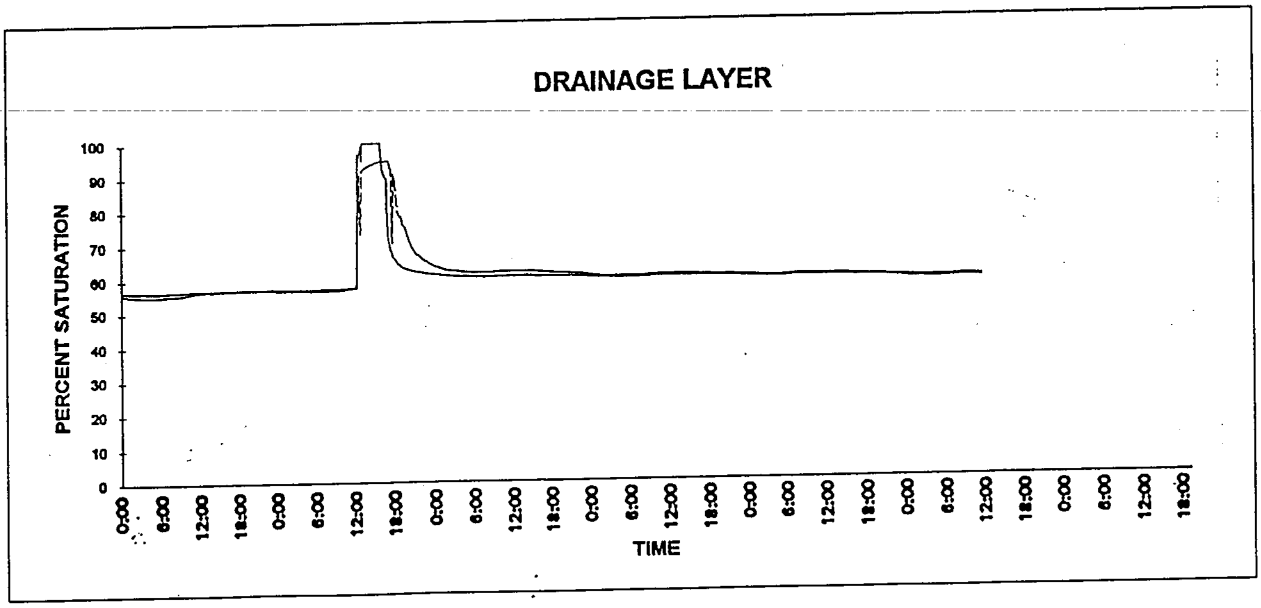




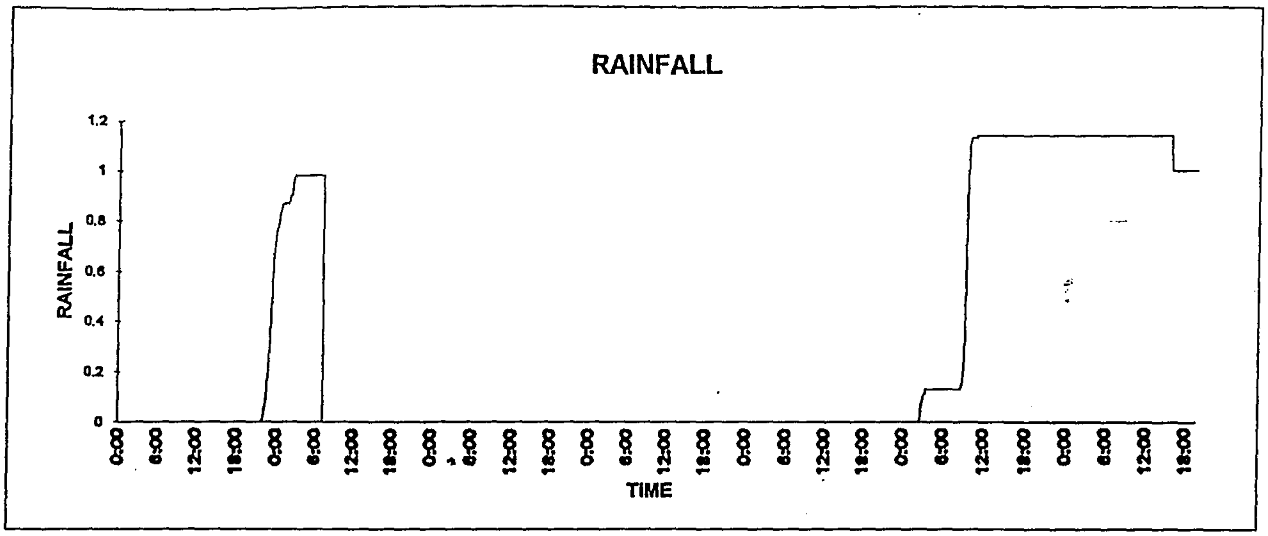
Interstate 459
Poor Drainage Location
4/26/94 - 5/2/94

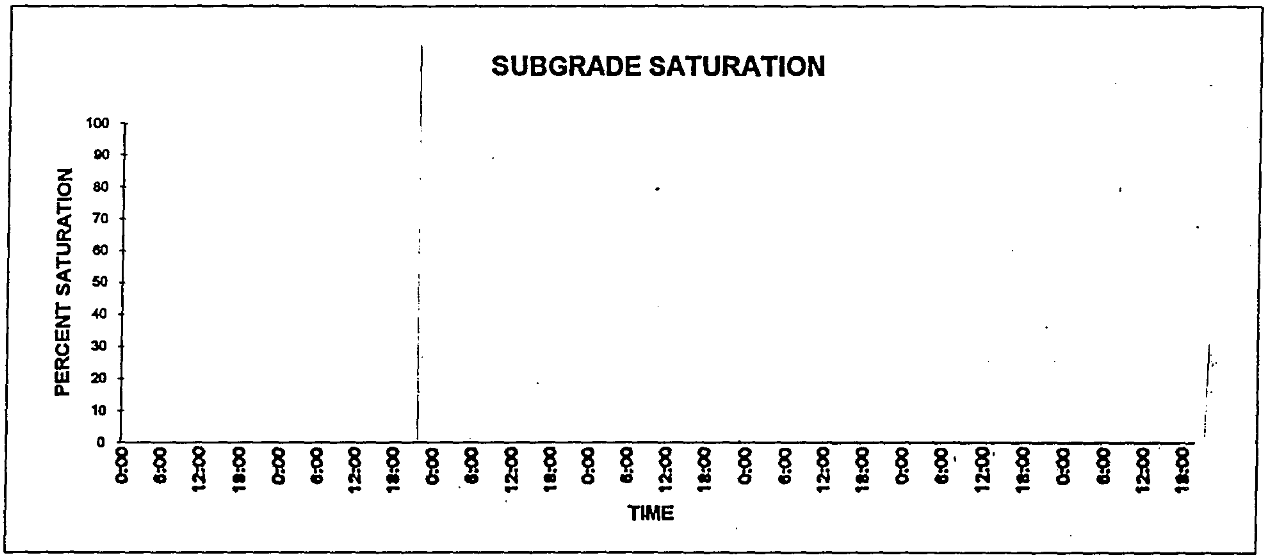


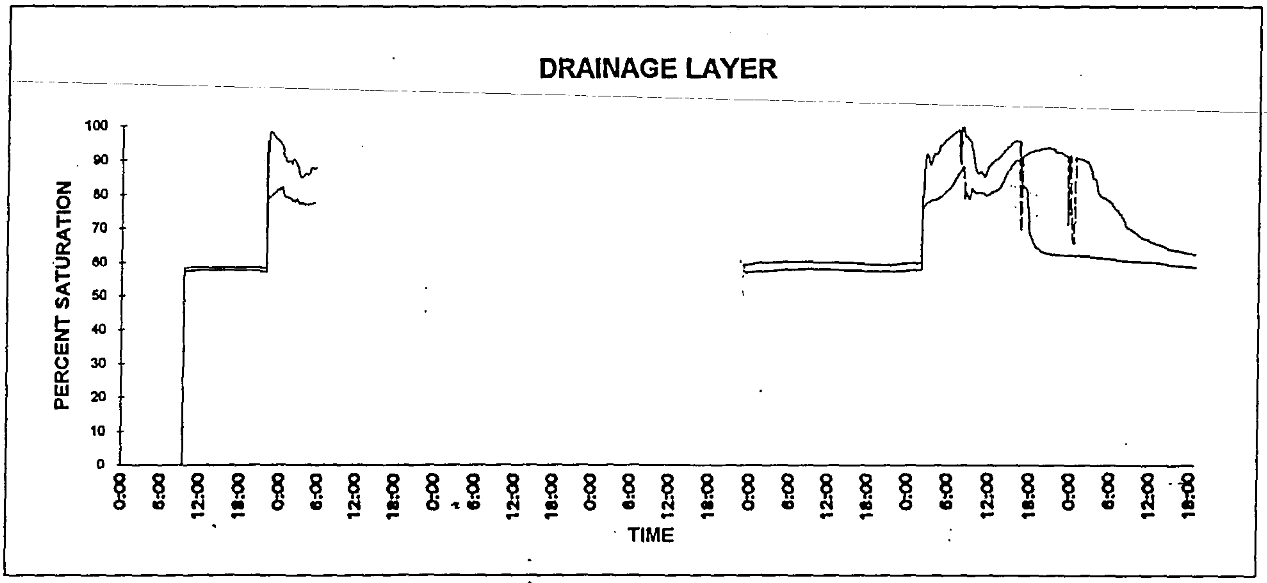




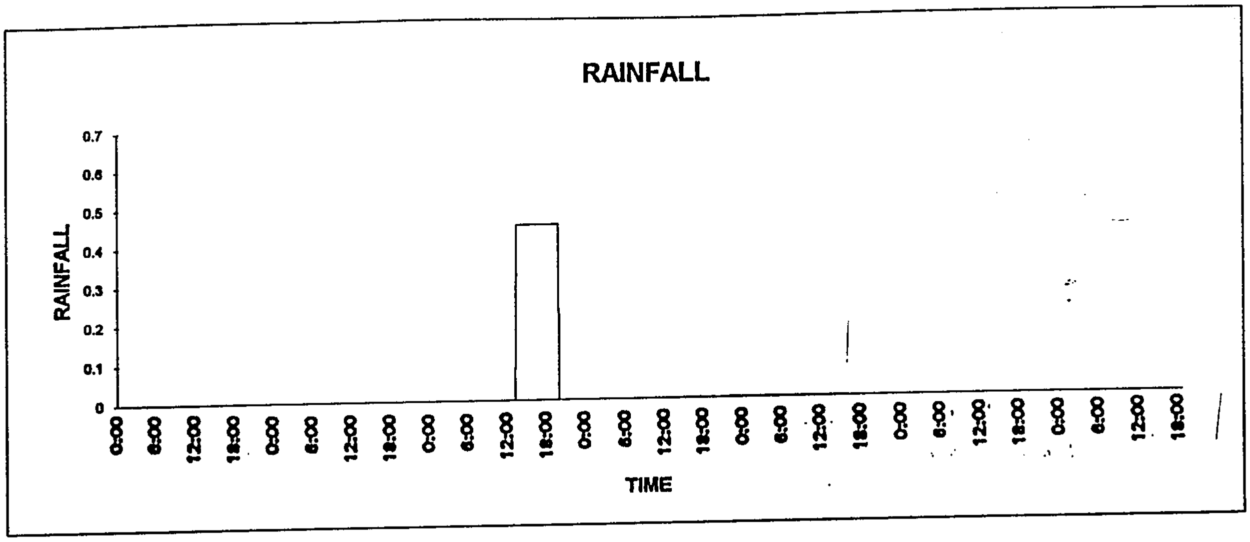
Interstate 459
Poor Drainage Location
5/3/94 - 5/9/94

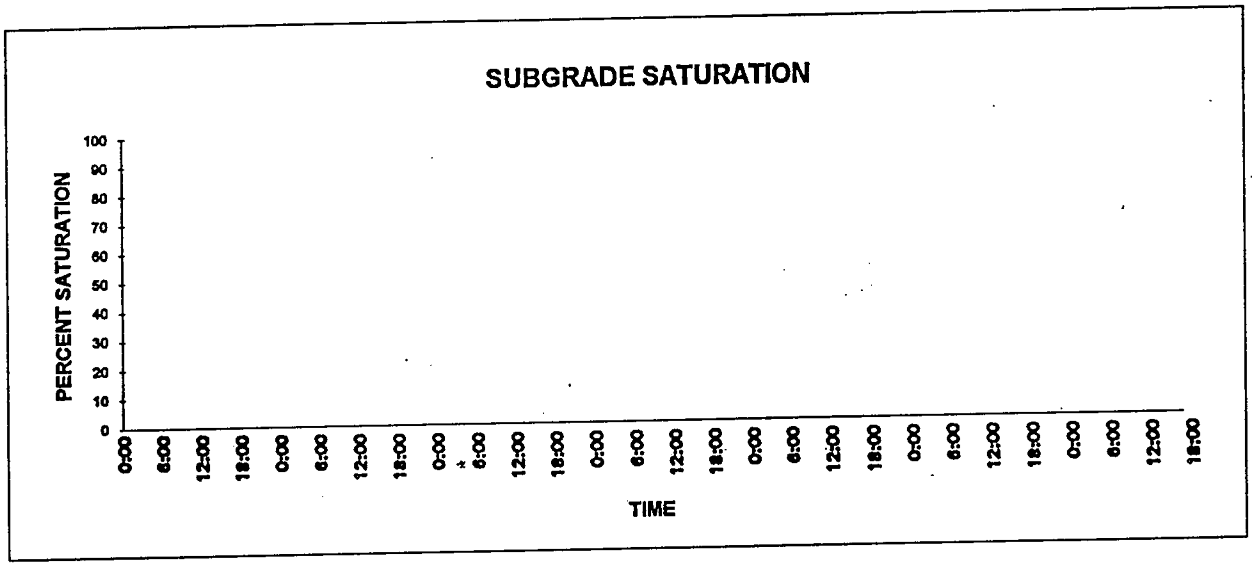


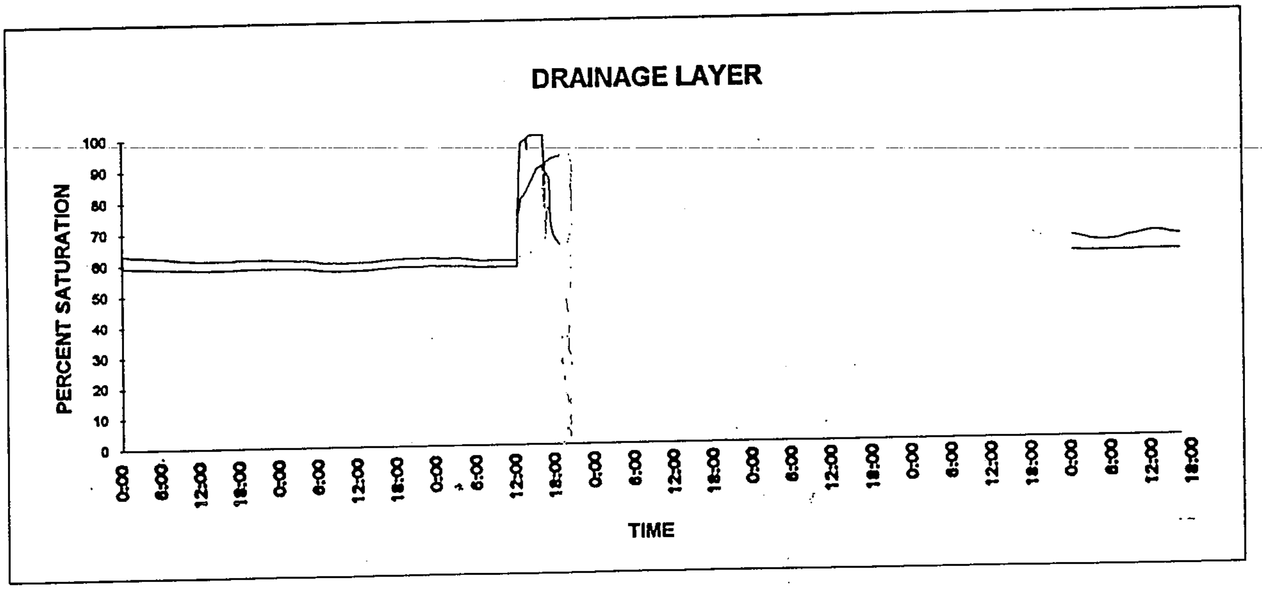




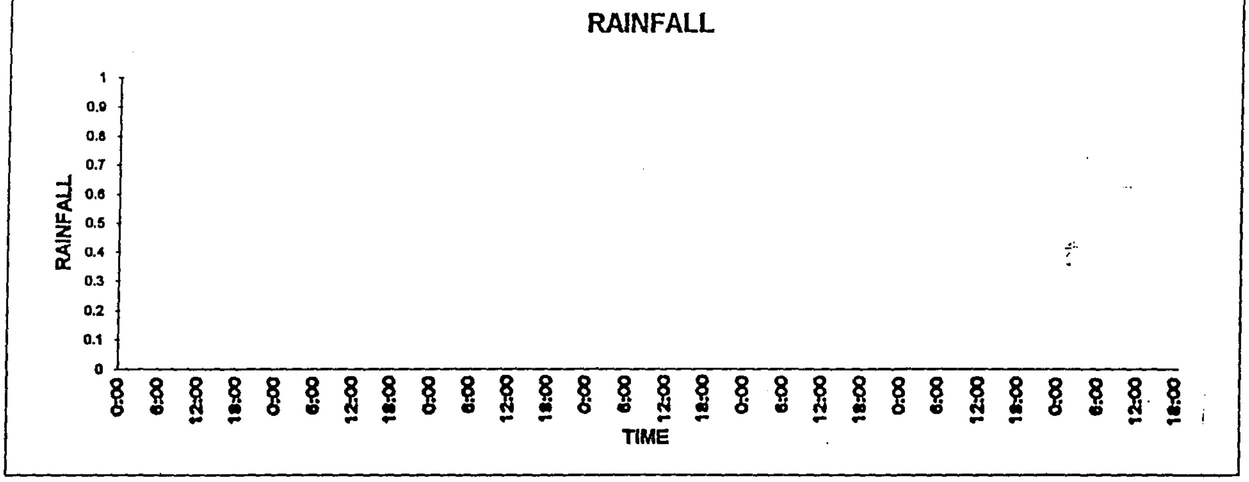
Interstate 459
Poor Drainage Location
5/10/94 - 5/16/94

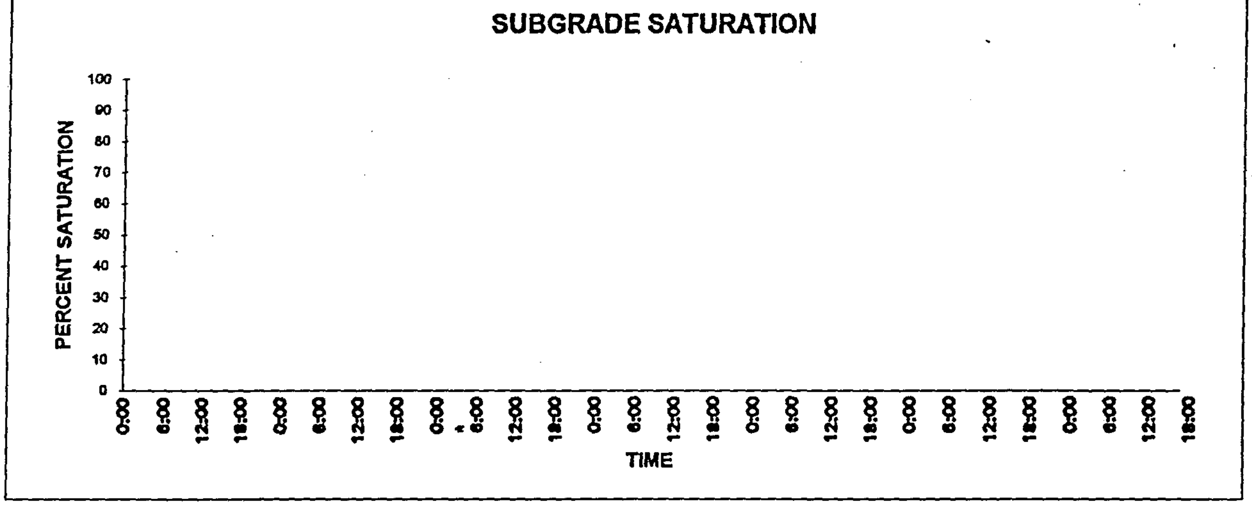


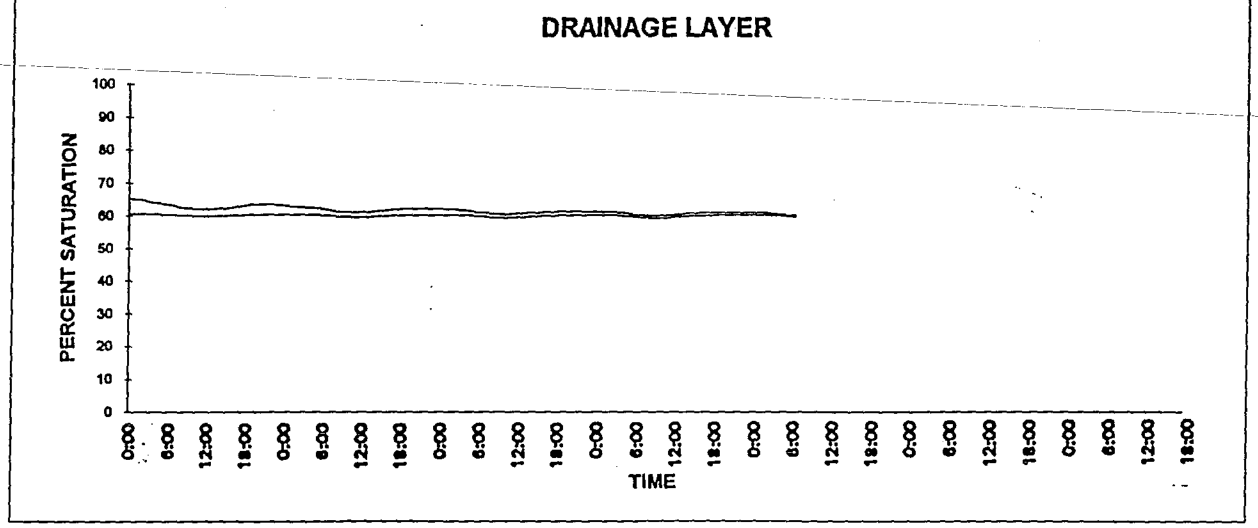




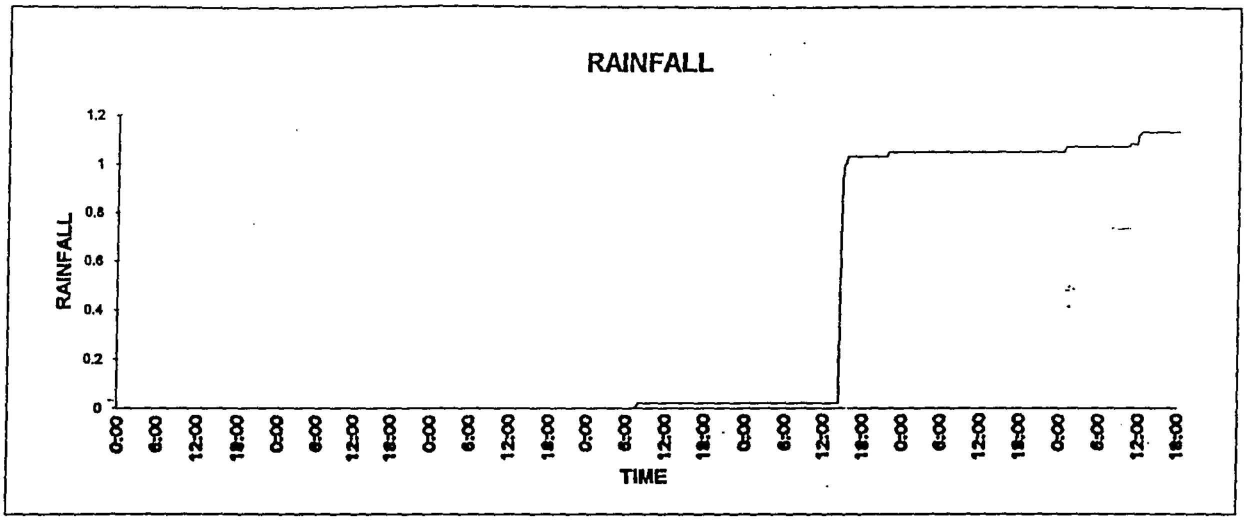
Interstate 459
Poor Drainage Location
5/17/94 - 5/23/94

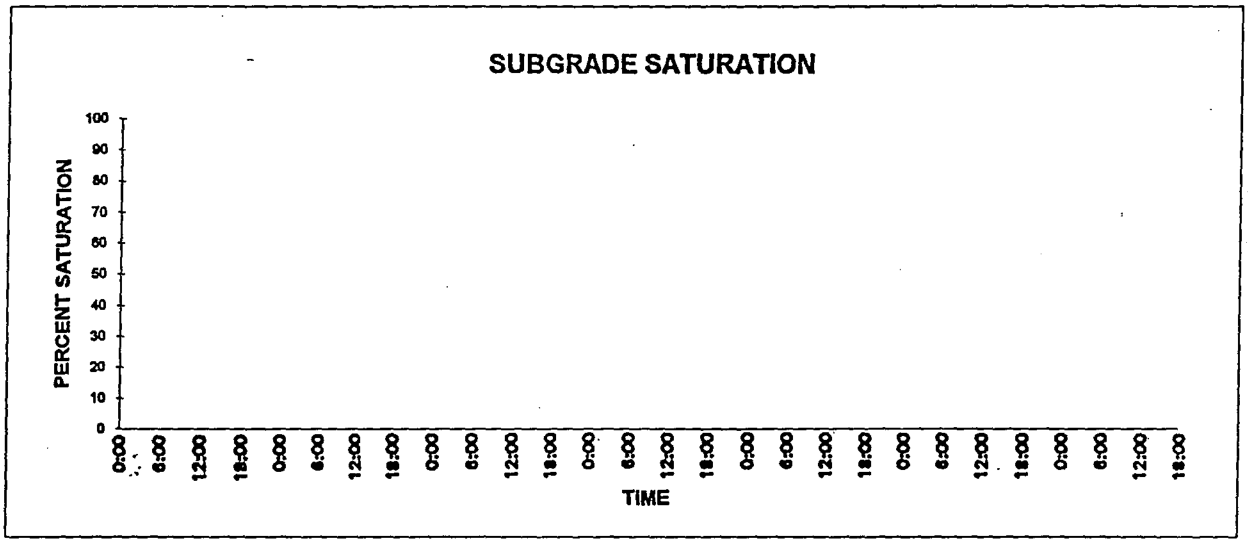


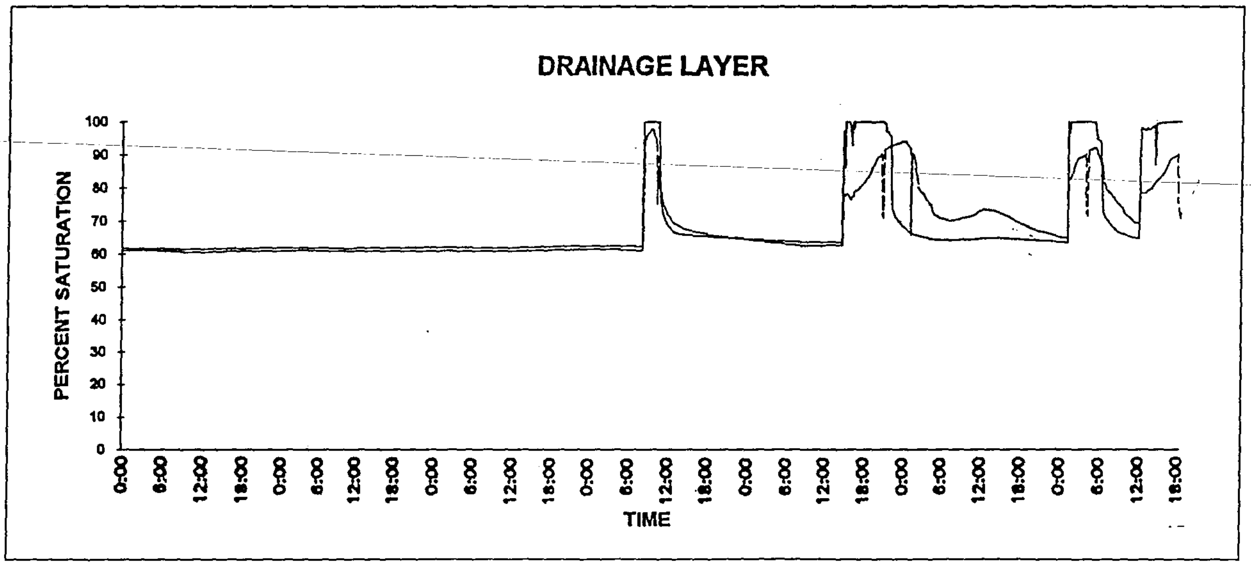




Interstate 459
Poor Drainage Location
6/3/94 - 6/9/94





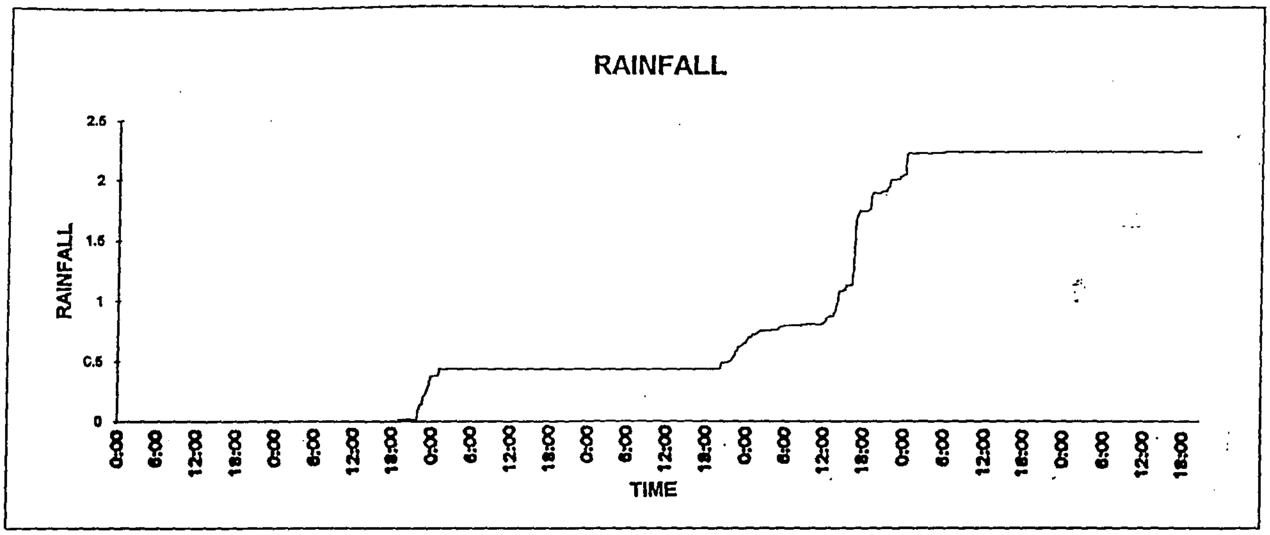


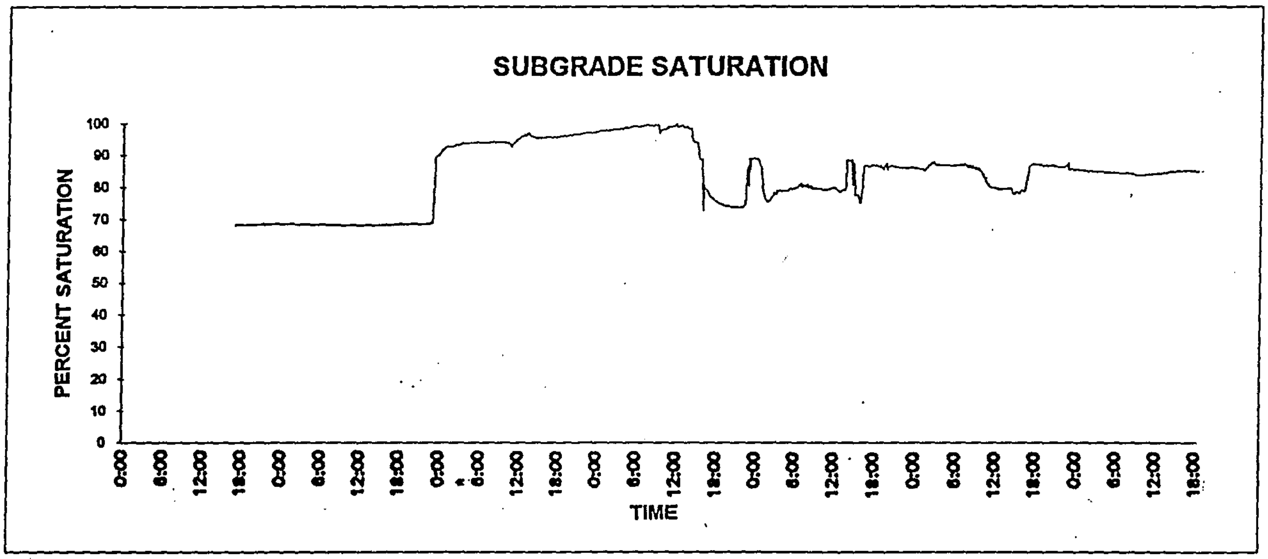
Highway 79

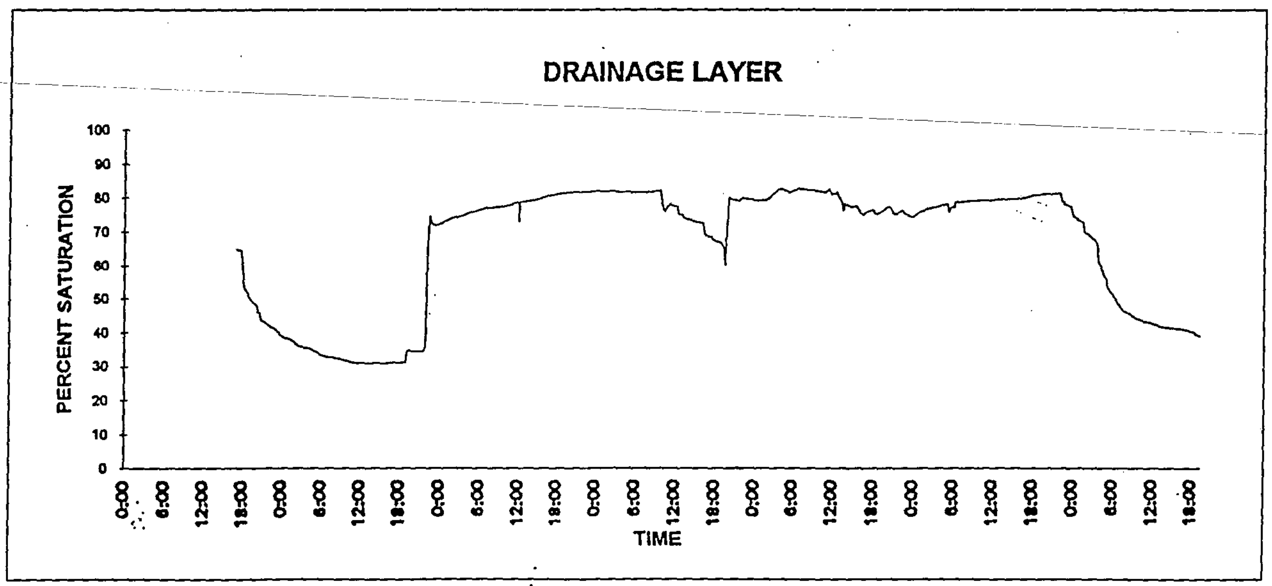
Good Drainage Location

Field Moisture Data

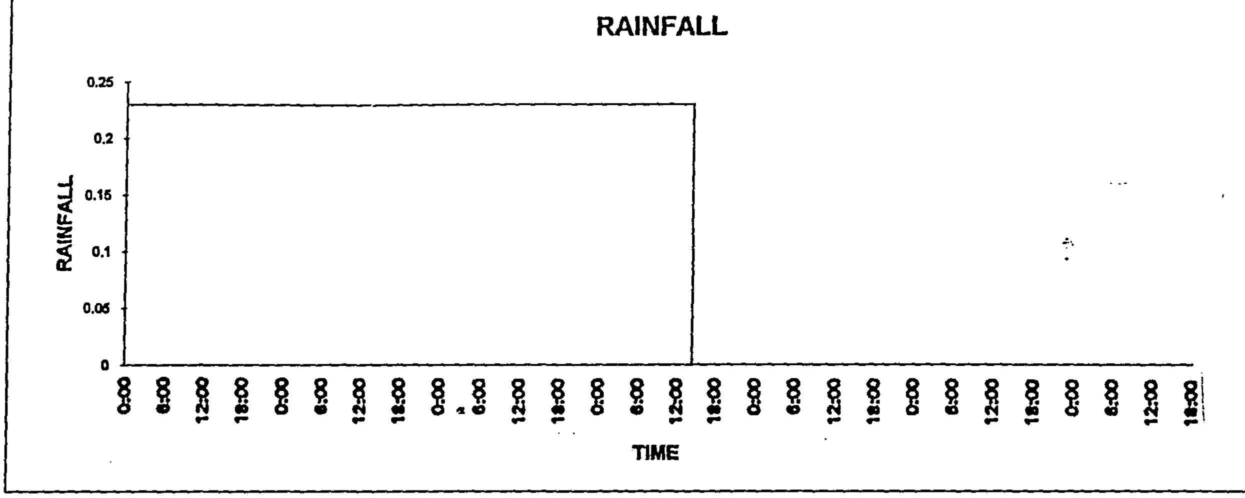
Highway 79
Good Drainage Location
2/6/94 - 2/12/94

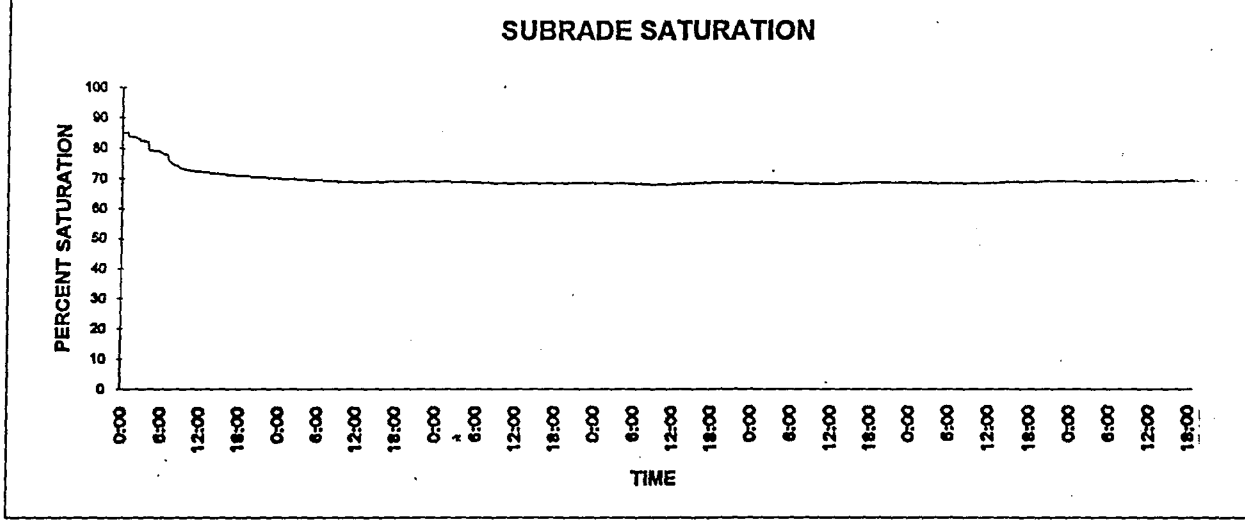


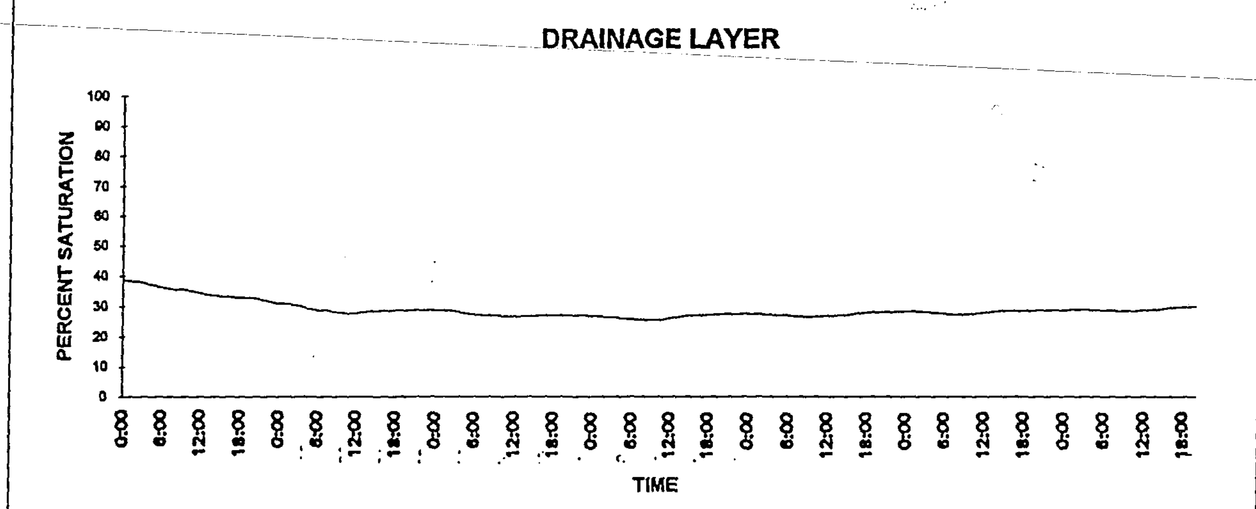




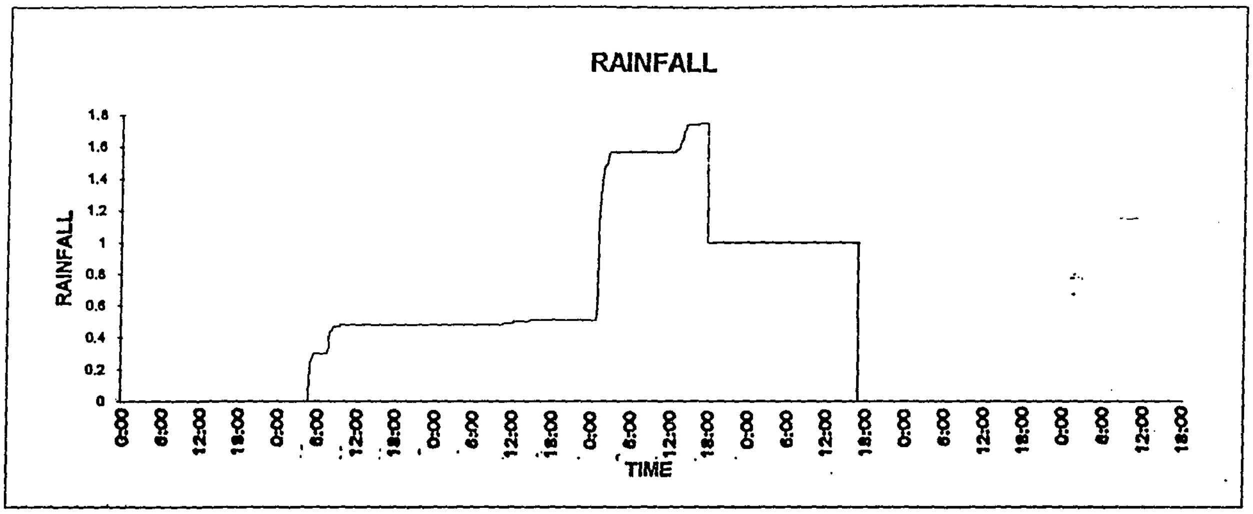
Highway 79
Good Drainage Location
2/13/94 - 2/19/94

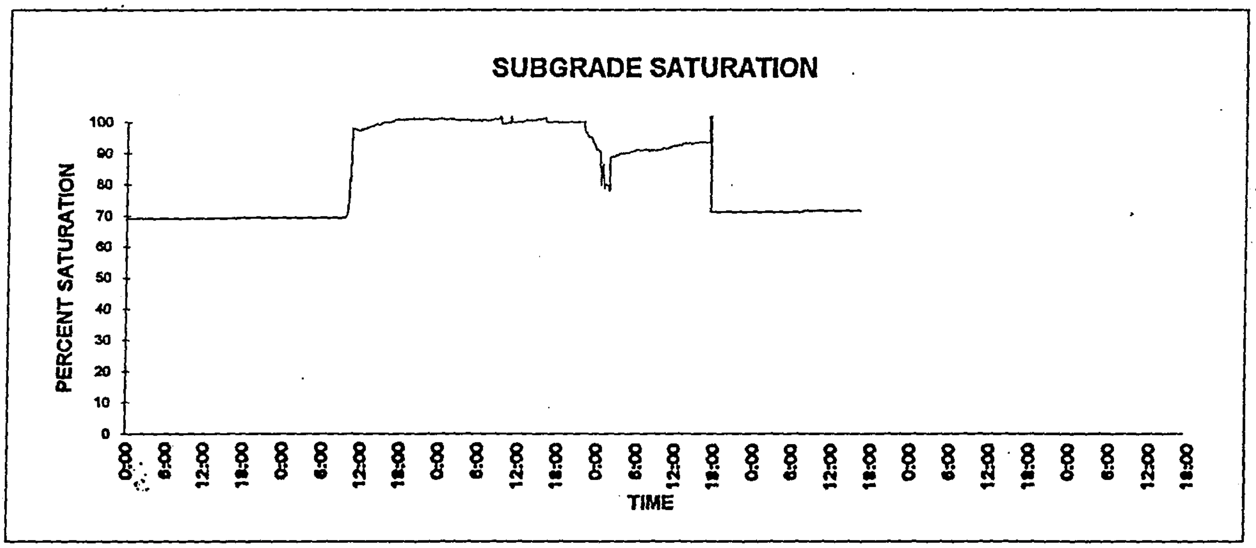


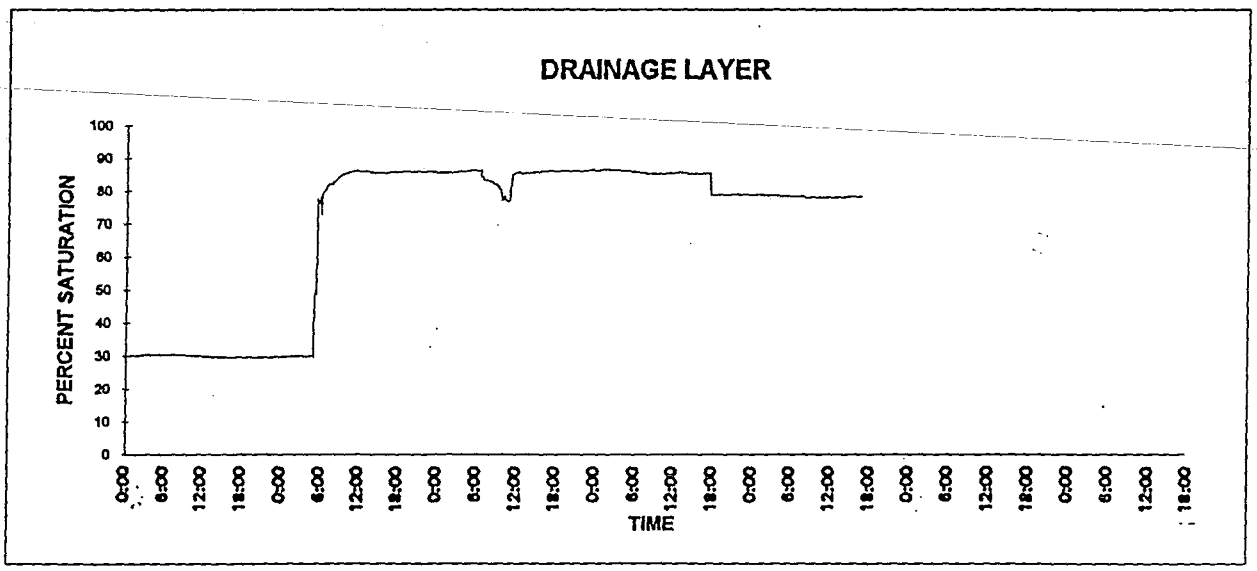




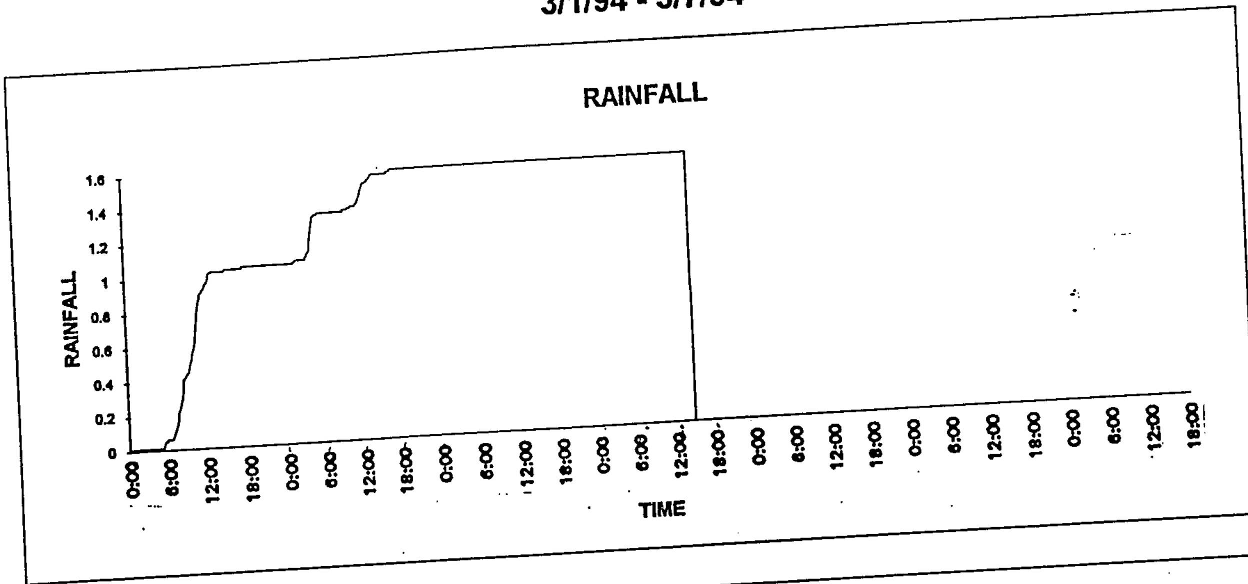
Highway 79
Good Drainage Location
2/20/94 - 2/26/94

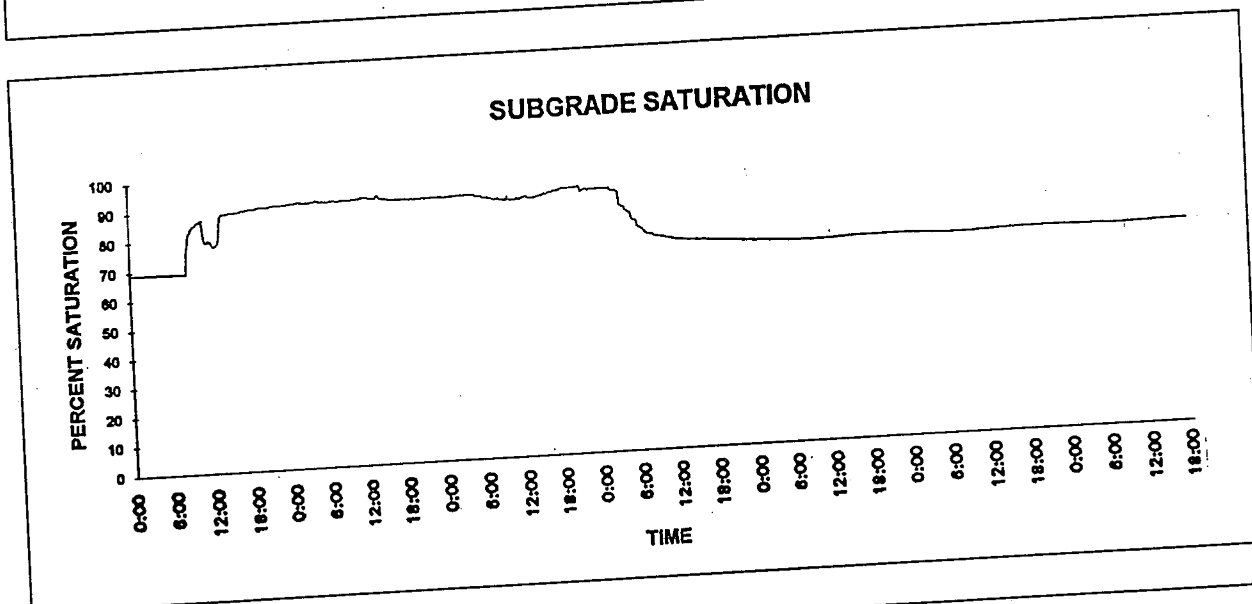


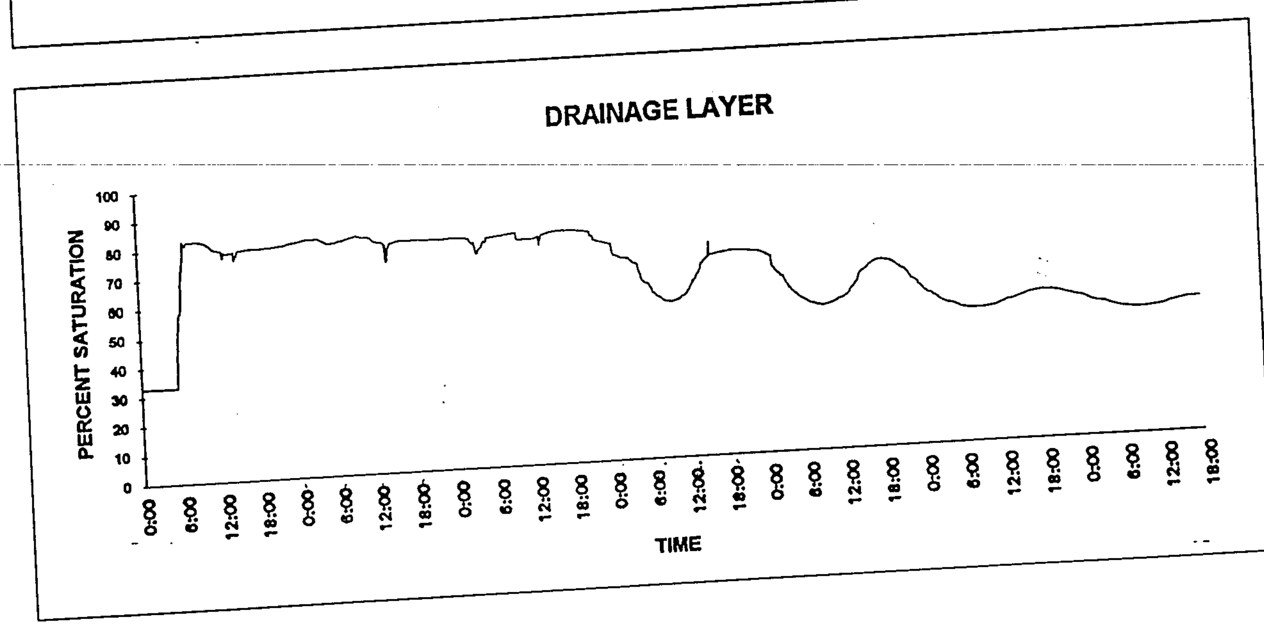




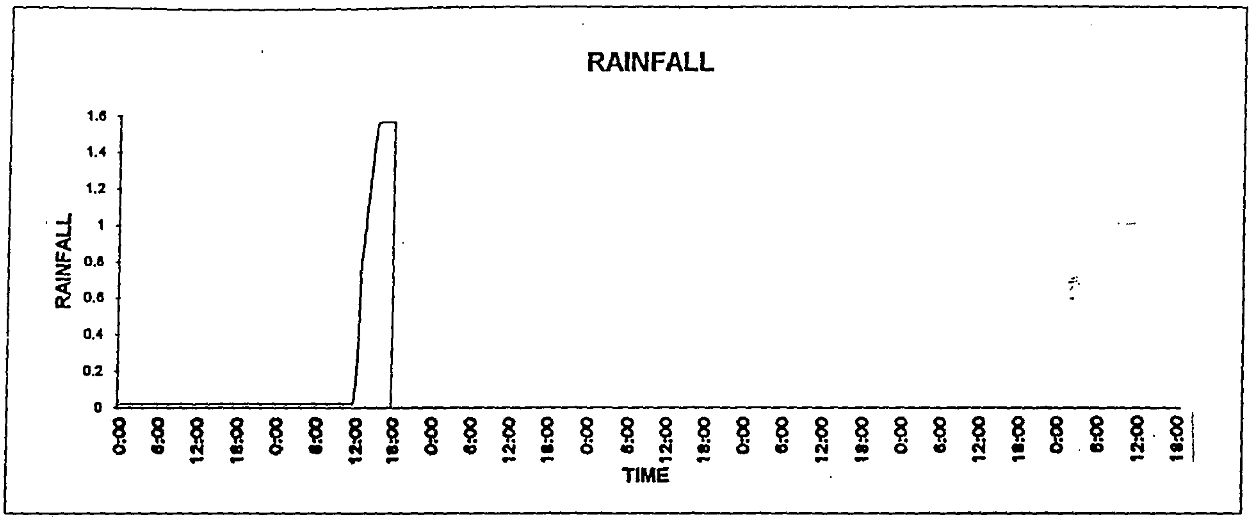
Highway 79
Good Drainage Location
3/1/94 - 3/7/94

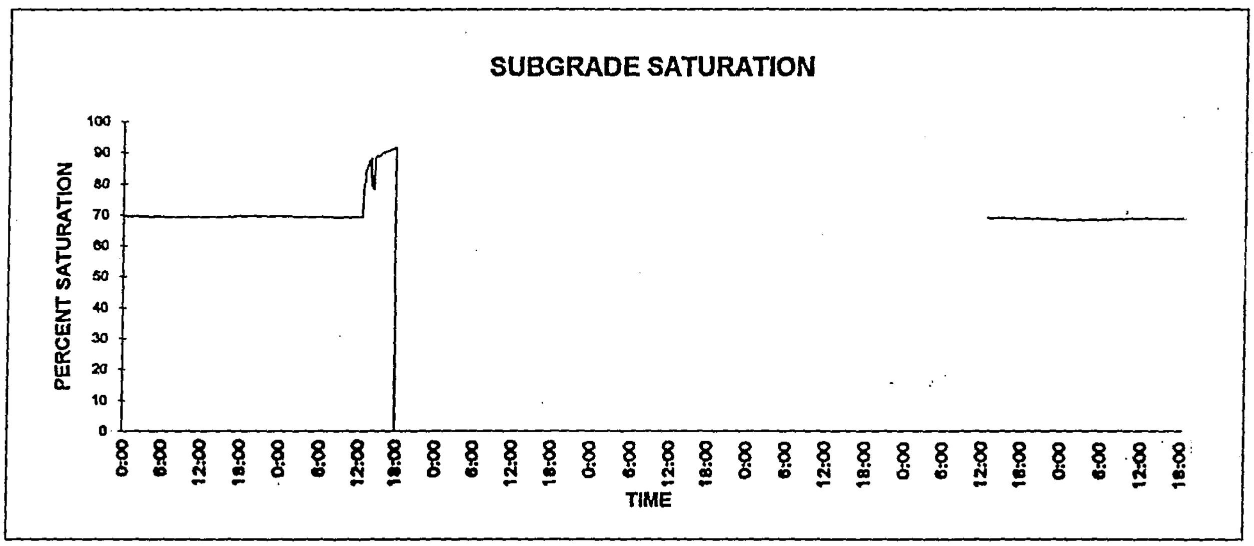


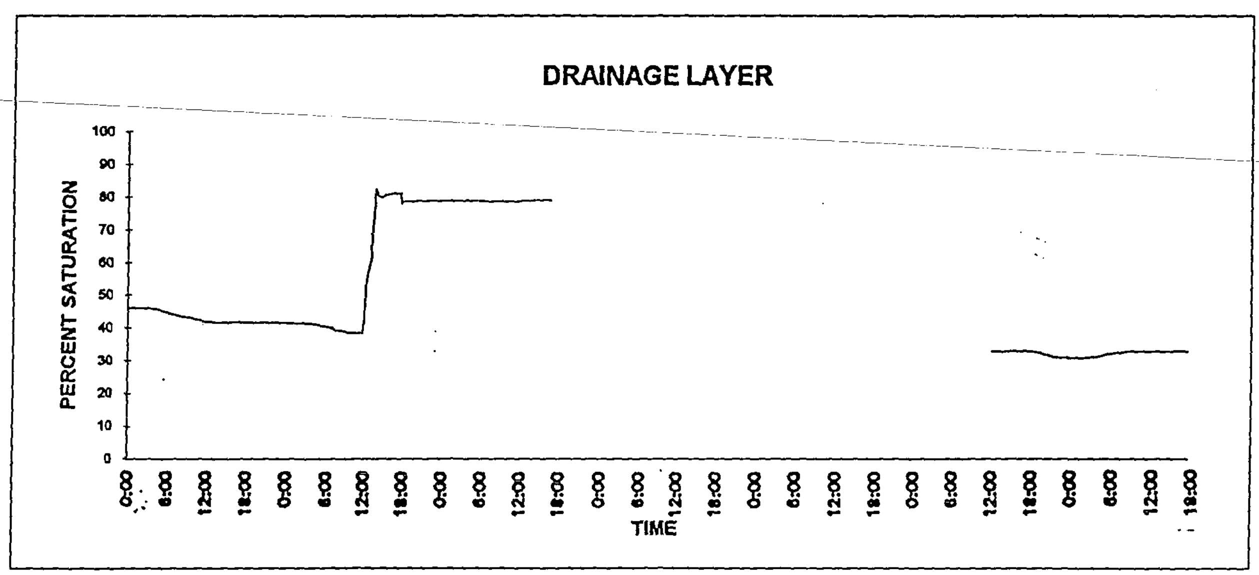




Highway 79
Good Drainage Location
3/8/94 - 3/14/94

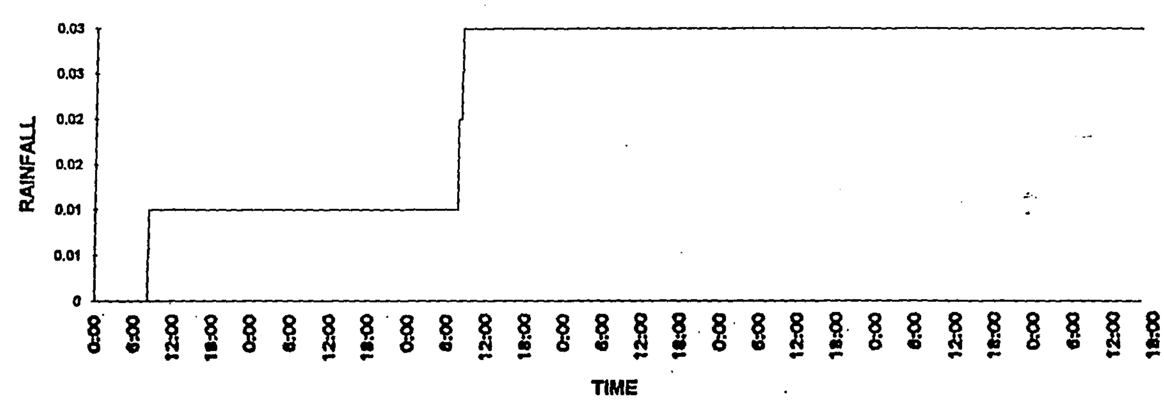




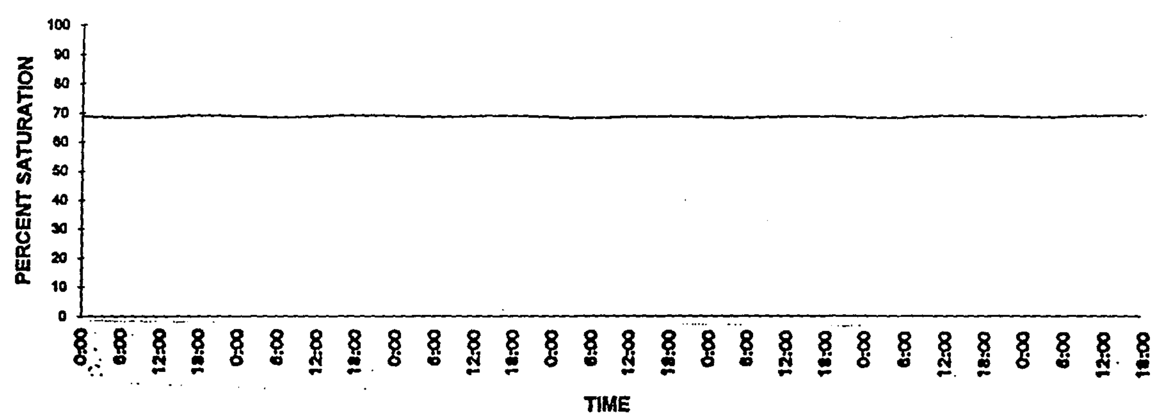


Highway 79
Good Drainage Location
3/15/94 - 3/21/94

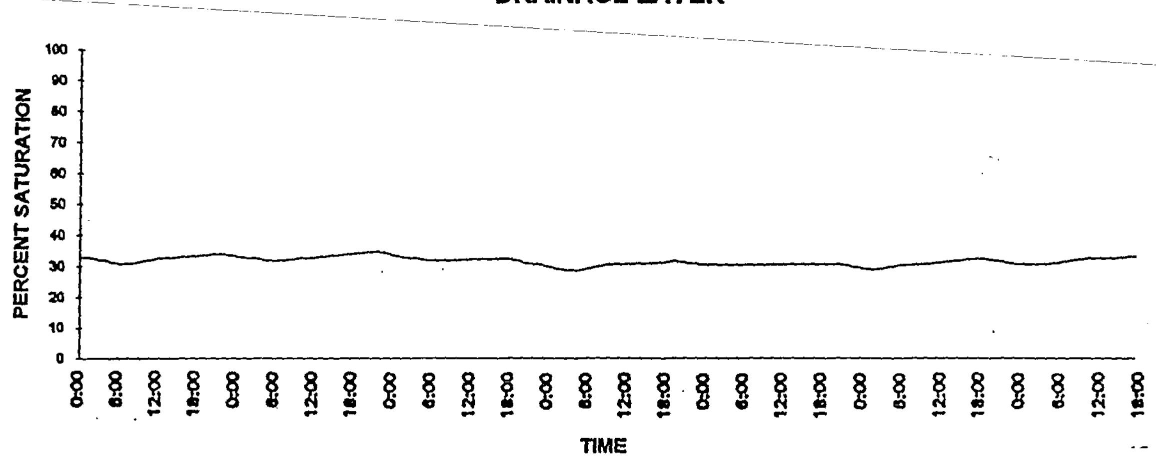
RAINFALL



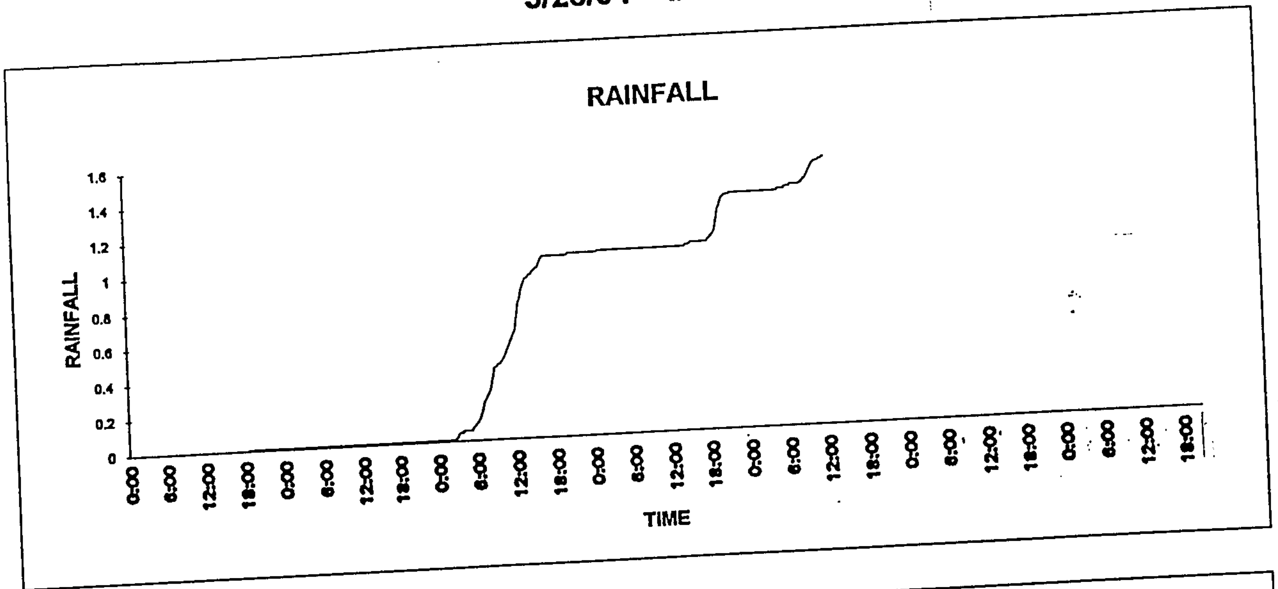
SUBGRADE SATURATION

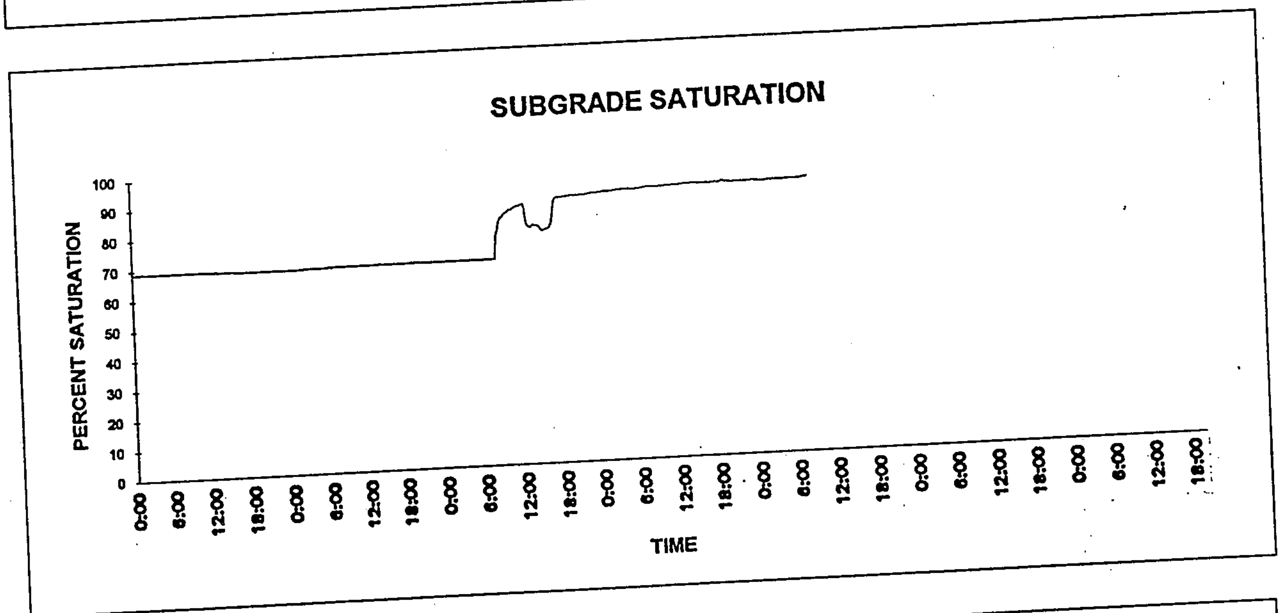


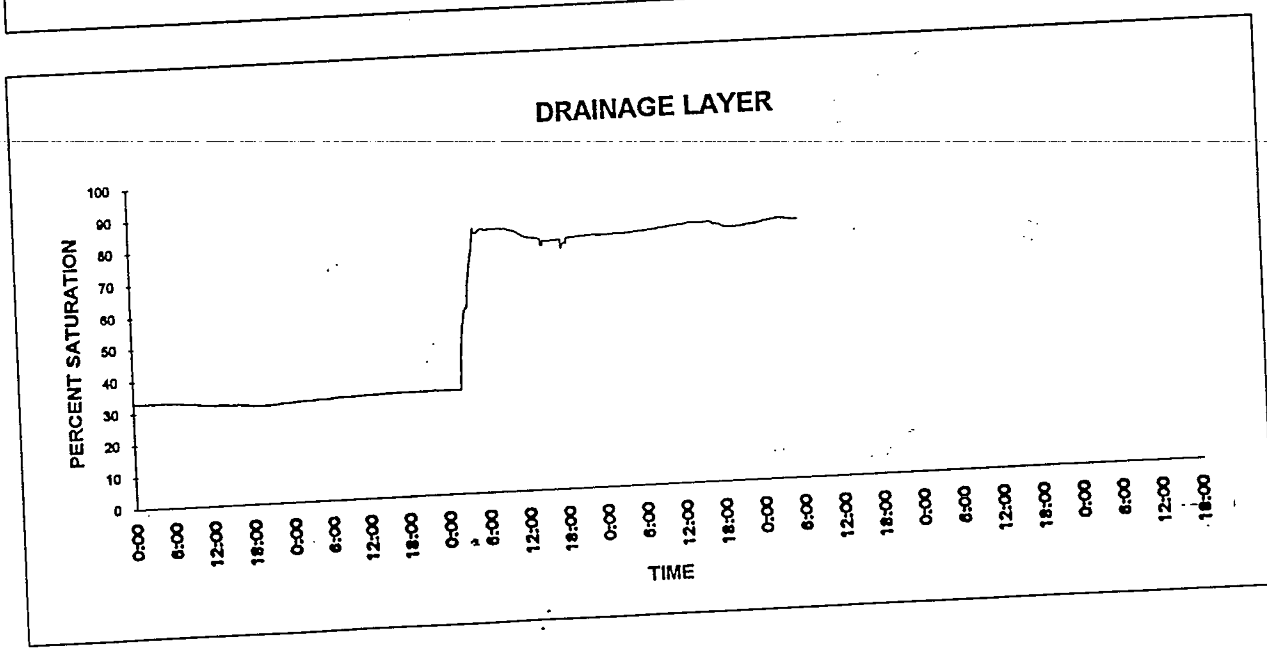
DRAINAGE LAYER



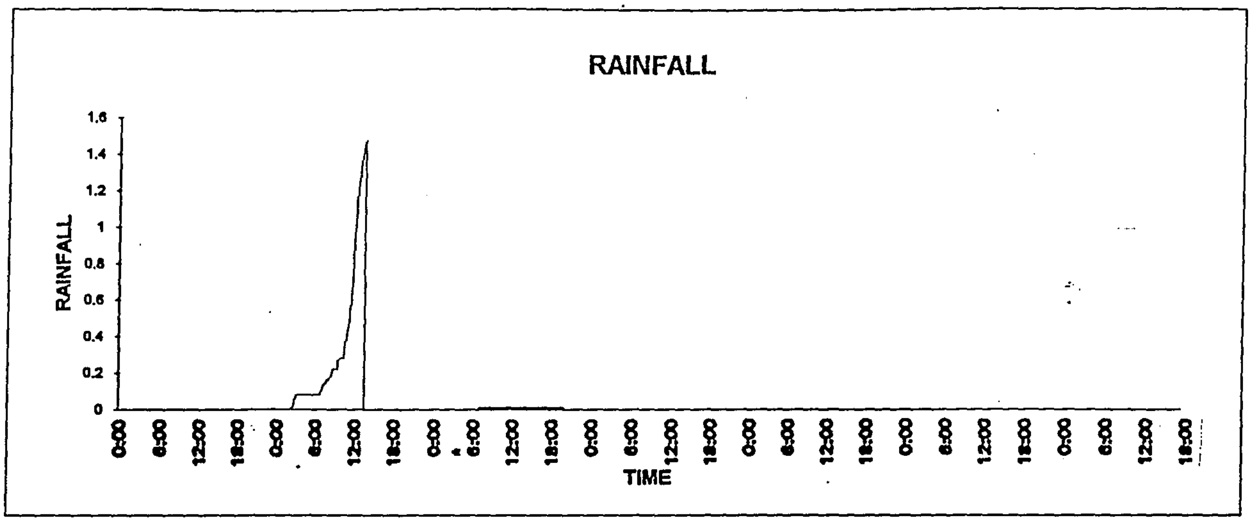
Highway 79
Good Drainage Location
3/28/94 - 4/3/94

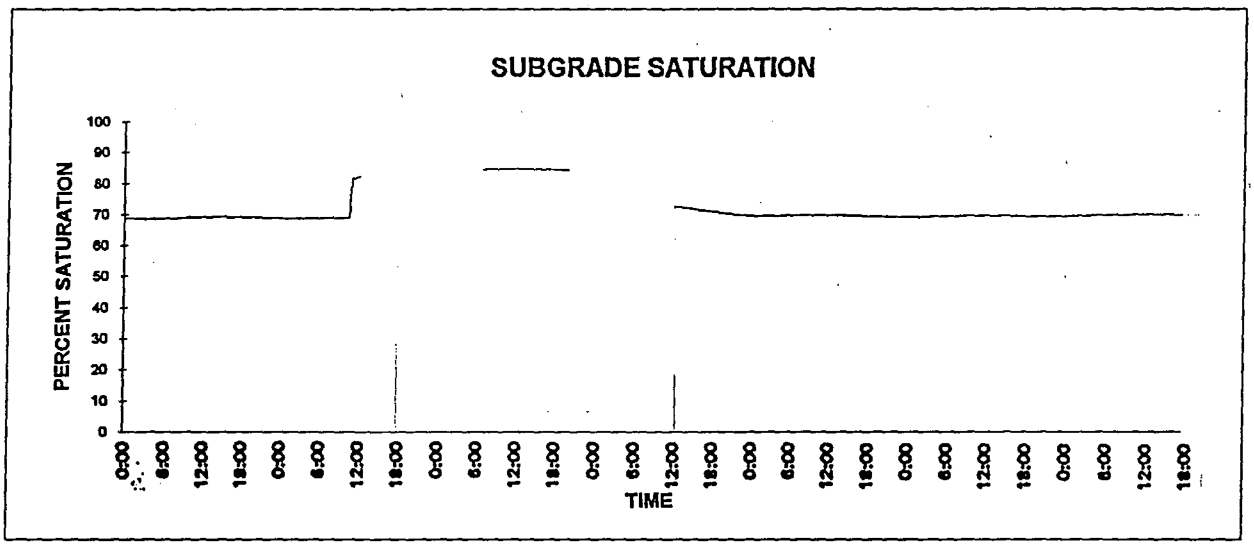


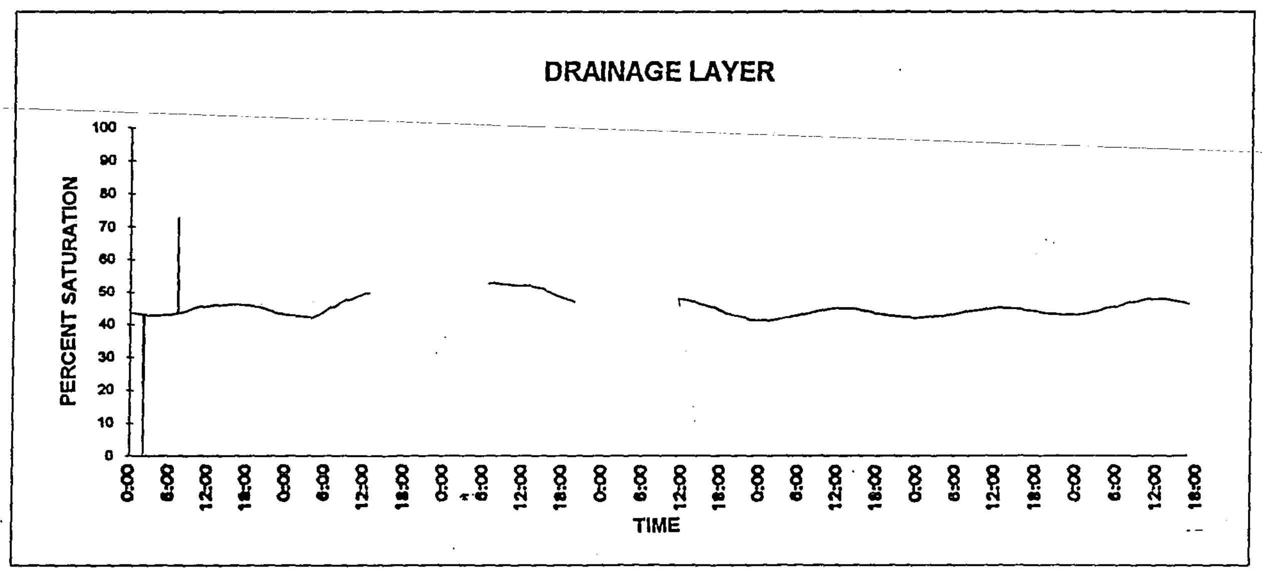




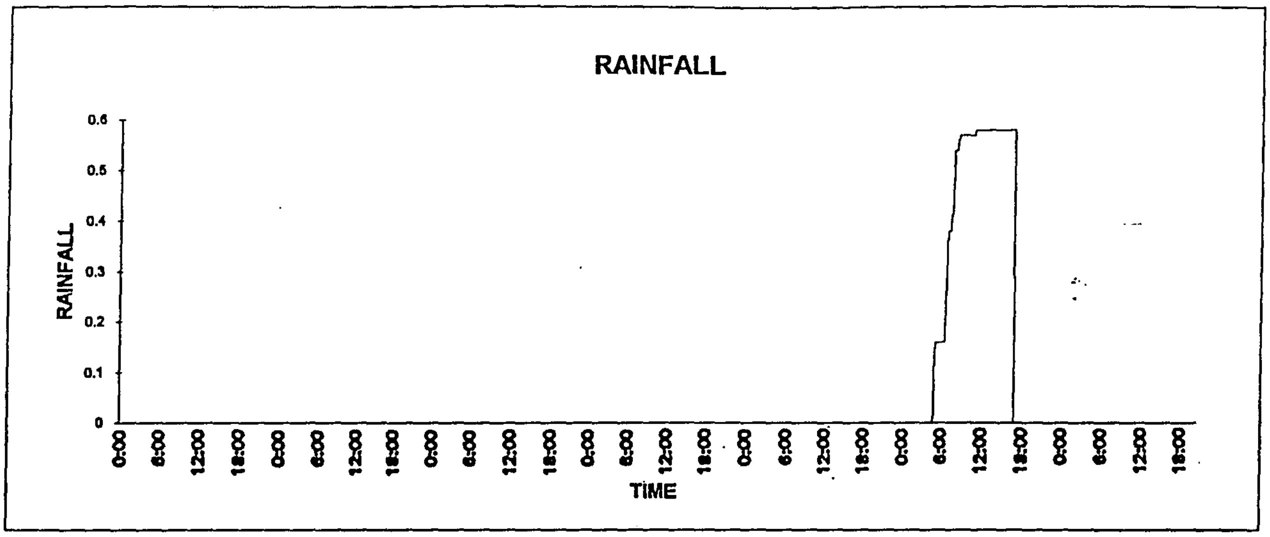
Highway 79
Good Drainage Location
4/4/94 - 4/10/94

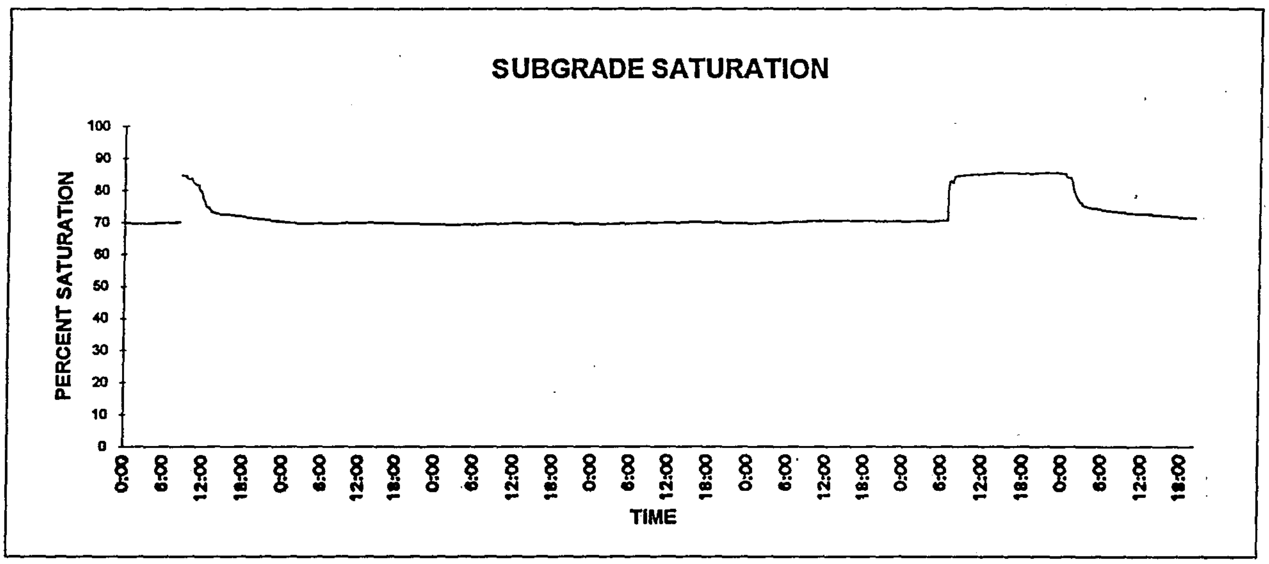


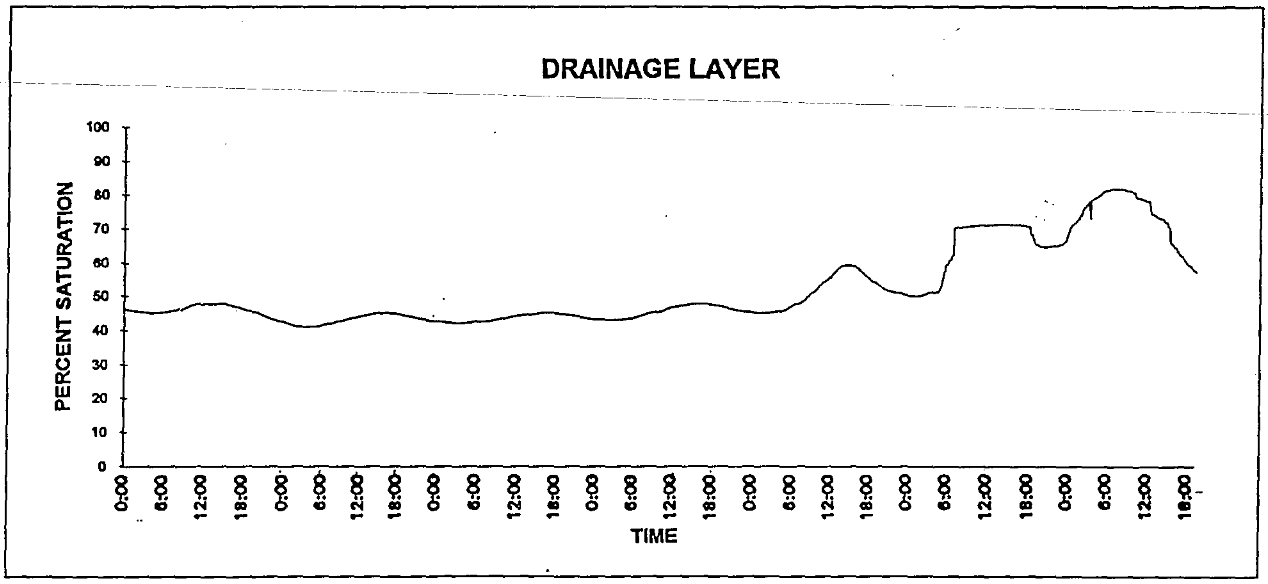




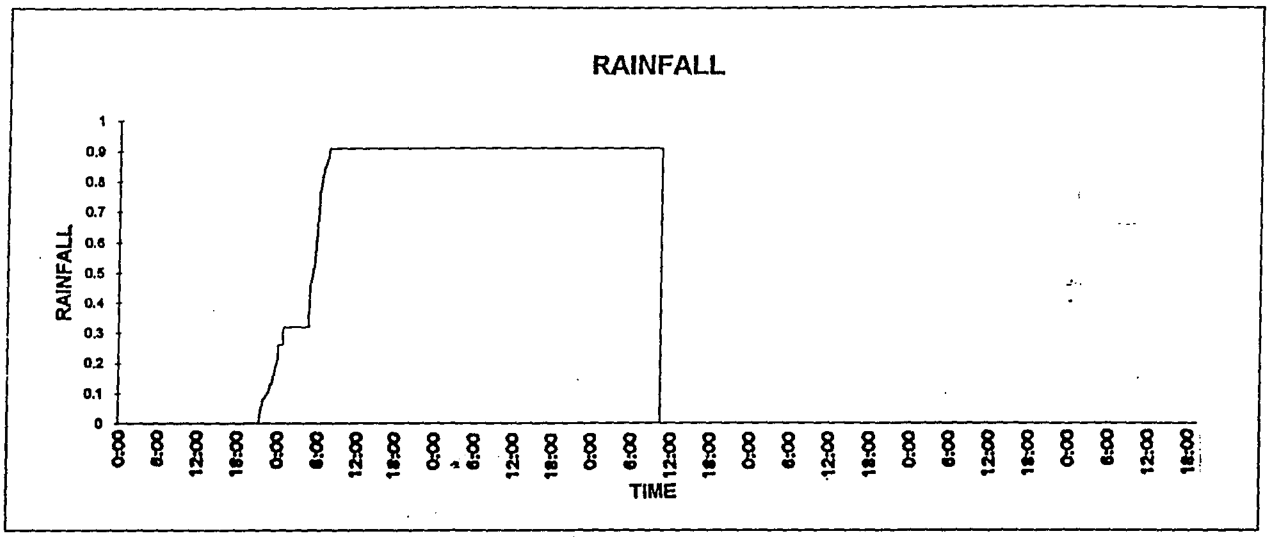
Highway 79
Good Drainage Location
4/11/94 - 4/17/94

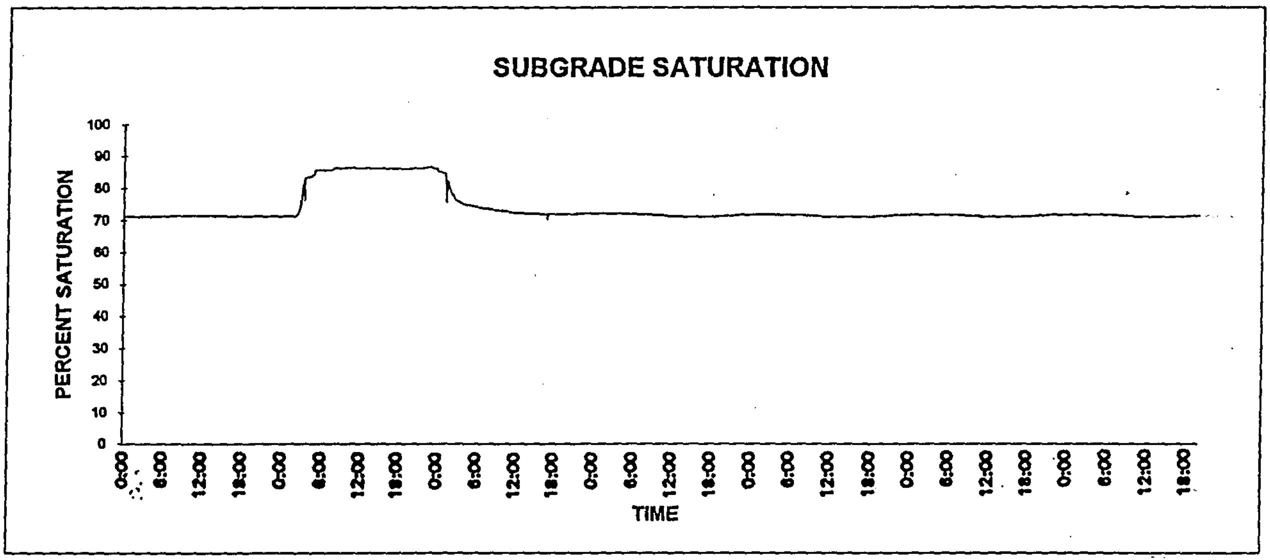


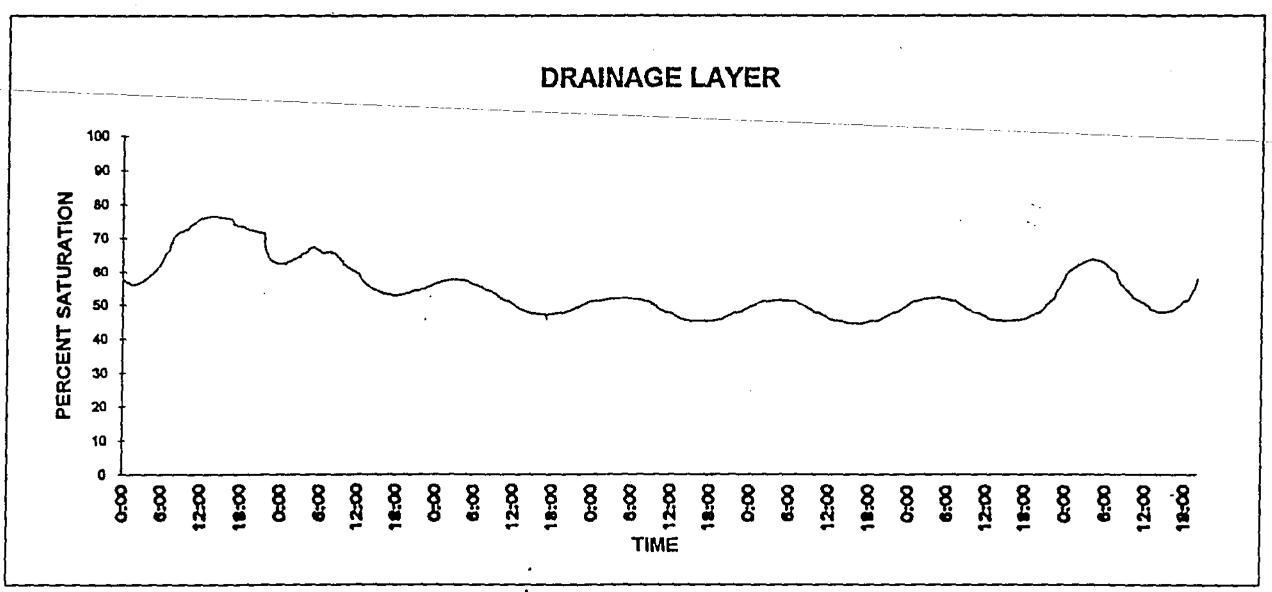




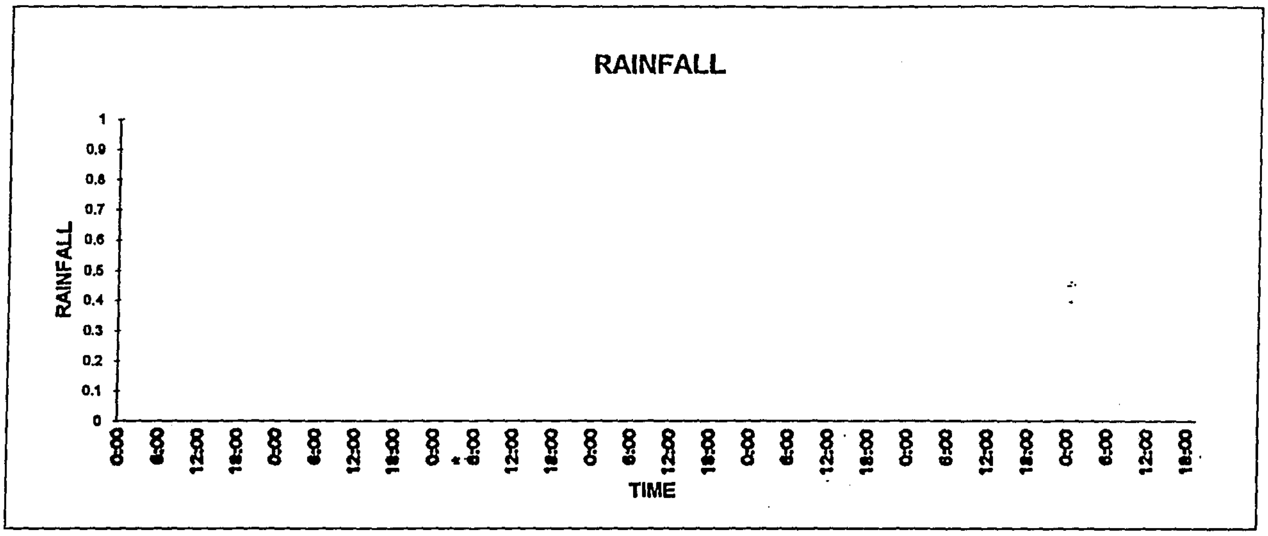
Highway 79
Good Drainage Location
4/18/94 - 4/24/94

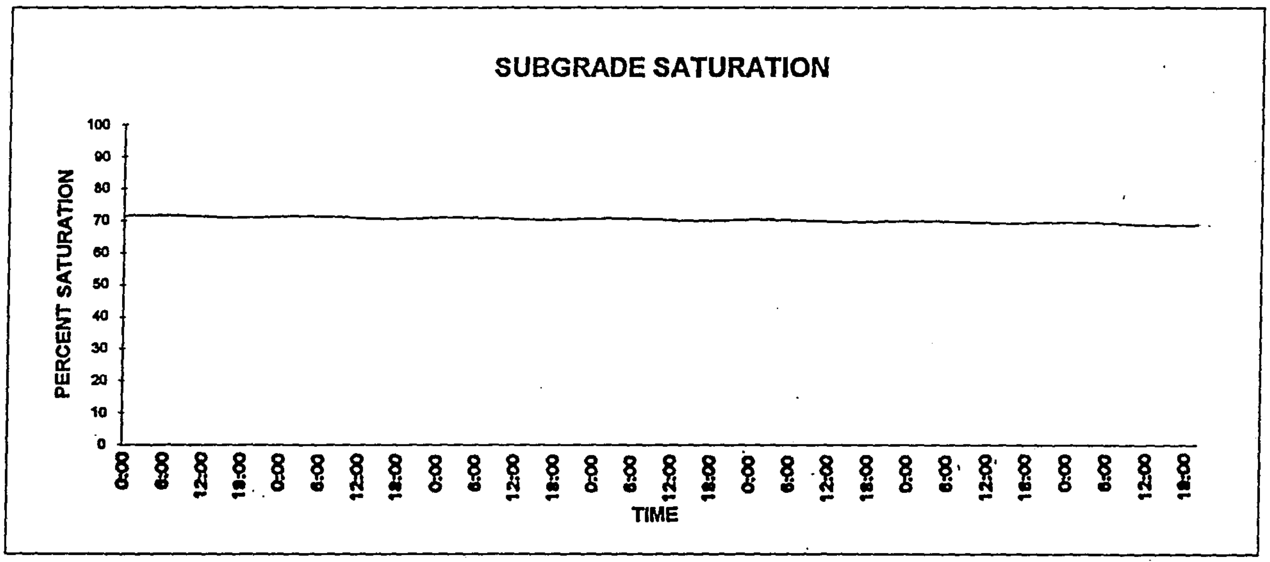


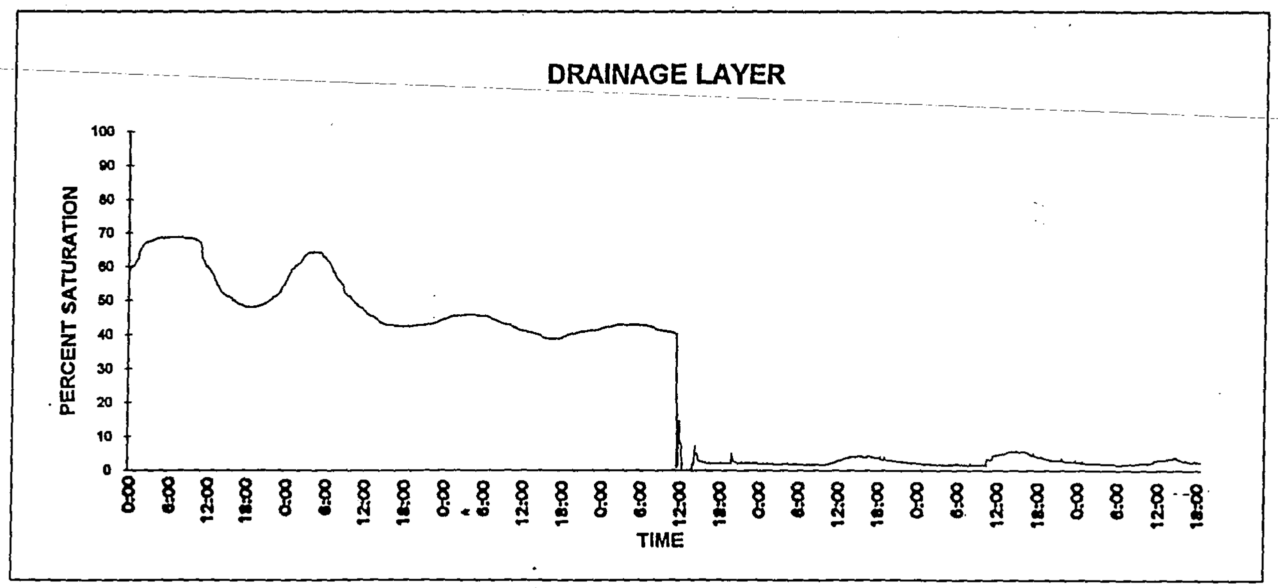




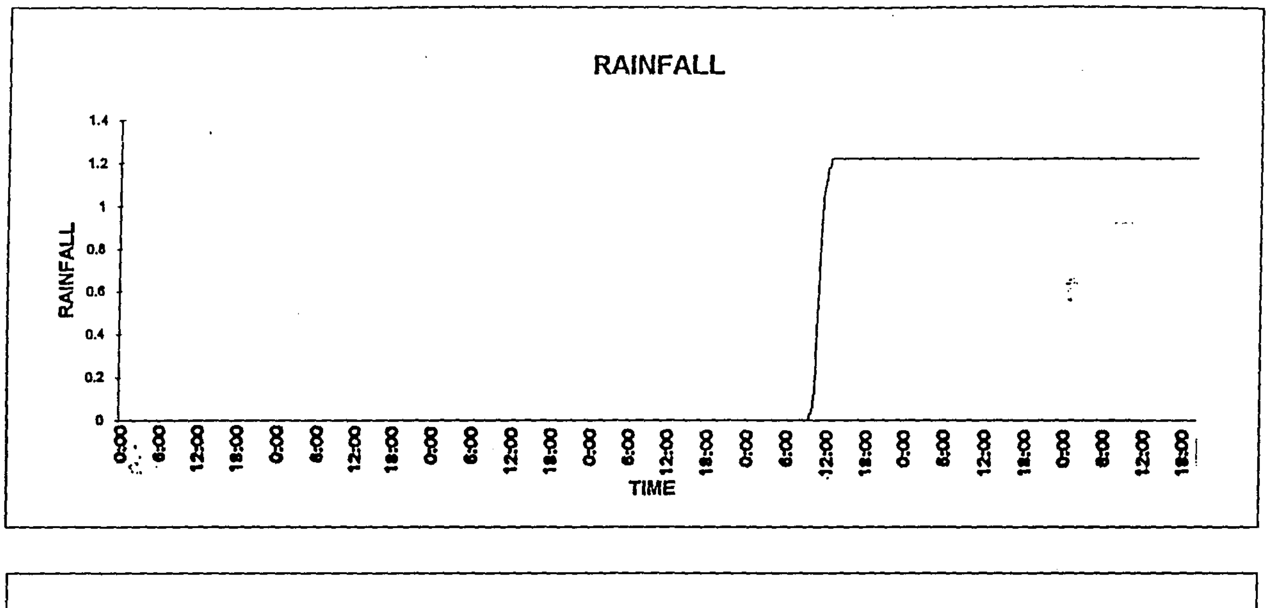
Highway 79
Good Drainage Location
4/25/94 - 5/1/94

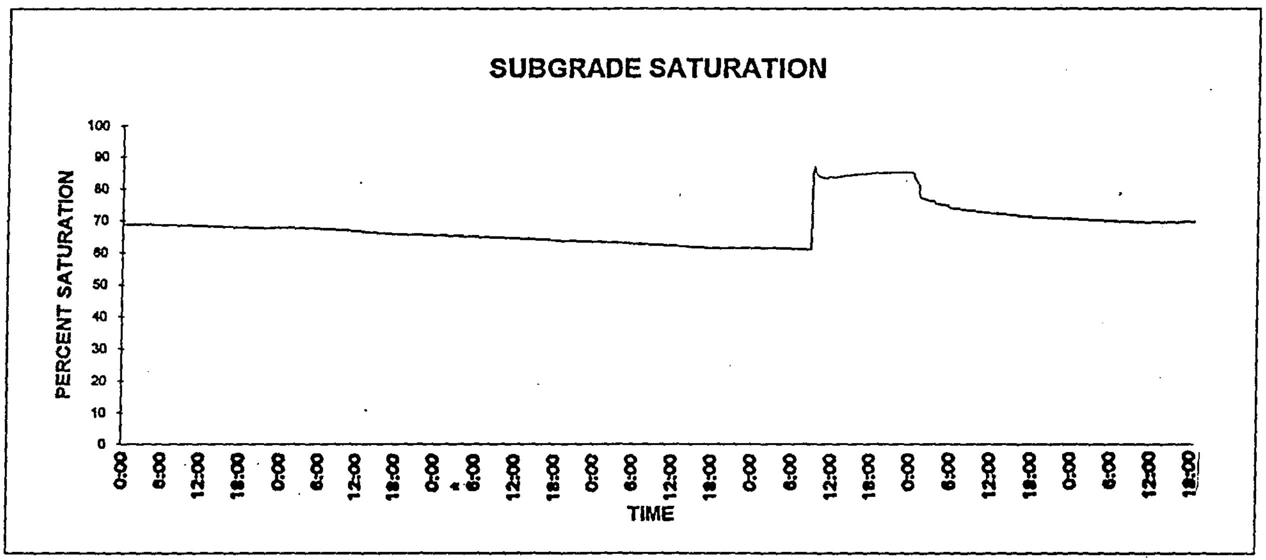


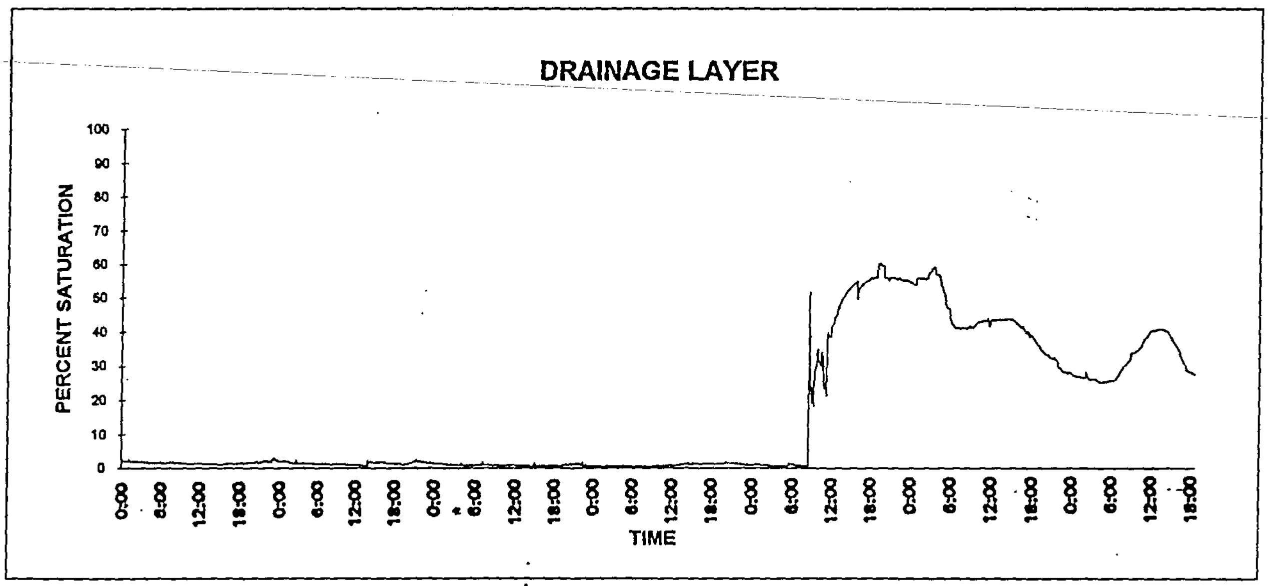




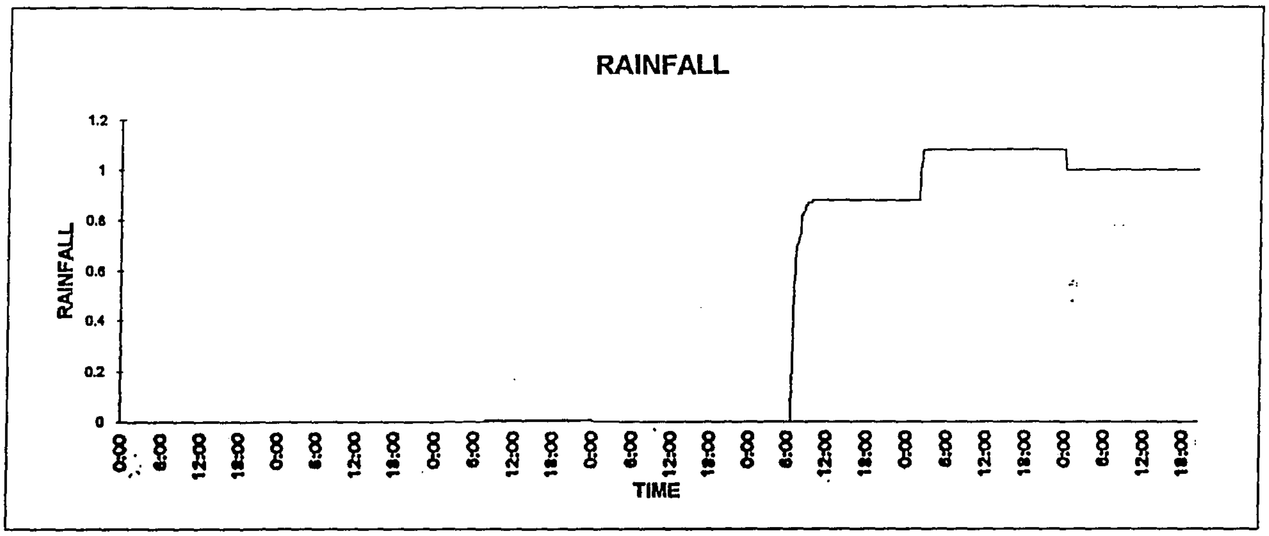
Highway 79
Good Drainage Location
5/2/94 - 5/8/94

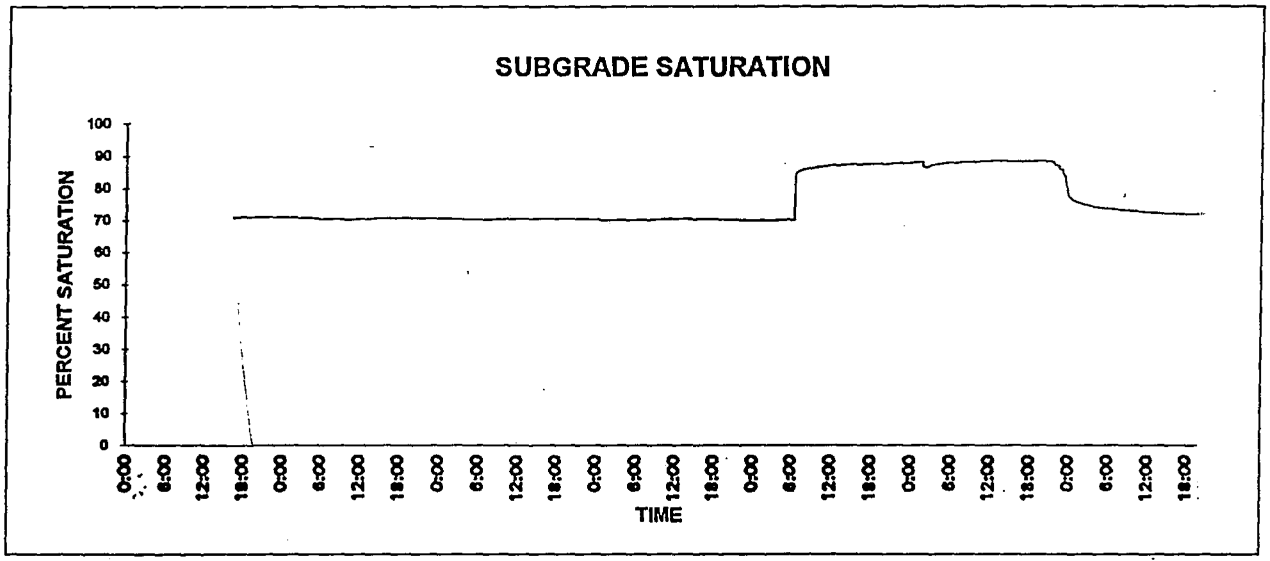


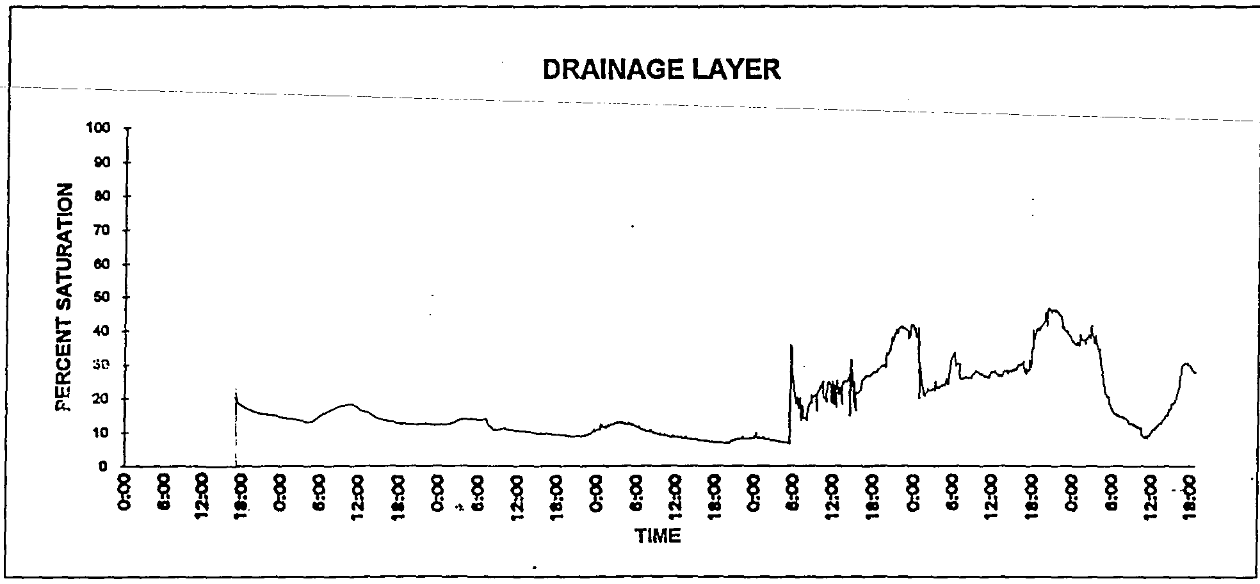




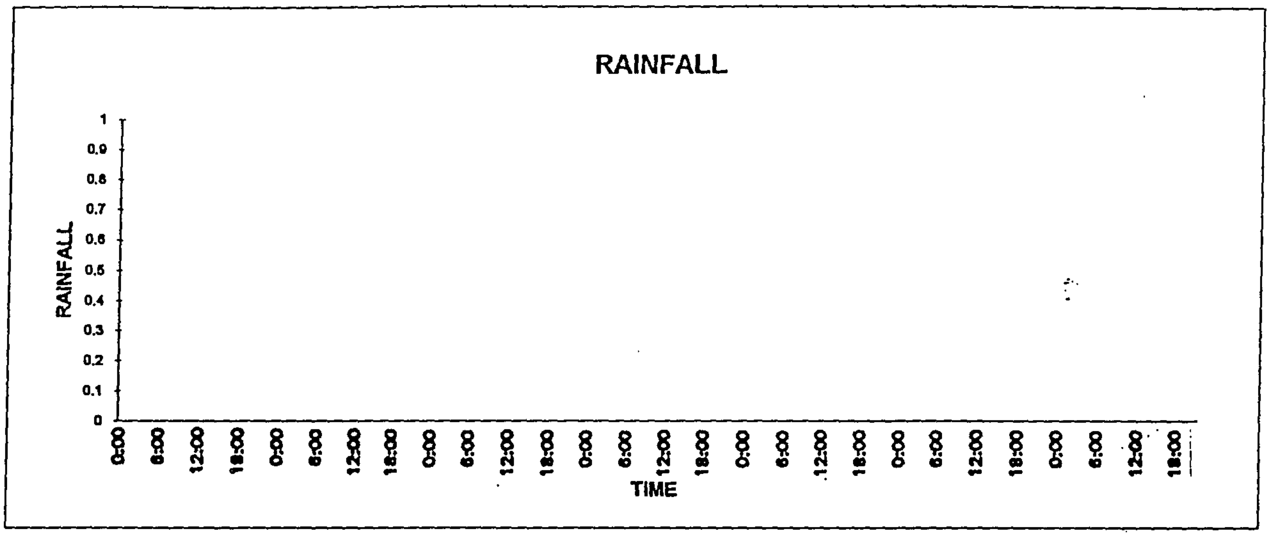
Highway 79
Good Drainage Location
5/10/94 - 5/16/94

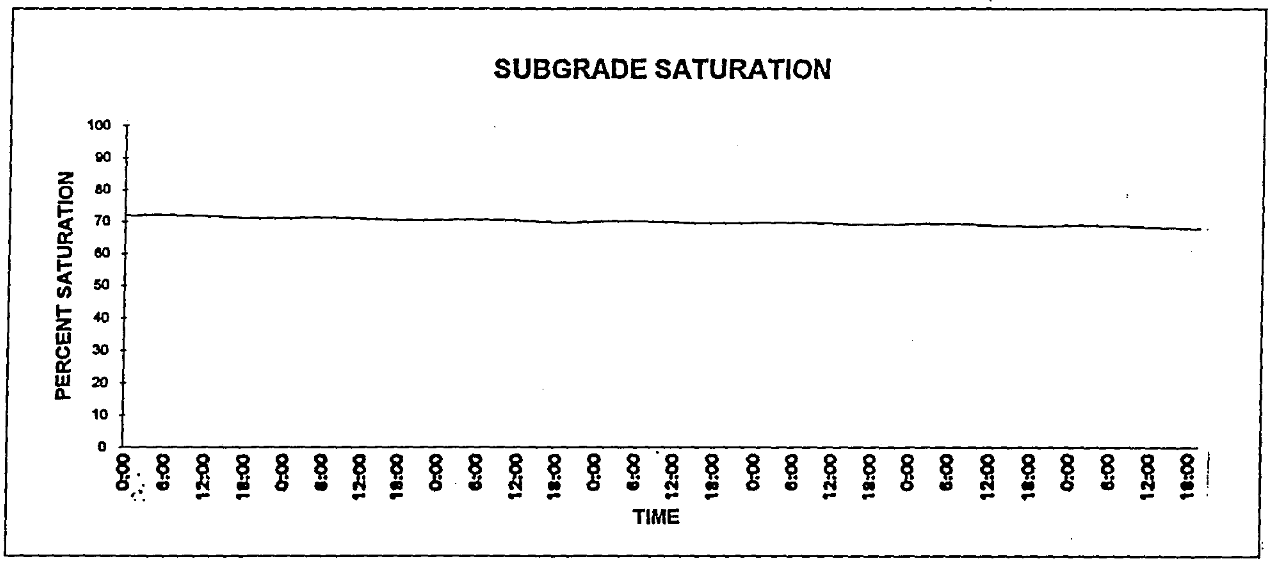


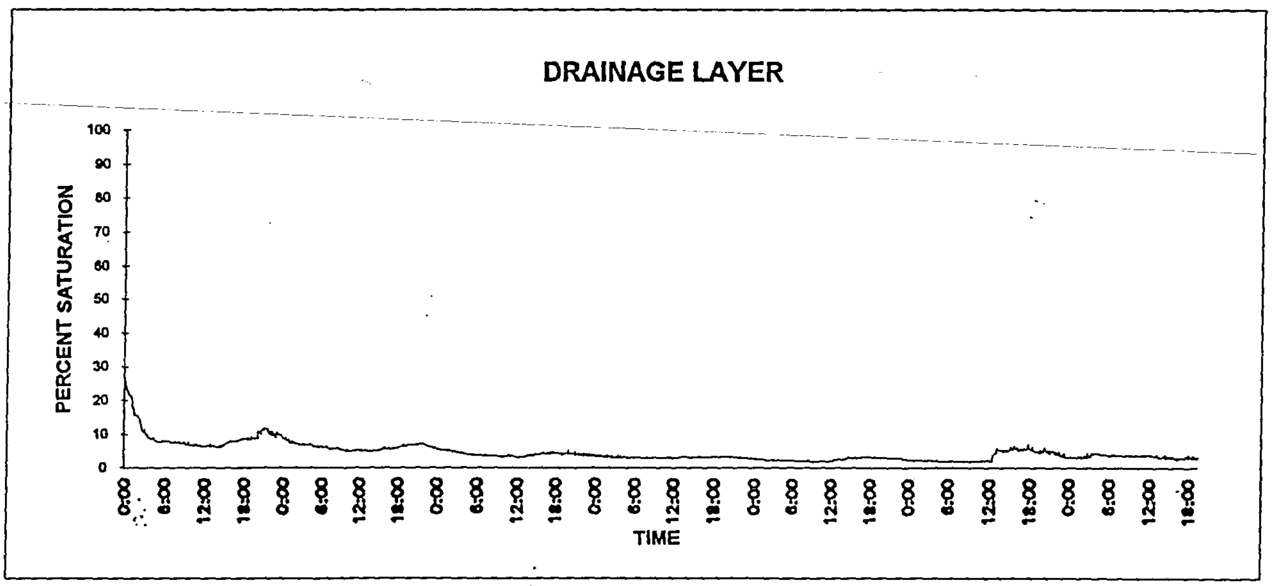




Highway 79
Good Drainage Location
5/17/94 - 5/23/94

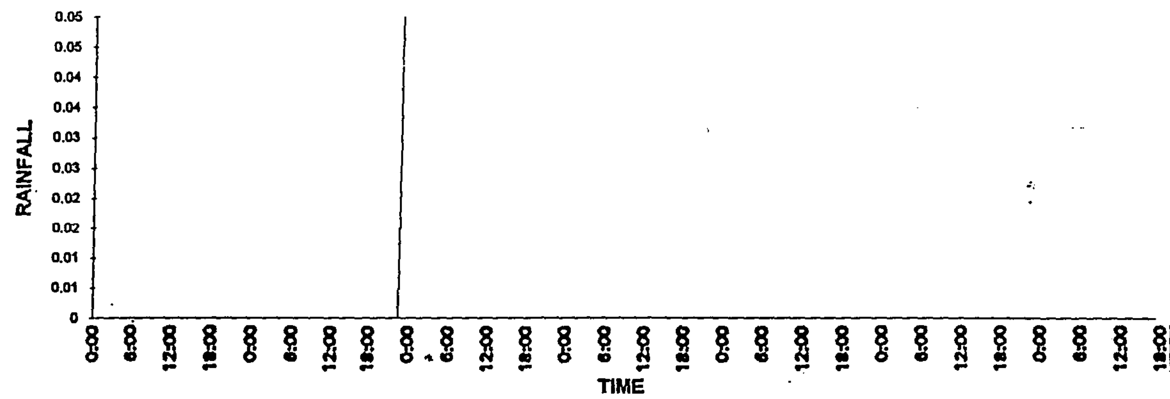




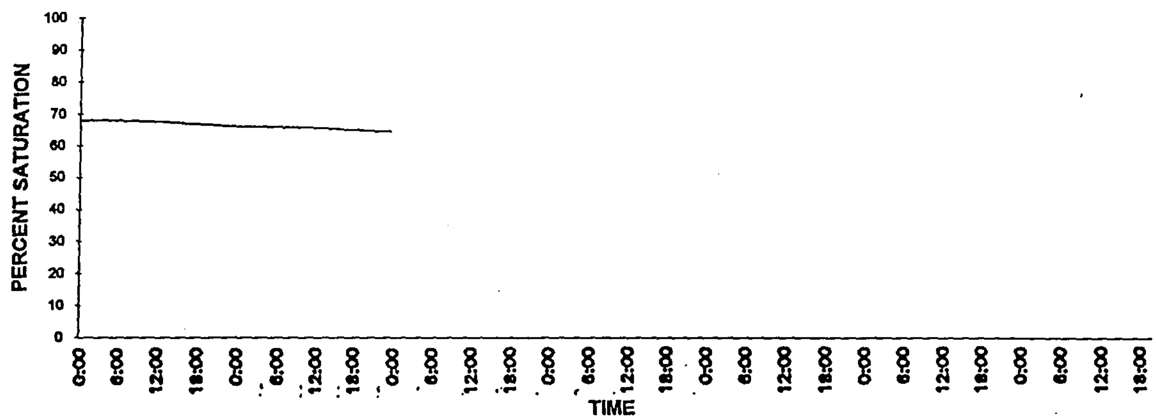


Highway 79
Good Drainage Location
5/24/94 - 5/30/94

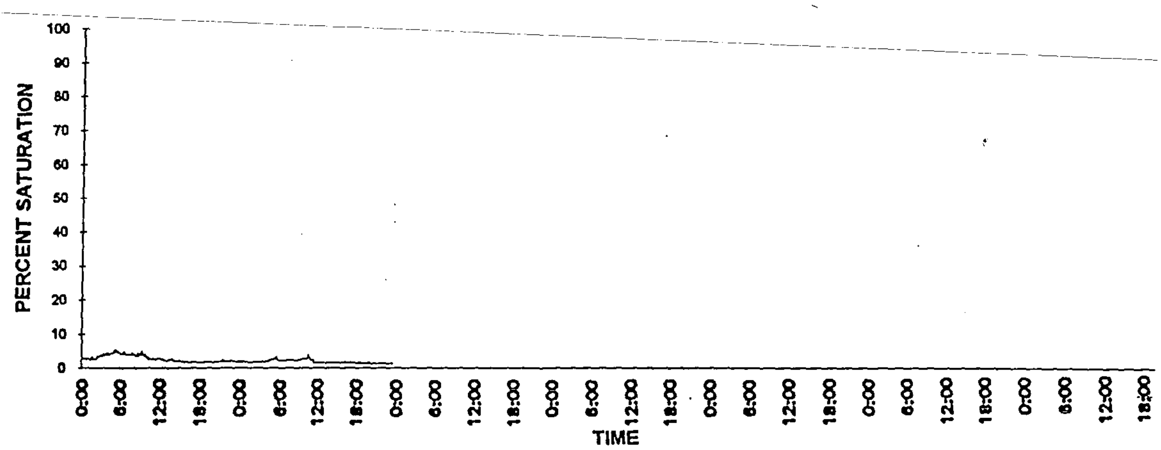
RAINFALL



SUBGRADE SATURATION



DRAINAGE LAYER

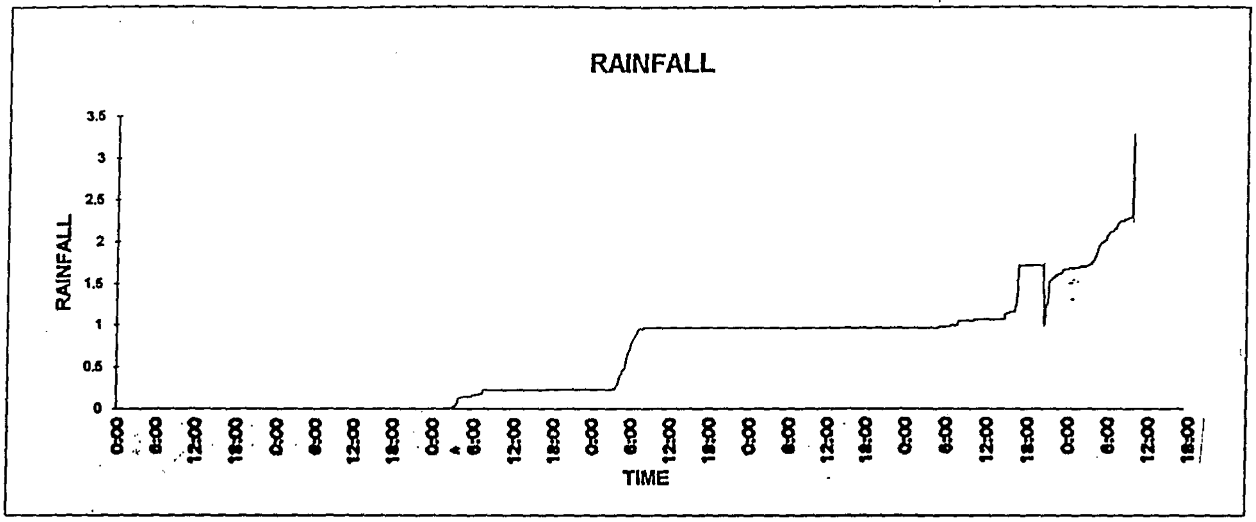


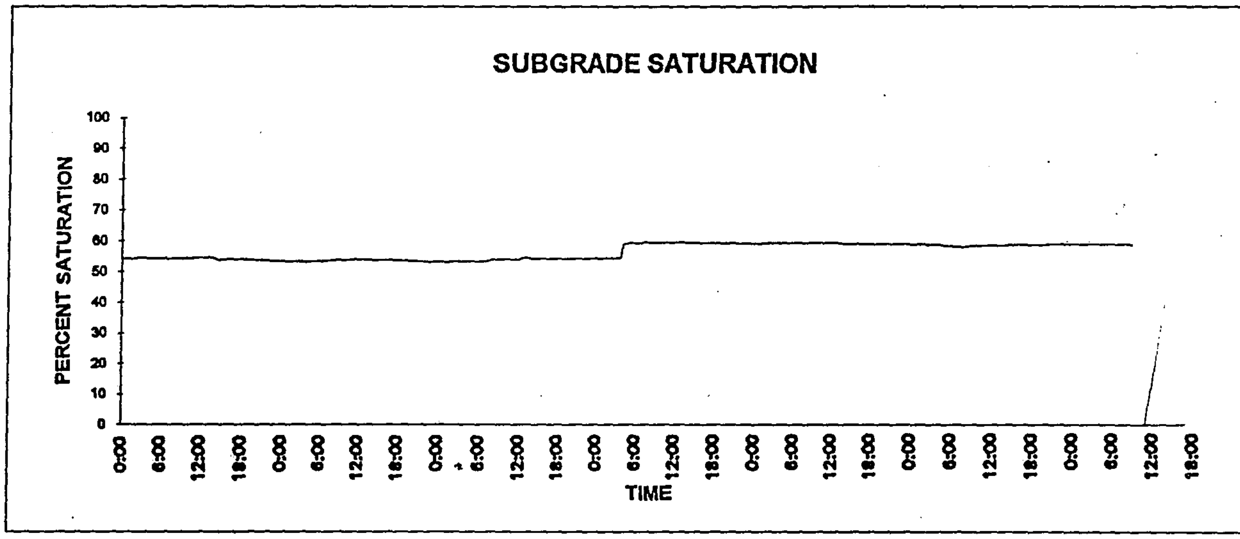
Highway 79

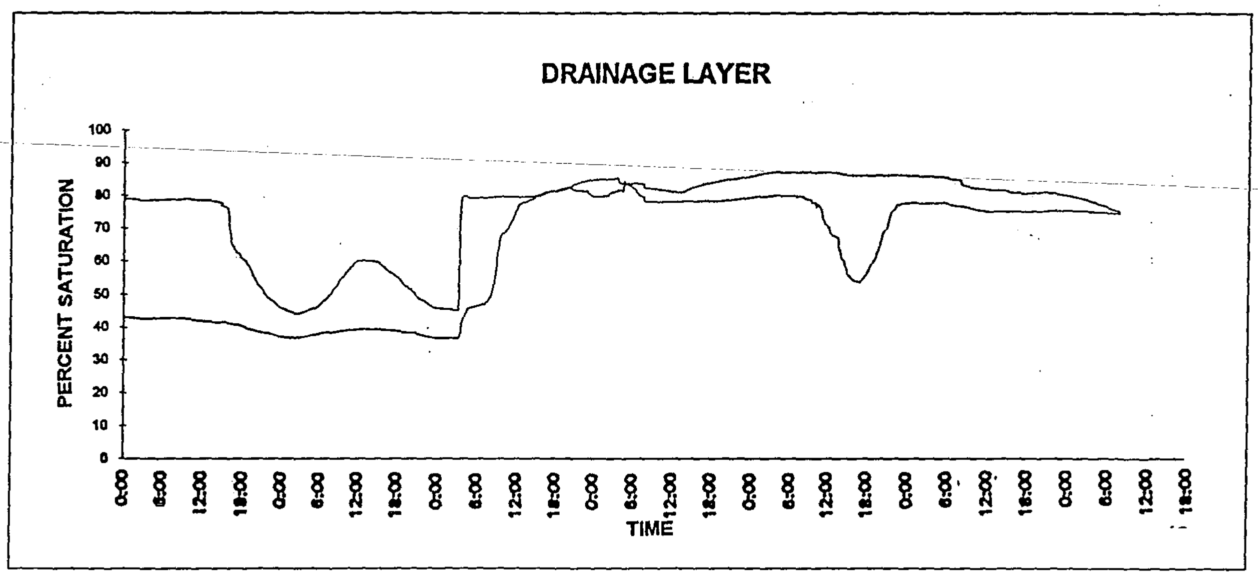
Poor Drainage Location

Field Moisture Data

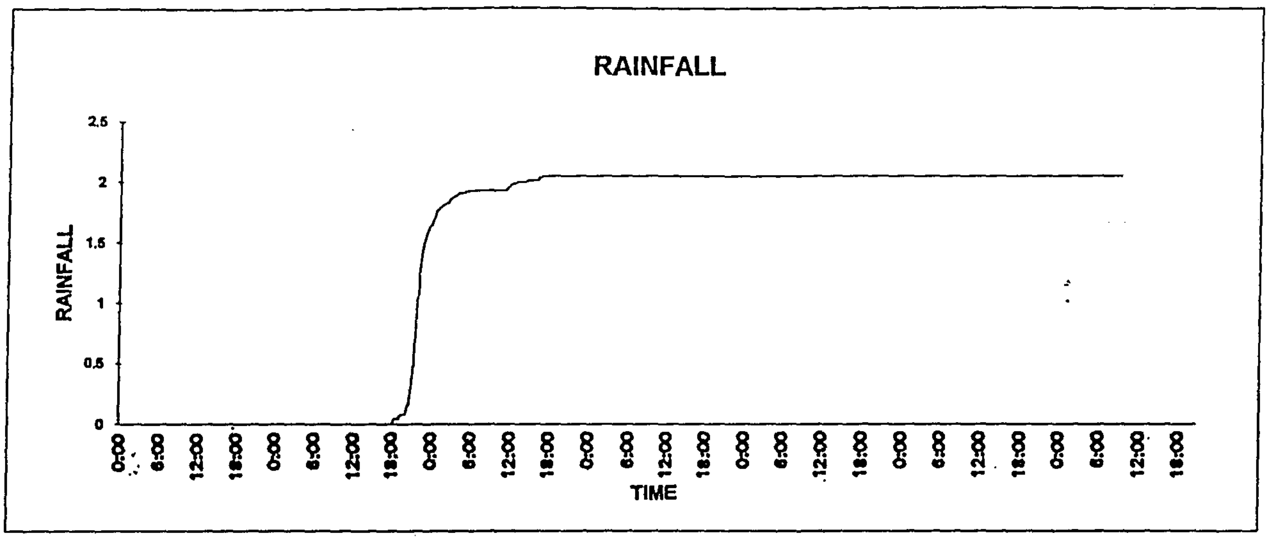
Highway 79
Poor Drainage Location
3/21/94 - 3/27/94

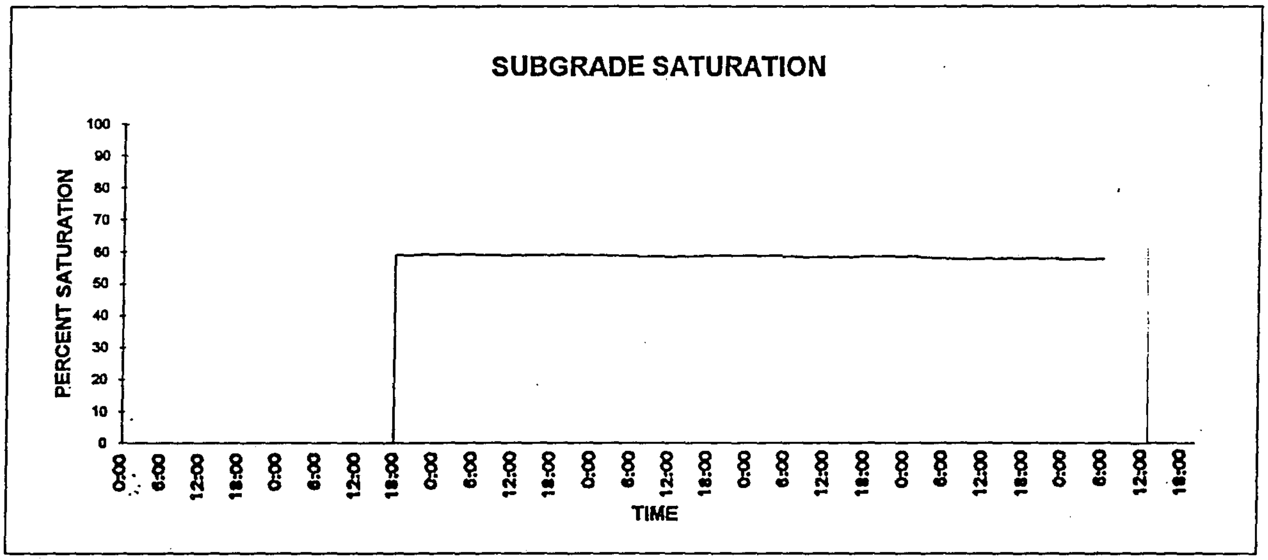


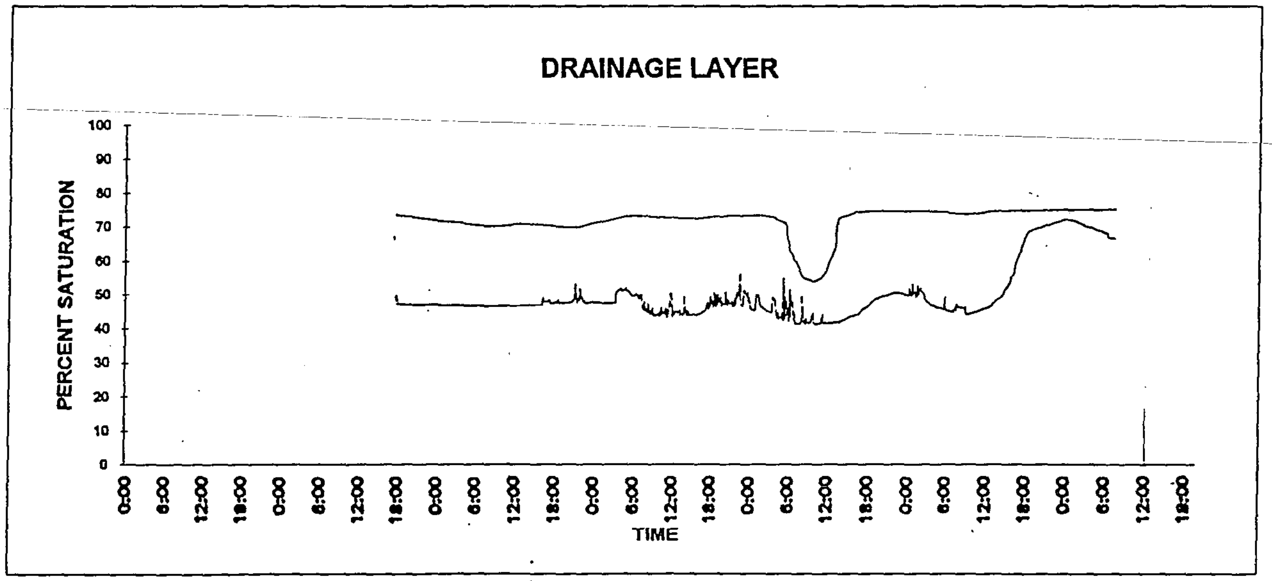




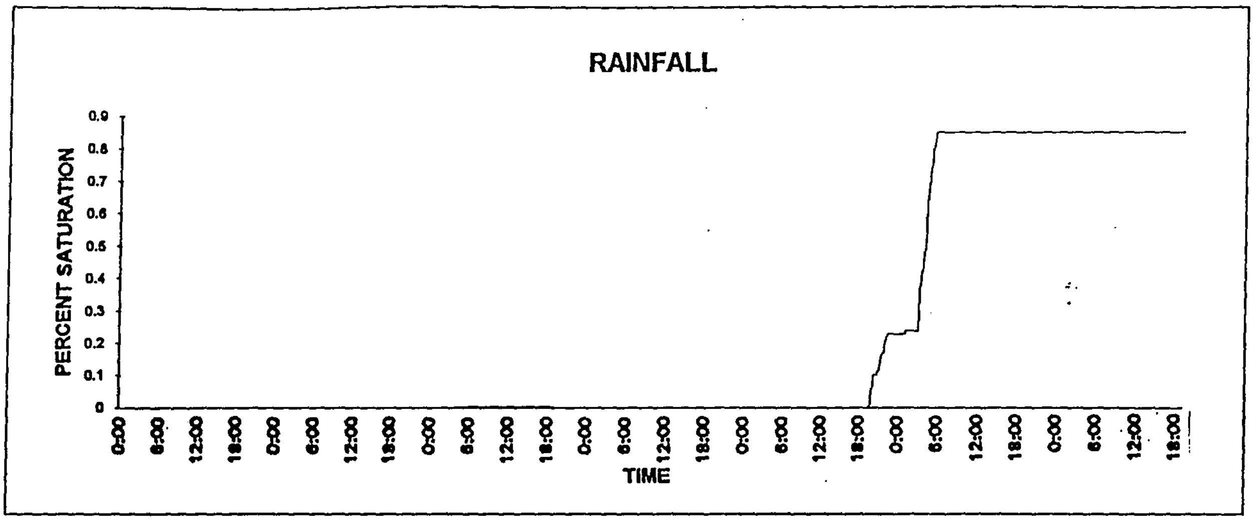
Highway 79
Poor Drainage Location
4/4/94 - 4/10/94

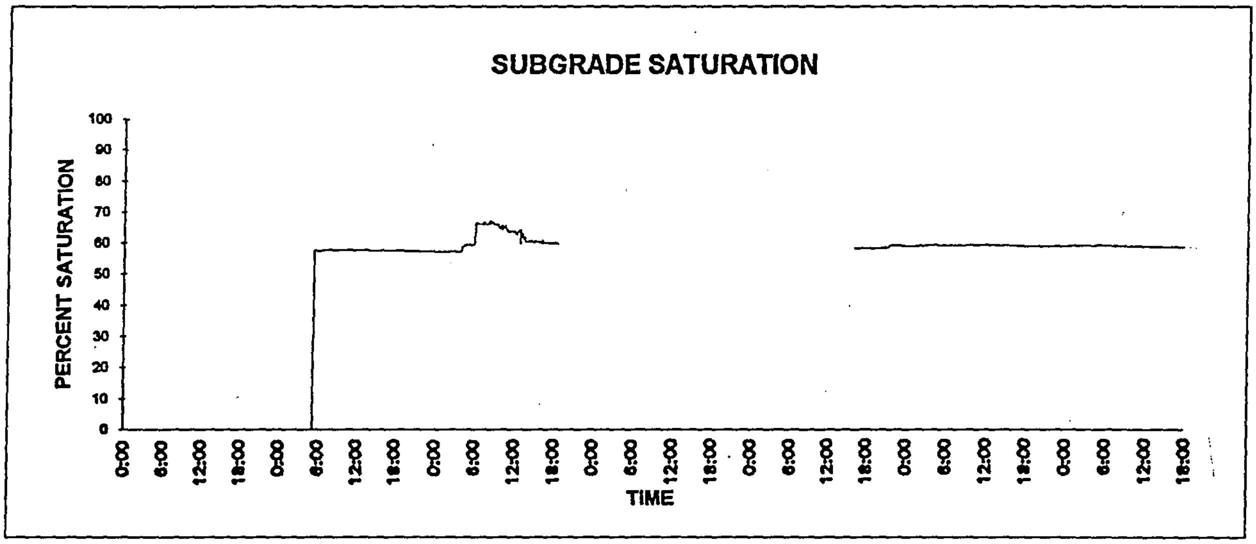


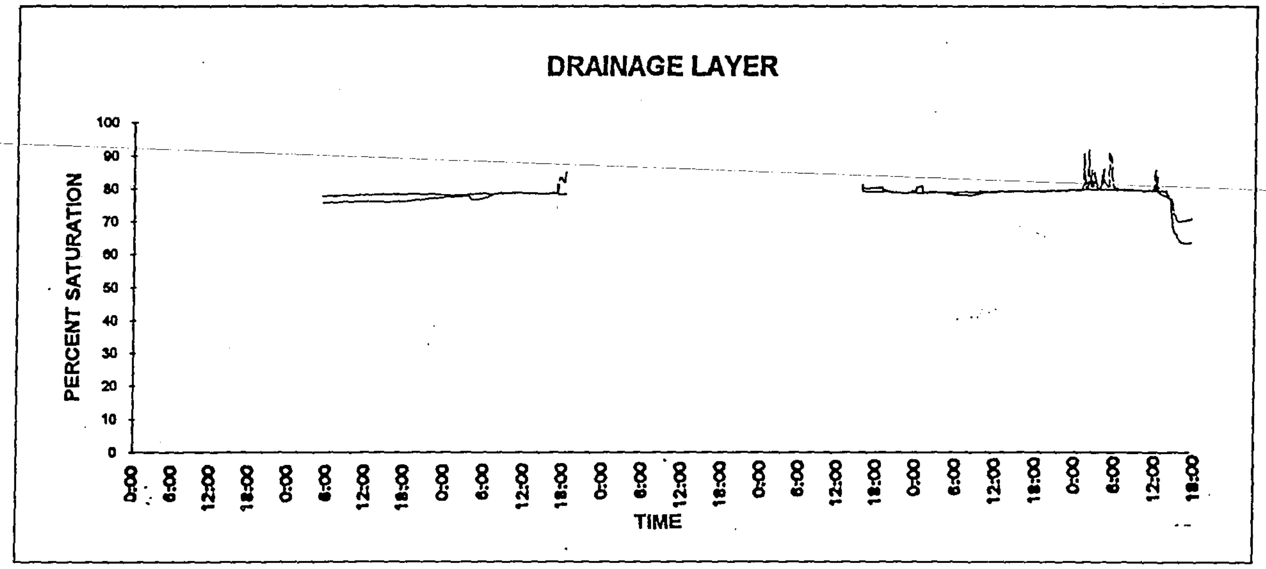




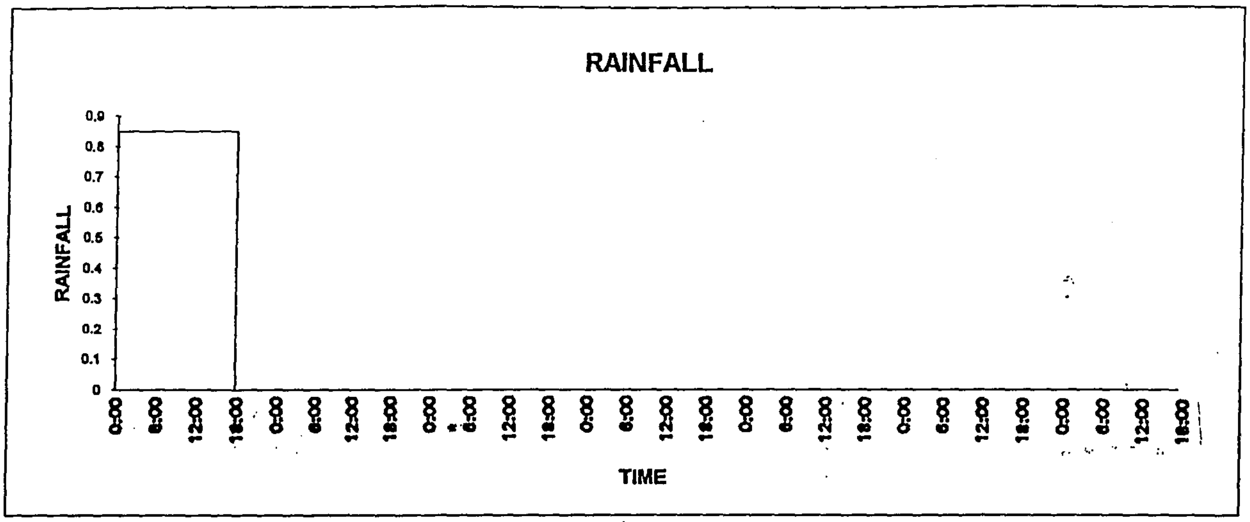
Highway 79
Poor Drainage Location
4/11/94 - 4/17/94

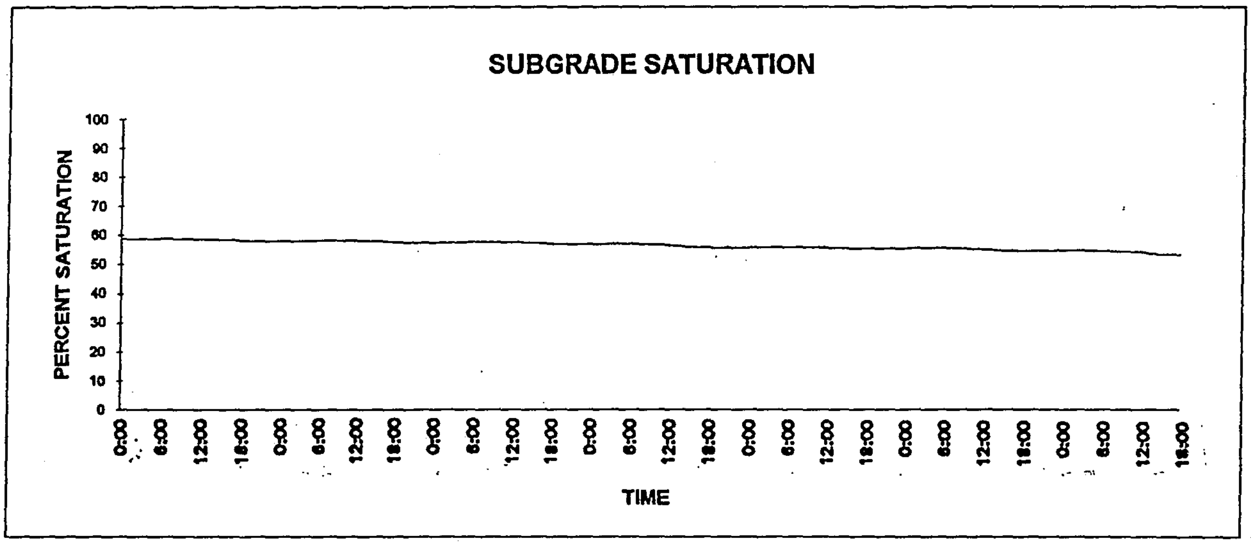


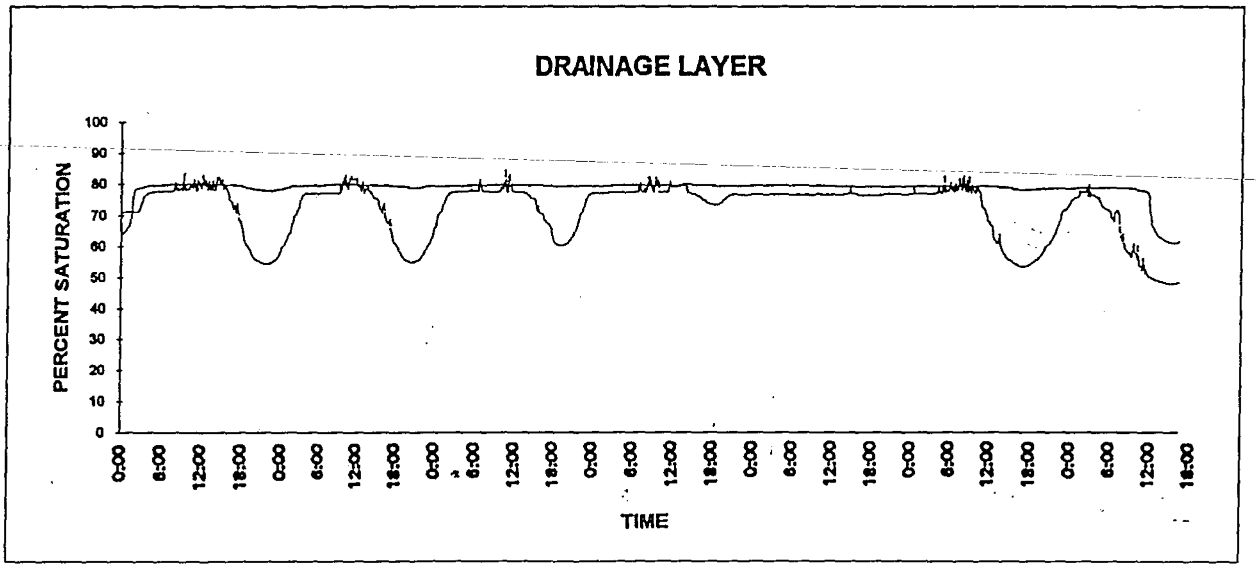




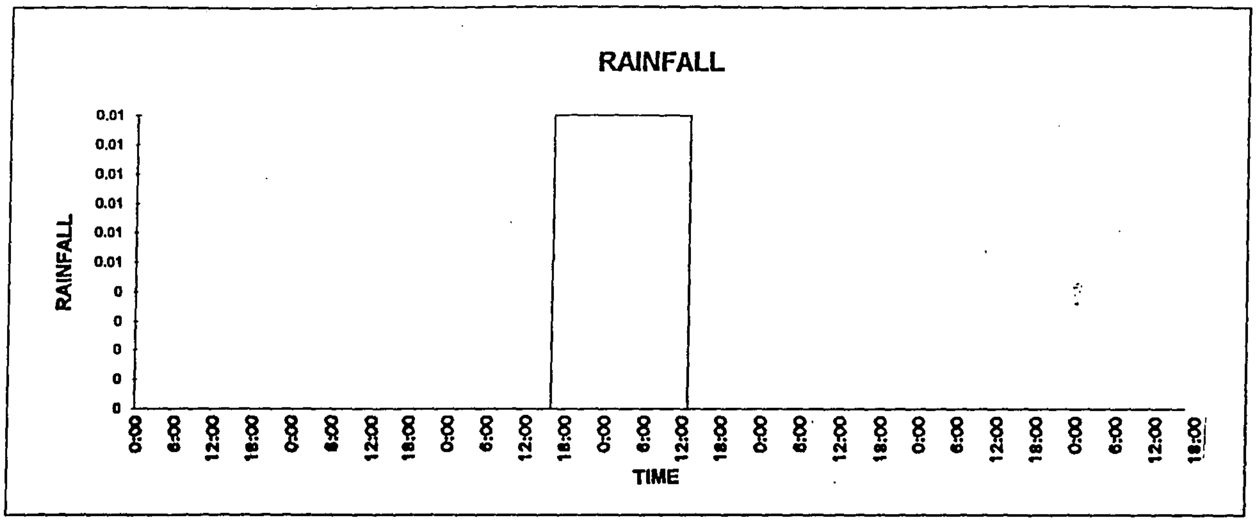
Highway 79
Poor Drainage Location
4/18/94 - 4/24/94

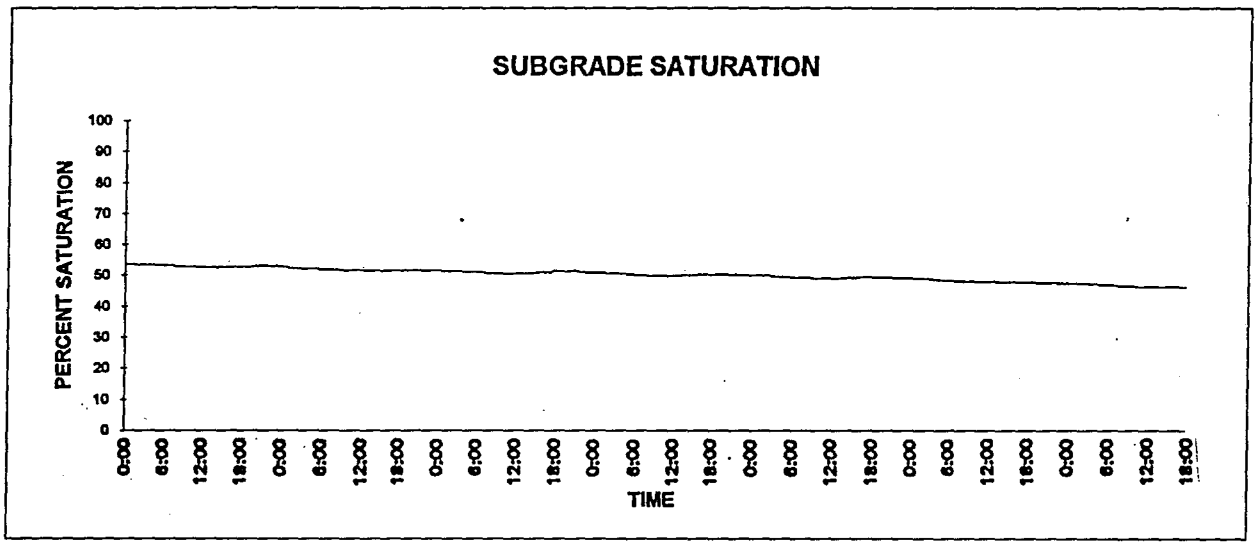


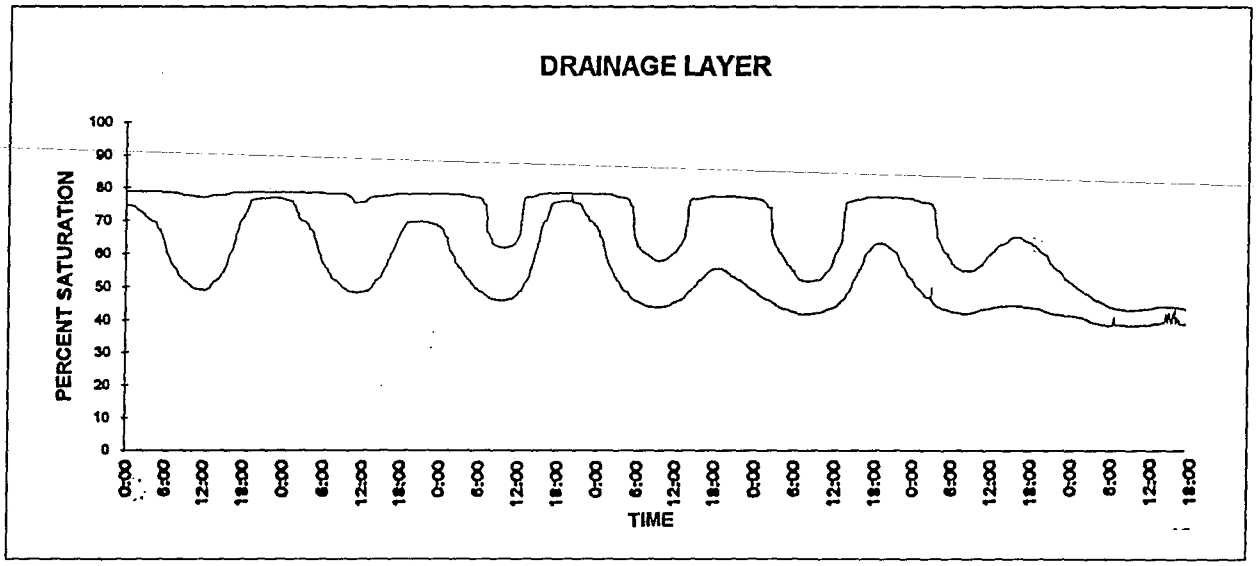




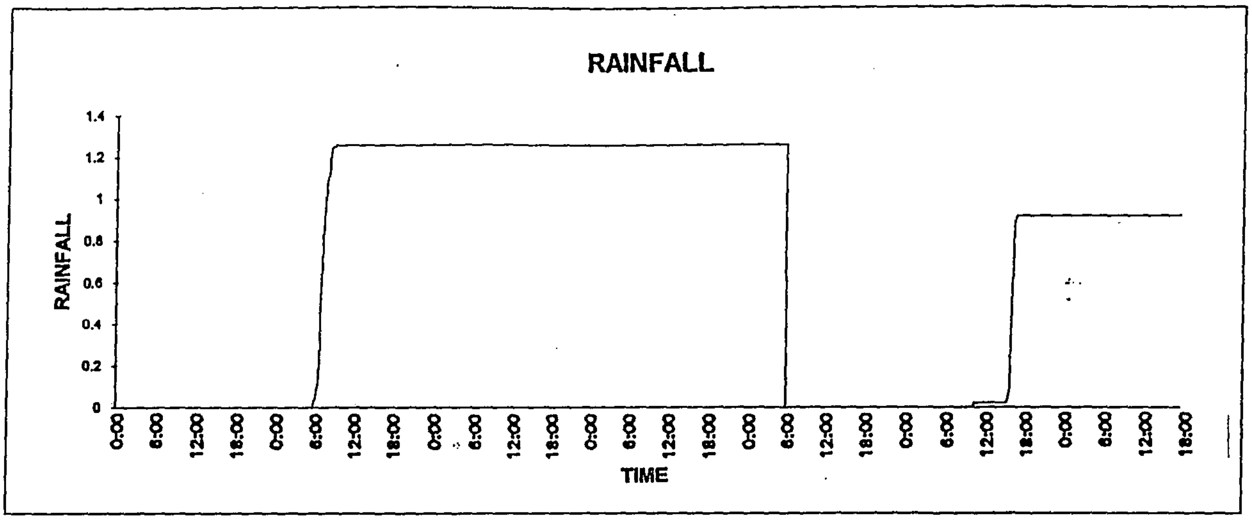
Highway 79
Poor Drainage Location
4/25/94 - 5/1/94

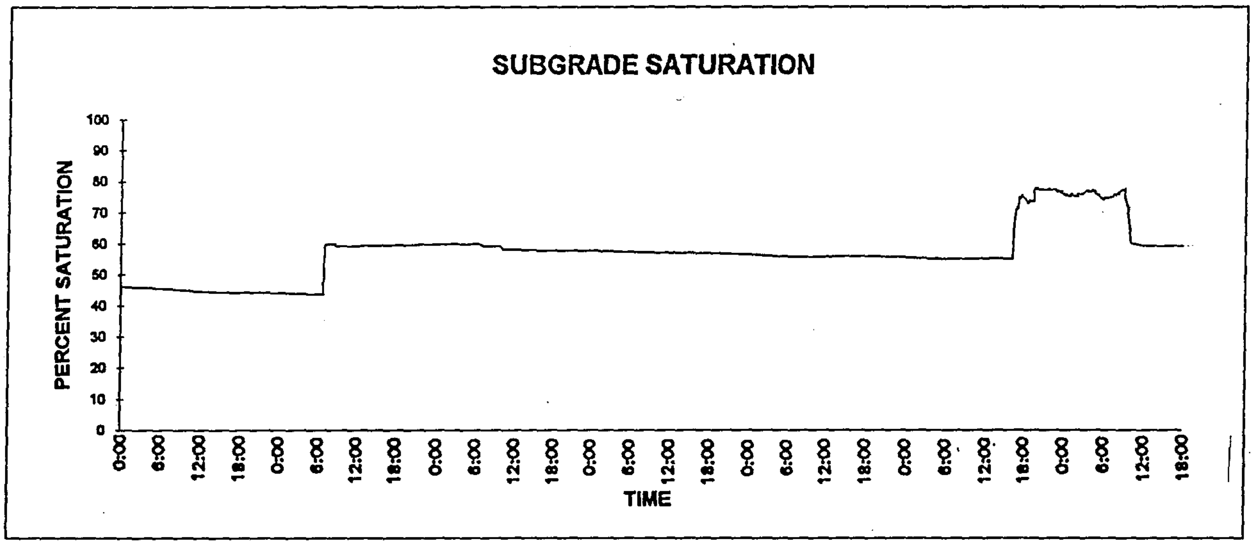


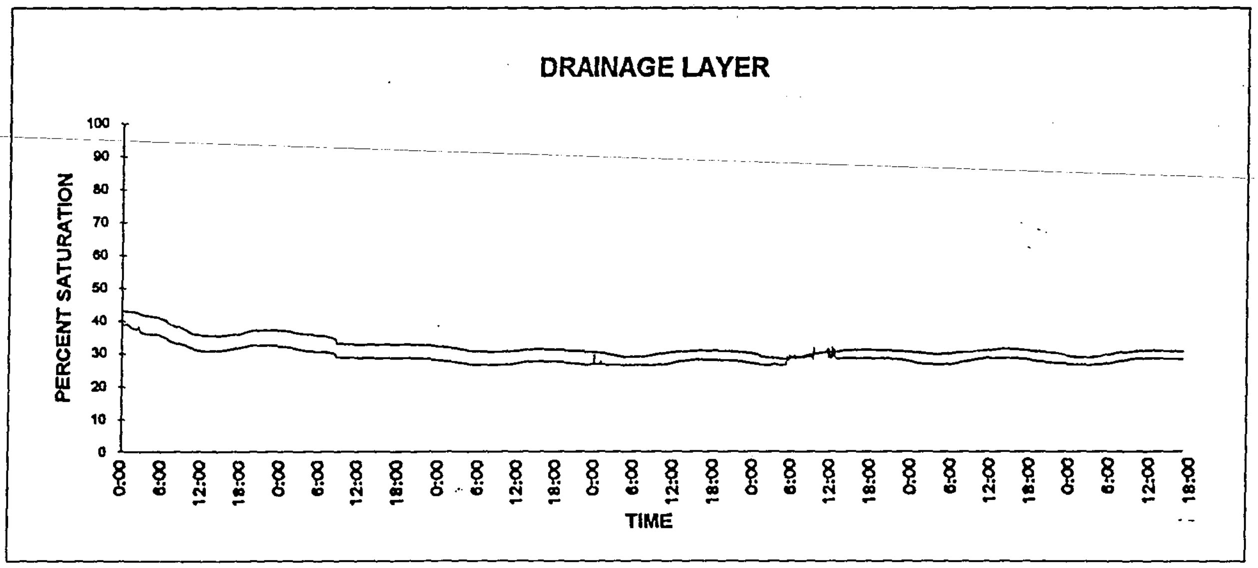




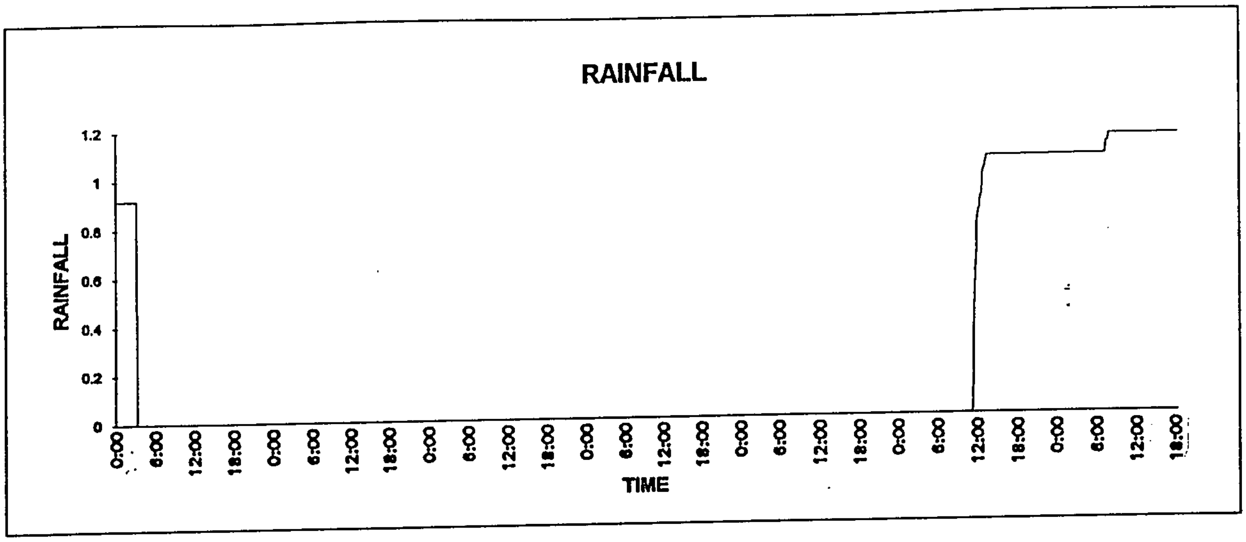
Highway 79
Poor Drainage Location
5/2/94 - 5/8/94

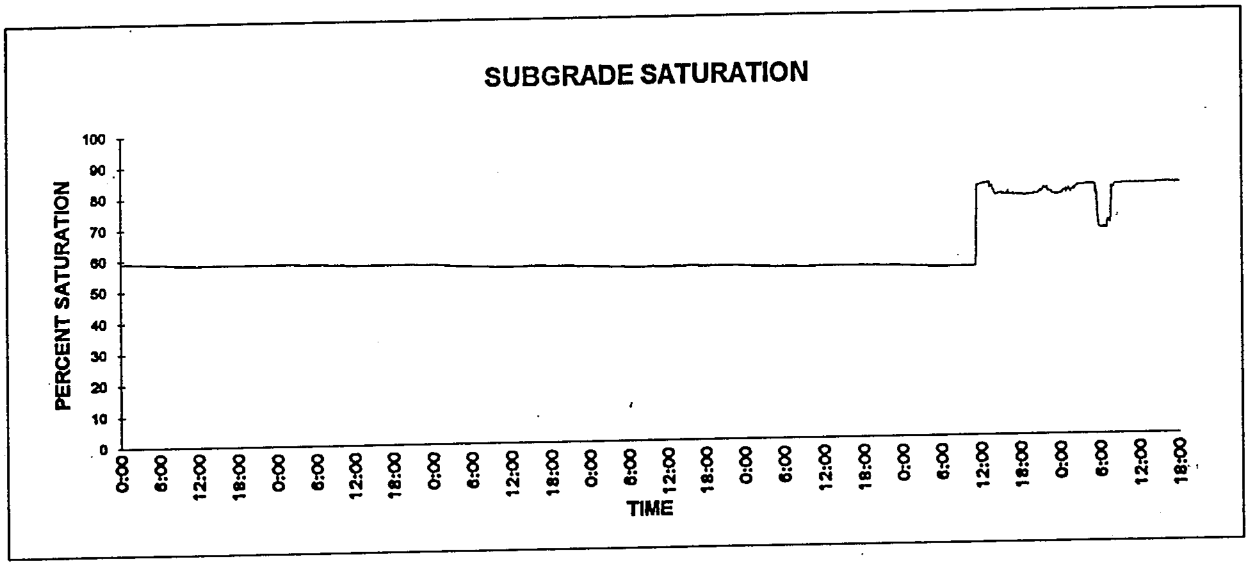


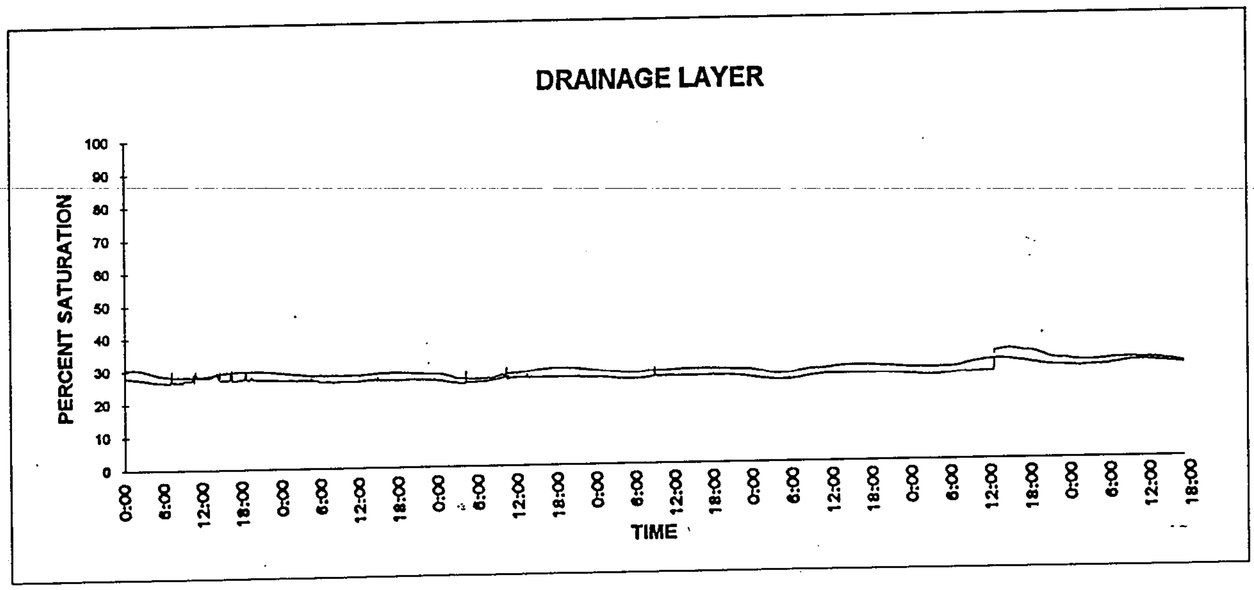




Highway 79
Poor Drainage Location
5/9/94 - 5/15/94

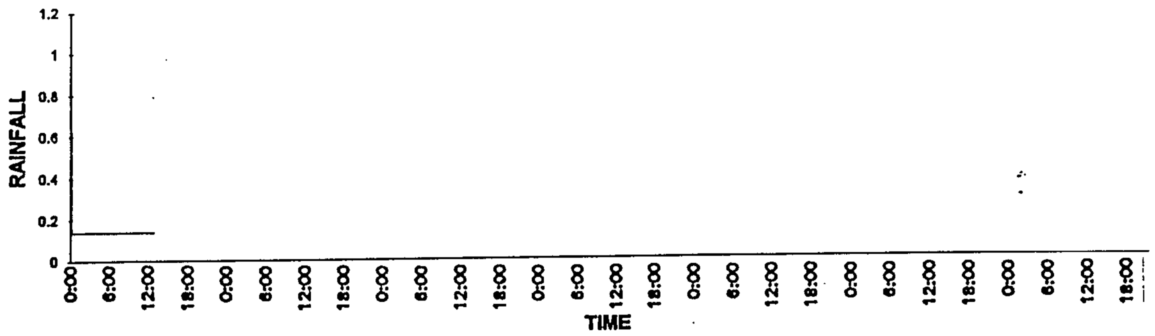




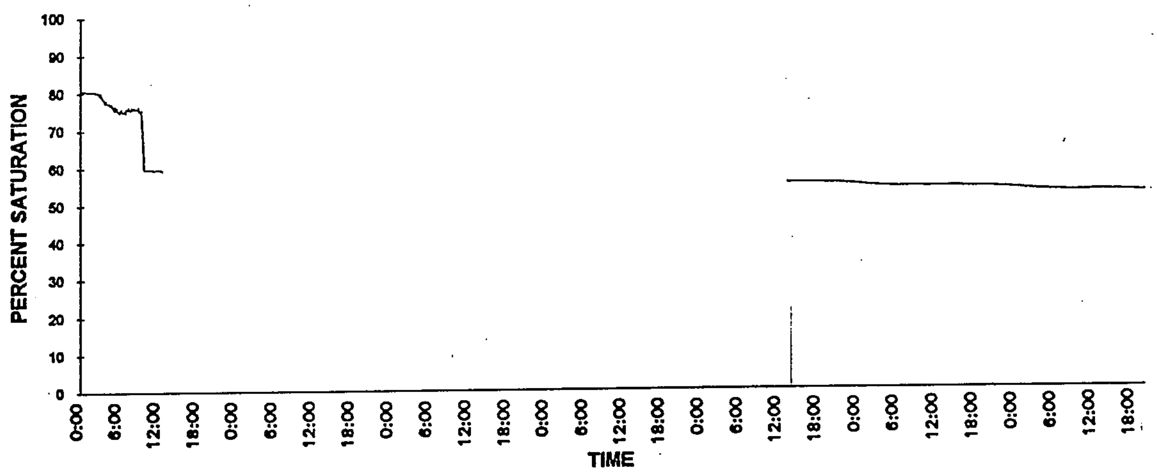


Highway 79
Poor Drainage Location
5/16/94 - 5/22/94

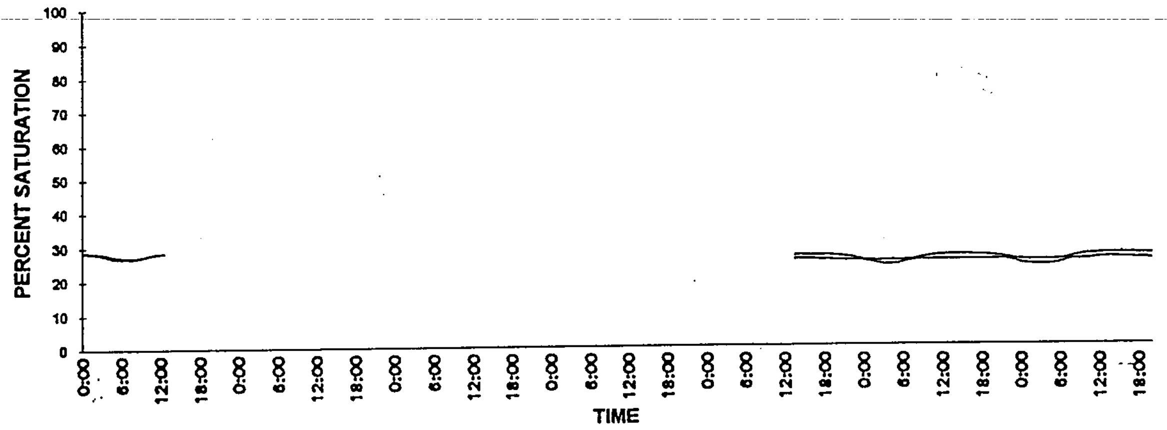
RAINFALL



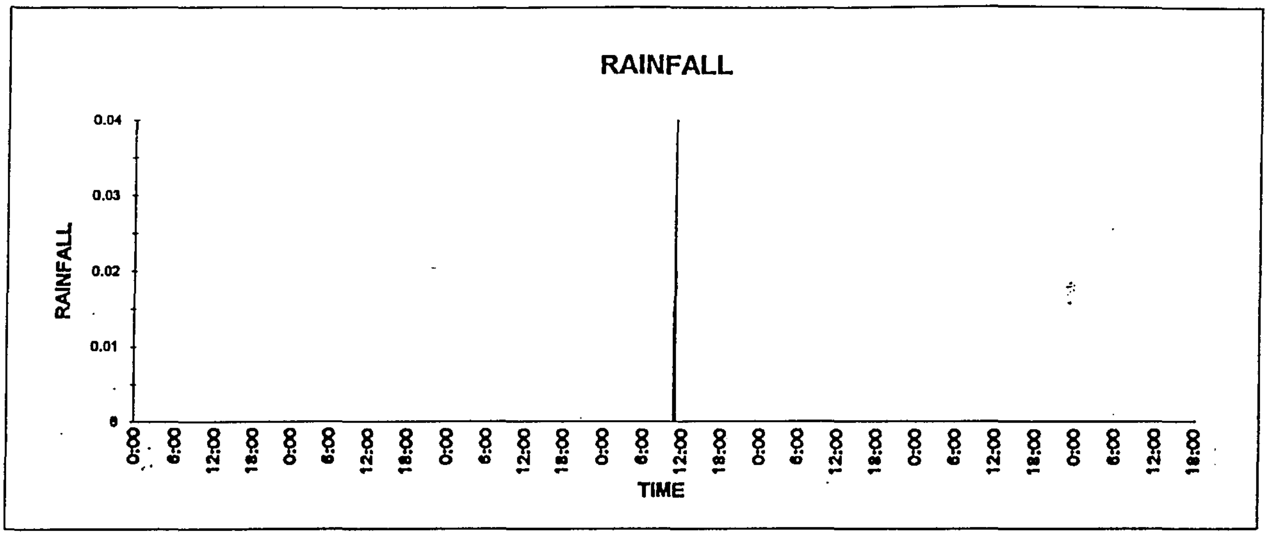
SUBGRADE SATURATION

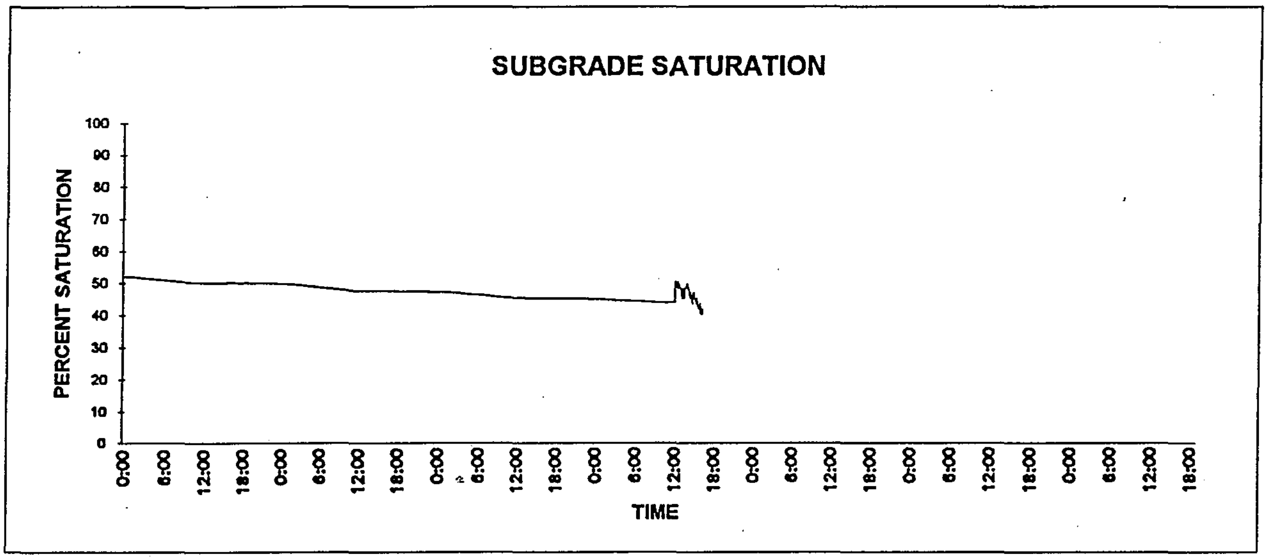


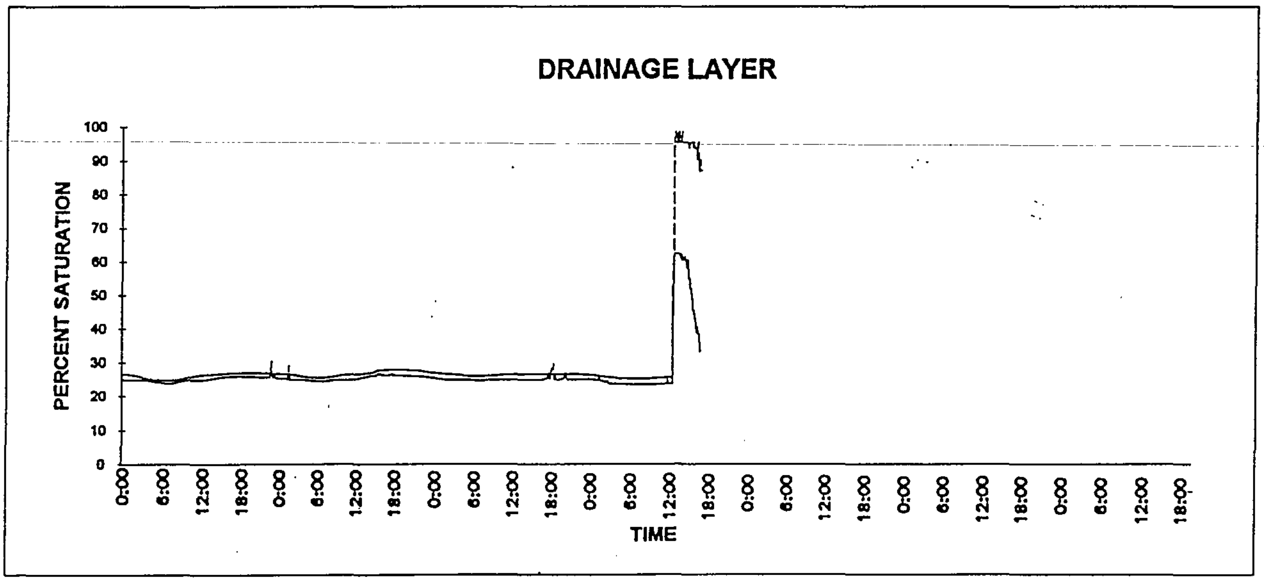
DRAINAGE LAYER

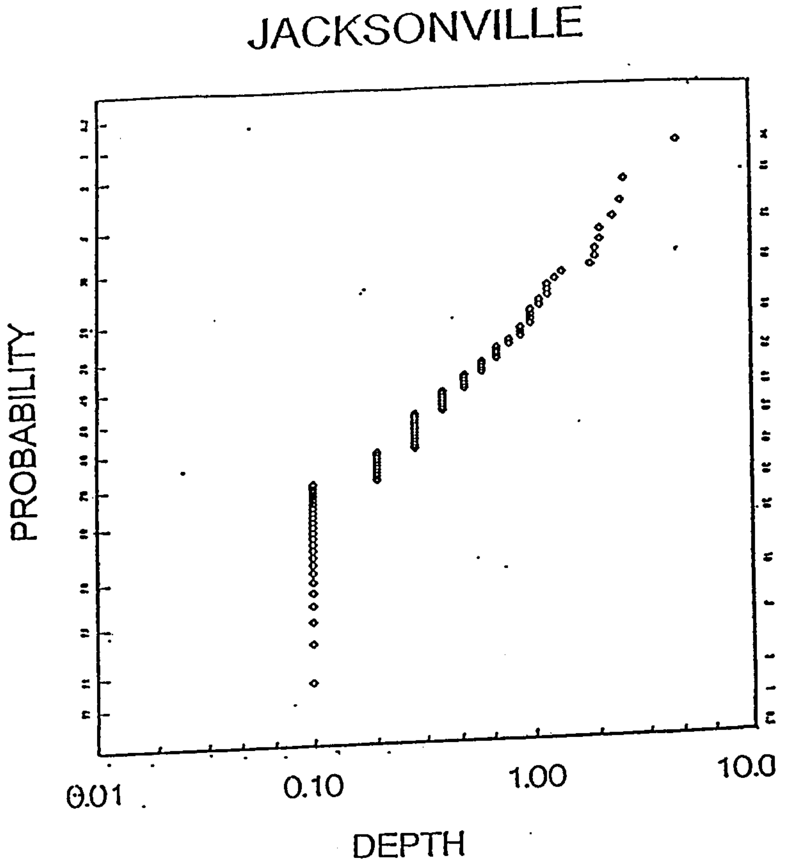


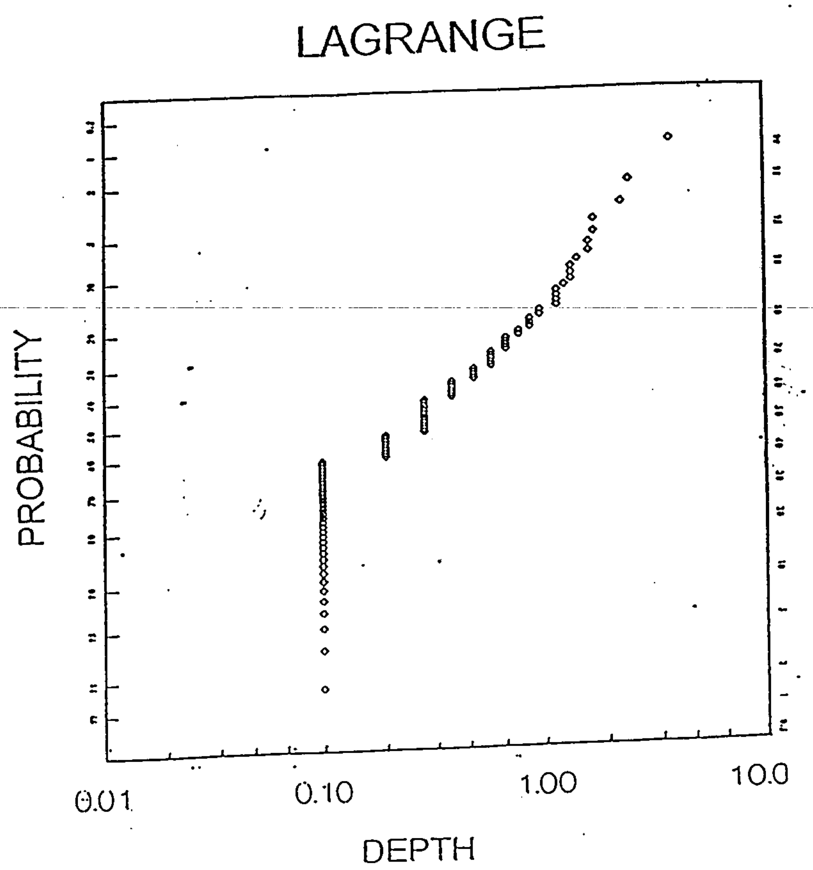
Highway 79
Poor Drainage Location
5/23/94 - 5/29/94



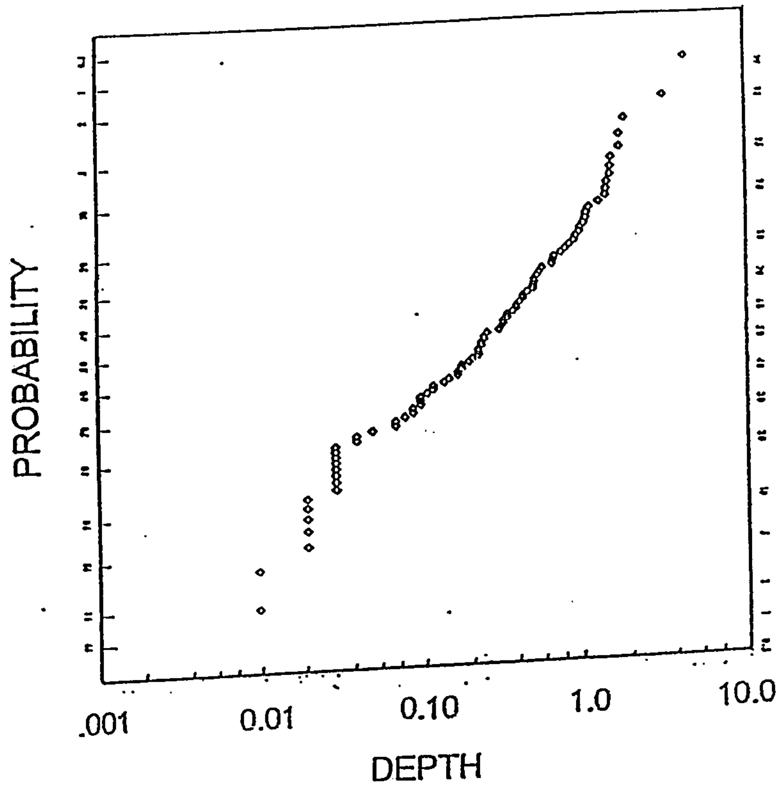




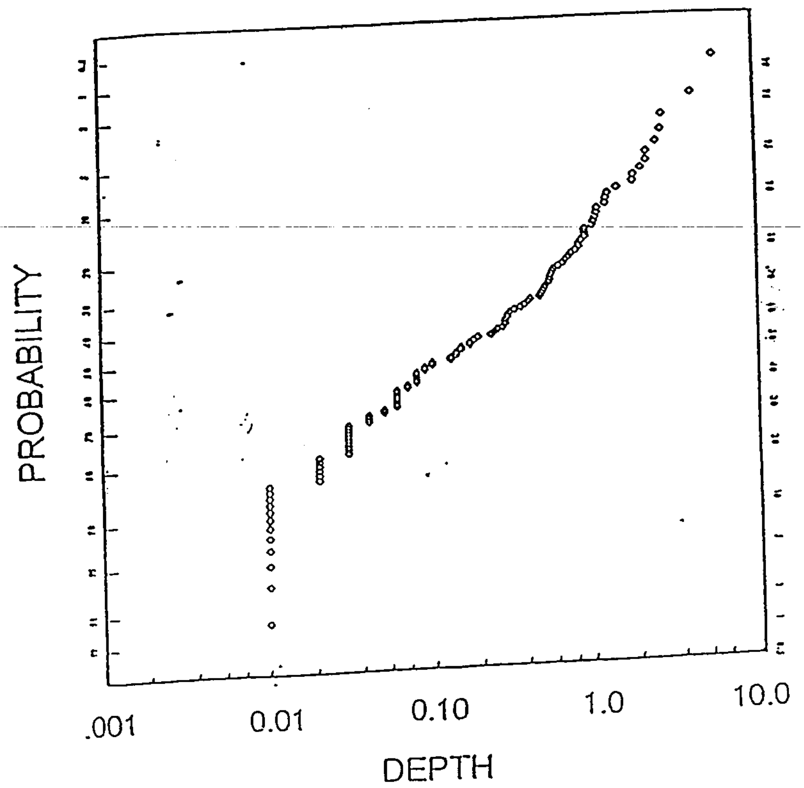

JACKSONVILLE
PROBABILITY
0.01
0.10
1.00
10.0
DEPTH


LAGRANGE
PROBABILITY
0.01
0.10
1.00
10.0
DEPTH

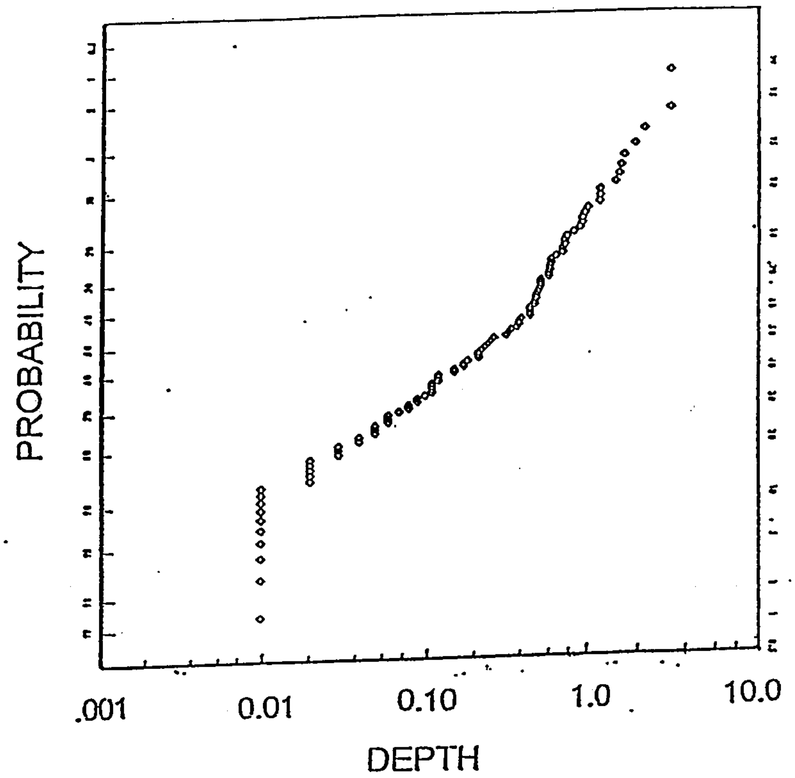
MERIDIAN



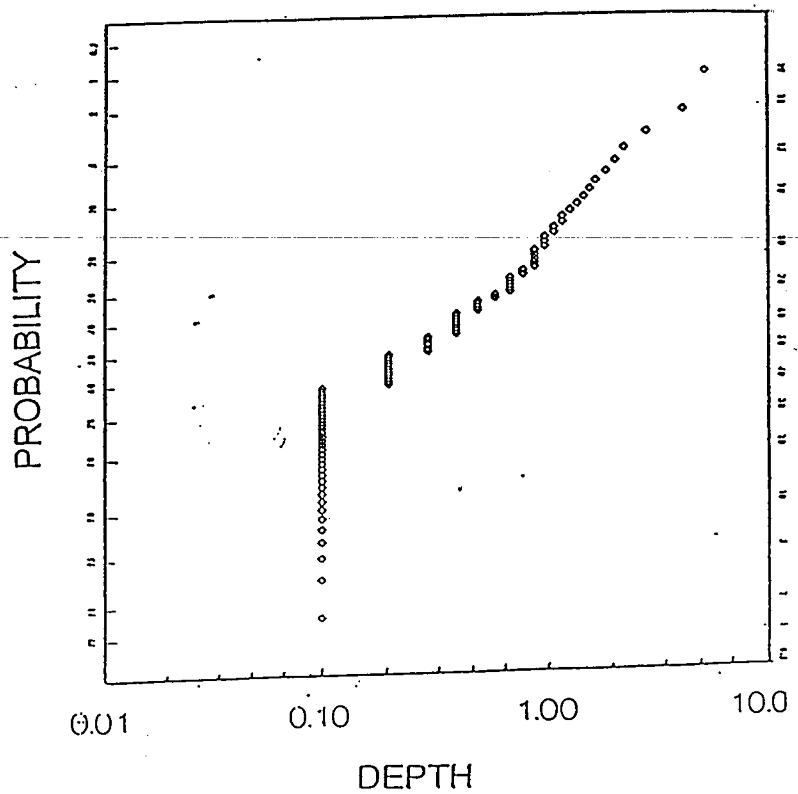
MOBILE



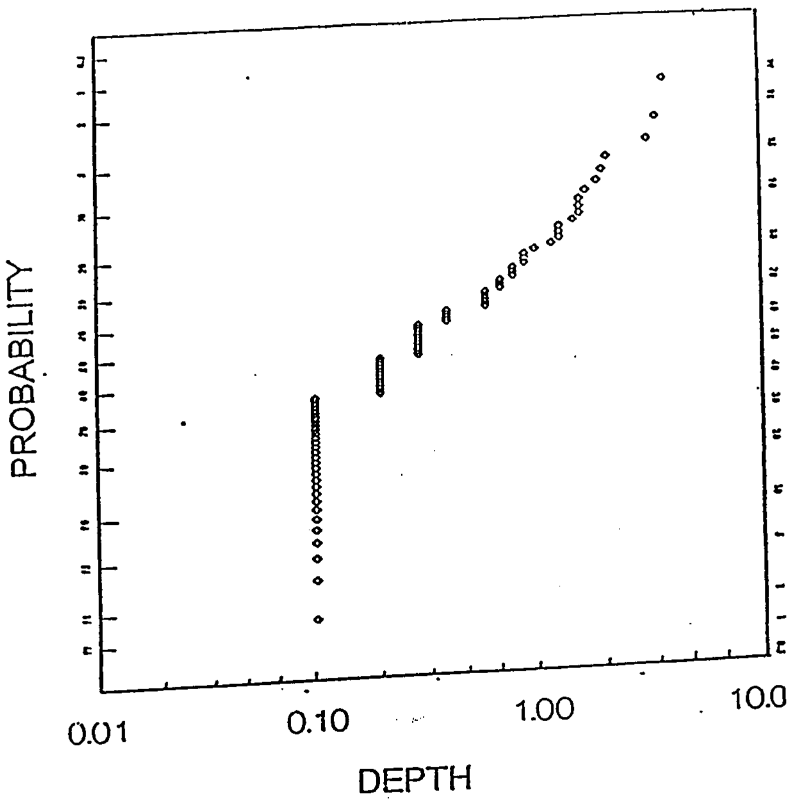
MONTGOMERY



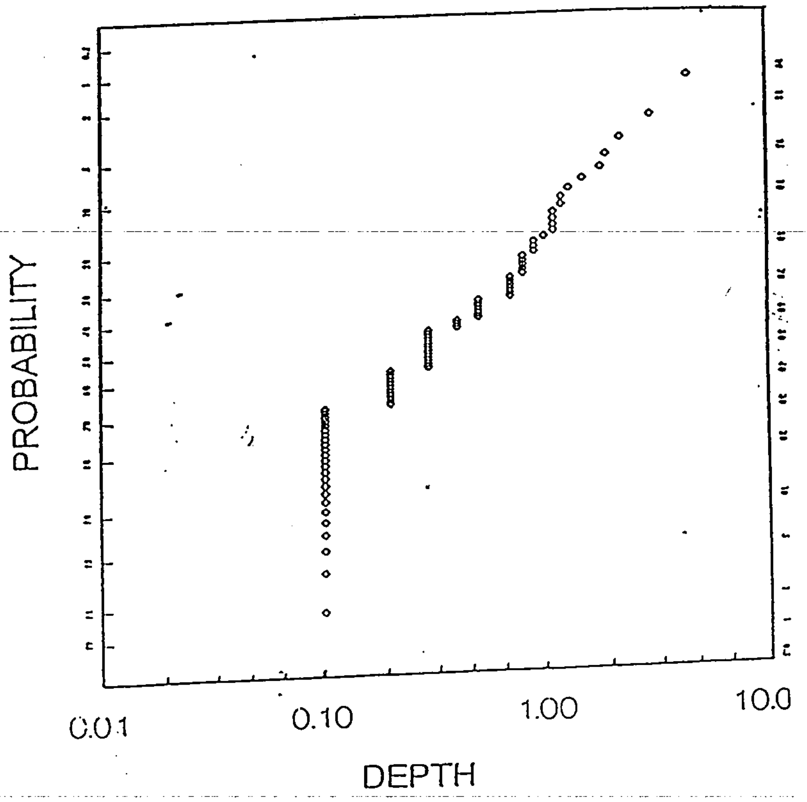
PETERMAN



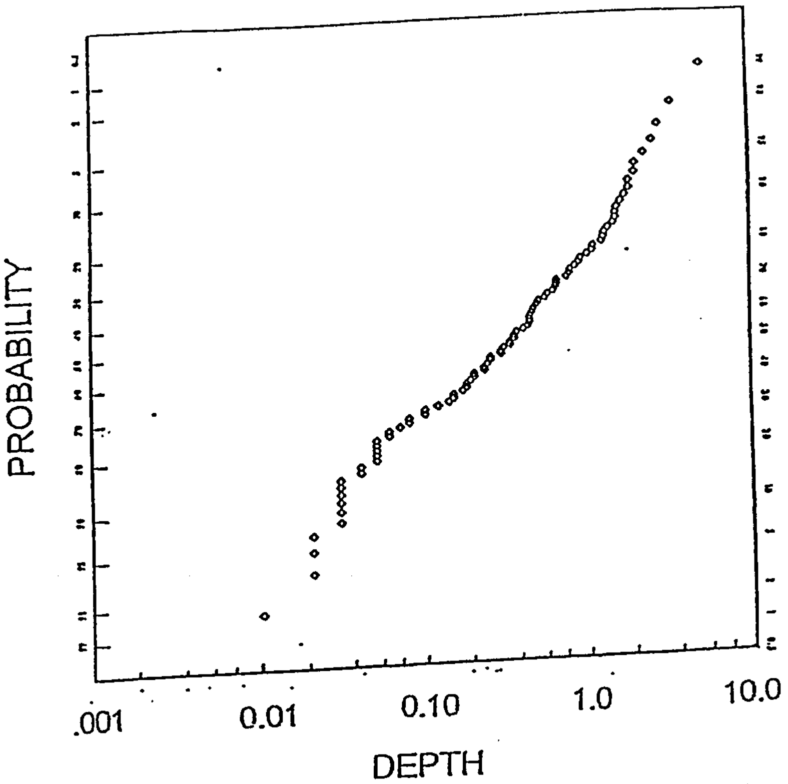
THOMASVILLE



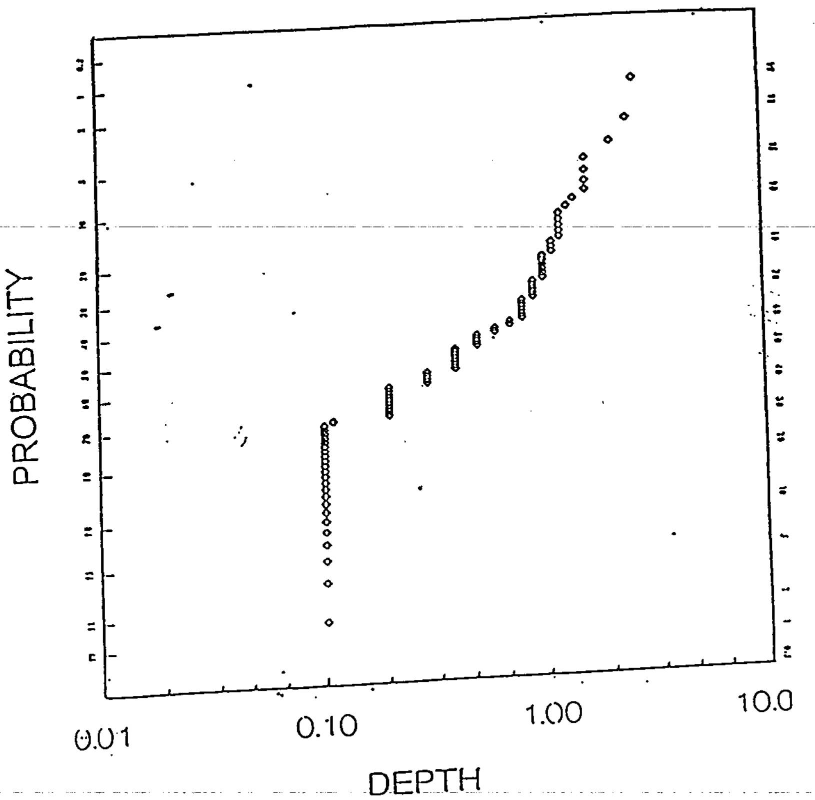
TROY

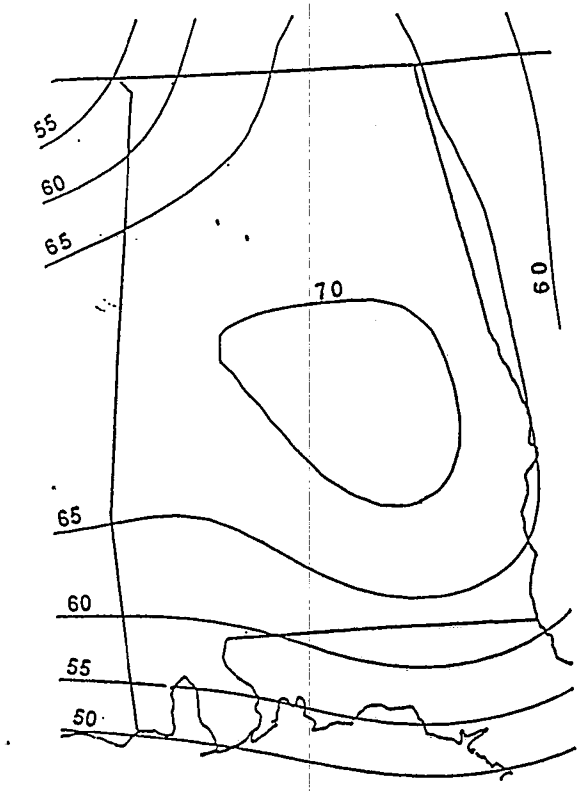


TUSCALOOSA

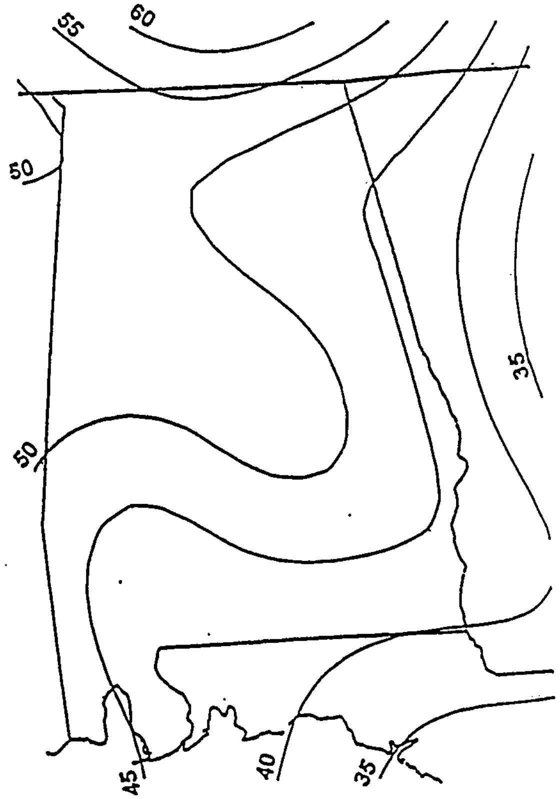


VERNON

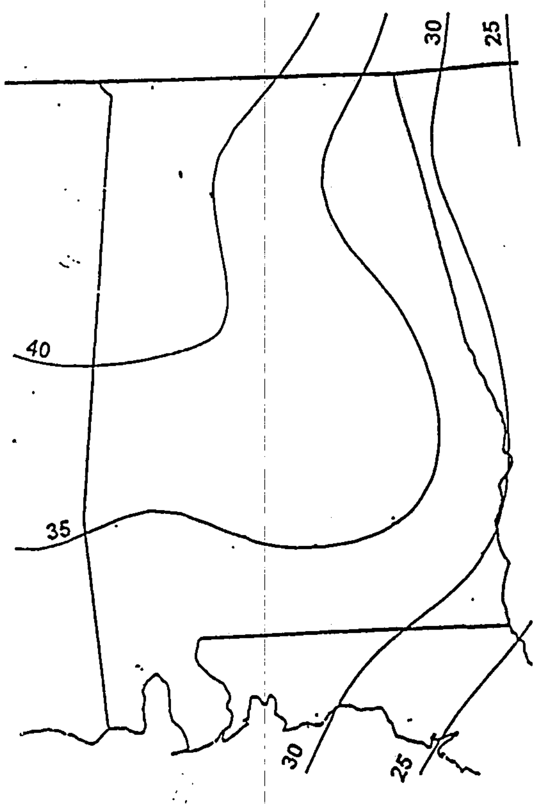




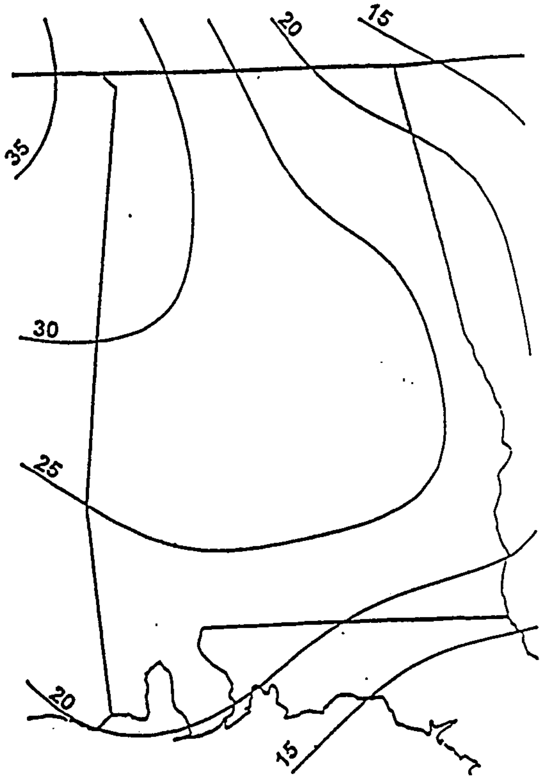
PROBABILITY OF RAIN DEPTH 0.10" OR GREATER



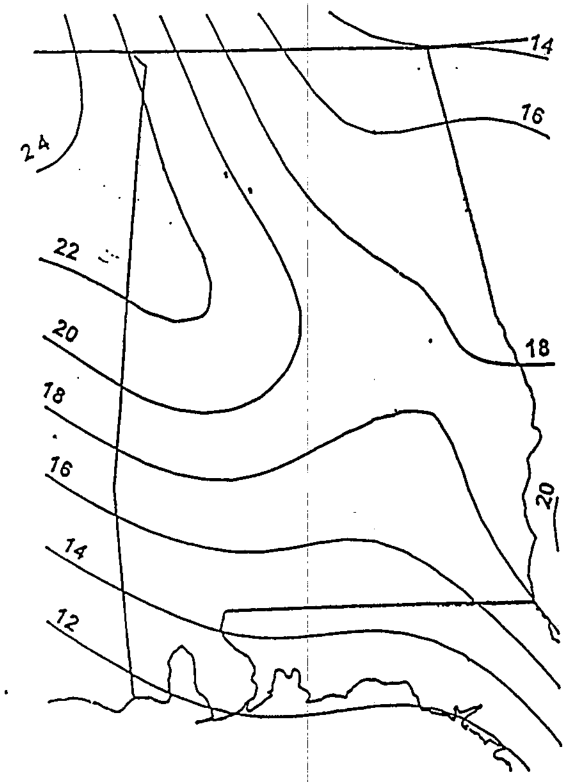
PROBABILITY OF RAIN DEPTH 0.25" OR GREATER



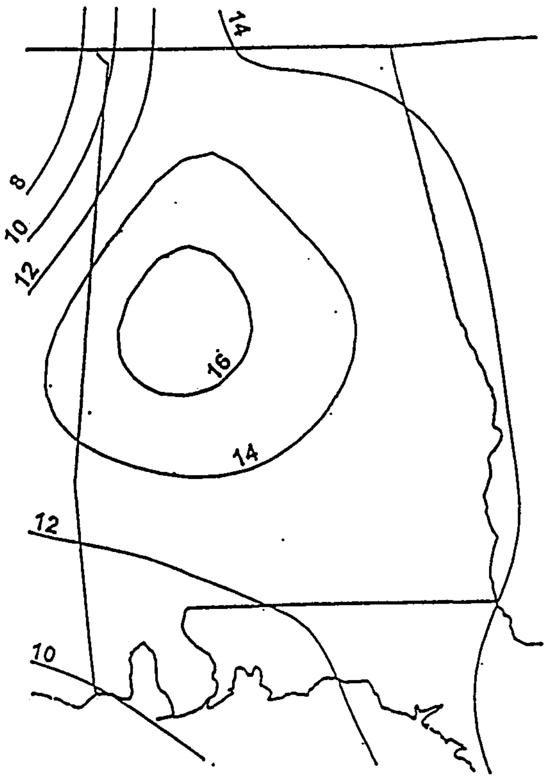
PROBABILITY OF RAIN DEPTH 0.50" OR GREATER



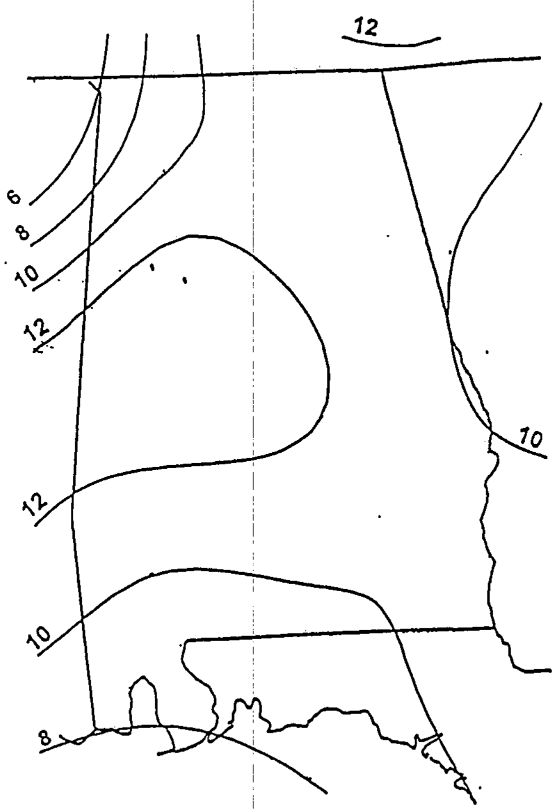
PROBABILITY OF RAIN DEPTH 0.75" OR GREATER



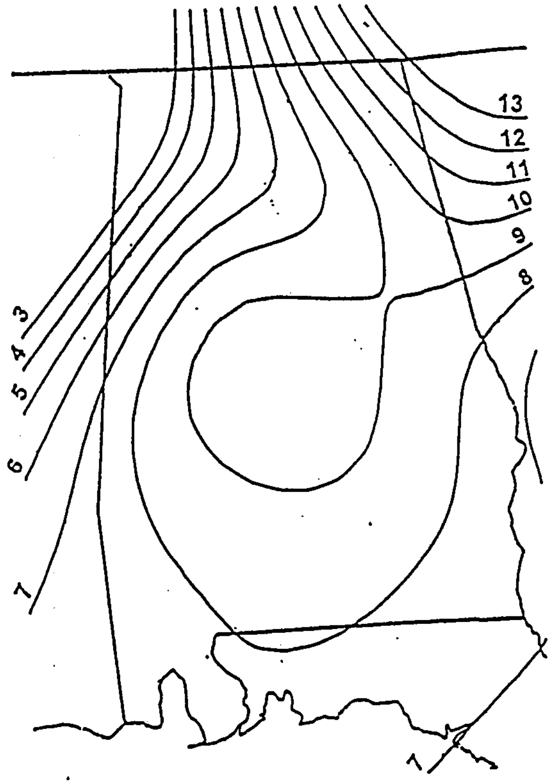
PROBABILITY OF RAIN DEPTH 1.00" OR GREATER



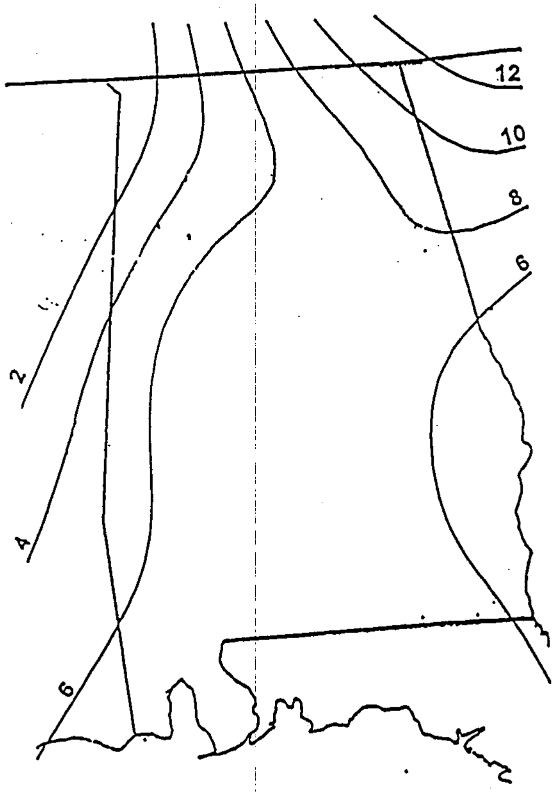
PROBABILITY OF RAIN DEPTH 1.25" OR GREATER



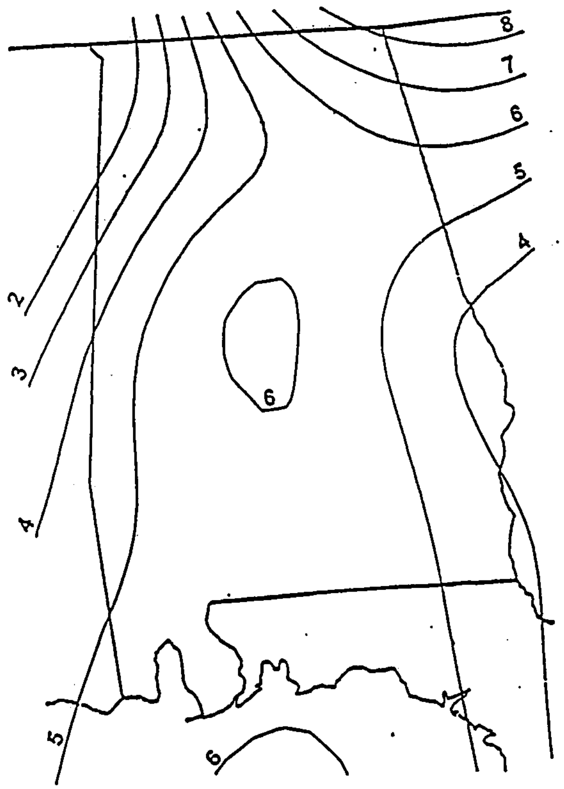
PROBABILITY OF RAIN DEPTH 1.50" OR GREATER



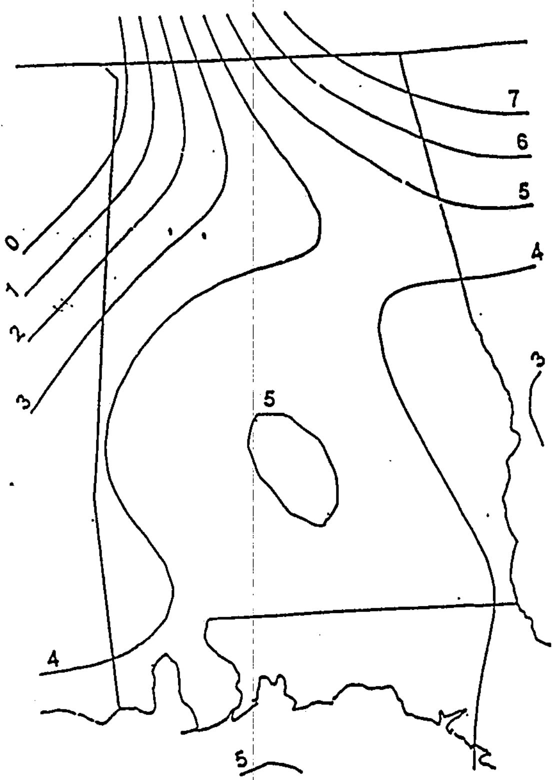
PROBABILITY OF RAIN DEPTH 1.75" OR GREATER



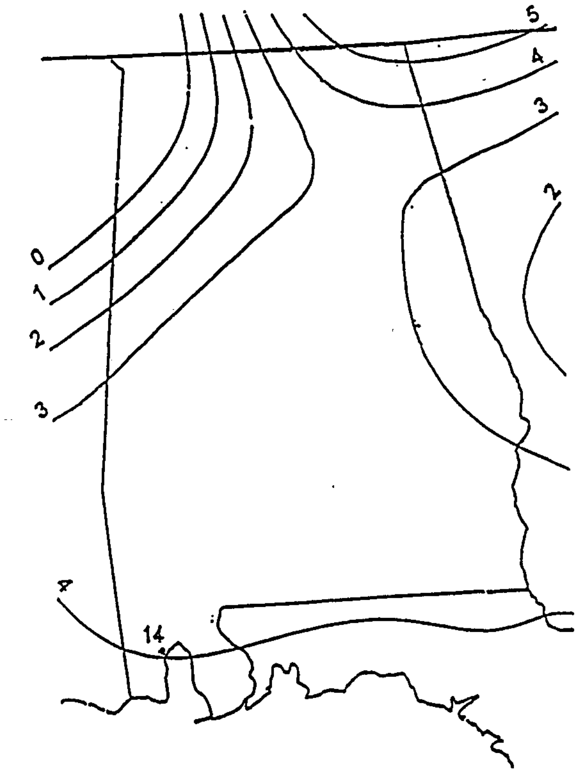
PROBABILITY OF RAIN DEPTH 2.00" OR GREATER



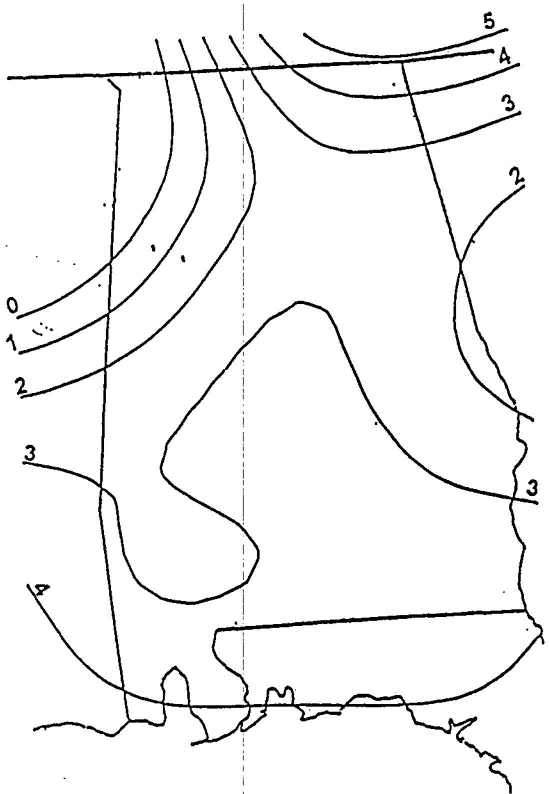
PROBABILITY OF RAIN DEPTH 2.25" OR GREATER



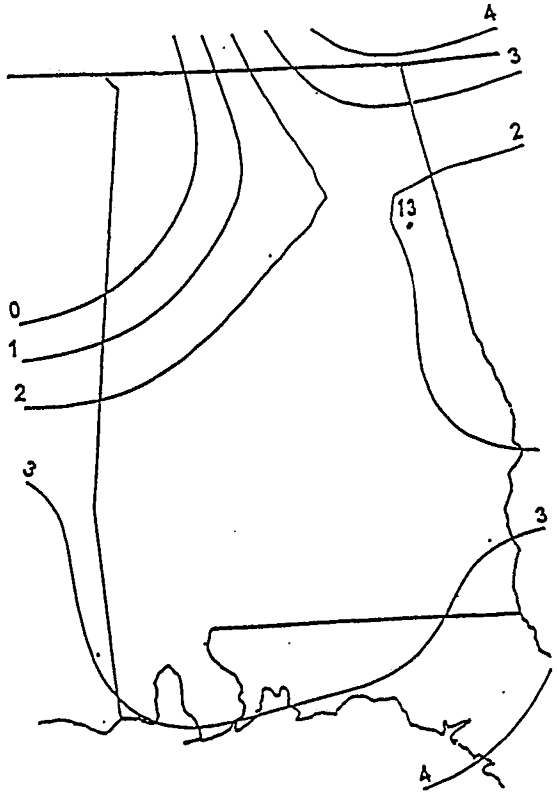
PROBABILITY OF RAIN DEPTH 2.50" OR GREATER



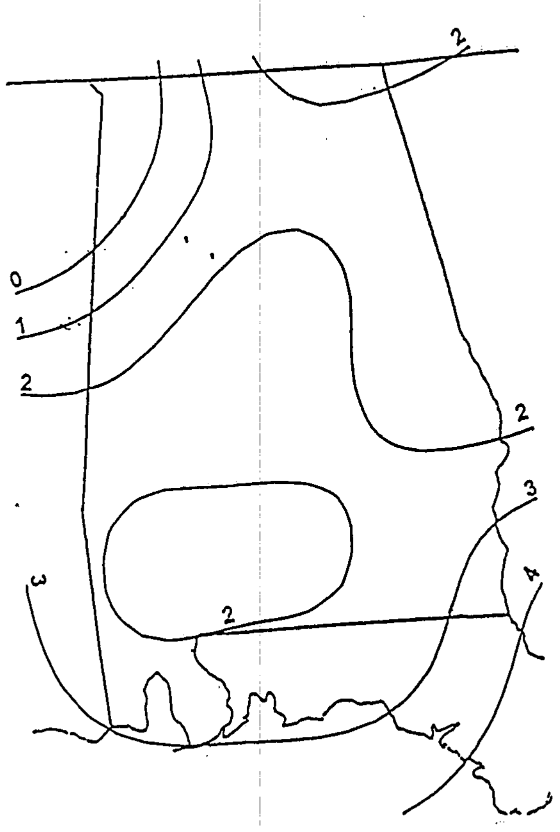
PROBABILITY OF RAIN DEPTH 2.75" OR GREATER



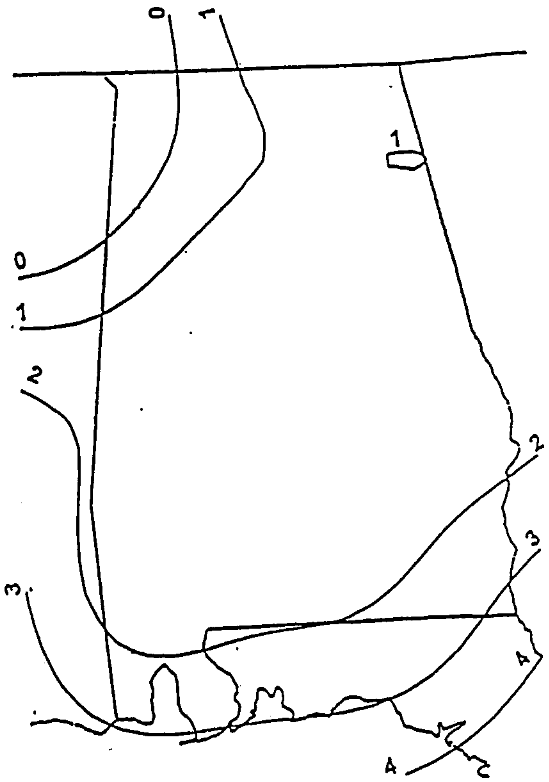
PROBABILITY OF RAIN DEPTH 3.00" OR GREATER



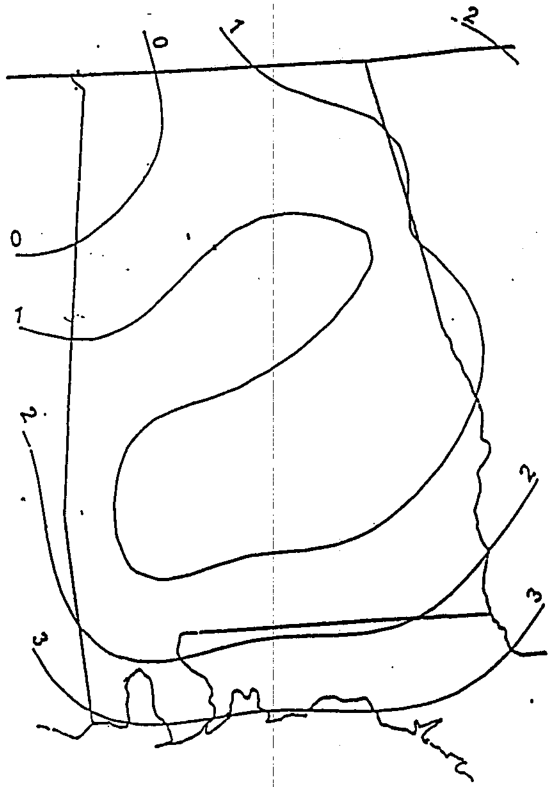
PROBABILITY OF RAIN DEPTH 3.25" OR GREATER



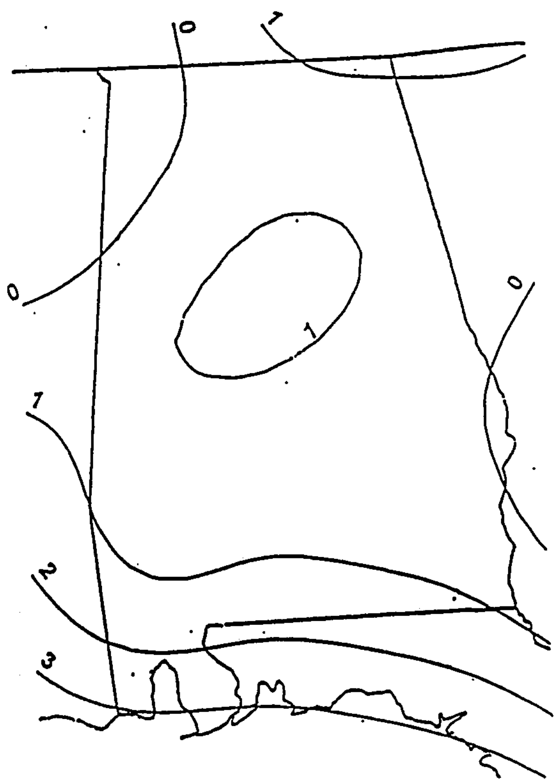
PROBABILITY OF RAIN DEPTH 3.50" OR GREATER



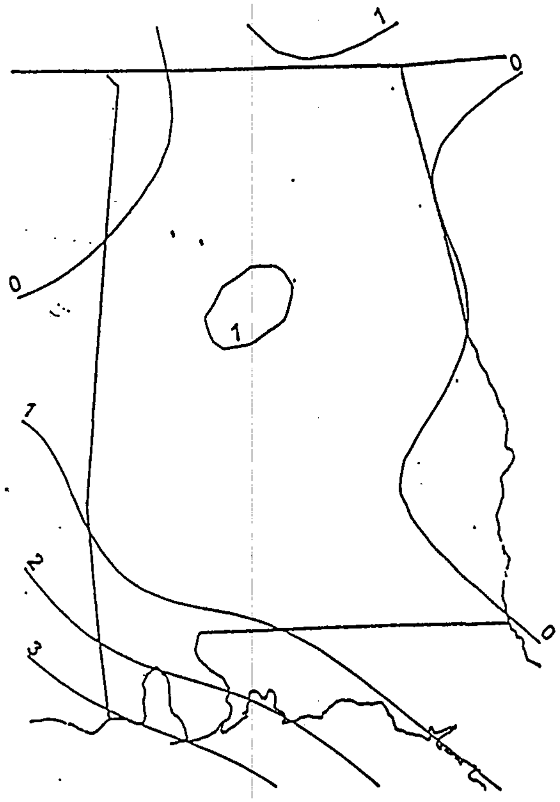
PROBABILITY OF RAIN DEPTH 3.75" OR GREATER



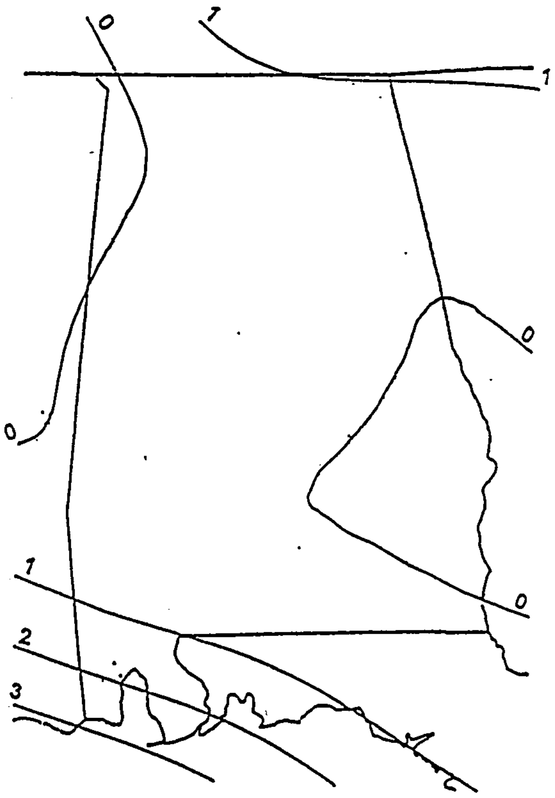
PROBABILITY OF RAIN DEPTH 4.00" OR GREATER



PROBABILITY OF RAIN DEPTH 4.25" OR GREATER

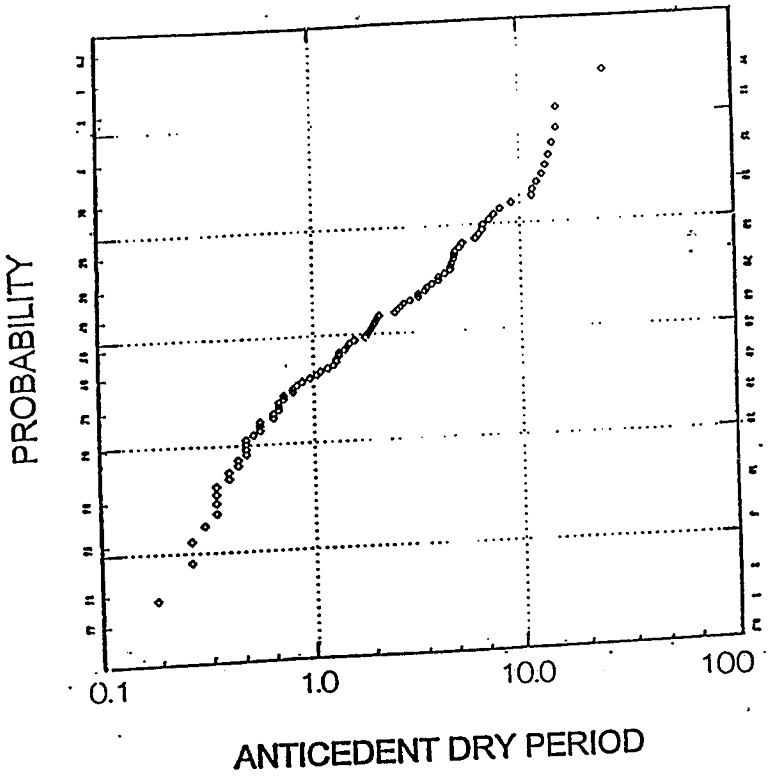


PROBABILITY OF RAIN DEPTH 4.50" OR GREATER

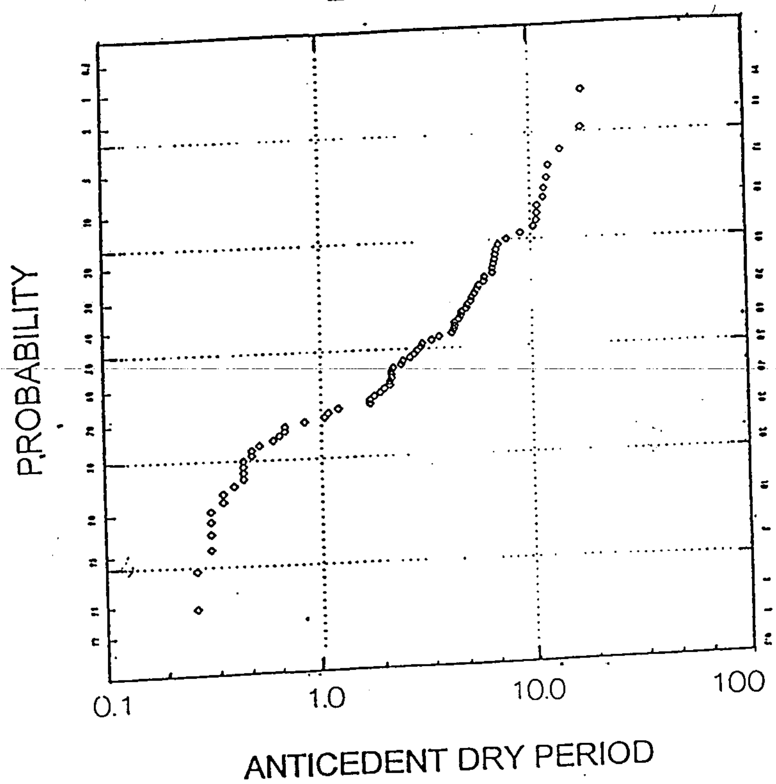


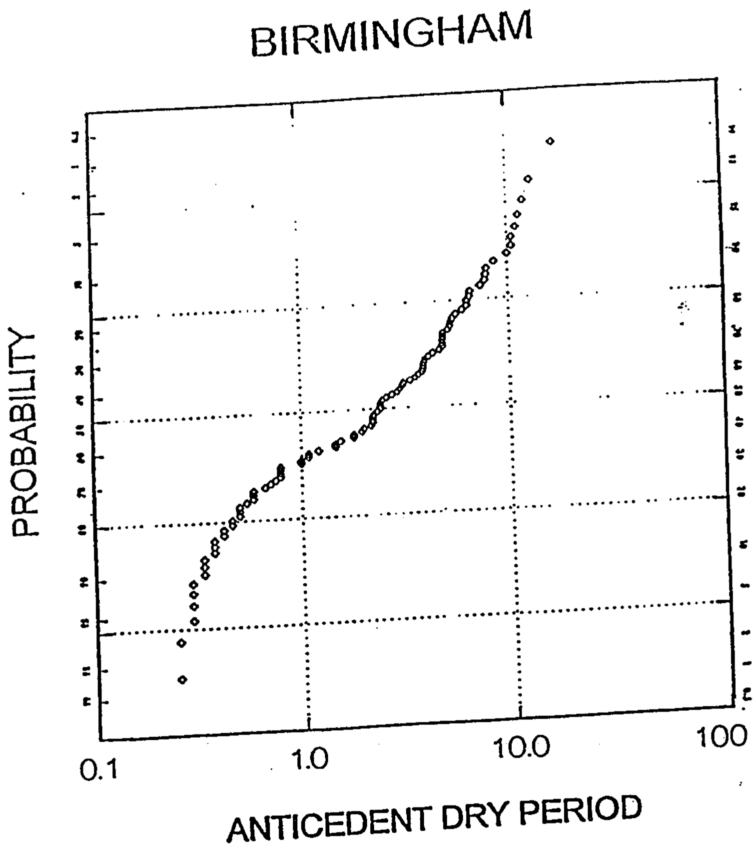
PROBABILITY OF RAIN DEPTH 5.00" OR GREATER

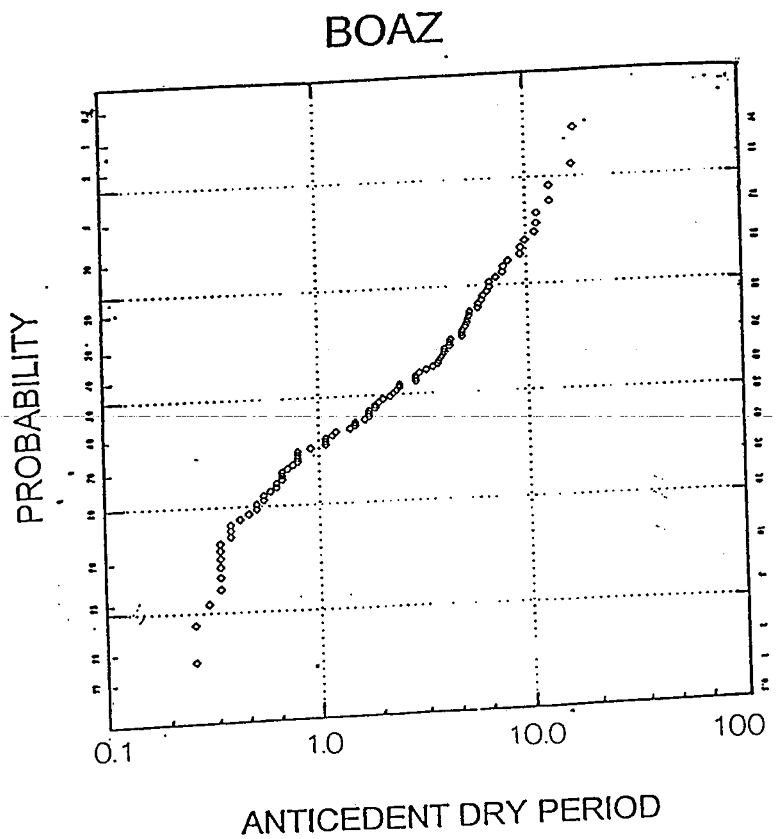
ABBEVILLE



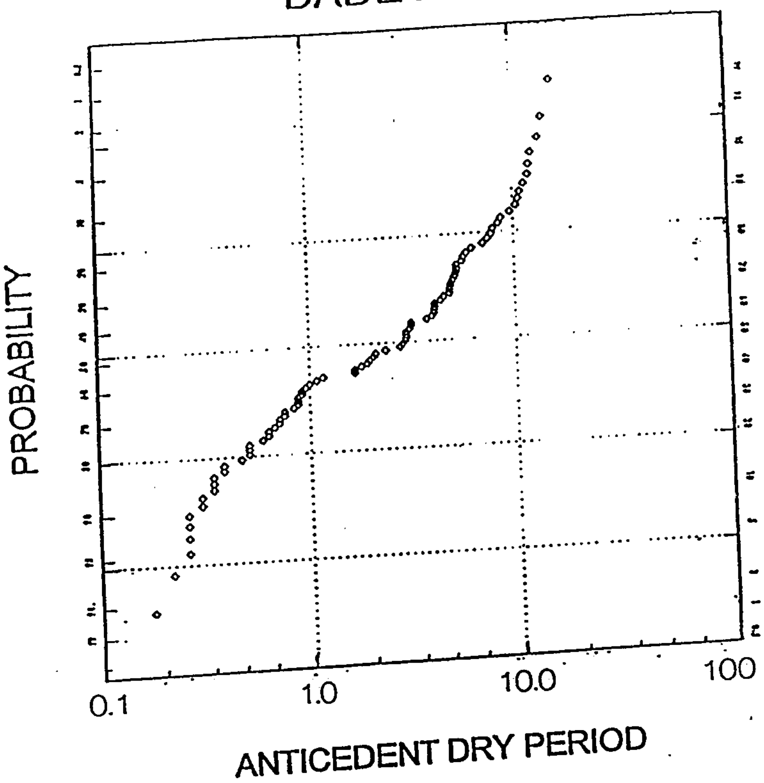
BERRY



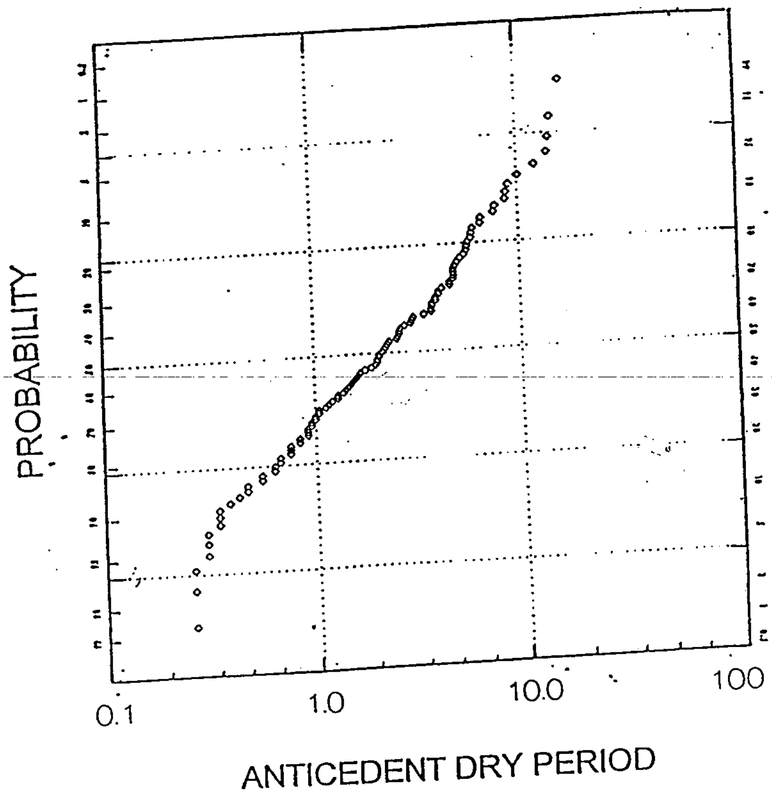

BIRMINGHAM
PROBABILITY
0.1
1.0
10.0
100
ANTICEDENT DRY PERIOD


BOAZ
PROBABILITY
0.1
1.0
10.0
100
ANTICEDENT DRY PERIOD

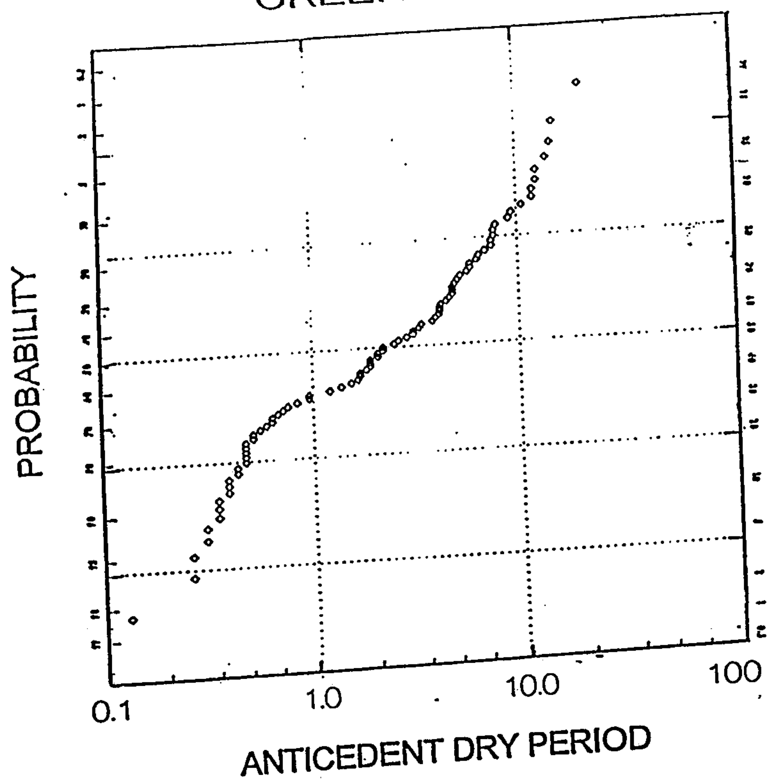
DADEVILLE



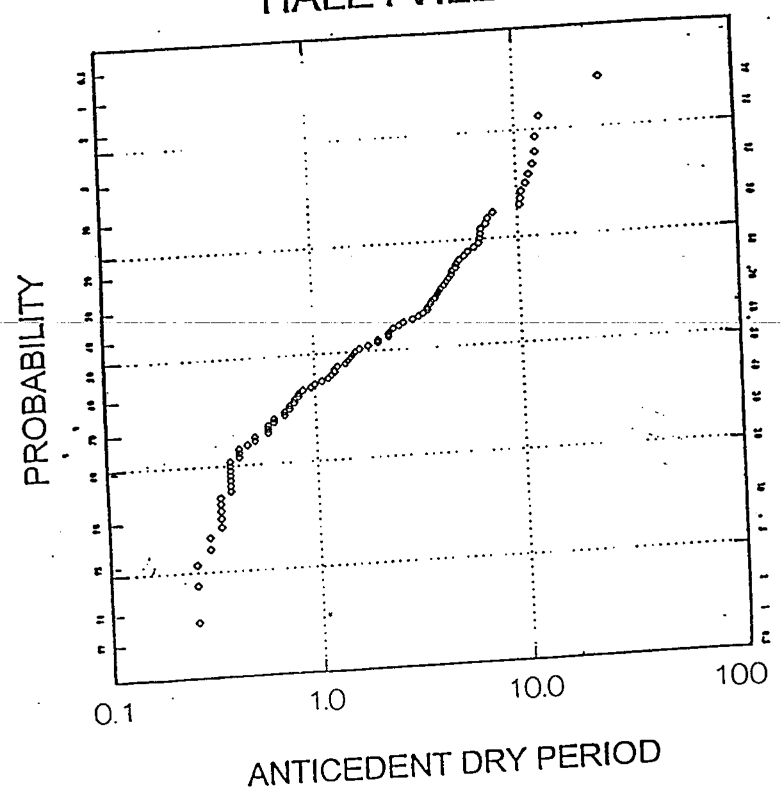
DOTHAN



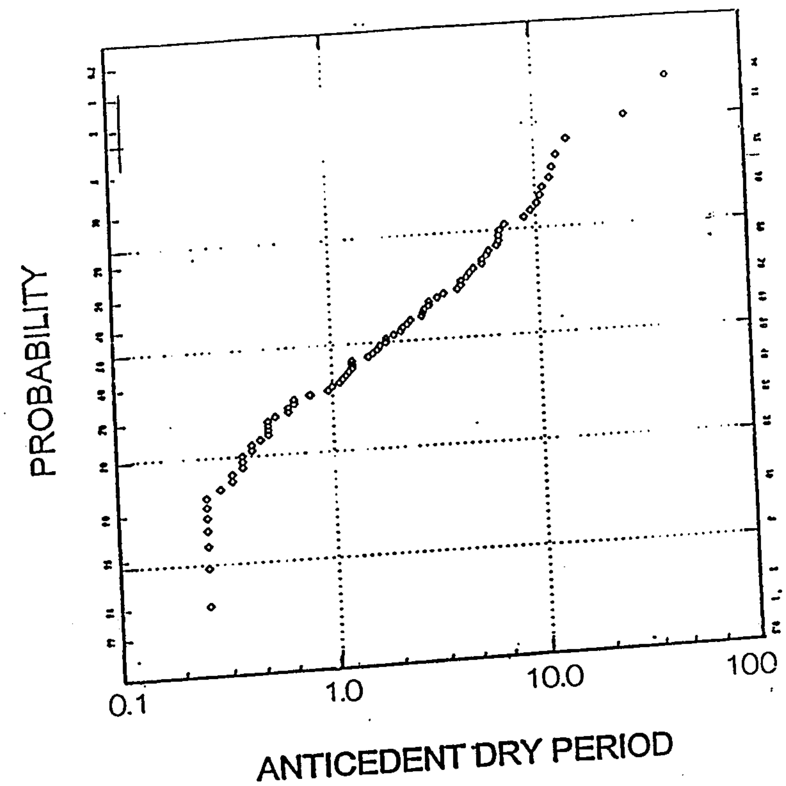
GREENVILLE



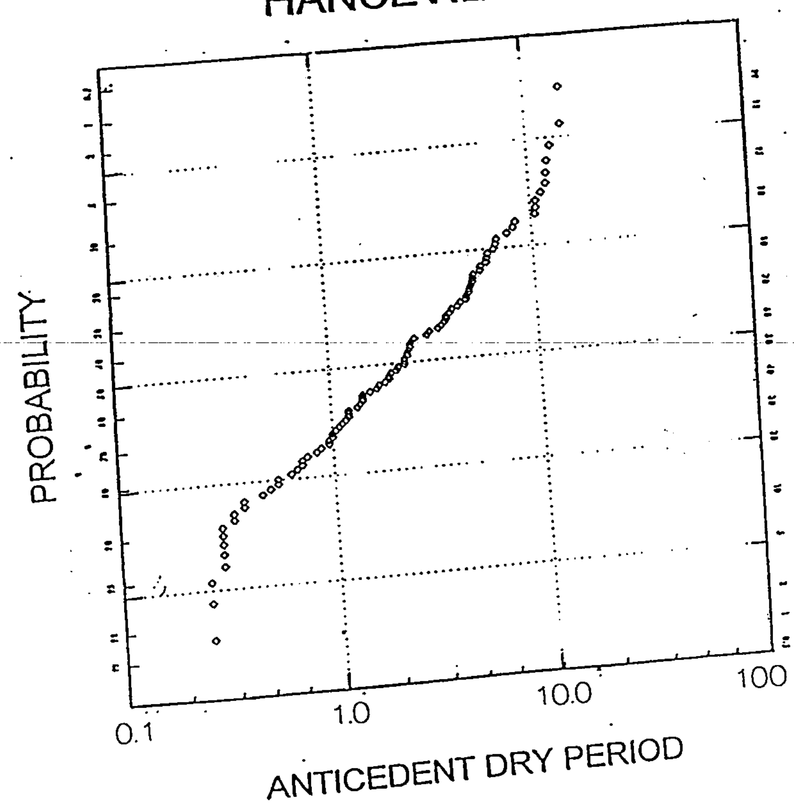
HALEYVILLE

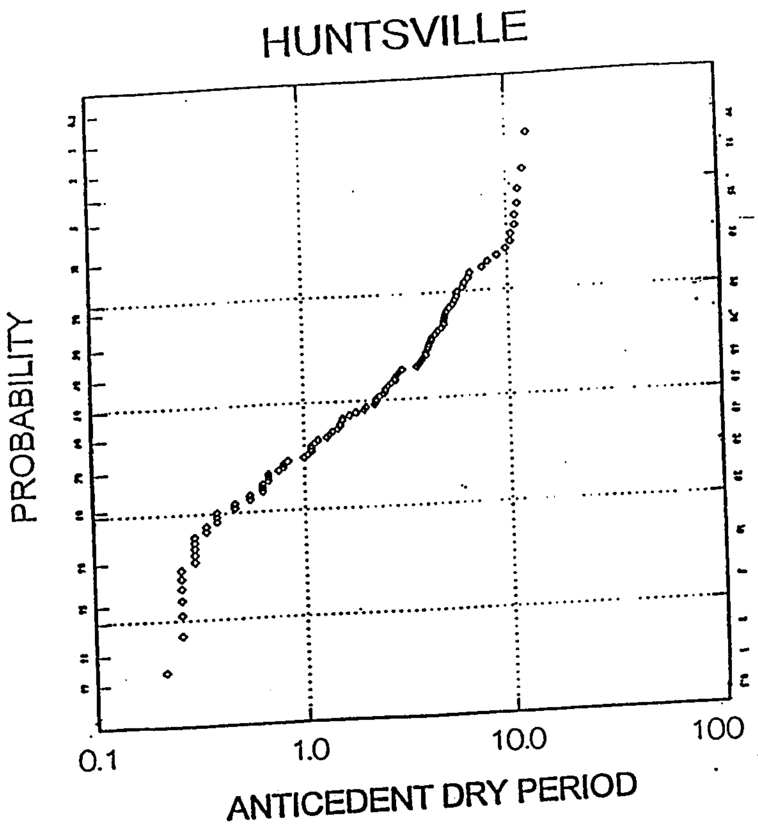


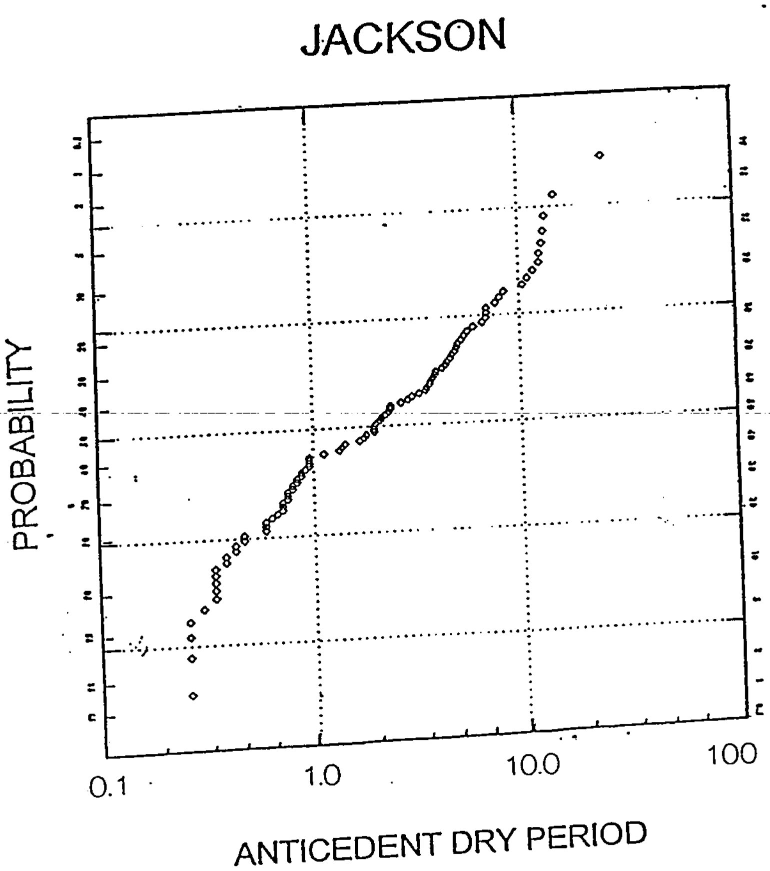
HAMILTON



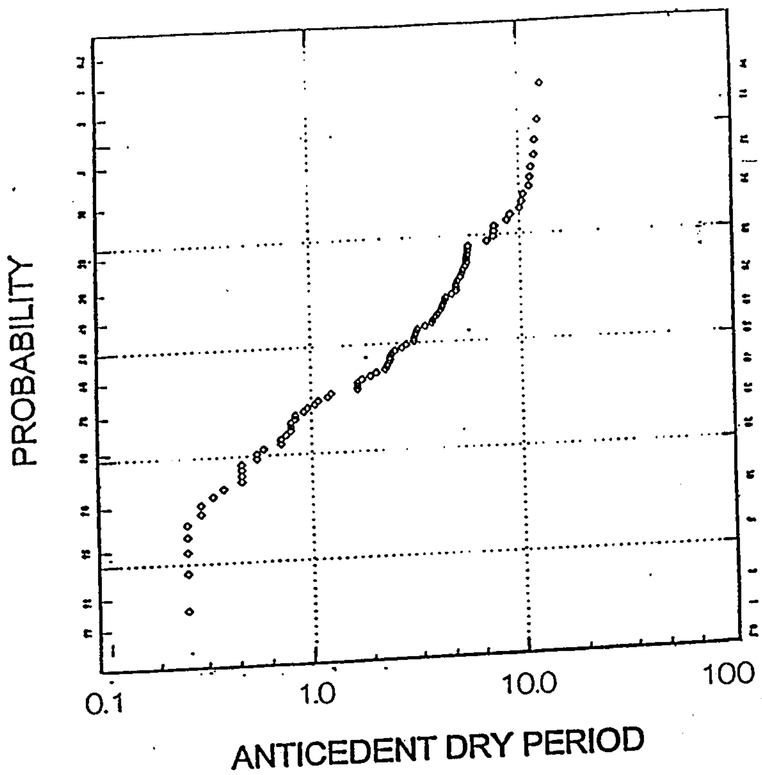
HANCEVILLE



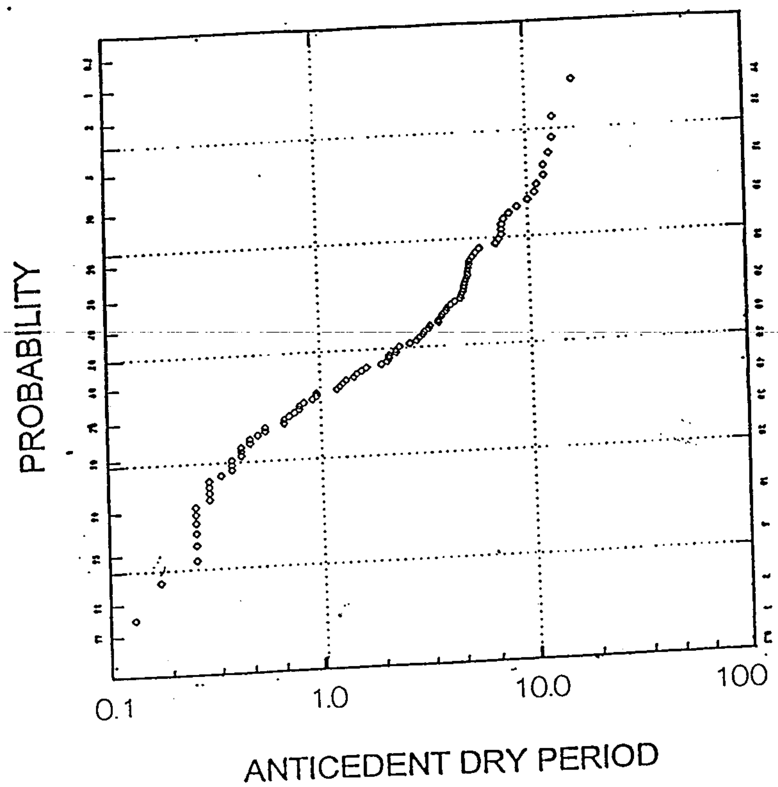

HUNTSVILLE
PROBABILITY
0.1
1.0
10.0
100
ANTICEDENT DRY PERIOD

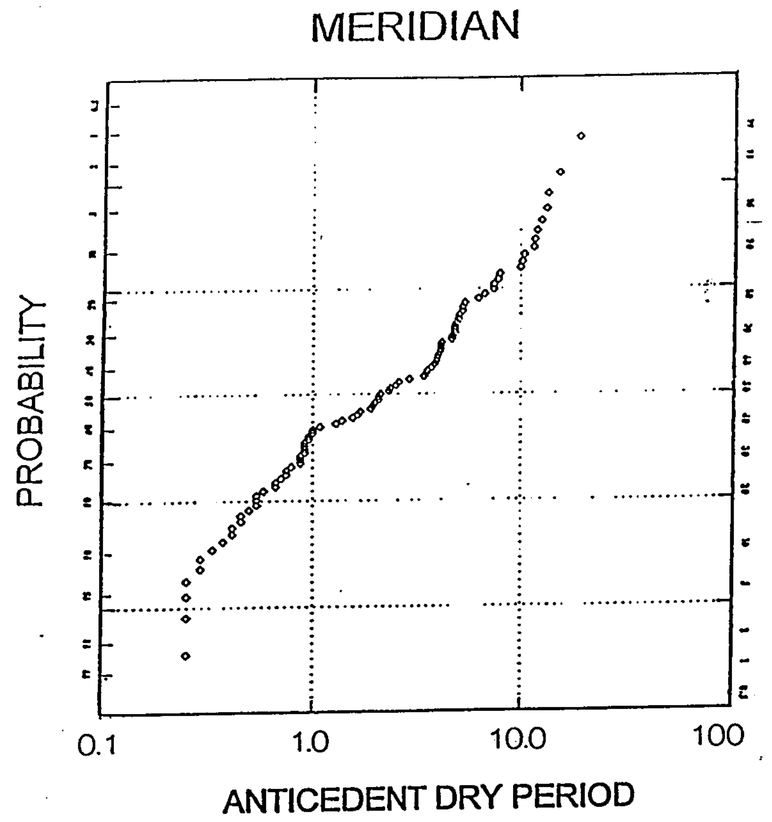

JACKSON
PROBABILITY
0.1
1.0
10.0
100
ANTICEDENT DRY PERIOD

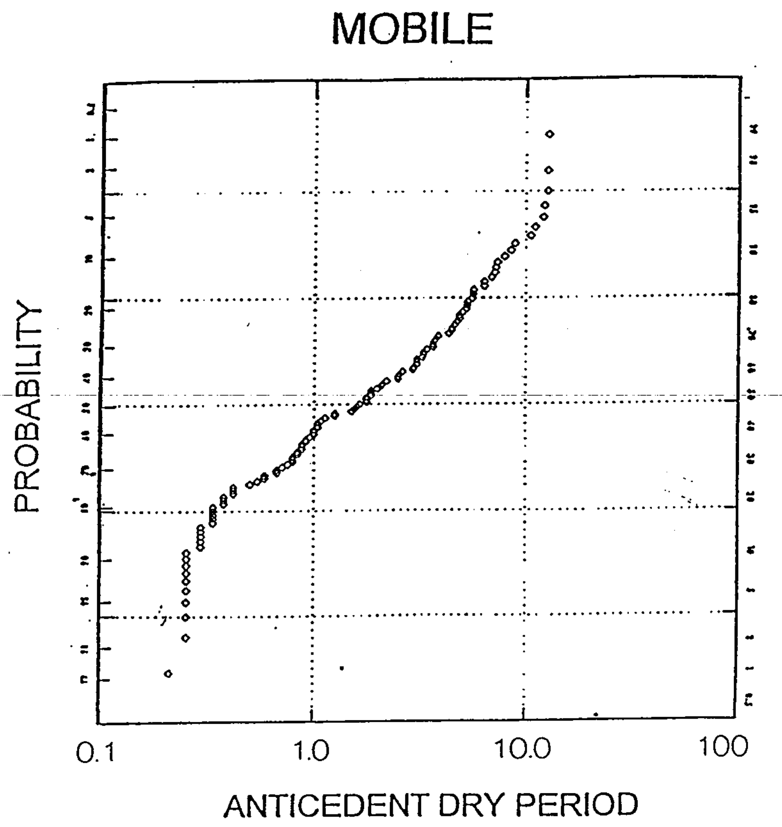
JACKSONVILLE



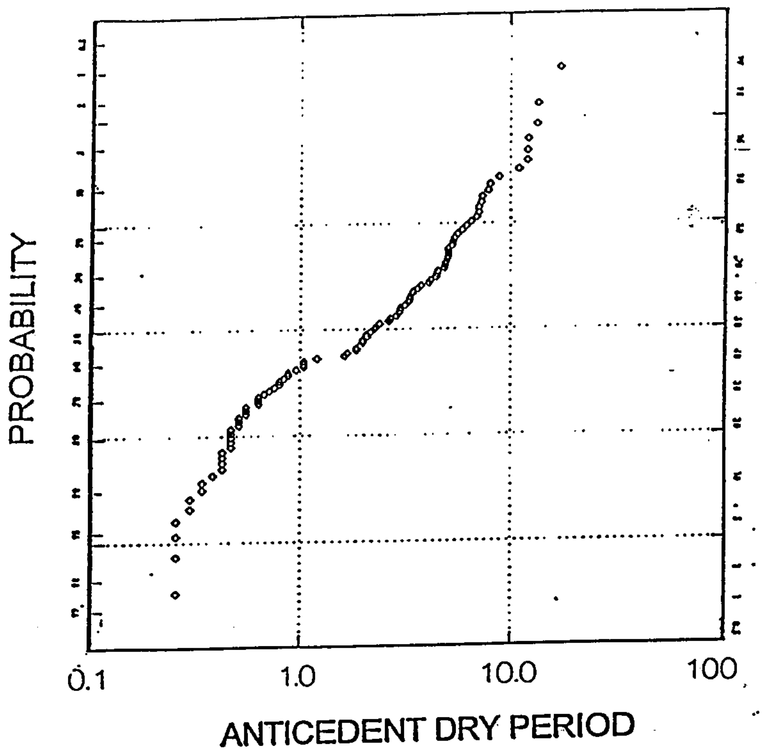
LAGRANGE



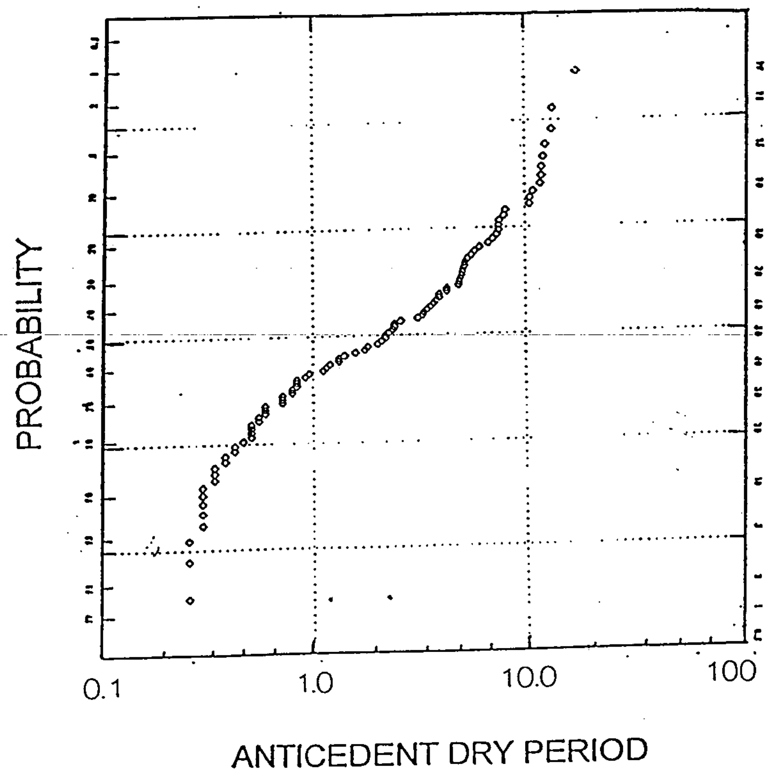

MERIDIAN
PROBABILITY
0.1
1.0
10.0
100
ANTICEDENT DRY PERIOD


MOBILE
PROBABILITY
0.1
1.0
10.0
100
ANTICEDENT DRY PERIOD

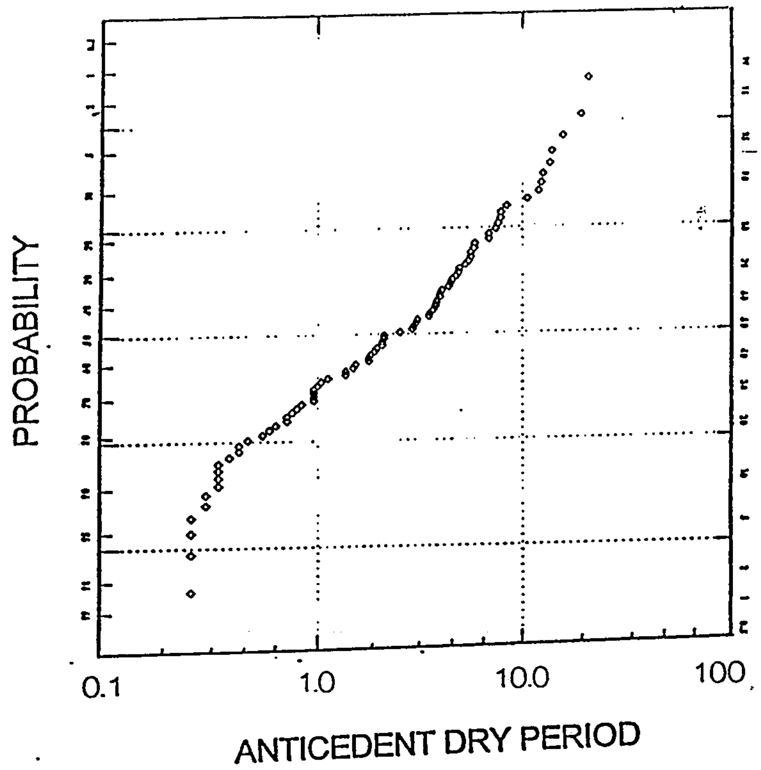
MONTGOMERY



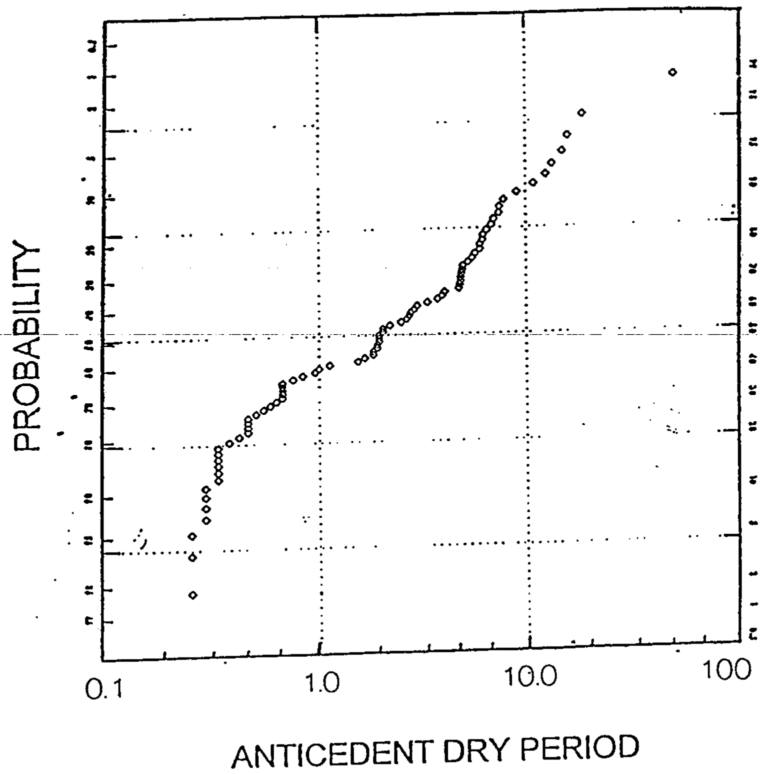
PETERMAN



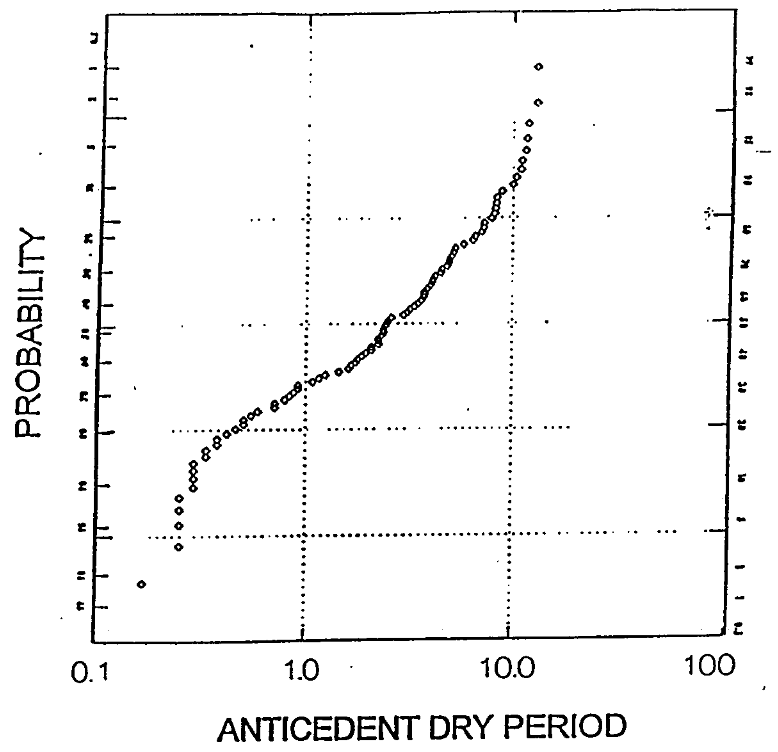
THOMASVILLE



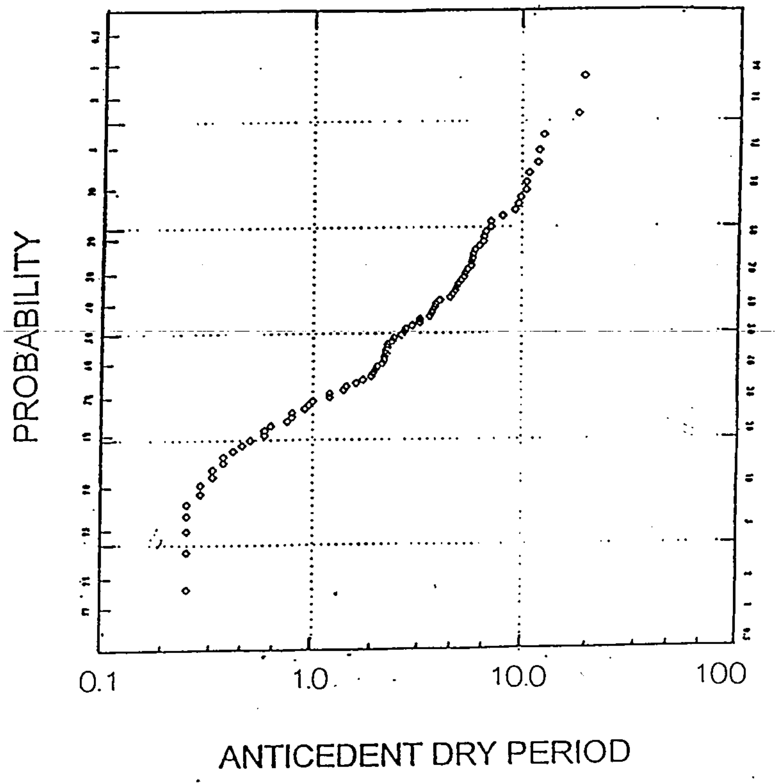
TROY

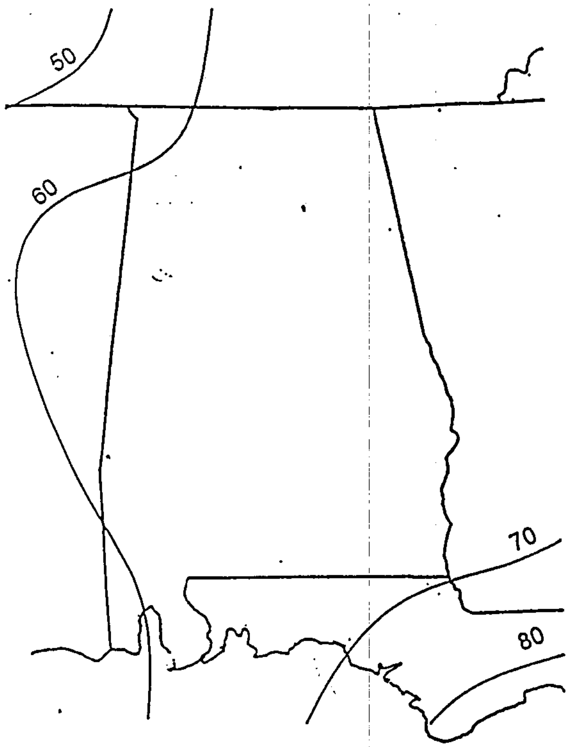


TUSCALOOSA

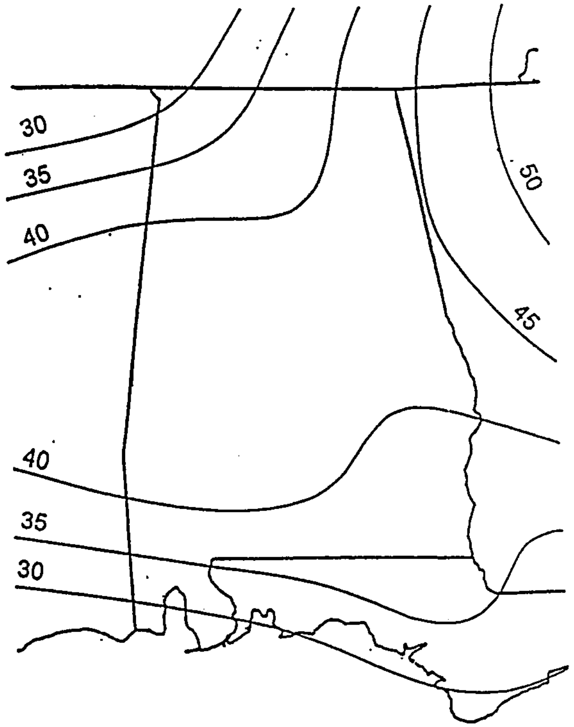


VERNON

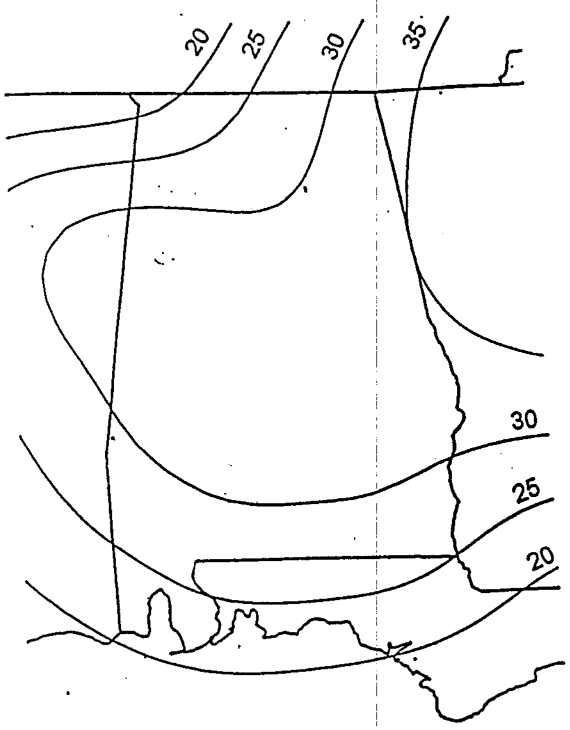




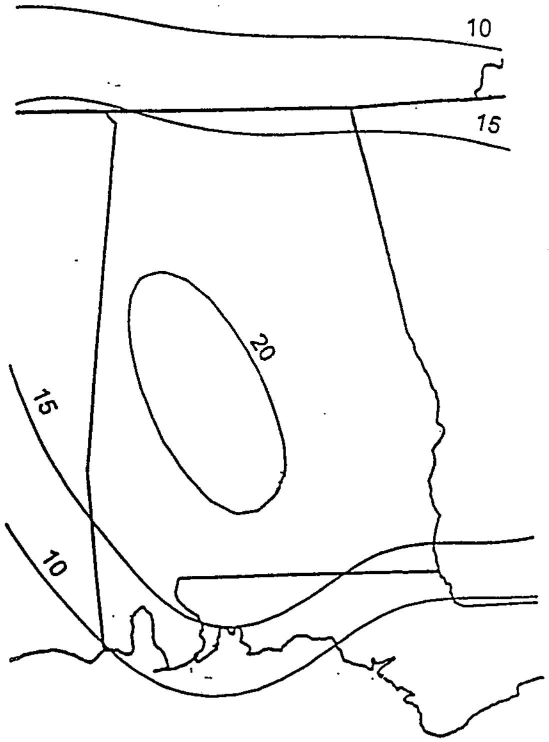
PROBABILITY CONTOUR FOR
1 DAY ANTICEDENT PERIOD



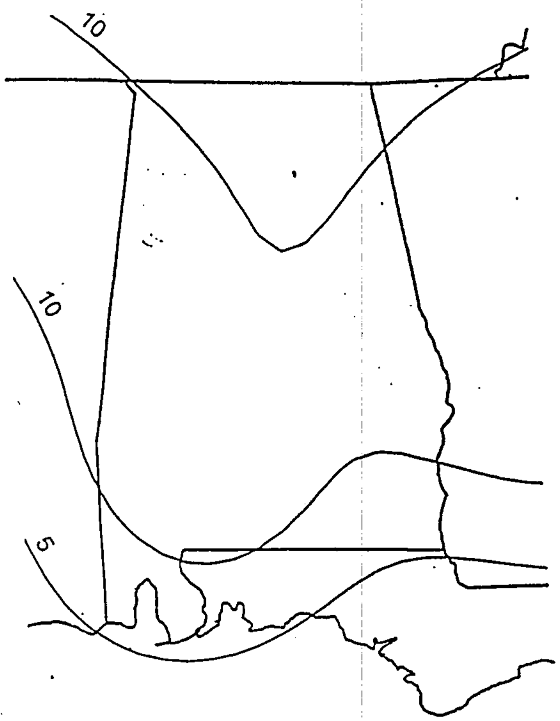
PROBABILITY CONTOUR FOR
3 DAY ANTICEDENT PERIOD



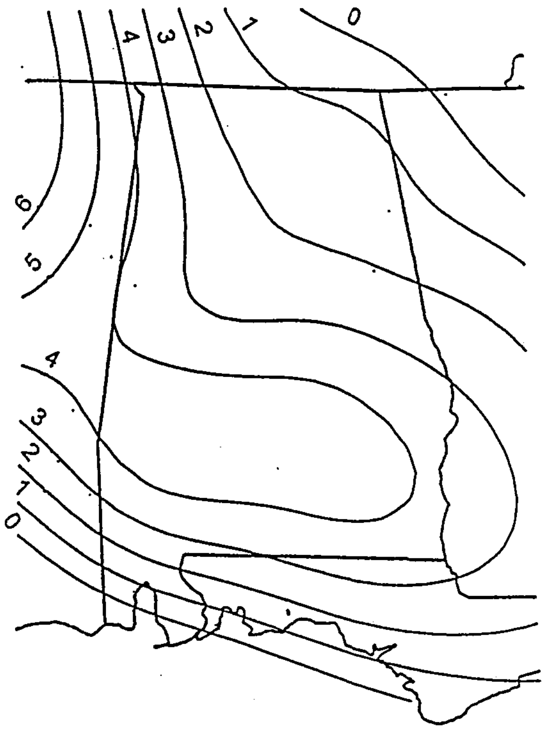
PROBABILITY CONTOUR FOR
5 DAY ANTICEDENT PERIOD



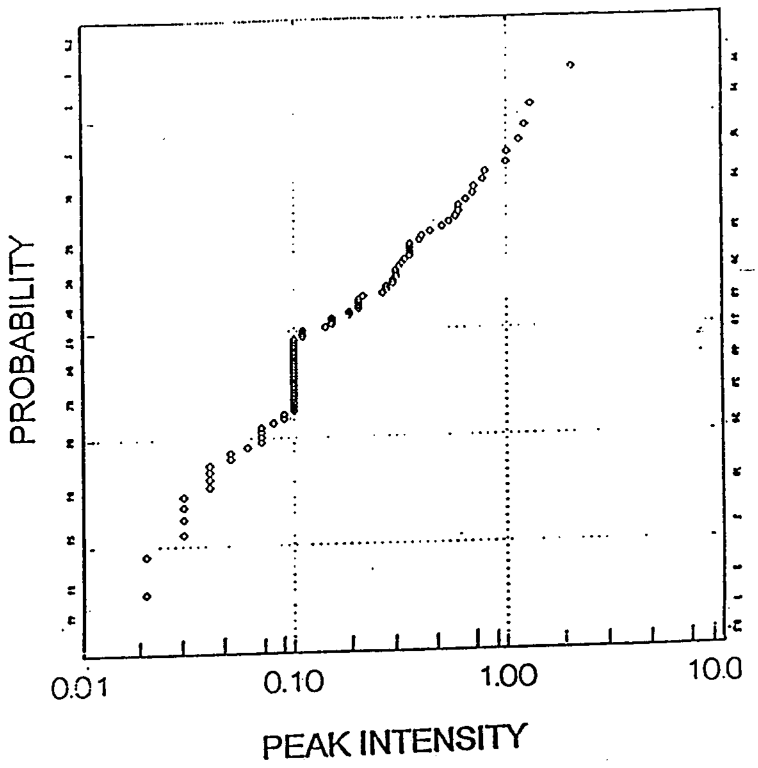
PROBABILITY CONTOUR FOR
7 DAY ANTICEDENT PERIOD



PROBABILITY CONTOUR FOR
10 DAY ANTICEDENT PERIOD



PROBABILITY CONTOUR FOR
15 DAY ANTICEDENT PERIOD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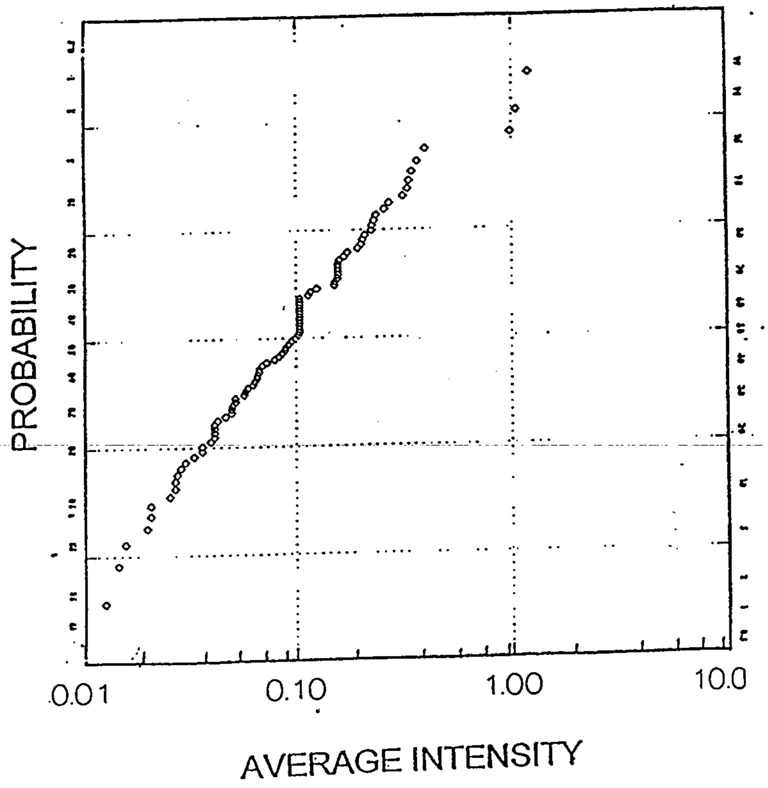
10.0
PEAK INTENSITY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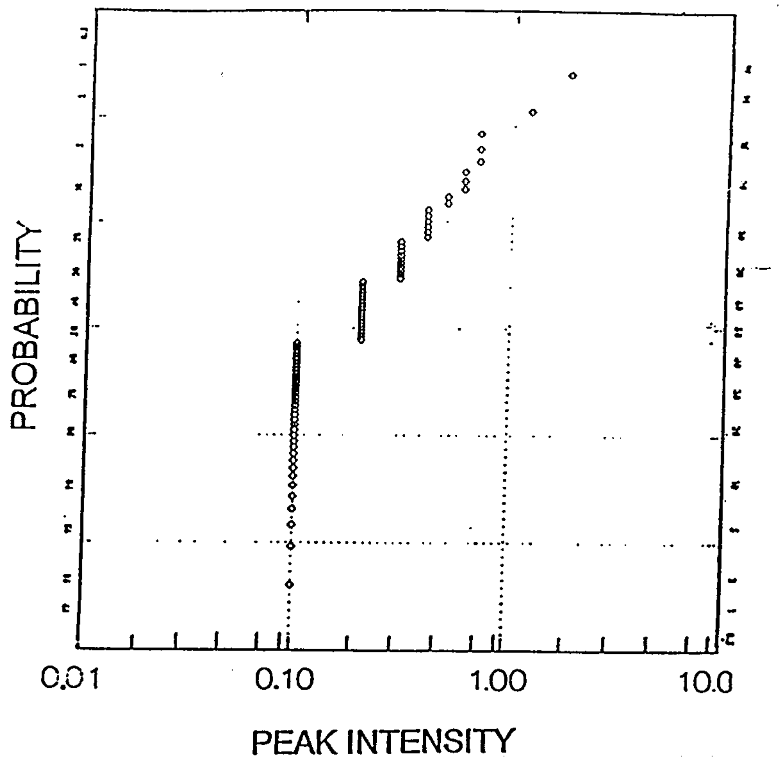
1.00
10.0
AVERAGE INTENSITY

ABBEVILLE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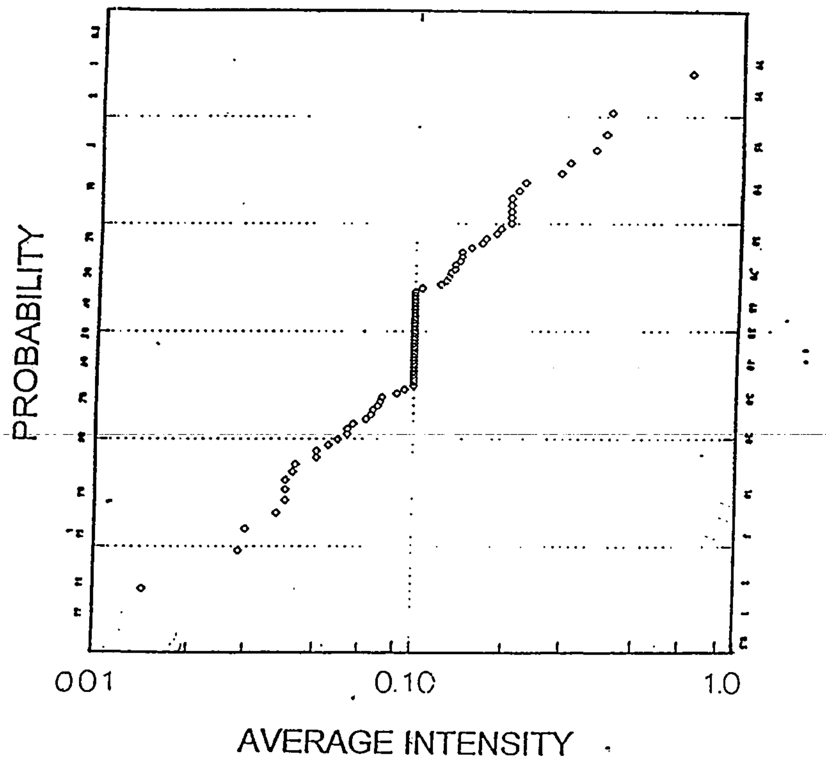
PEAK INTENSITY


PROBABILITY
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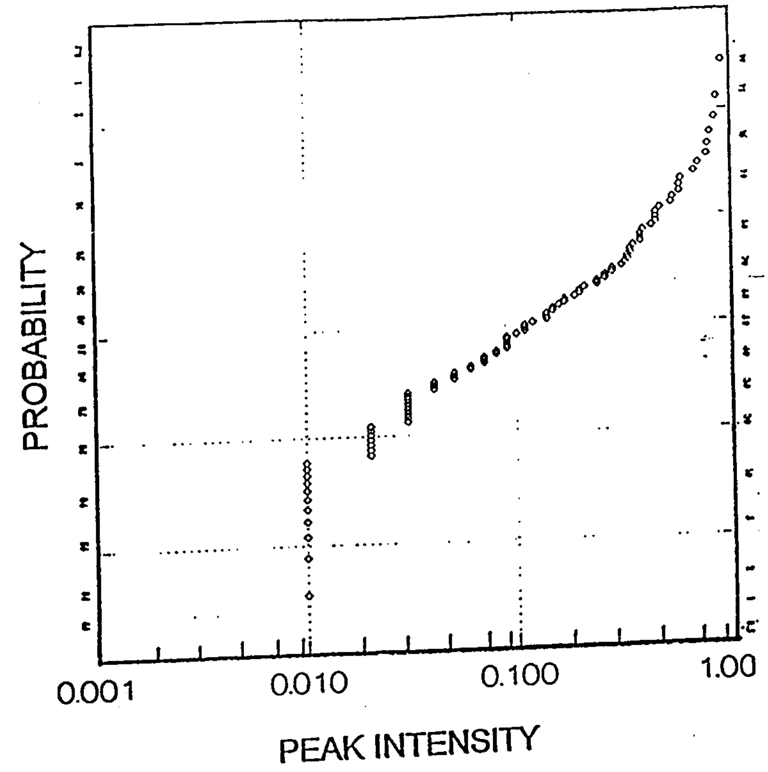
1.0
AVERAGE INTENSITY

BERRY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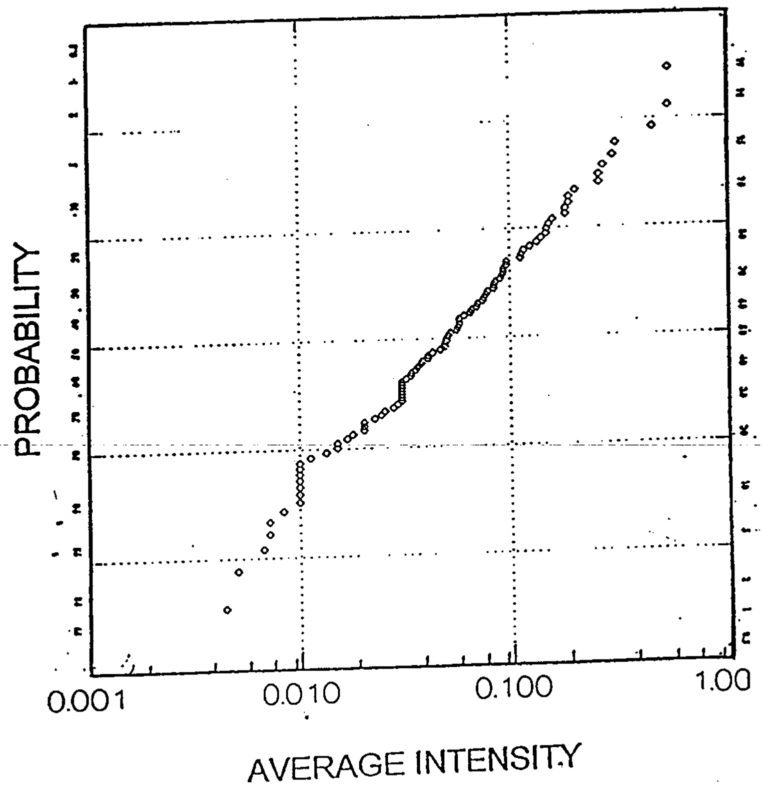
1.00
PEAK INTENSITY


PROBABILITY
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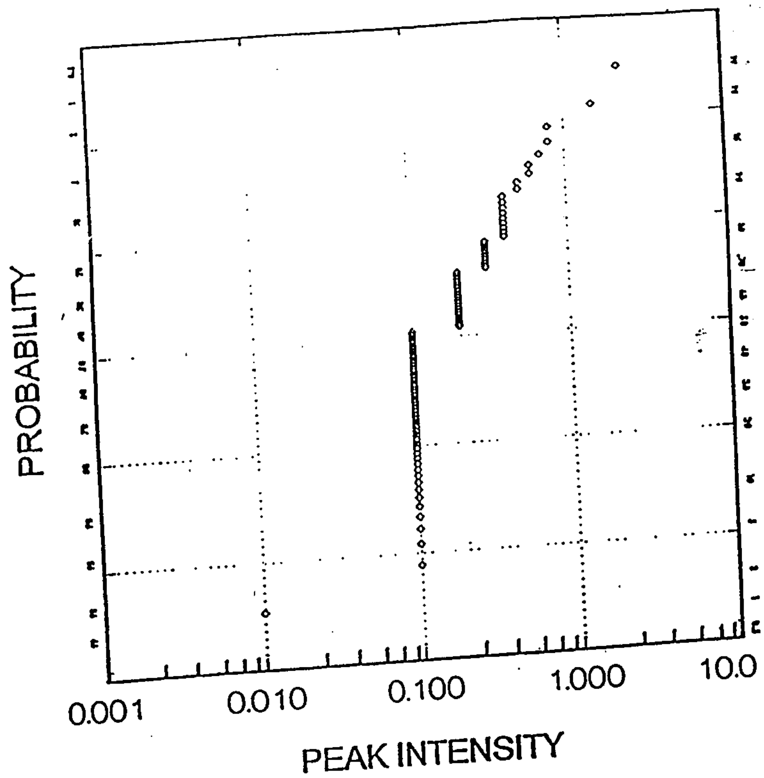
1.00
AVERAGE INTENSITY

BIRMINGHAM


PROBABILITY
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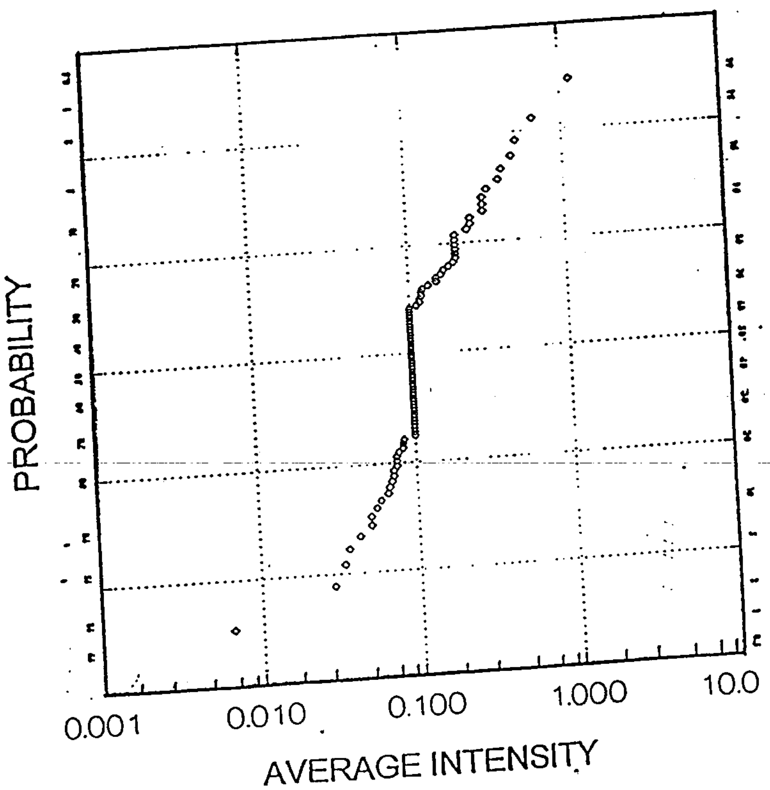
0.100
1.000
10.0
PEAK INTENSITY


PROBABILITY
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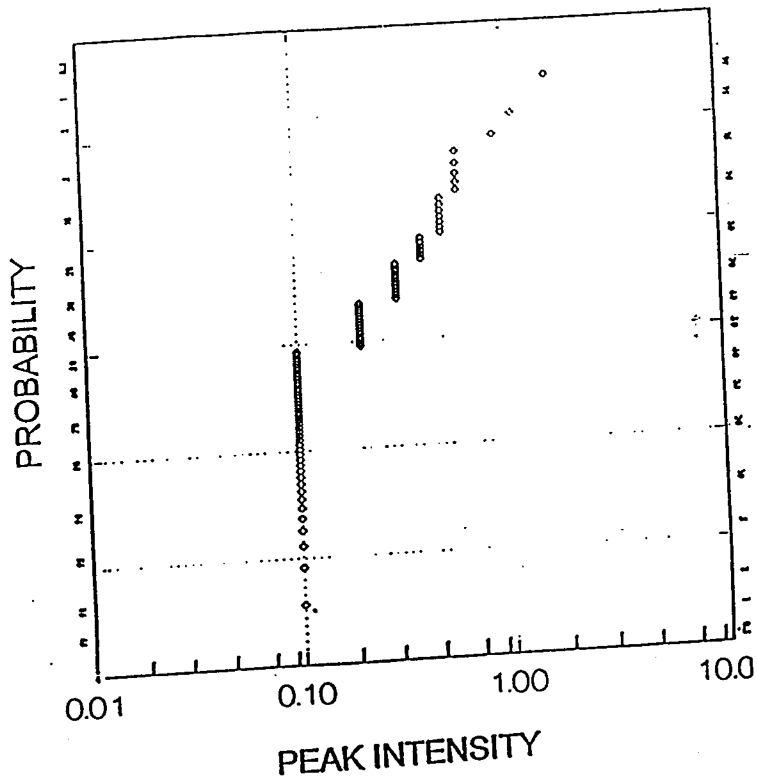
0.100
1.000
10.0
AVERAGE INTENSITY

BOAZ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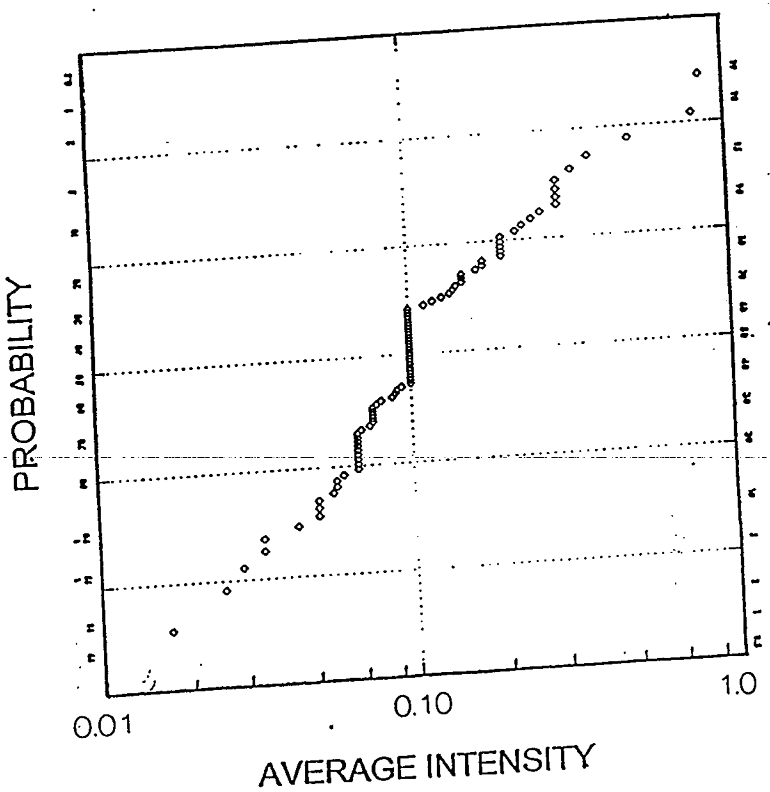
1.00
10.0
PEAK INTENSITY


PROBABILITY
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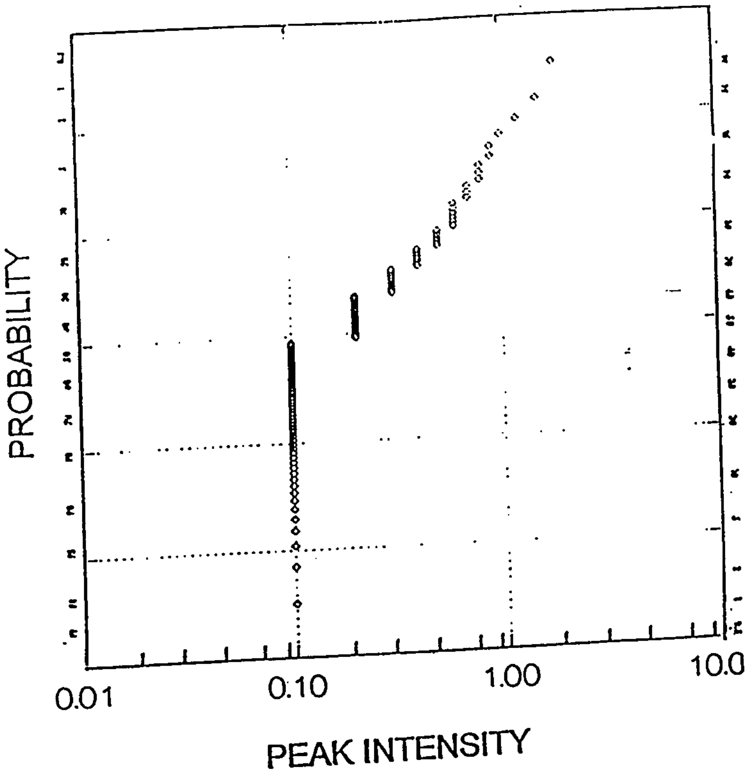
0.10
1.0
AVERAGE INTENSITY

DADEVILLE



PROBABILITY
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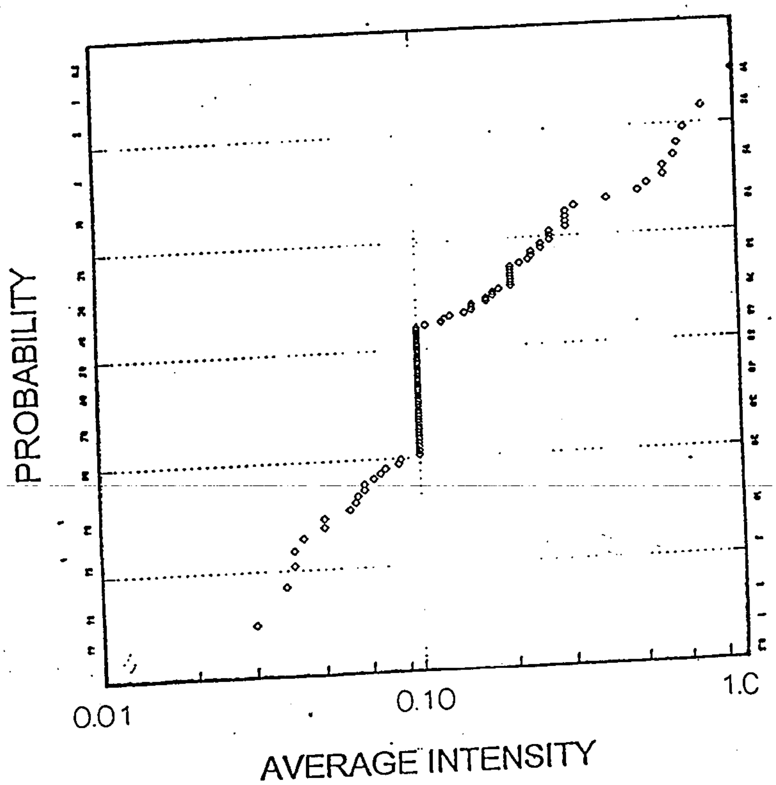
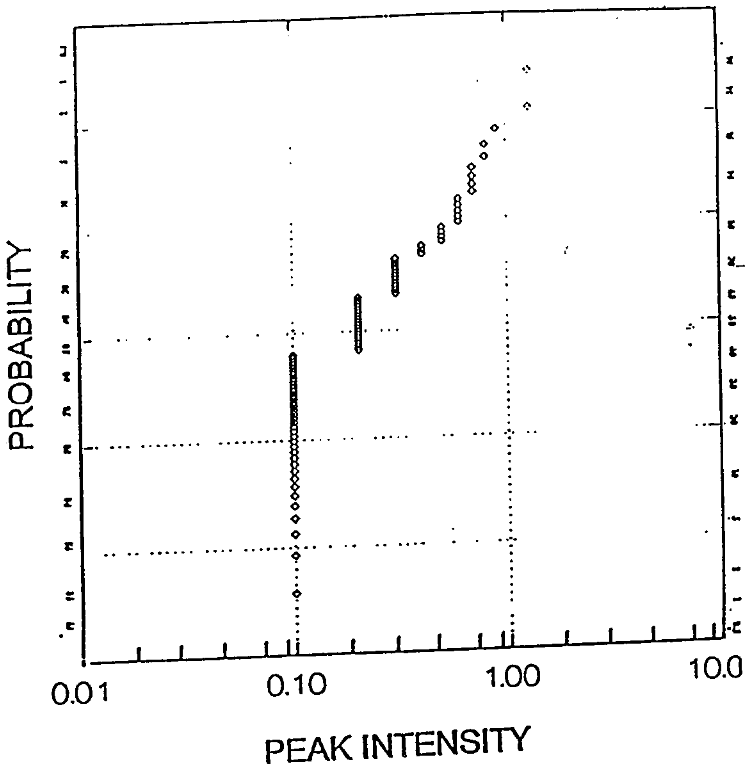
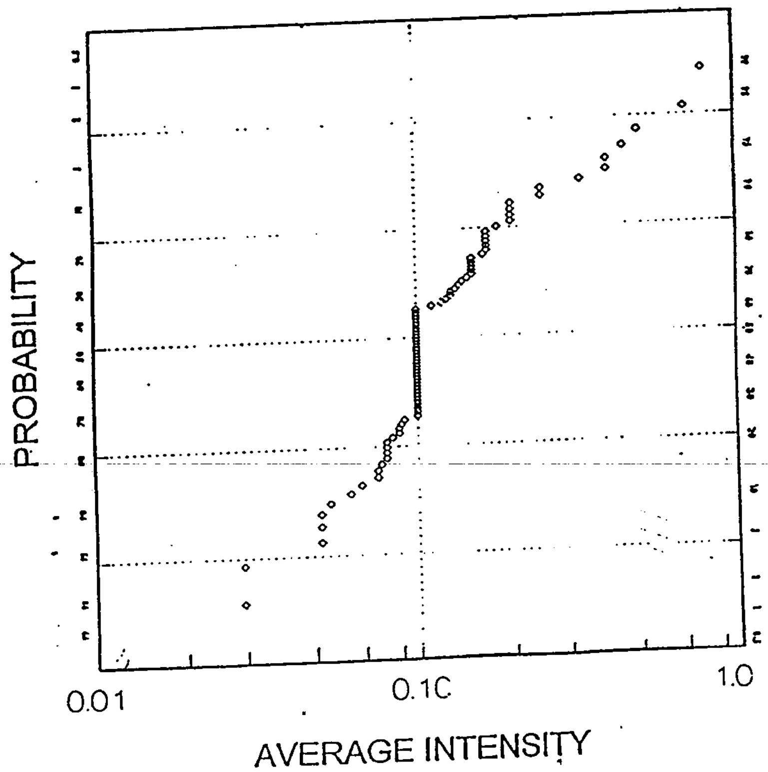
0.10
1.00
10.0
PEAK INTENSITY



PROBABILITY
0.01
0.10
1.0
AVERAGE INTENSITY

DOTHAN


PROBABILITY
0.01
0.10
1.00
10.0
PEAK INTENSITY


PROBABILITY
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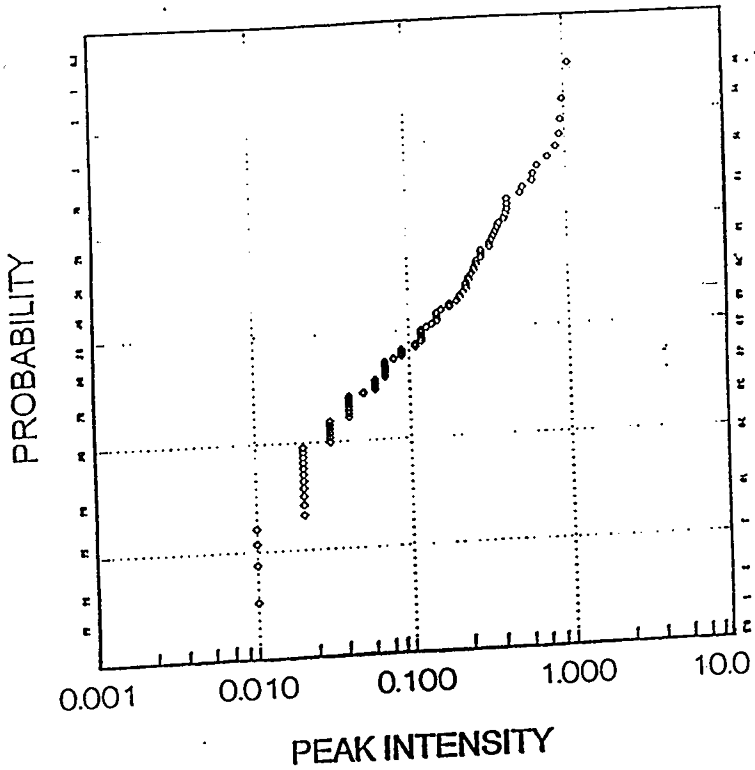
0.1C
1.0
AVERAGE INTENSITY

GREENVILLE


PROBABILITY
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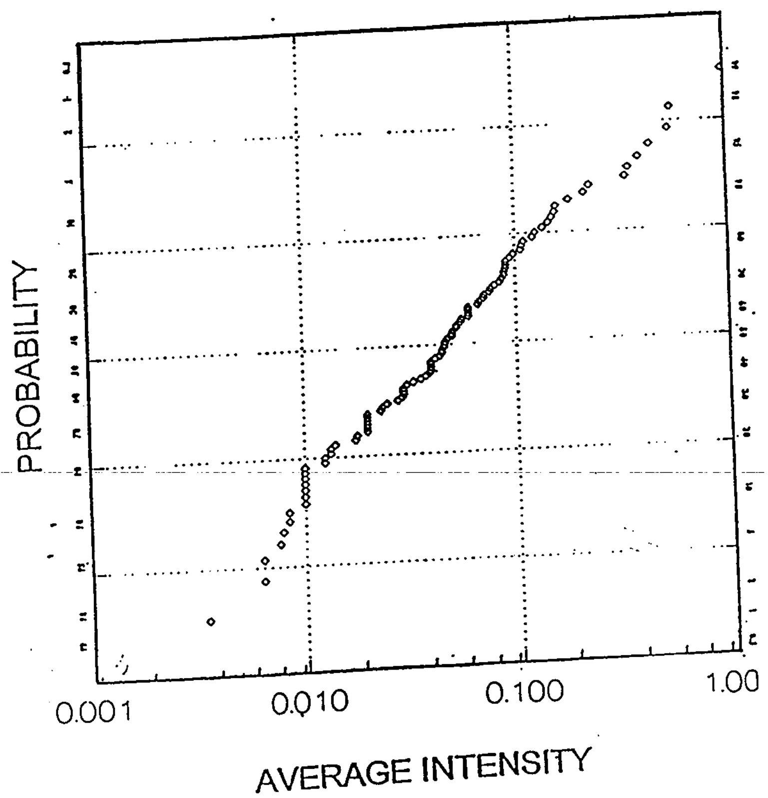
0.100
1.000
10.0
PEAK INTENSITY


PROBABILITY
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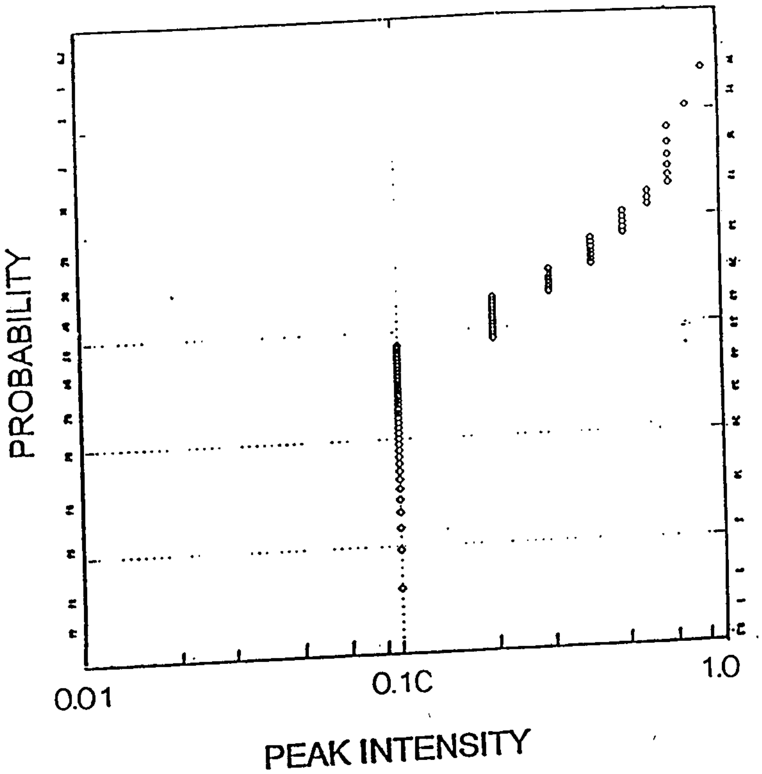
0.100
1.00
AVERAGE INTENSITY

HALEYVILLE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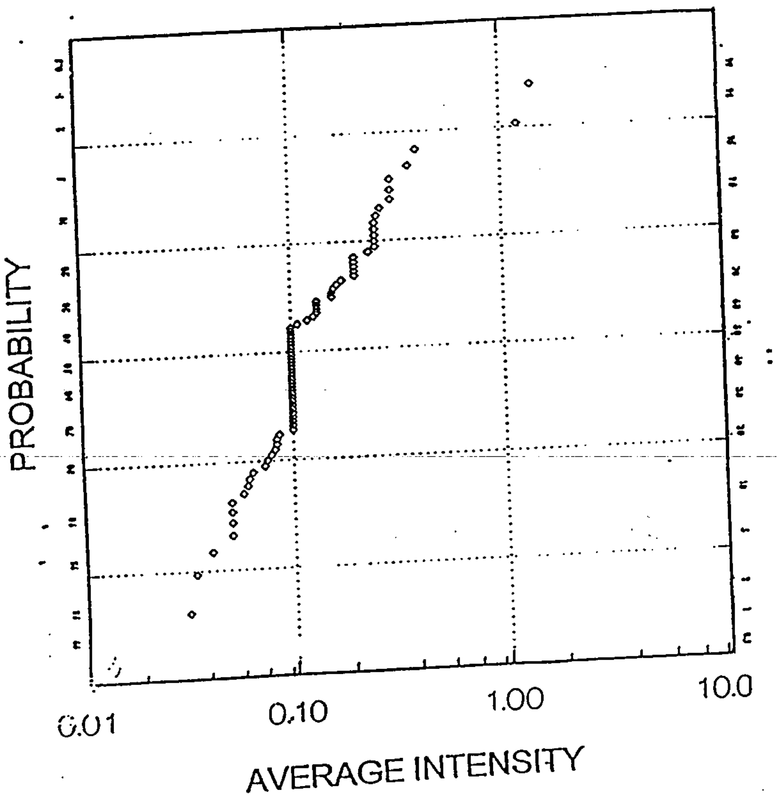
PEAK INTENSITY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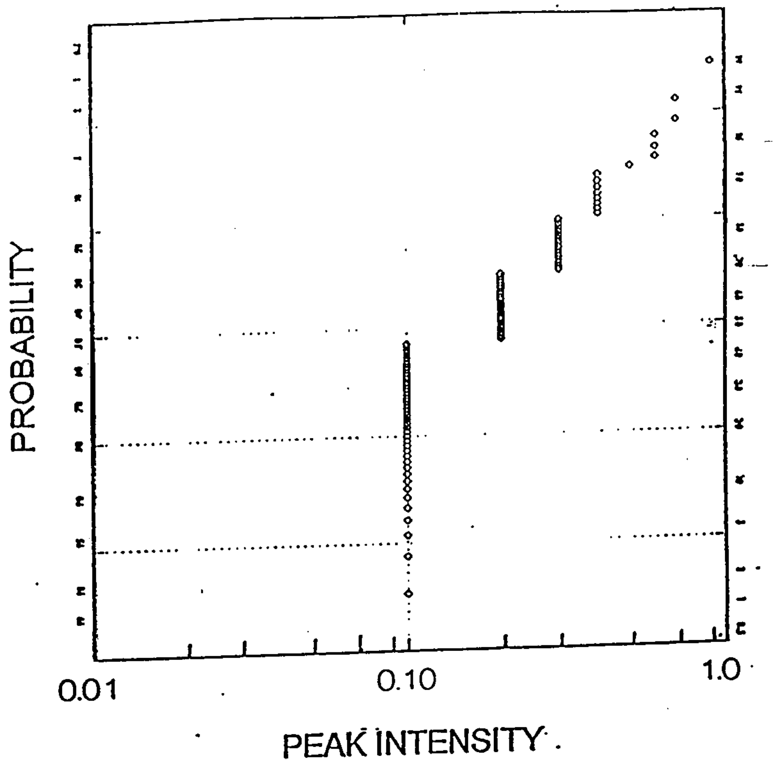
1.00
10.0
AVERAGE INTENSITY

HAMILTON


PROBABILITY
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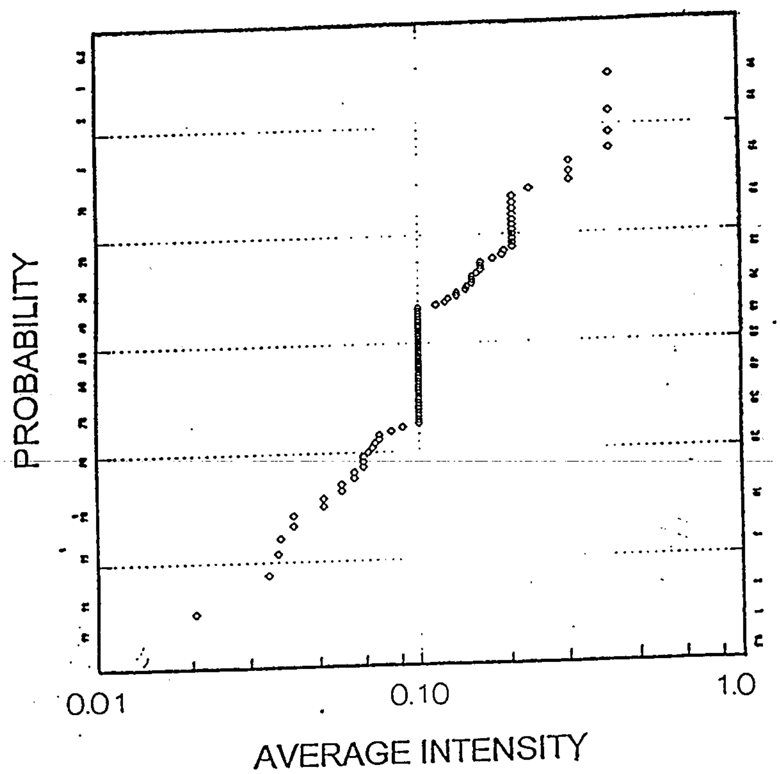
PEAK INTENSITY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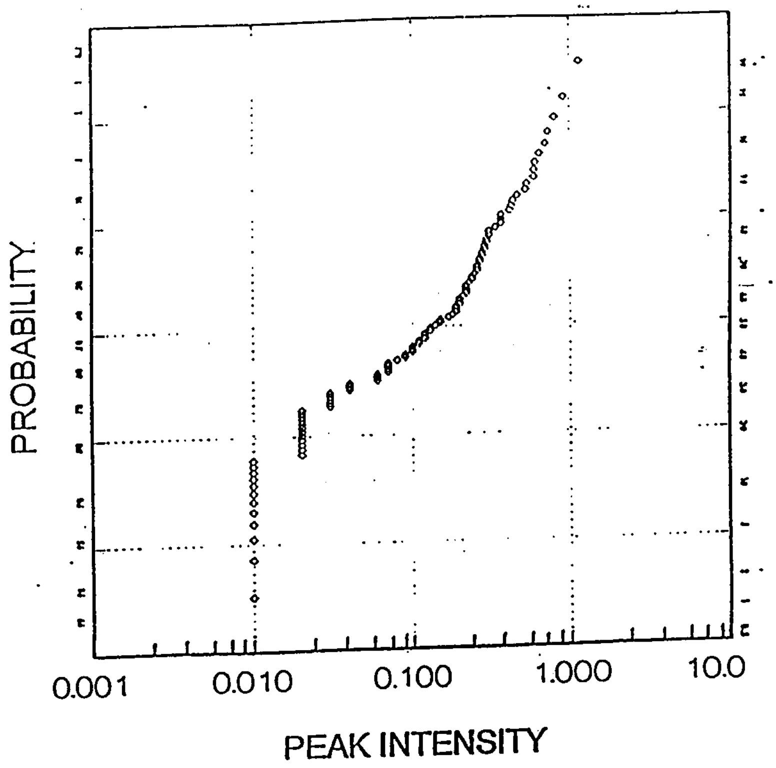
1.0
AVERAGE INTENSITY

HANCEVILLE



PROBABILITY
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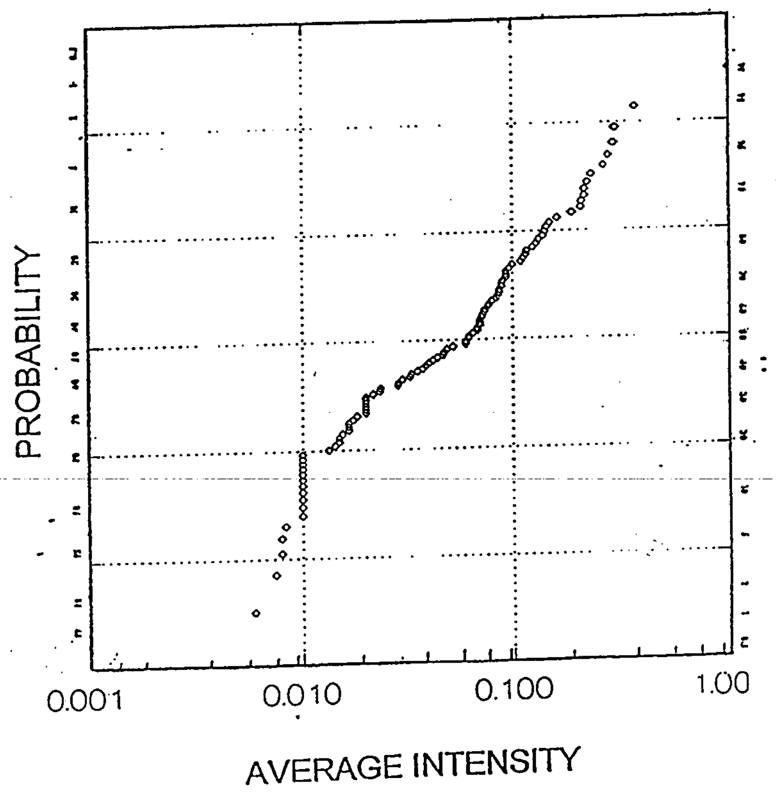
0.100
1.000
10.0
PEAK INTENSITY



PROBABILITY
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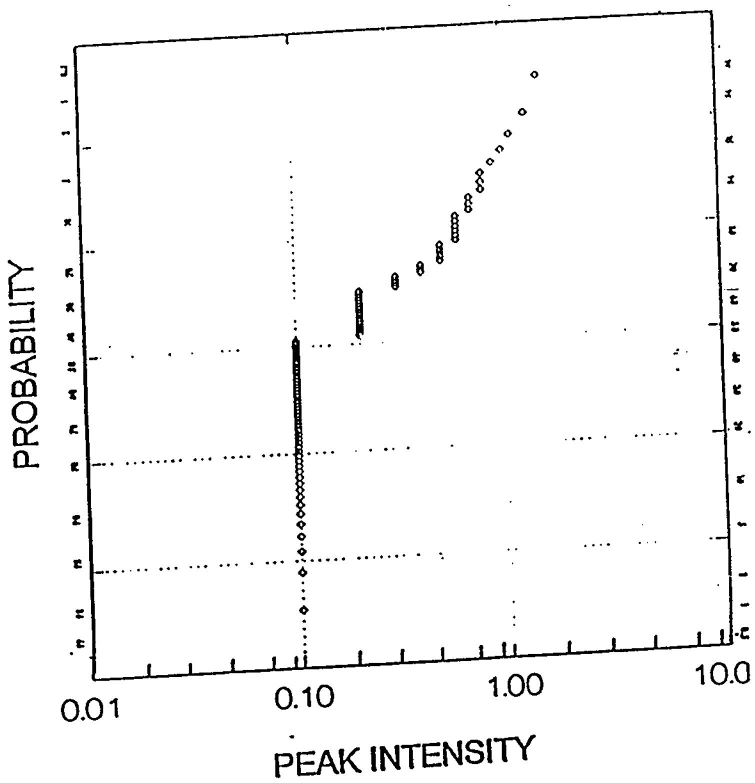
AVERAGE INTENSITY

HUNTSVILLE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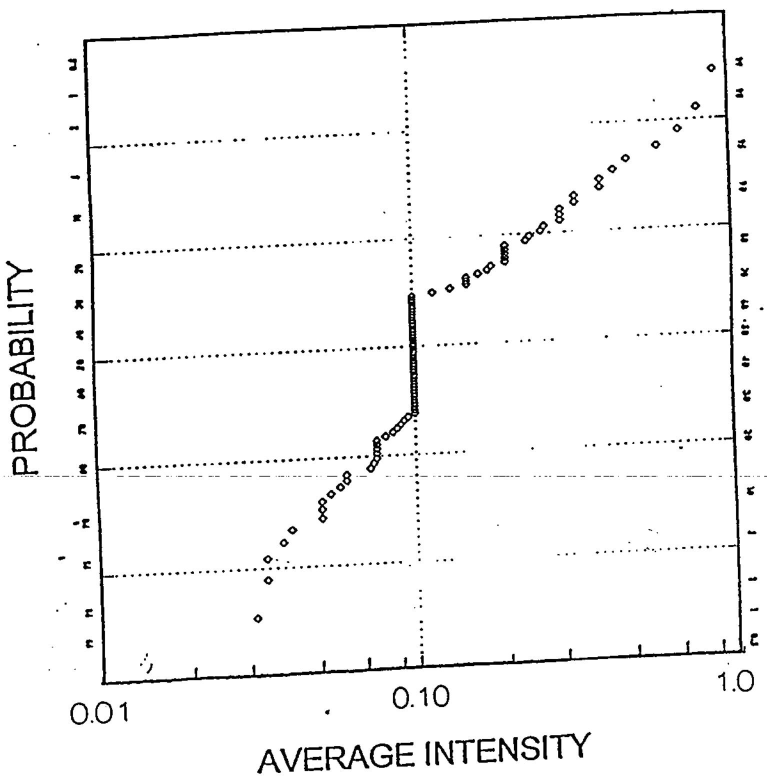
10.0
PEAK INTENSITY


PROBABILITY
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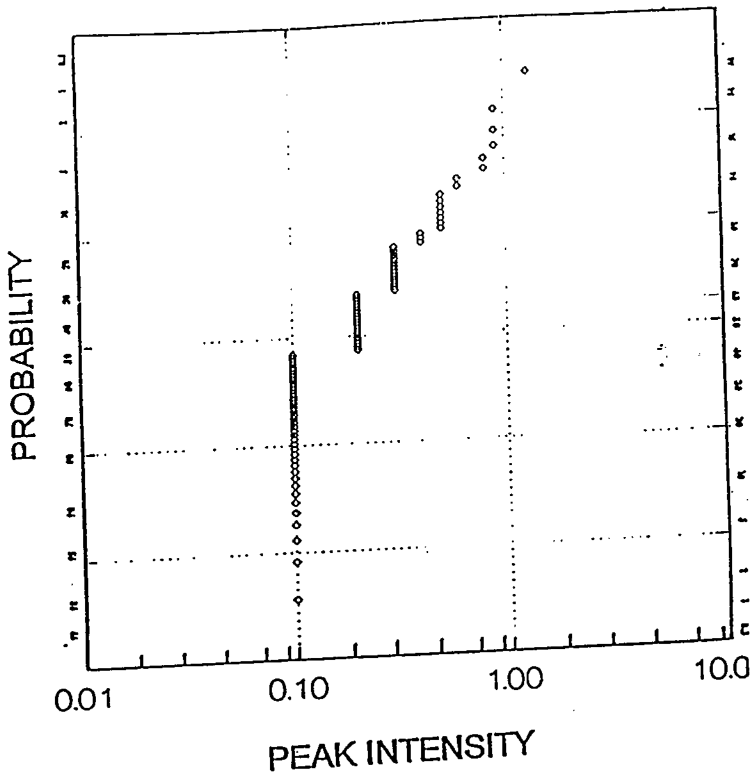
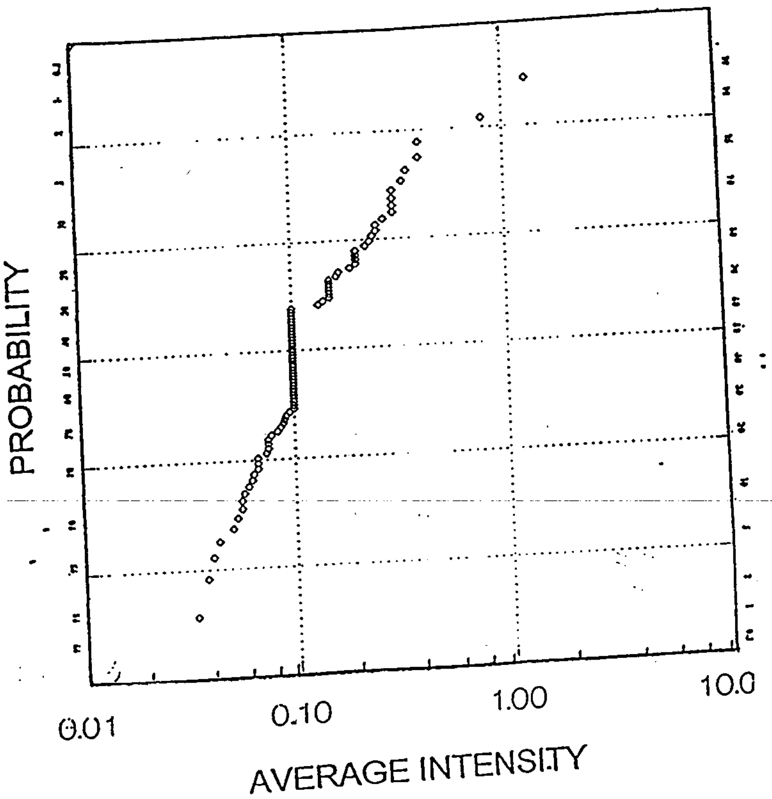
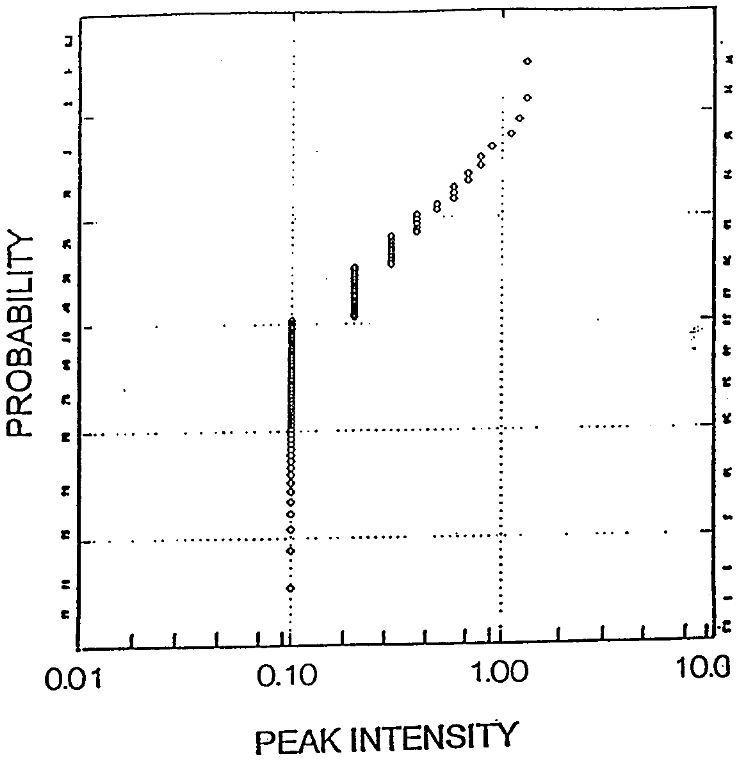
1.0
AVERAGE INTENSITY

JACKSON





JACKSONVILLE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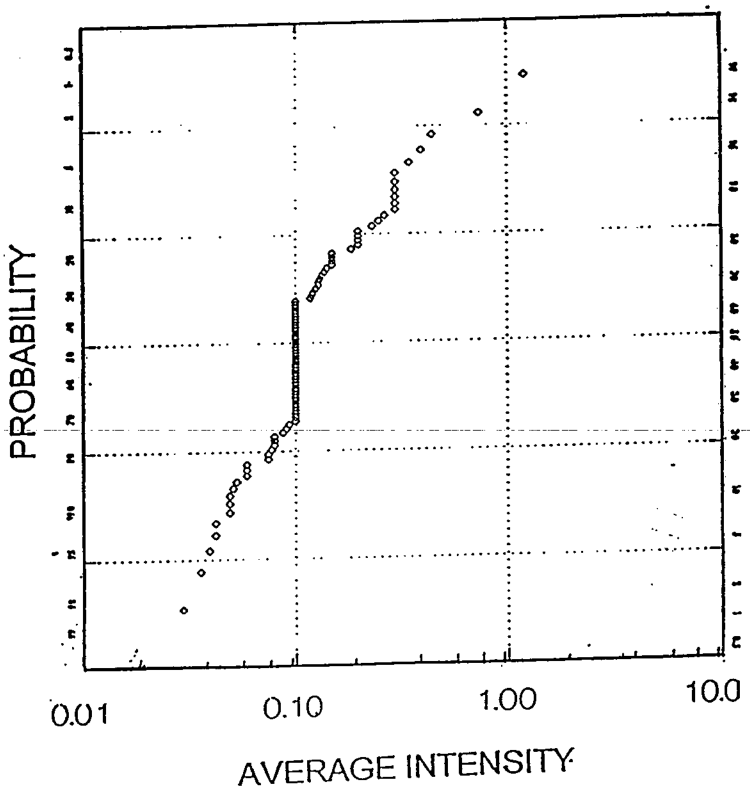
PEAK INTENSITY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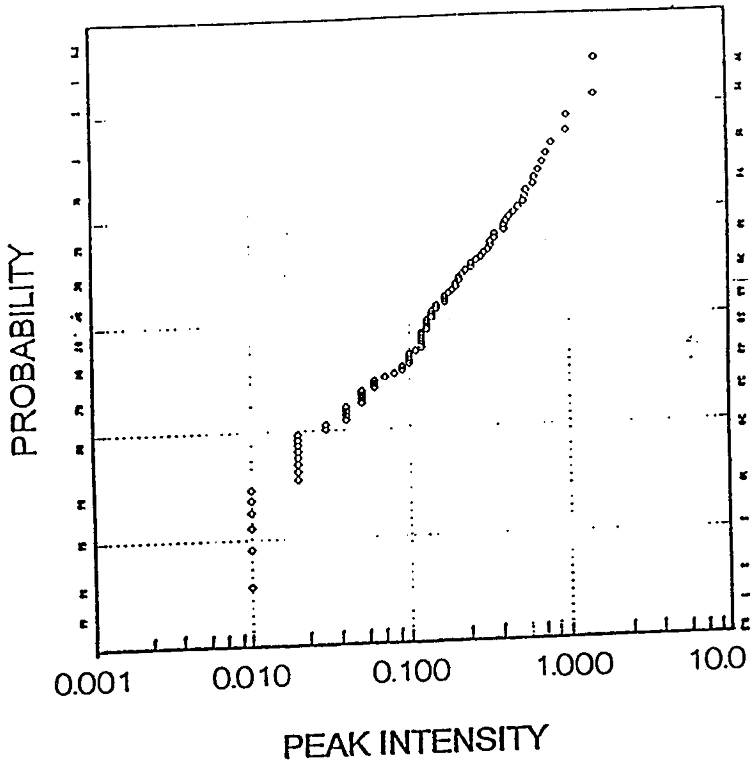
10.0
AVERAGE INTENSITY

LAGRANGE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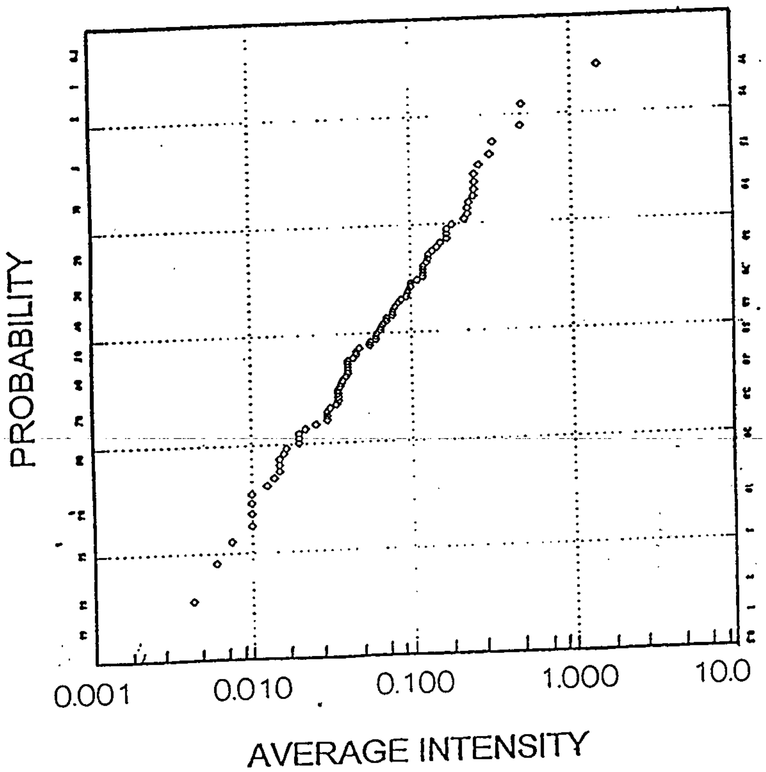
10.0
PEAK INTENSITY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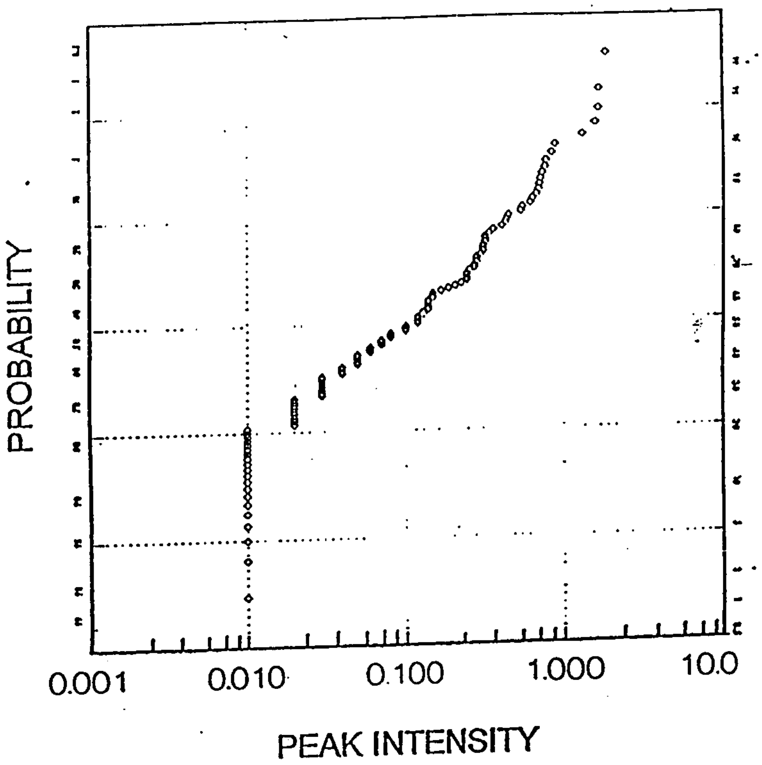
10.0
AVERAGE INTENSITY

MERIDIAN


PROBABILITY
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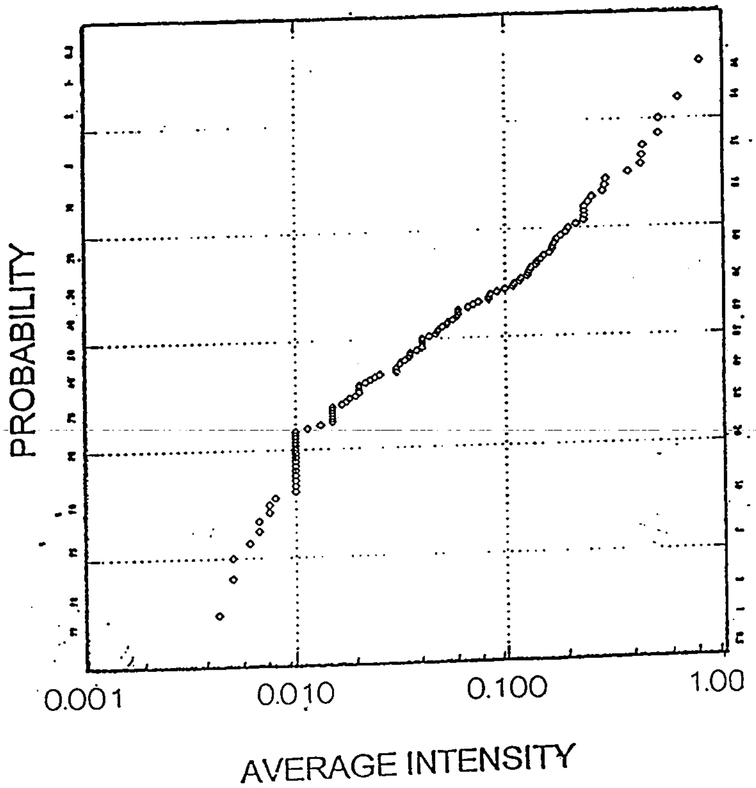
1.000
10.0
PEAK INTENSITY


PROBABILITY
0.001
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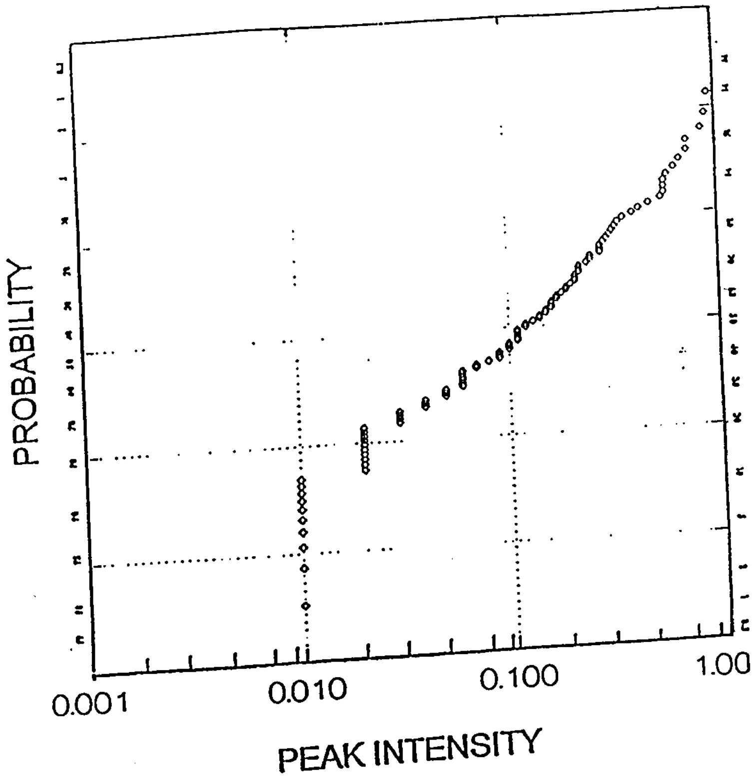
0.100
1.00
AVERAGE INTENSITY

MOBILE


PROBABILITY
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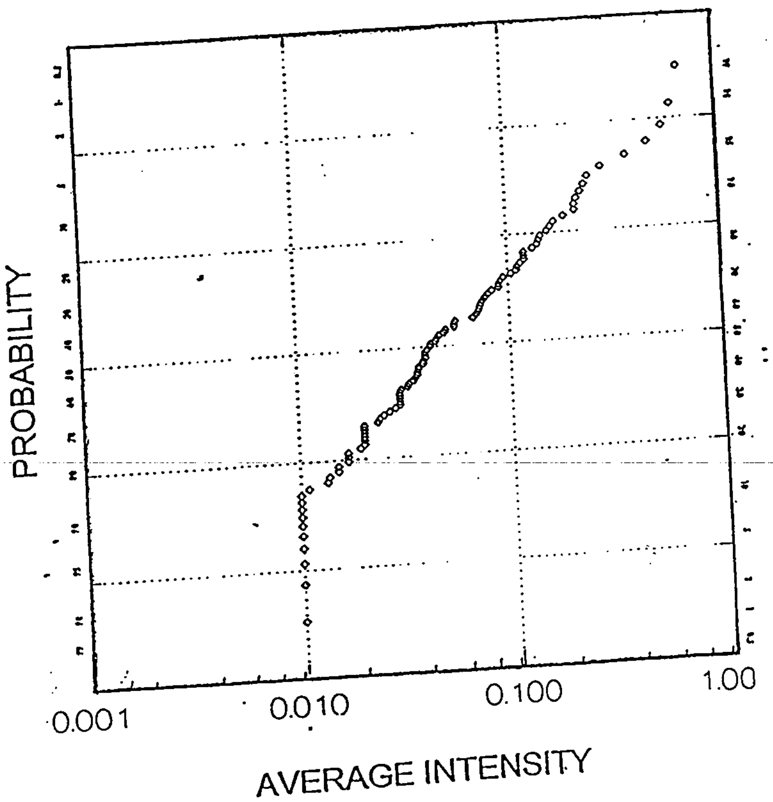
1.00
PEAK INTENSITY


PROBABILITY
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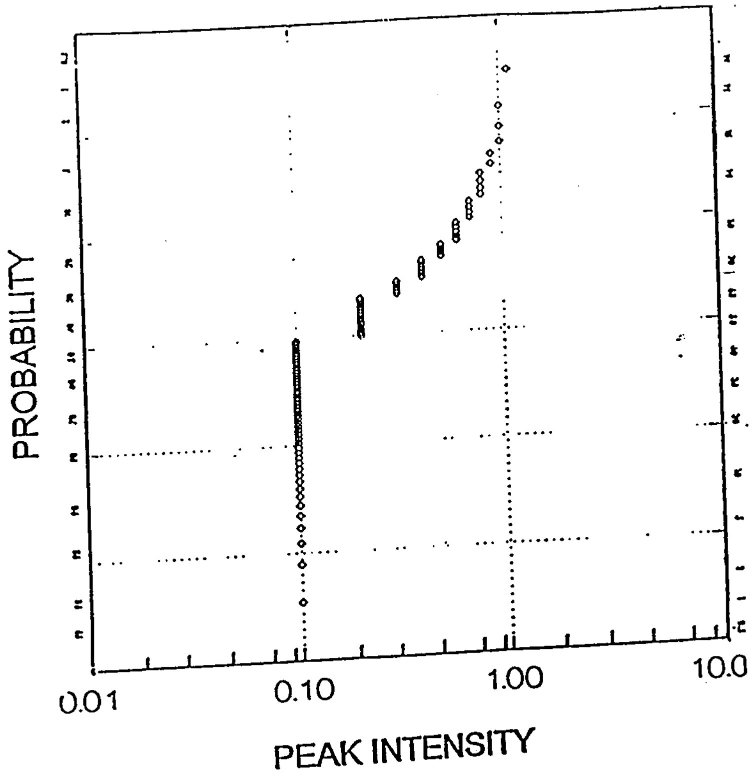
0.100
1.00
AVERAGE INTENSITY

MONTGOMERY


PROBABILITY
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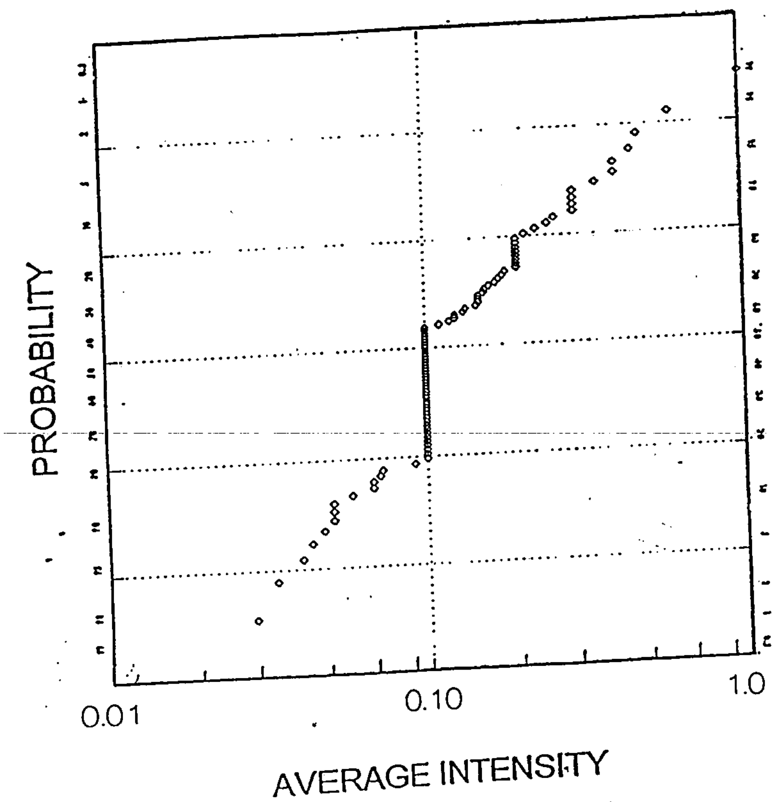
1.00
10.0
PEAK INTENSITY


PROBABILITY
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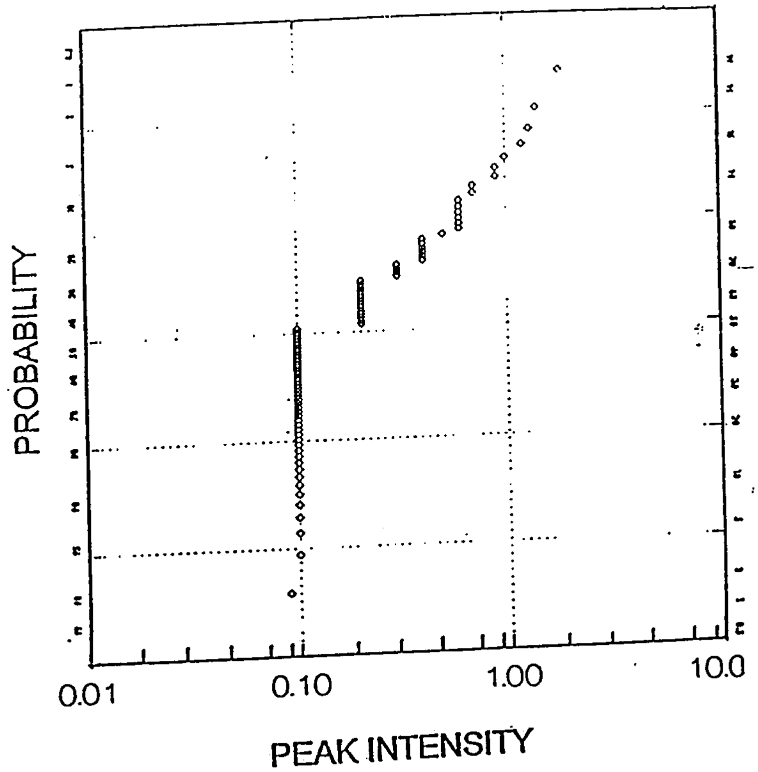
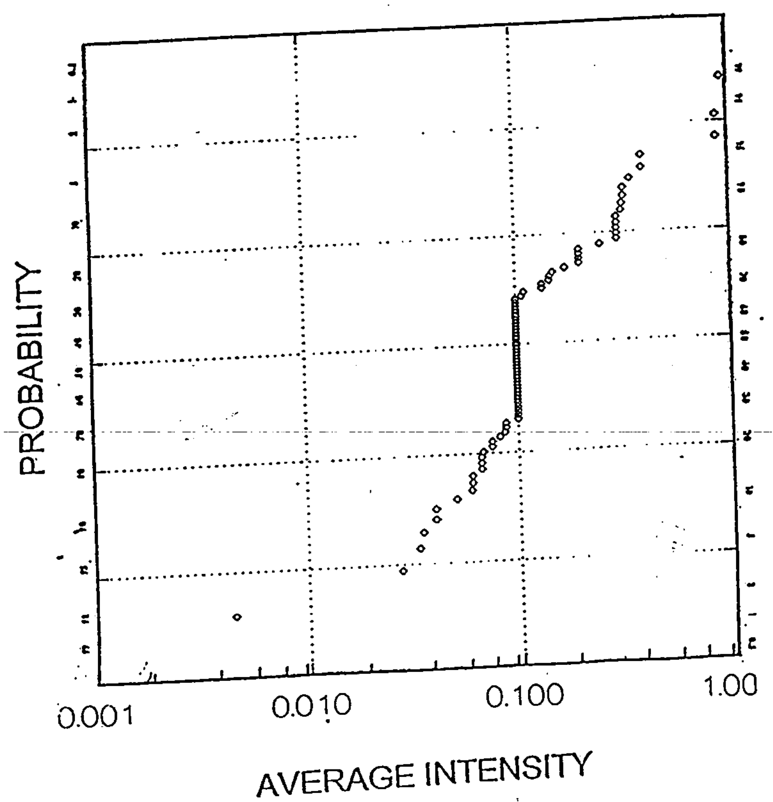
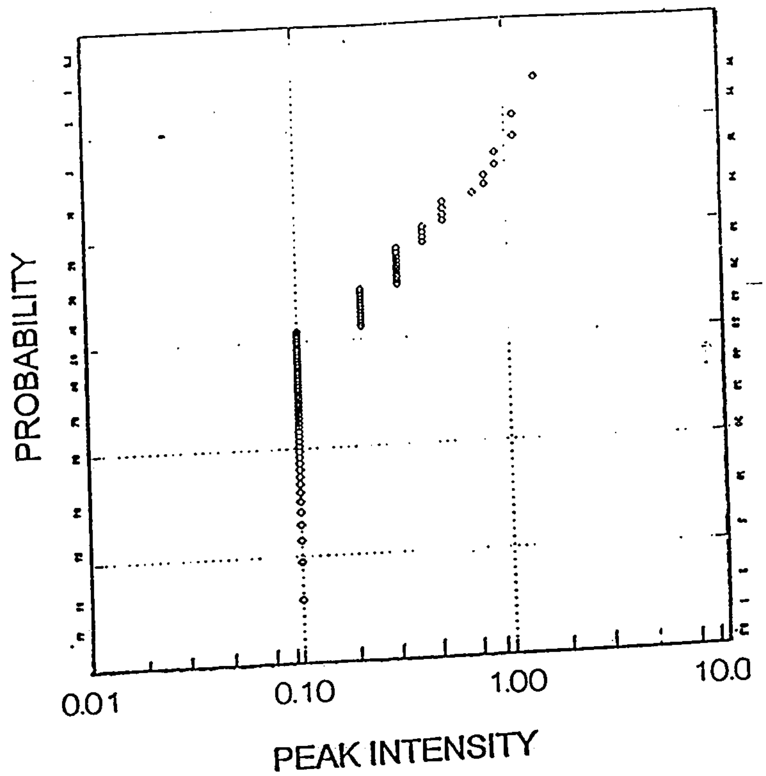
AVERAGE INTENSITY

PETERMAN





THOMASVILLE


PROBABILITY
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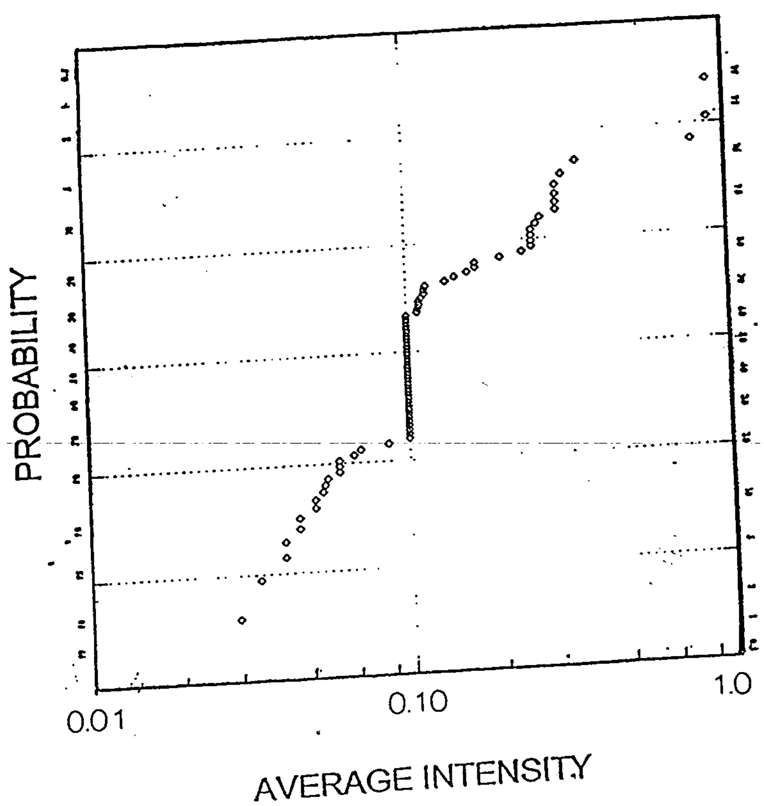
PEAK INTENSITY


PROBABILITY
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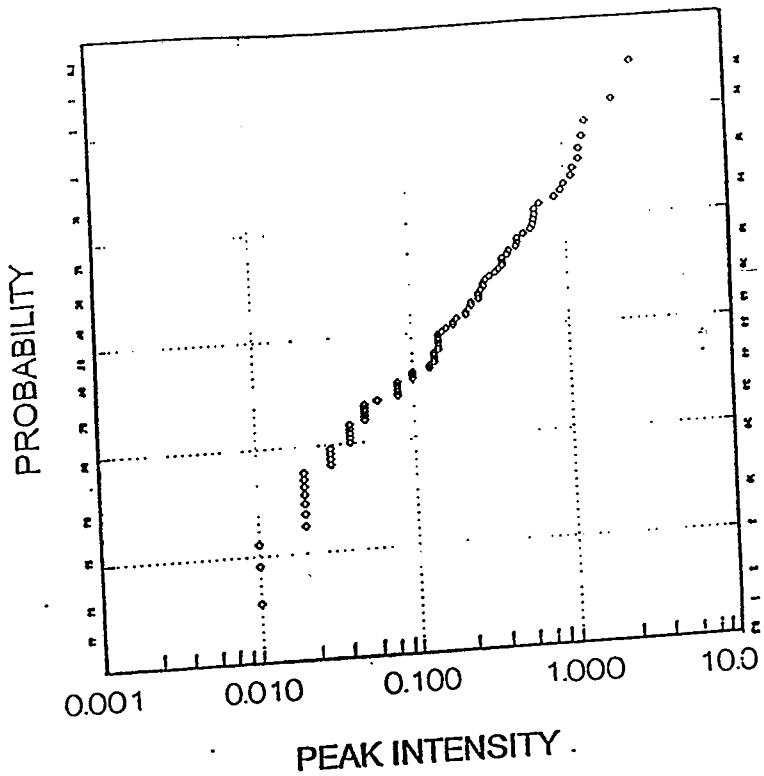
1.0
AVERAGE INTENSITY

TROY


PROBABILITY
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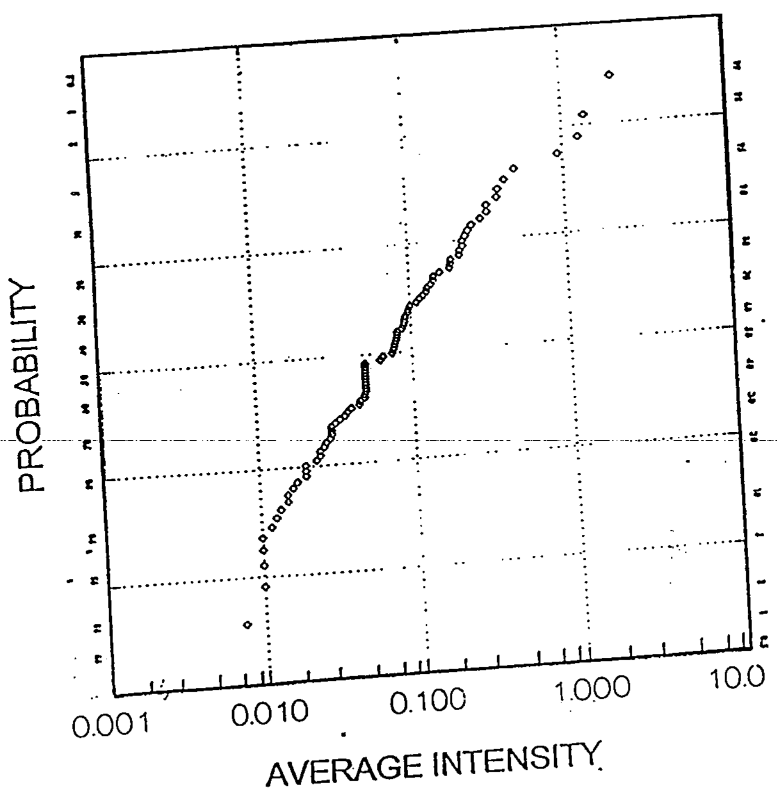
10.0
PEAK INTENSITY


PROBABILITY
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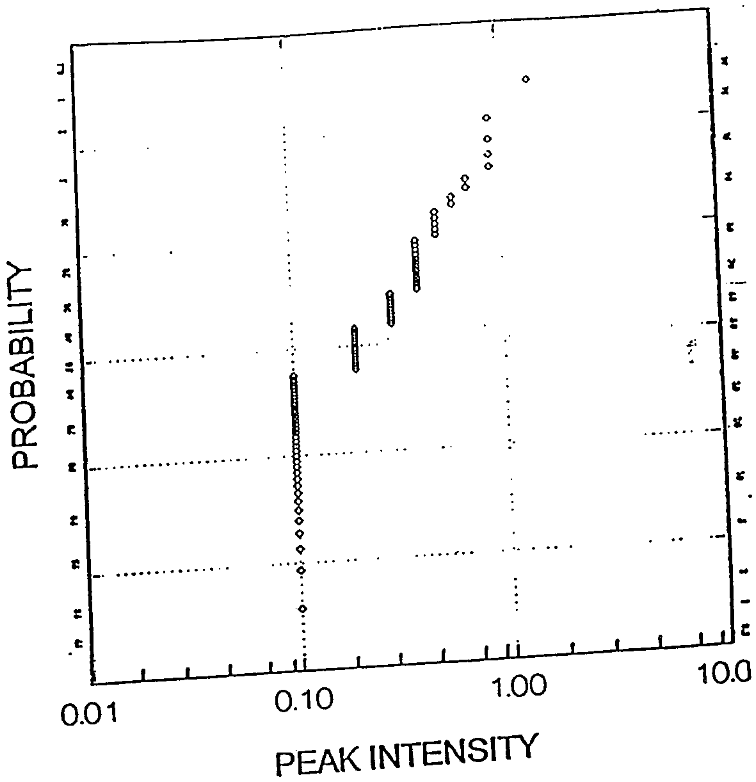
10.0
AVERAGE INTENSITY

TUSCALOOSA


PROBABILITY
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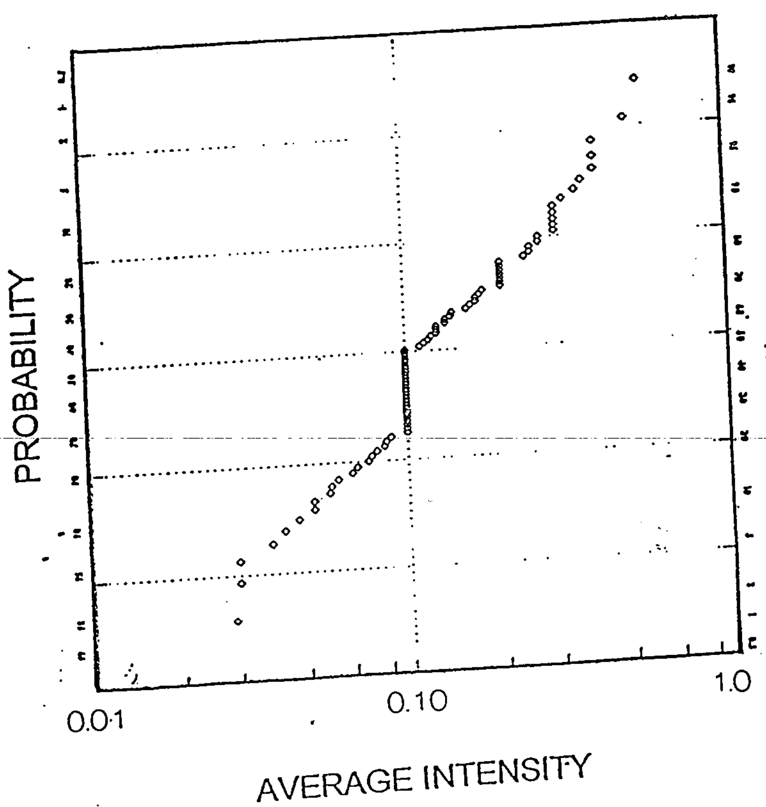
10.0
PEAK INTENSITY


PROBABILITY
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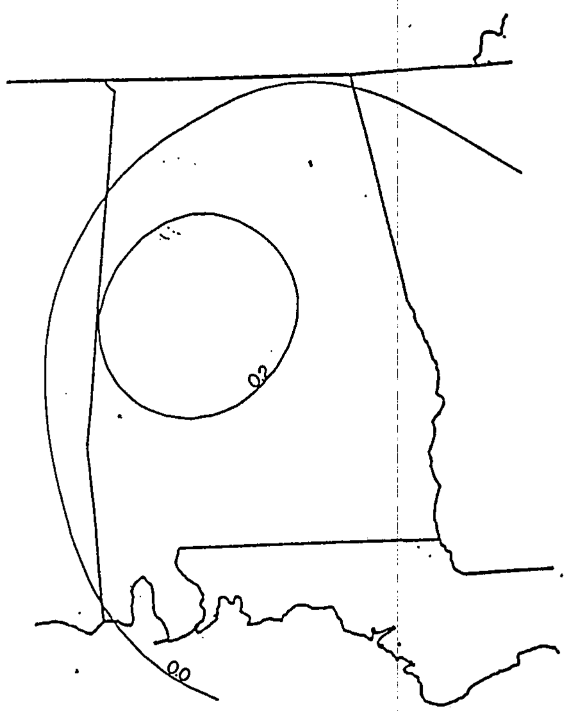
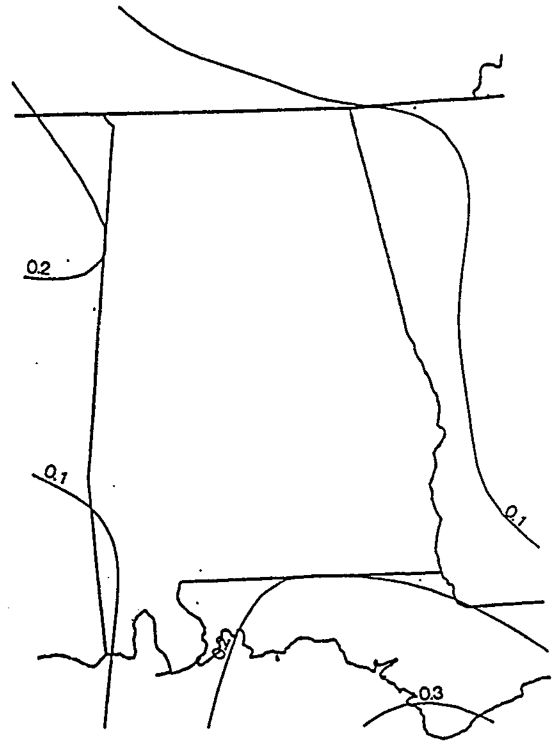
1.0
AVERAGE INTENSITY

VERNON



CONTOUR OF
AVERAGE INTENSITY



CONTOUR OF
PEAK INTENSITY

APPENDIX B

FIELD INFILTRATION TEST RESULTS

INTERSTATE 459
GOOD DRAINAGE LOCATION

* Tests 1, 3, and 5 : simple crack (0.125" wide, .025" deep) across diameter.
* Tests with cracks present had seepage through cracks.

TEST 1:
Diameter = 6.75 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
7D13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	0.03	0.03	15.43
11.00	2.00	1.00	0.13	0.10	5.14
10.00	3.00	1.00	0.27	0.13	3.86
9.00	4.00	1.00	0.42	0.15	3.43

TEST 2:
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.75	0.25	0.25	0.98	0.98	0.17
12.50	0.50	0.25	3.20	2.22	0.07
12.25	0.75	0.25	9.12	5.92	0.03
12.38	0.75	0.00	23.00	13.88	0.00
12.38	0.75	0.00	38.00	15.00	0.00

TEST 3 :
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.50	0.50	0.50	0.13	0.13	2.44
12.00	1.00	0.50	0.23	0.10	3.26
11.50	1.50	0.50	0.45	0.22	1.50
11.00	2.00	0.50	0.70	0.25	1.30
10.50	2.50	0.50	0.97	0.27	1.22
10.00	3.00	0.50	1.33	0.37	0.89
9.50	3.50	0.50	1.75	0.42	0.78
9.00	4.00	0.50	2.22	0.47	0.70

TEST 4:
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.75	0.25	0.25	0.20	0.20	0.81
12.50	0.50	0.25	0.62	0.42	0.39
12.25	0.75	0.25	1.27	0.65	0.25
12.00	1.00	0.25	2.32	1.05	0.16
11.75	1.25	0.25	11.25	8.93	0.02
11.75	1.25	0.00	42.00	30.75	0.00

TEST 5 :
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.50	0.50	0.50	0.12	0.12	2.79
12.00	1.00	0.50	0.25	0.13	2.44
11.50	1.50	0.50	0.43	0.18	1.78
11.00	2.00	0.50	0.60	0.17	1.95
10.50	2.50	0.50	0.72	0.12	2.79
10.00	3.00	0.50	1.00	0.28	1.15
9.50	3.50	0.50	1.23	0.23	1.40
9.00	4.00	0.50	1.43	0.20	1.63

TEST 6:
Diameter = 6.50 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.75	0.25	0.25	2.33	2.33	0.06
12.50	0.50	0.25	5.92	3.58	0.04
12.50	0.50	0.00	40.00	34.08	0.00

INTERSTATE 459
POOR DRAINAGE LOCATION

- First two attempts failed.
- Tests 1, 2, and 3 : located around moisture sensor locations. Possible overlay.
- Test 4 : located on obvious overlay.

TEST 1:
Diameter = 7.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	0.18	0.18	2.61
11.00	2.00	1.00	0.35	0.17	2.87
10.00	3.00	1.00	0.52	0.17	2.87
9.00	4.00	1.00	0.65	0.13	3.59
8.00	5.00	1.00	0.83	0.18	2.61

TEST 2:
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	1.62	1.62	0.40
11.00	2.00	1.00	9.72	8.10	0.08

TEST 3 :
Diameter = 6.50 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00		0.00	0.00	N/A
12.00	1.00	1.00	0.13	0.13	4.16
11.00	2.00	1.00	0.23	0.09	6.05
10.00	3.00	1.00	0.35	0.12	4.56
9.00	4.00	1.00	0.56	0.21	2.64

TEST 4:
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.13	0.88	0.88	16.67	16.67	0.03

HIGHWAY 79
GOOD DRAINAGE LOCATION

* Two inch overlay approximately one week ago.
* Wearing surface has not yet been applied.
* Pavement is very porous - water is "piping" down into pavement and then back up around seal.

TEST 1:
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	0.05	0.05	13.02
11.00	2.00	1.00	0.12	0.07	9.77
10.00	3.00	1.00	0.17	0.05	13.02

TEST 2:
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	0.02	0.02	39.06
11.00	2.00	1.00	0.03	0.02	39.06
10.00	3.00	1.00	0.05	0.02	39.06

TEST 3:
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	0.02	0.02	39.06
11.00	2.00	1.00	0.03	0.02	39.06
10.00	3.00	1.00	0.05	0.02	39.06

TEST 4:
Diameter = 5.75 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	0.08	0.08	8.51
11.00	2.00	1.00	0.19	0.11	6.54
10.00	3.00	1.00	0.32	0.13	5.45
9.00	4.00	1.00	0.45	0.13	5.67

TEST 5 :
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	0.03	0.03	16.69
11.00	2.00	1.00	0.05	0.01	48.23
10.00	3.00	1.00	0.13	0.08	7.86
9.00	4.00	1.00	0.18	0.05	12.64

TEST 6 :
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	0.03	0.03	25.37
11.00	2.00	1.00	0.06	0.03	20.56
10.00	3.00	1.00	0.09	0.04	18.17
9.00	4.00	1.00	0.13	0.04	17.21

HIGHWAY 79
POOR DRAINAGE LOCATION

TEST 1:
Diameter = 6.5 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	6.07	6.07	0.09
11.75	1.25	0.25	15.50	9.43	0.01

TEST 2:
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.00	1.00	1.00	7.13	7.13	0.09
11.75	1.25	0.25	13.33	6.20	0.03
11.56	1.44	0.19	19.50	6.17	0.02

TEST 3:
Diameter = 5.75 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.75	0.25	0.25	0.22	0.22	0.82
12.50	0.50	0.25	0.70	0.48	0.37
12.25	0.75	0.25	1.23	0.53	0.33
12.00	1.00	0.25	1.93	0.70	0.25
11.75	1.25	0.25	2.80	0.87	0.20
11.50	1.50	0.25	4.45	1.65	0.11
11.25	1.75	0.25	5.10	0.65	0.27
11.00	2.00	0.25	6.75	1.65	0.11
10.75	2.25	0.25	10.35	3.60	0.05
10.63	2.38	0.13	21.83	11.48	0.01

TEST 4:
Diameter = 5.75 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00 222 0.00		0.00	N/A
12.75	0.25	0.25	0.40	0.40	0.44
12.50	0.50	0.25	1.23	0.83	0.21
12.25	0.75	0.25	2.95	1.72	0.10
12.00	1.00	0.25	6.07	3.12	0.06
11.75	1.25	0.25	11.62	5.55	0.03
11.50	1.50	0.25	16.50	4.88	0.04

TEST 5:
Diameter = 6.00 in.

Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.75	0.25	0.25	0.47	0.47	0.35
12.50	0.50	0.25	1.17	0.70	0.23
12.25	0.75	0.25	2.32	1.15	0.14
12.00	1.00	0.25	4.38	2.07	0.08
11.75	1.25	0.25	8.30	3.92	0.04

TEST 6:
Diameter = 7.25 in.

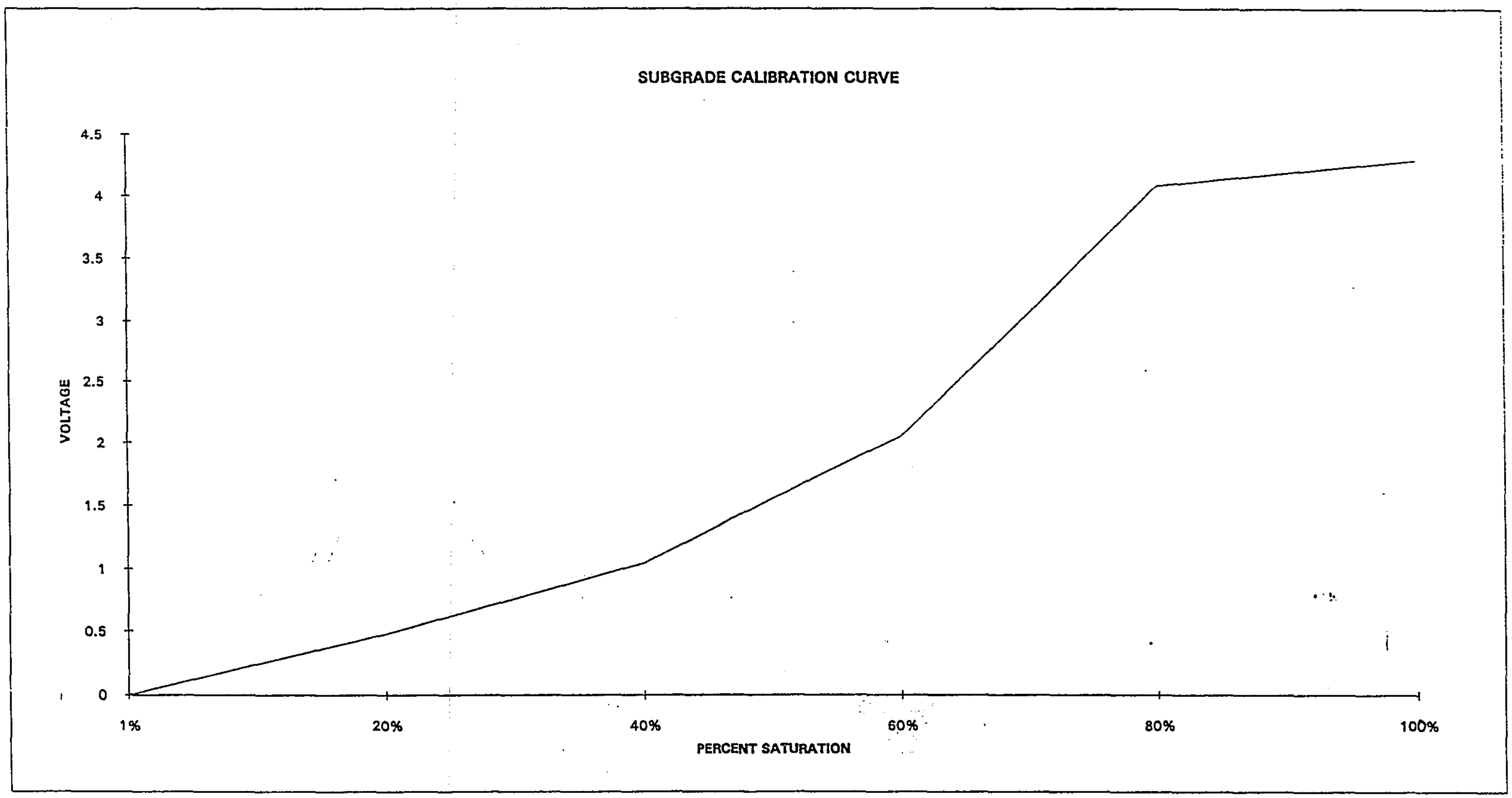
Head (in.)	Head Drop (in.)	Incremental Head Drop (in.)	Time (min.)	Incremental Time (min.)	Incremental Infiltration Rate (in/hr)
13.00	0.00	0.00	0.00	0.00	N/A
12.75	0.25	0.25	0.15	0.15	0.74
12.50	0.50	0.25	0.53	0.38	0.29
12.25	0.75	0.25	1.22	0.68	0.16
12.00	1.00	0.25	2.67	1.45	0.08
11.75	1.25	0.25	7.95	5.28	0.02
10.94	2.06	0.81	15.50	7.55	0.05

APPENDIX C

FIELD MOISTURE MEASUREMENTS

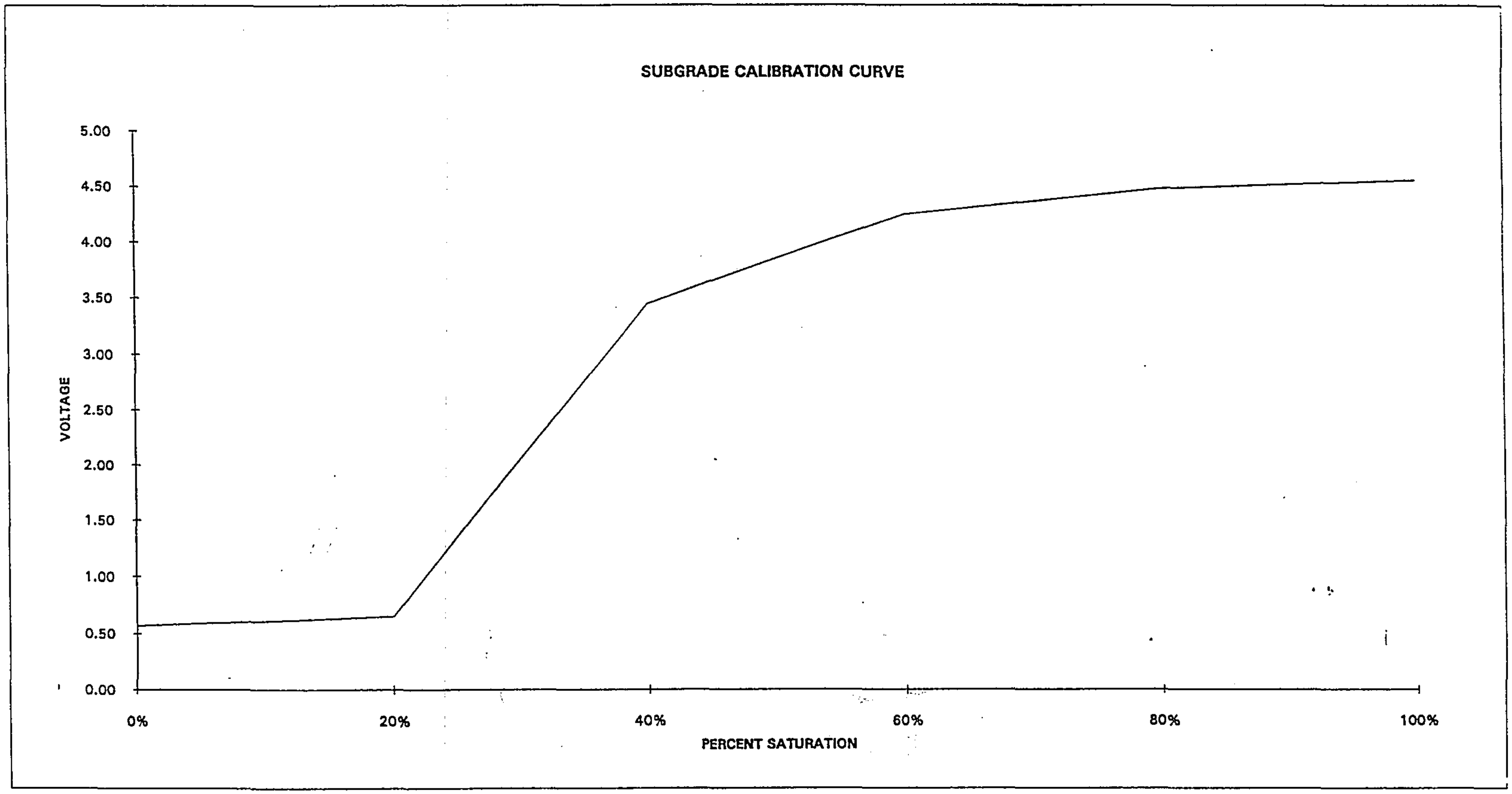
I 459 / GOOD DRAINAGE
SUBGRADE CALIBRATION CURVE

% SATURATION	VOLTAGE
1%	0.001
20%	0.479
40%	1.05
60%	2.05
80%	4.08
100%	4.29



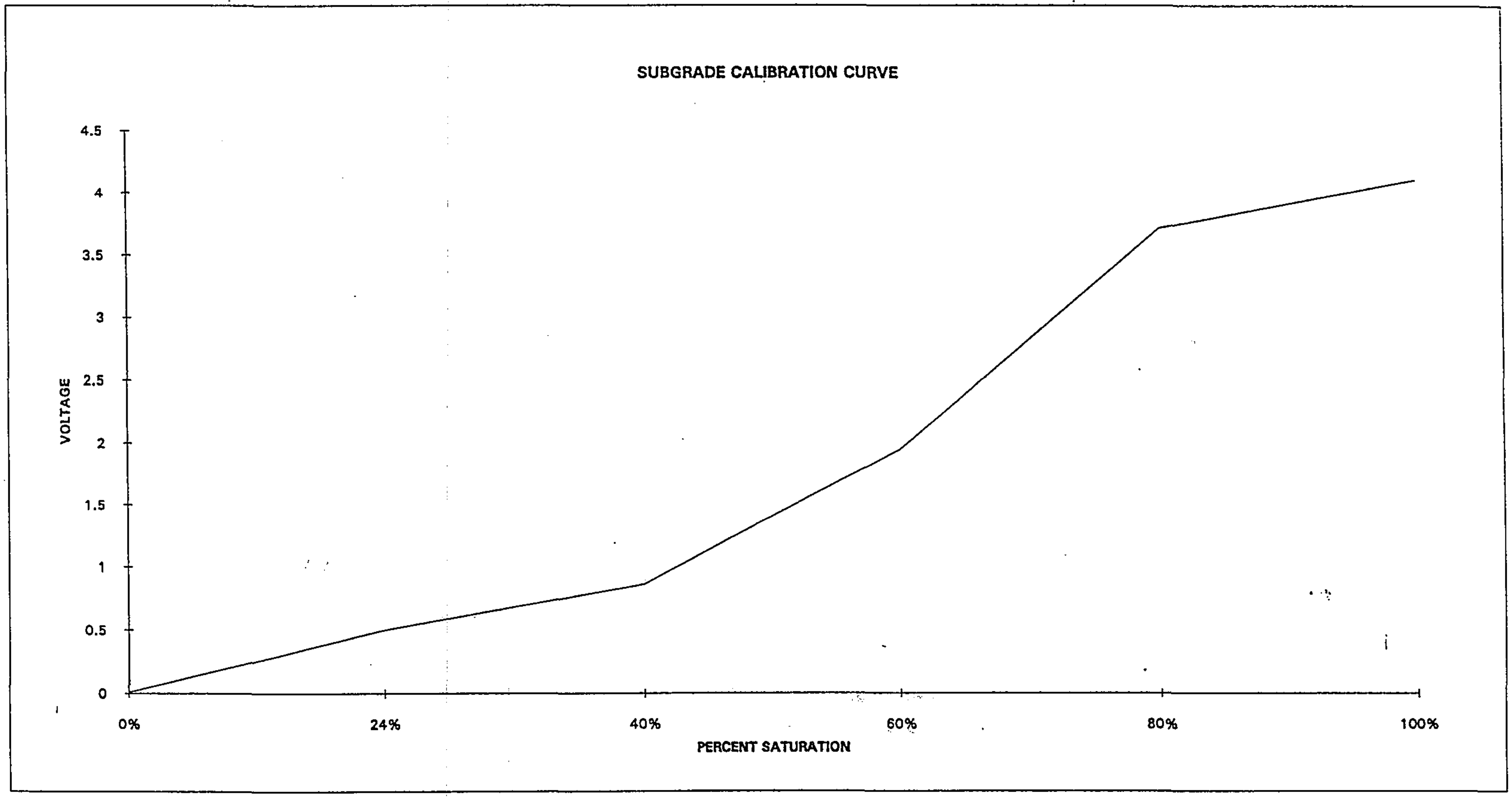
I 459 / POOR DRAINAGE
SUBGRADE CALIBRATION CURVE

% SATURATION	VOLTAGE
0%	0.57
20%	0.65
40%	3.44
60%	4.24
80%	4.47
100%	4.55



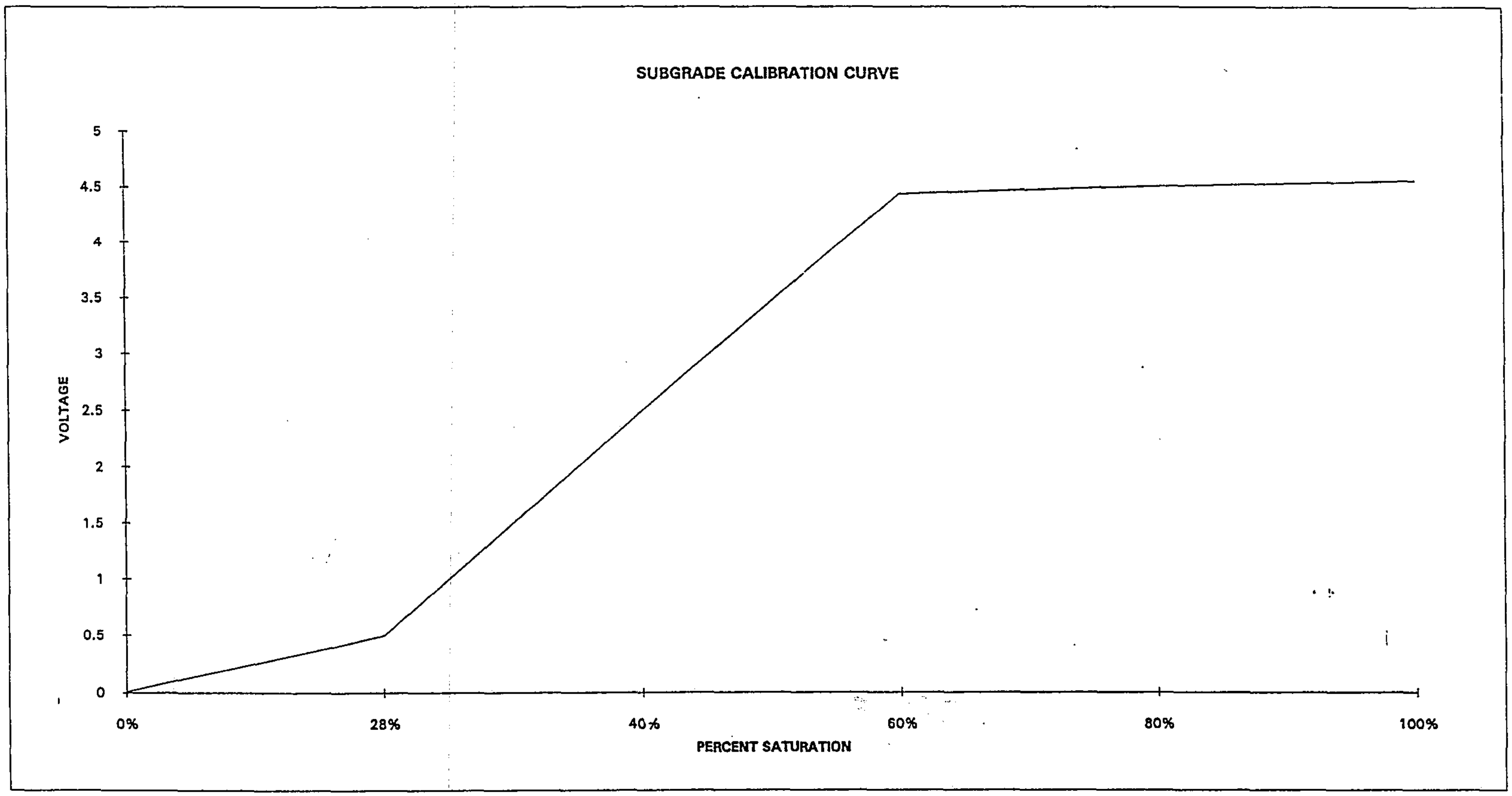
HIGHWAY 79 / GOOD DRAINAGE
SUBGRADE CALIBRATION CURVE

% SATURATION	VOLTAGE
0%	0.01
24%	0.502
40%	0.862
60%	1.94
80%	3.71
100%	4.1



HIGHWAY 79 / POOR DRAINAGE
SUBGRADE CALIBRATION CURVE

% SATURATION	VOLTAGE
0%	0.01
28%	0.497
40%	2.49
60%	4.43
80%	4.5
100%	4.55



Interstate - 459

Good Drainage Location

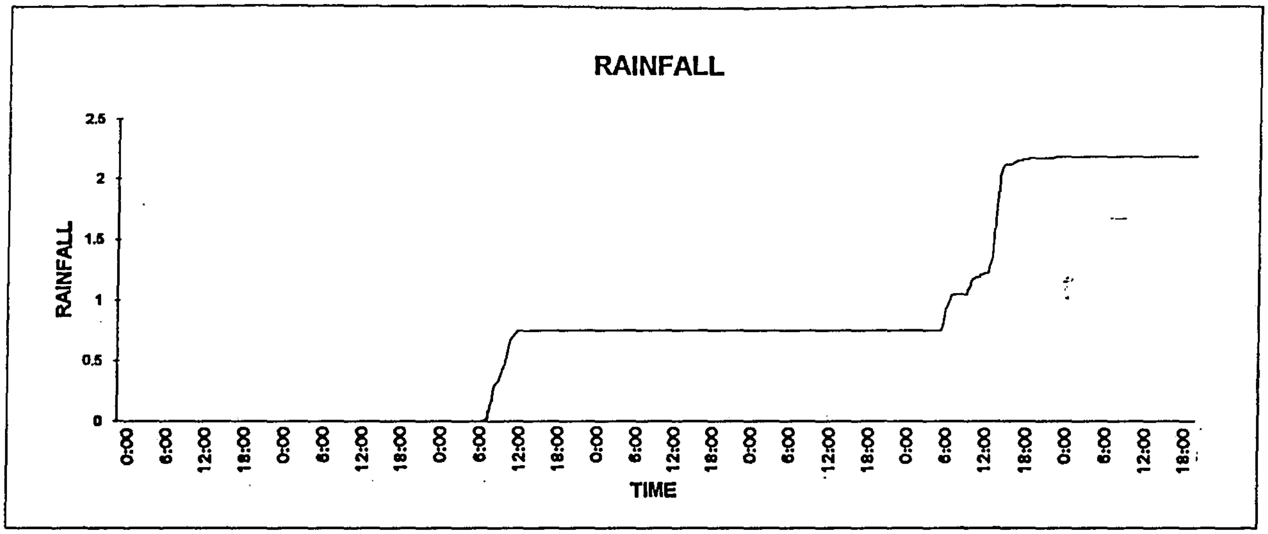
Field Moisture Data

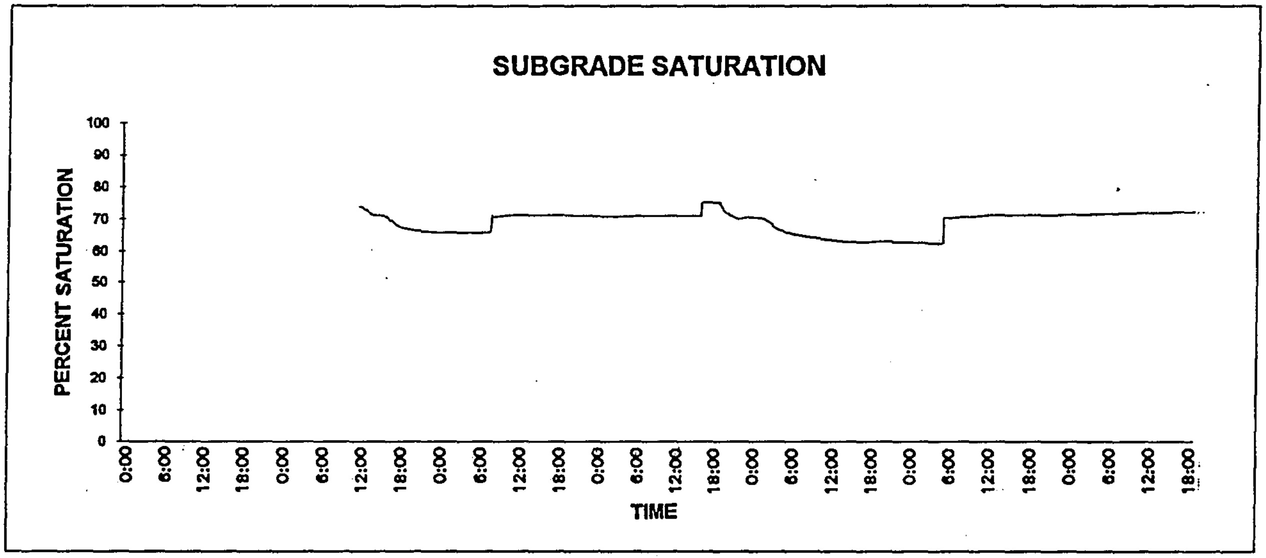
Interstate - 459

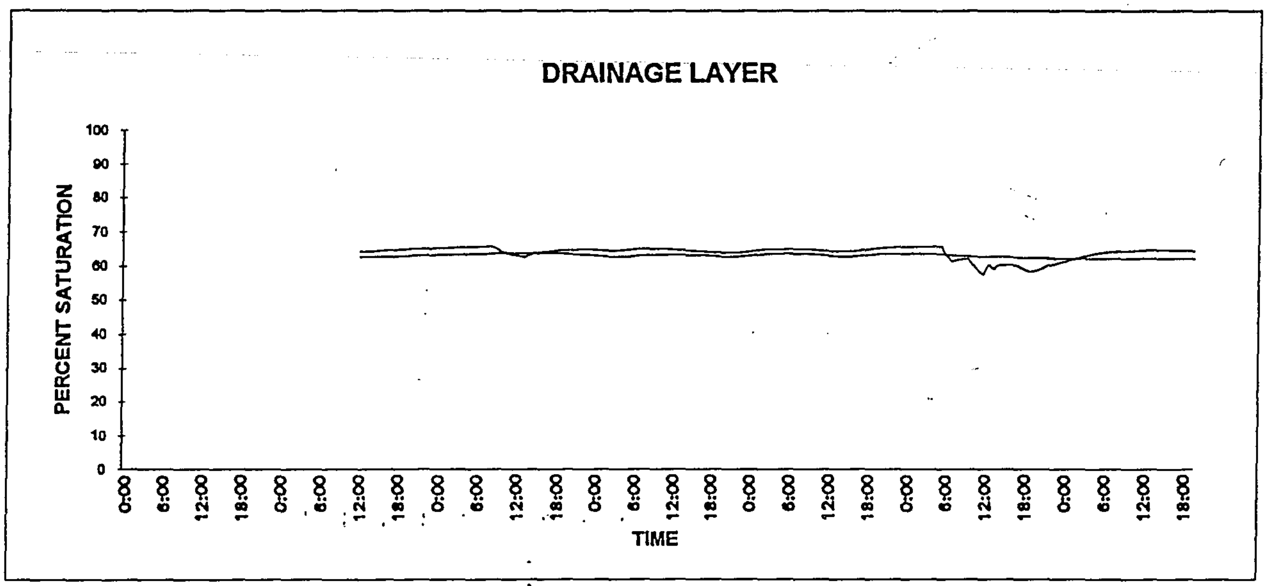
Poor Drainage Location

Field Moisture Data

Interstate 459
Good Drainage Location
1/4/94 - 1/10/94

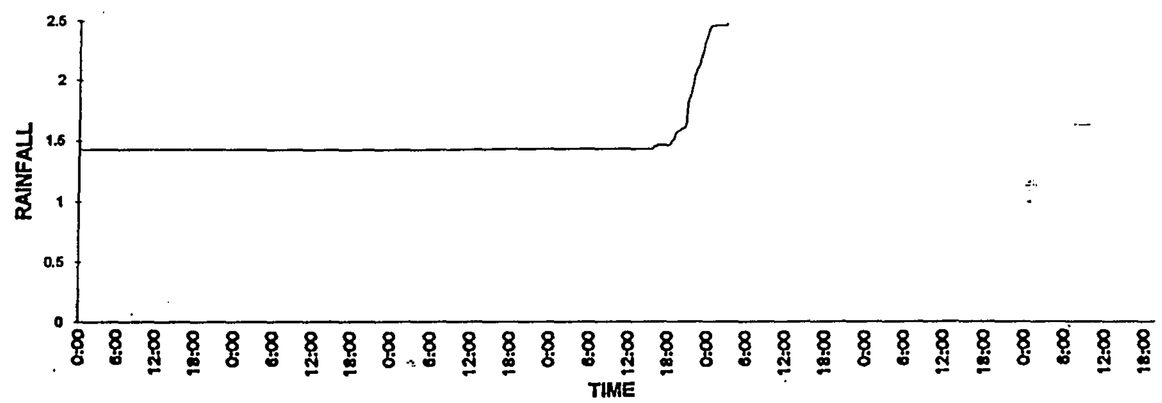




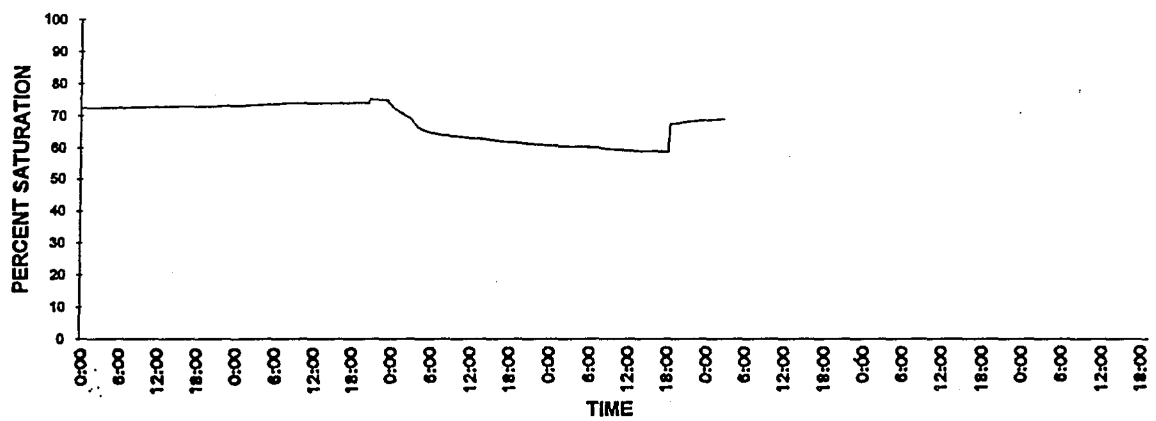


Interstate 459
Good Drainage Location
1/11/94 - 1/17/94

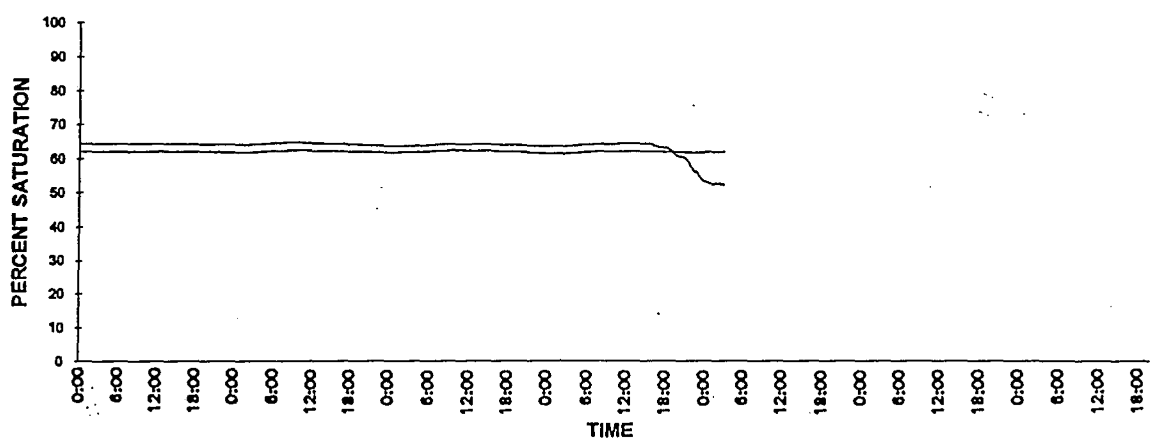
RAINFALL



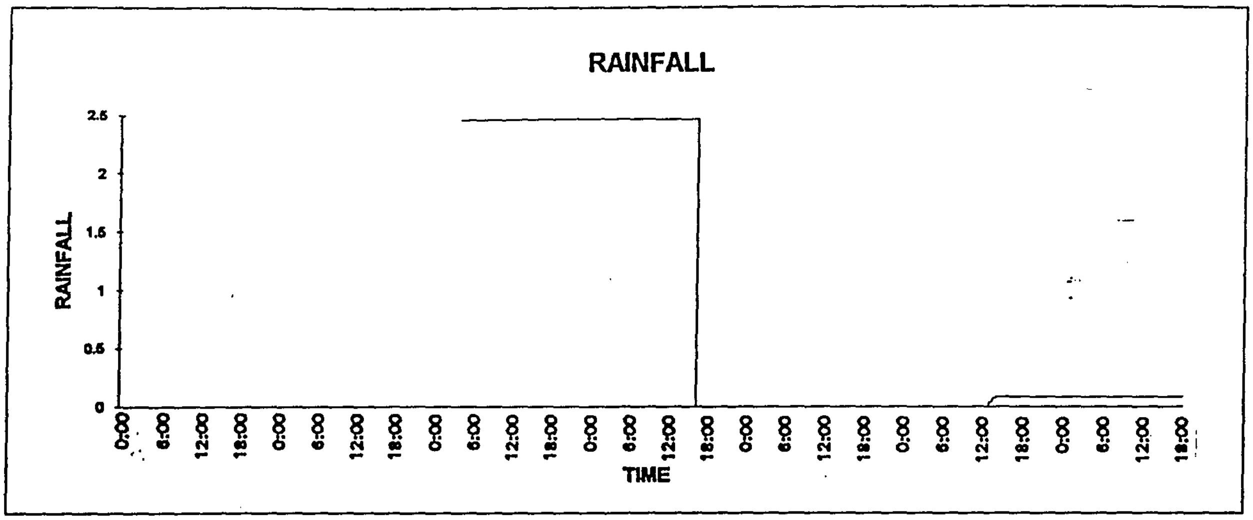
SUBGRADE SATURATION

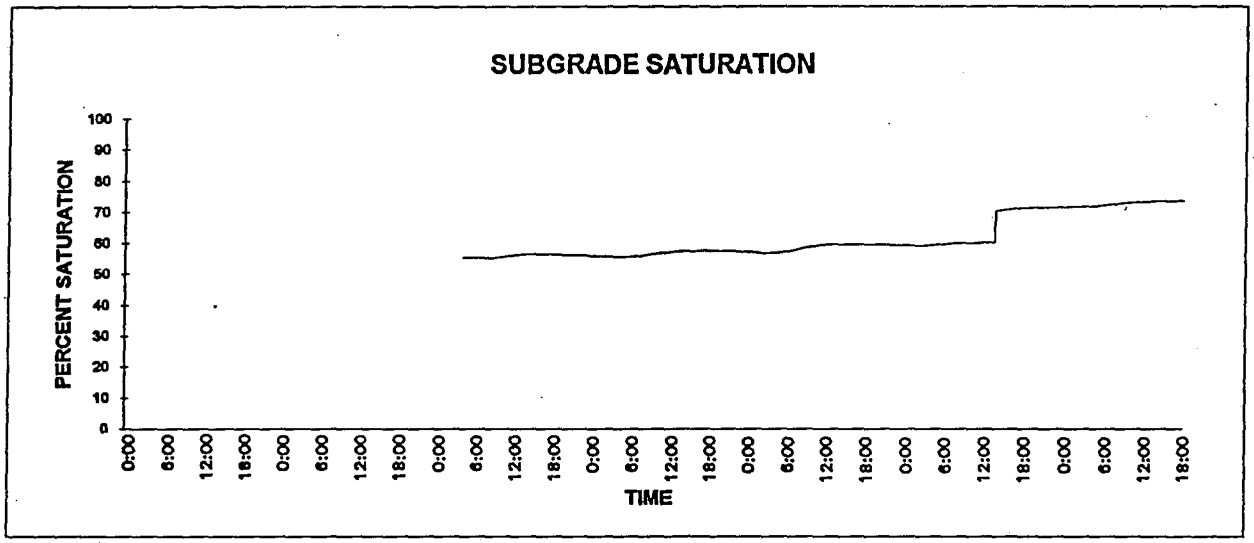


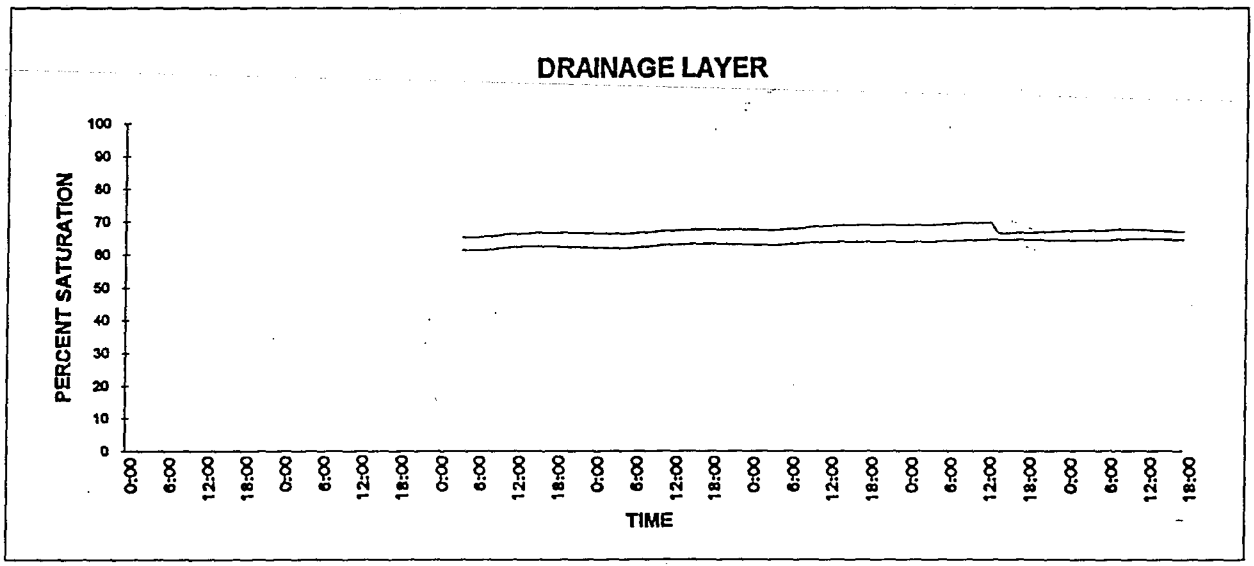
DRAINAGE LAYER



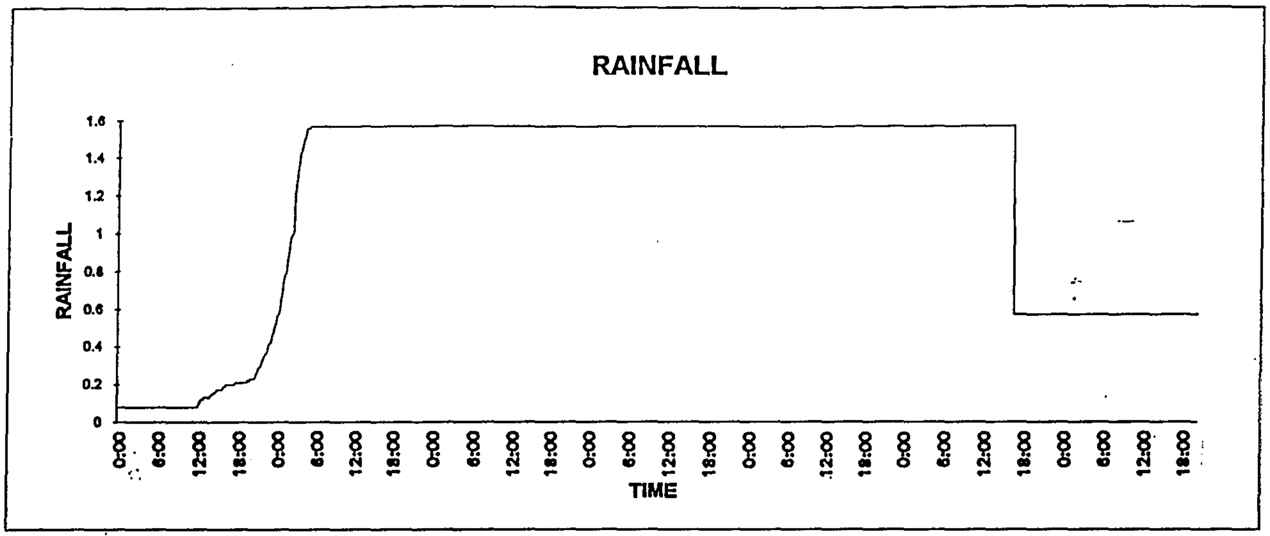
Interstate 459
Good Drainage Location
1/18/94 - 1/24/94

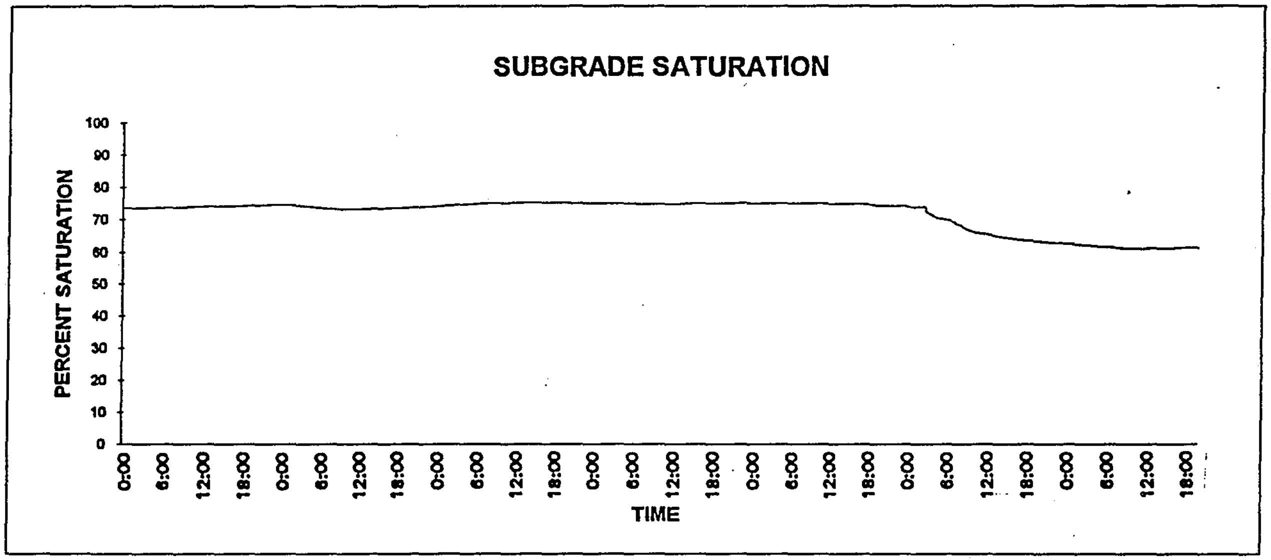


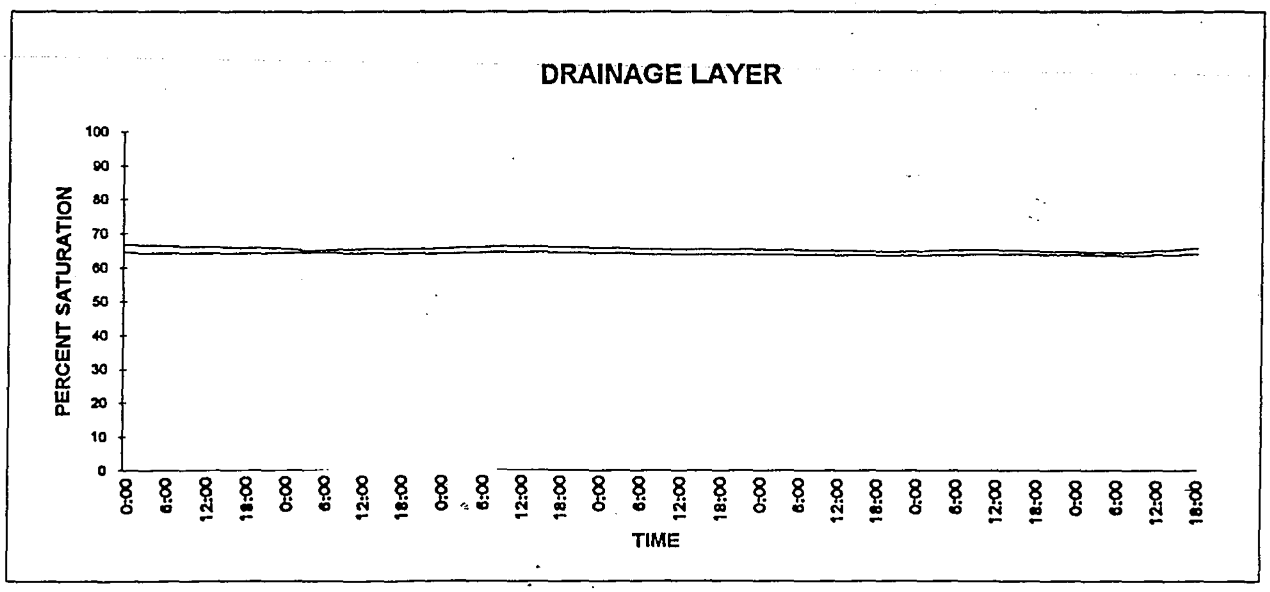




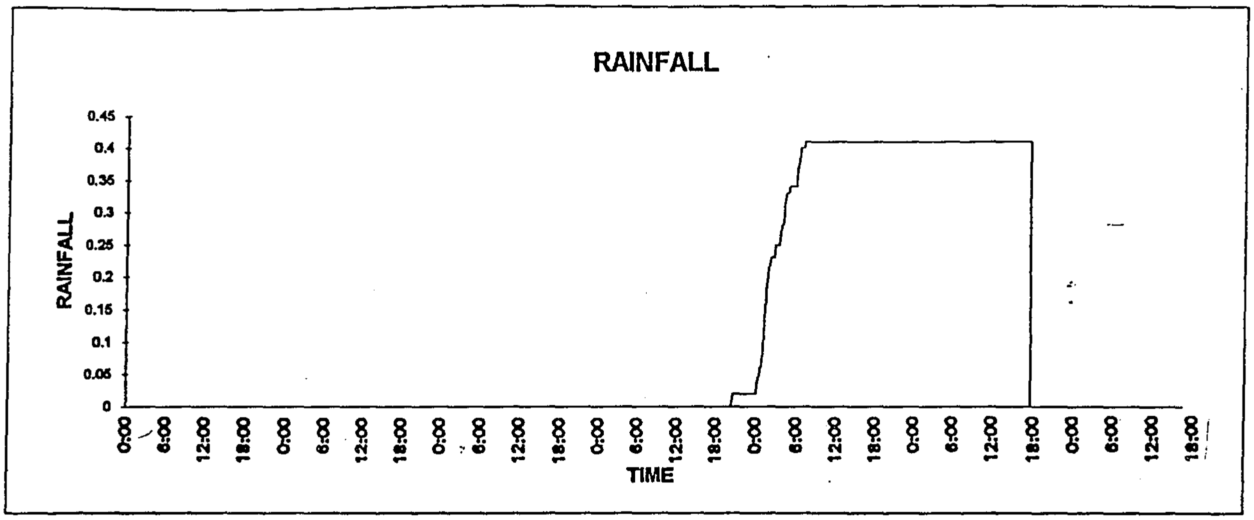
Interstate 459
Good Drainage Location
1/25/94 - 1/31/94

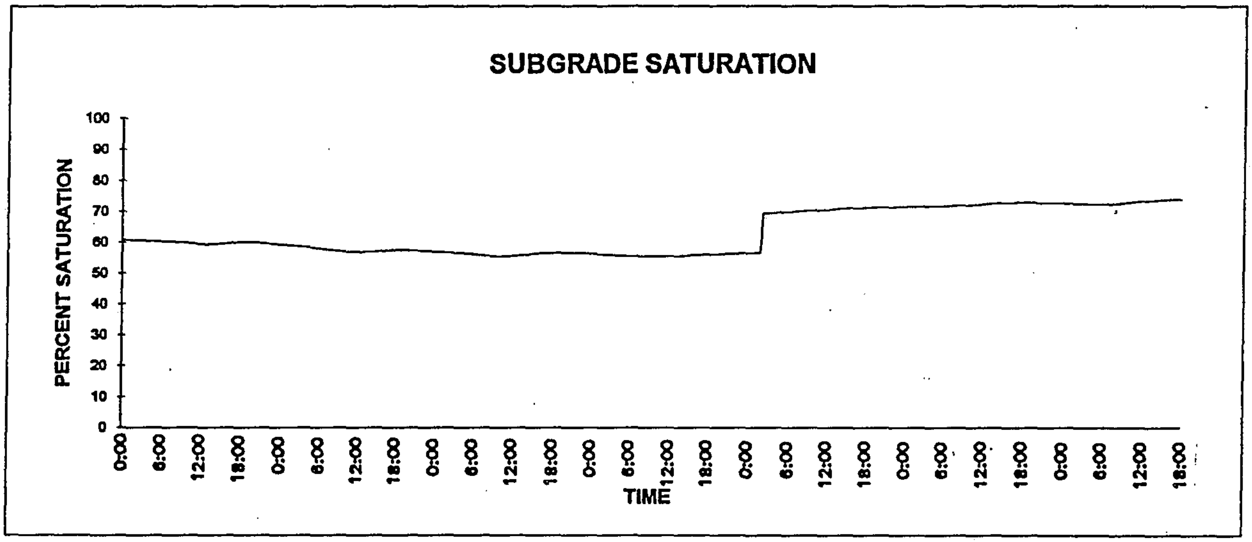


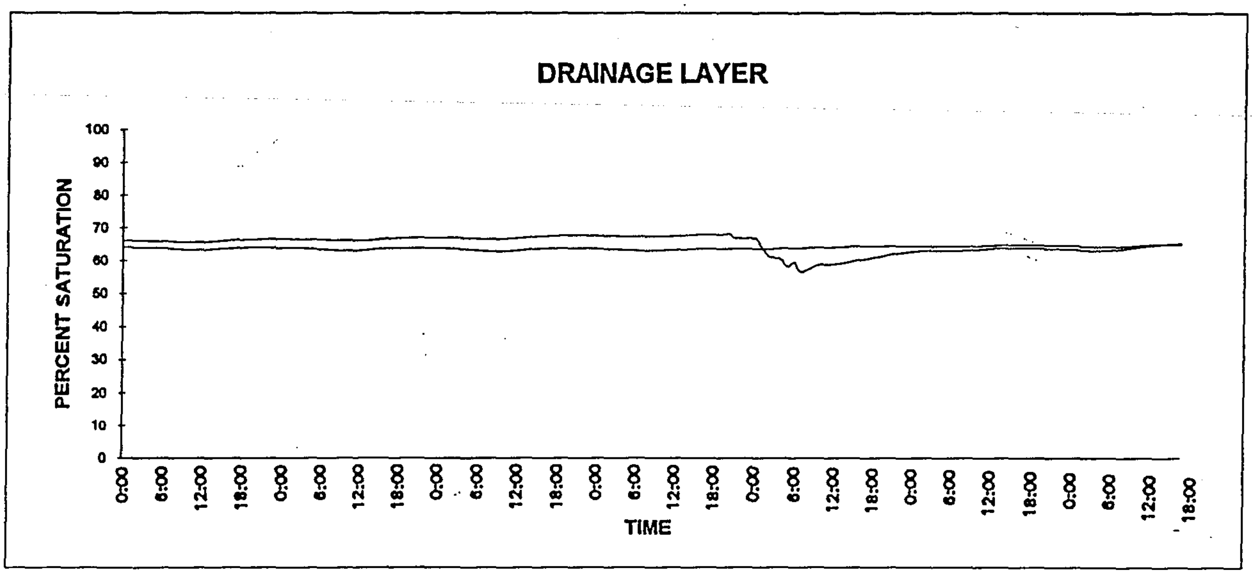




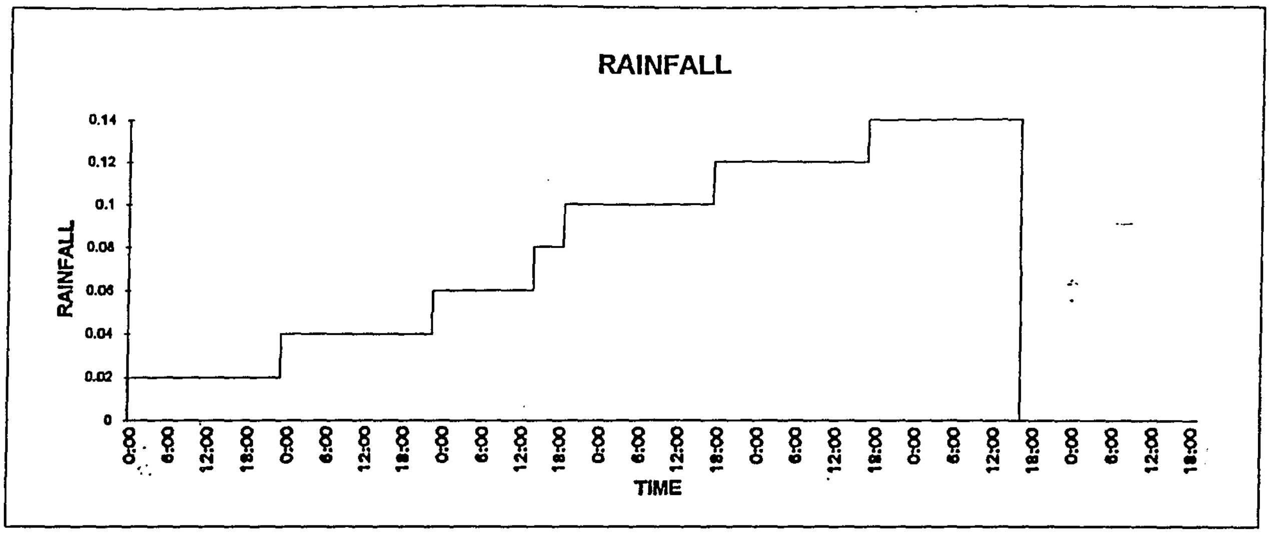
Interstate 459
Good Drainage Location
2/1/94 - 2/7/94

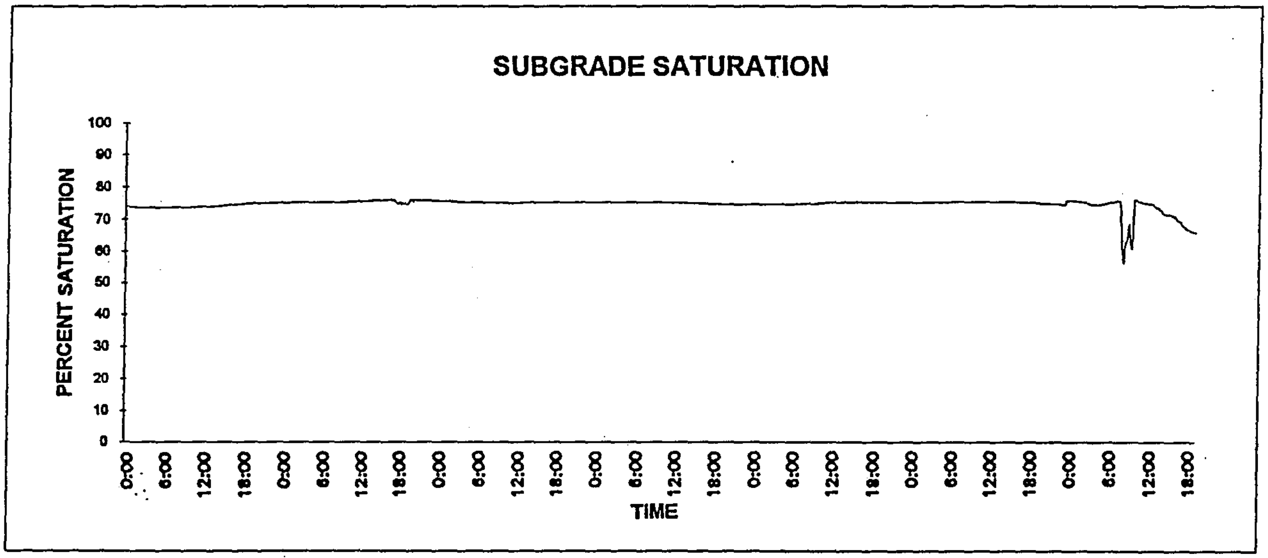


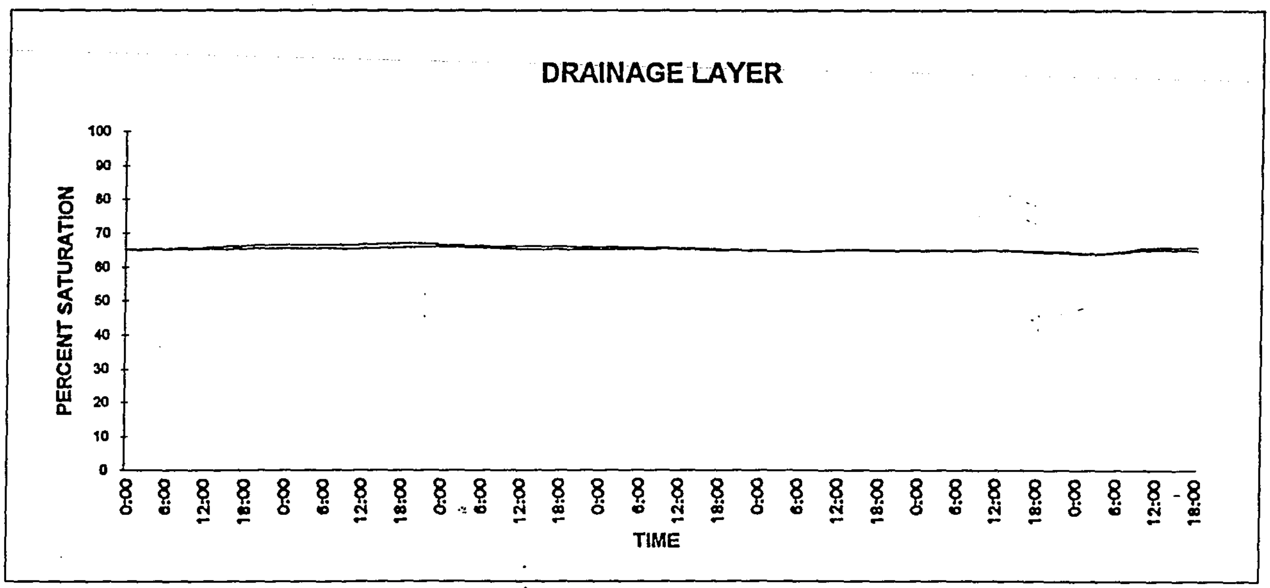




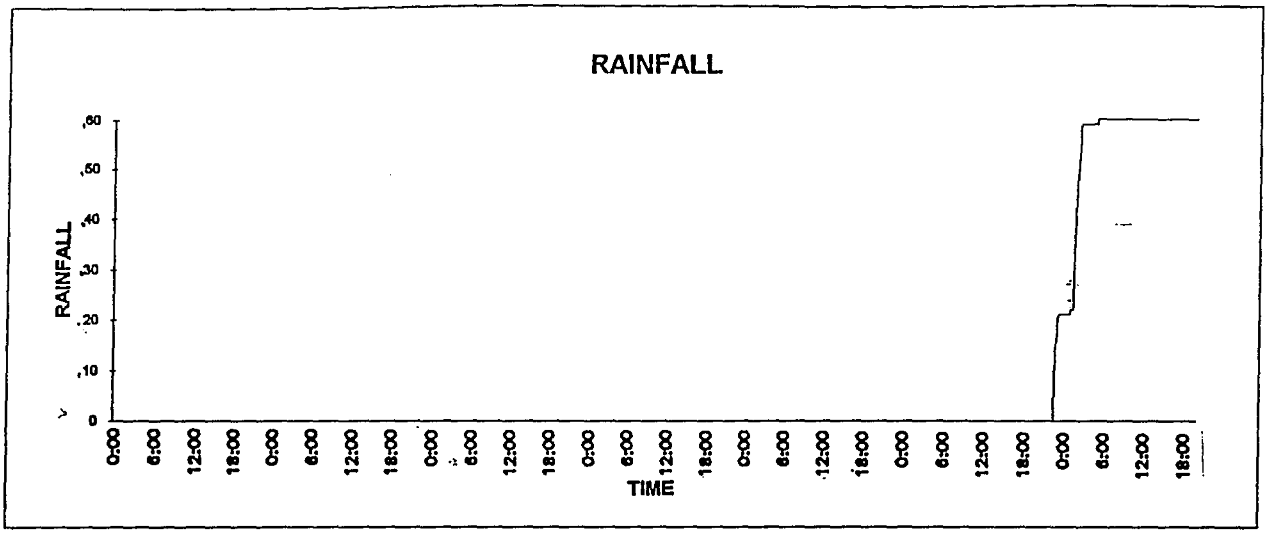
Interstate 459
Good Drainage Location
2/8/94 - 2/14/94

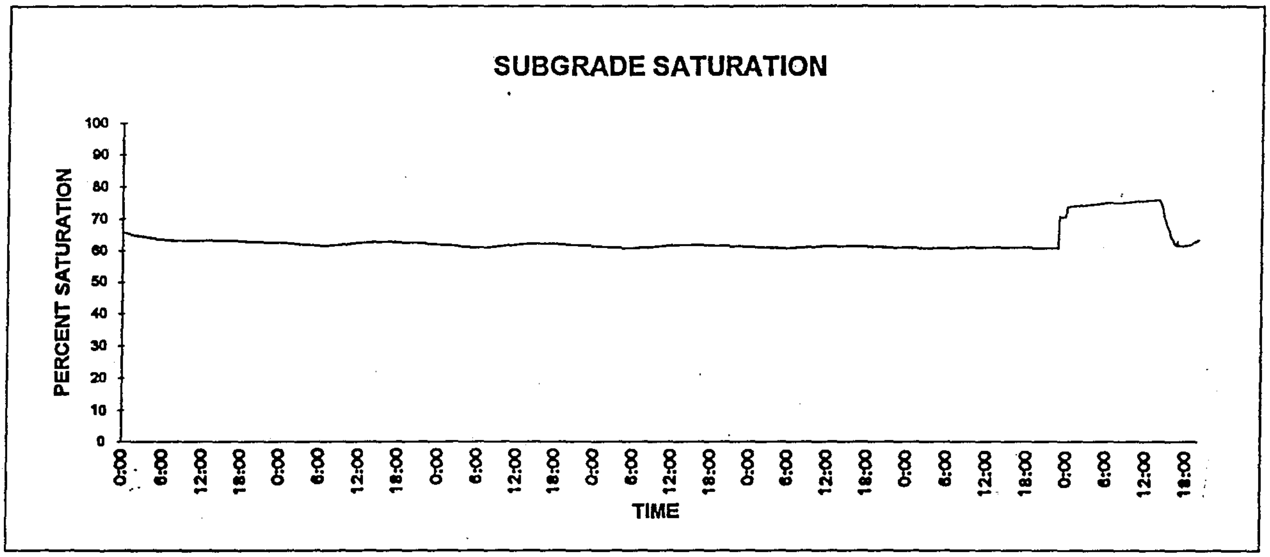


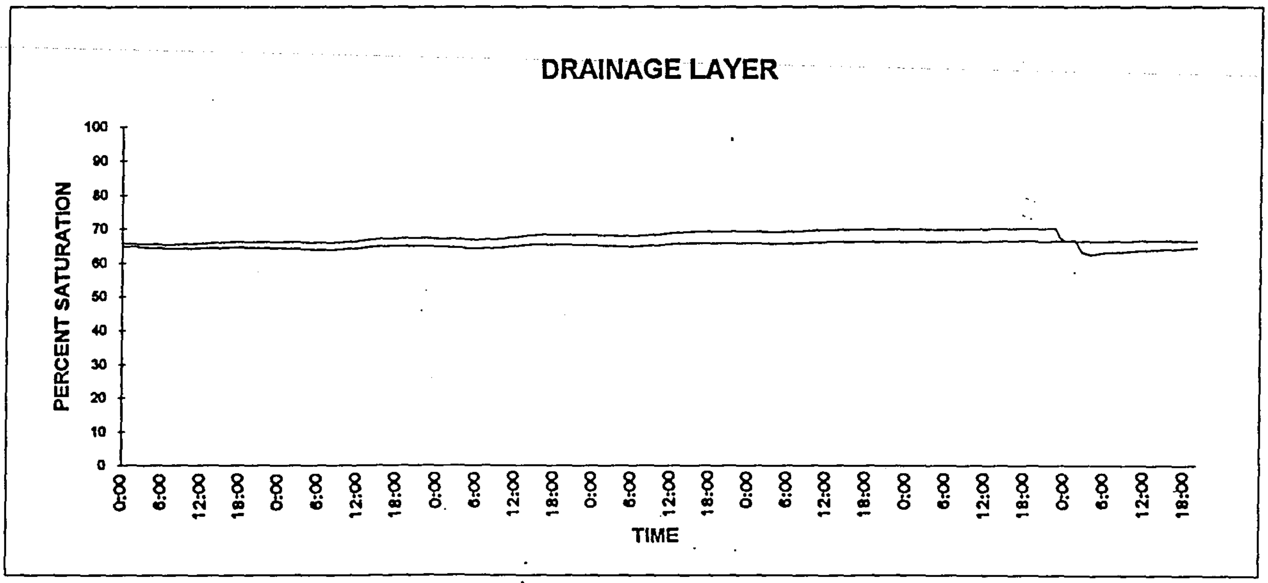




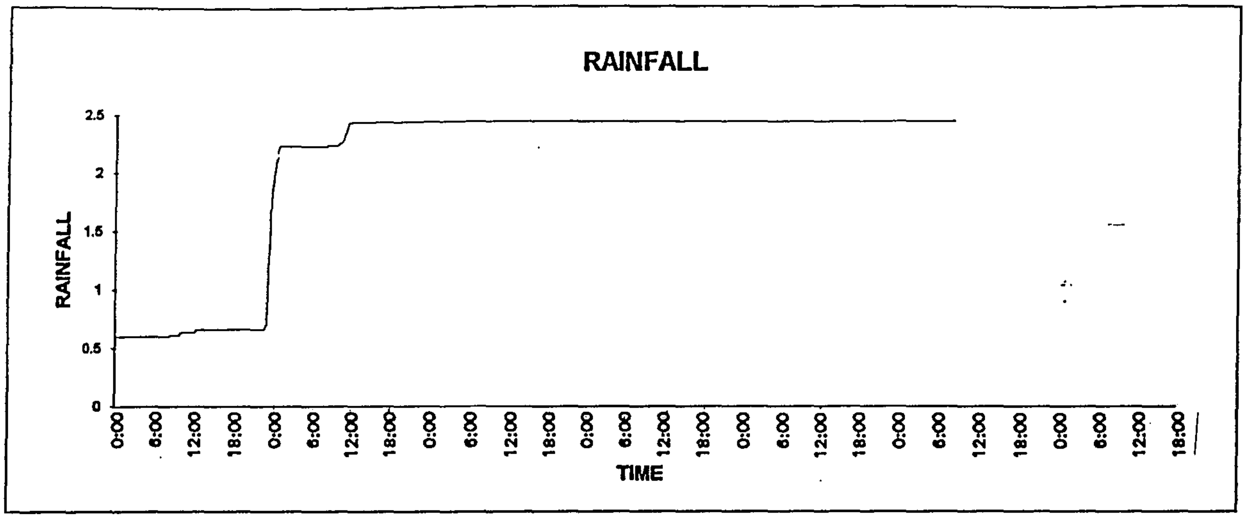
Interstate 459
Good Drainage Location
2/15/94 - 2/21/94

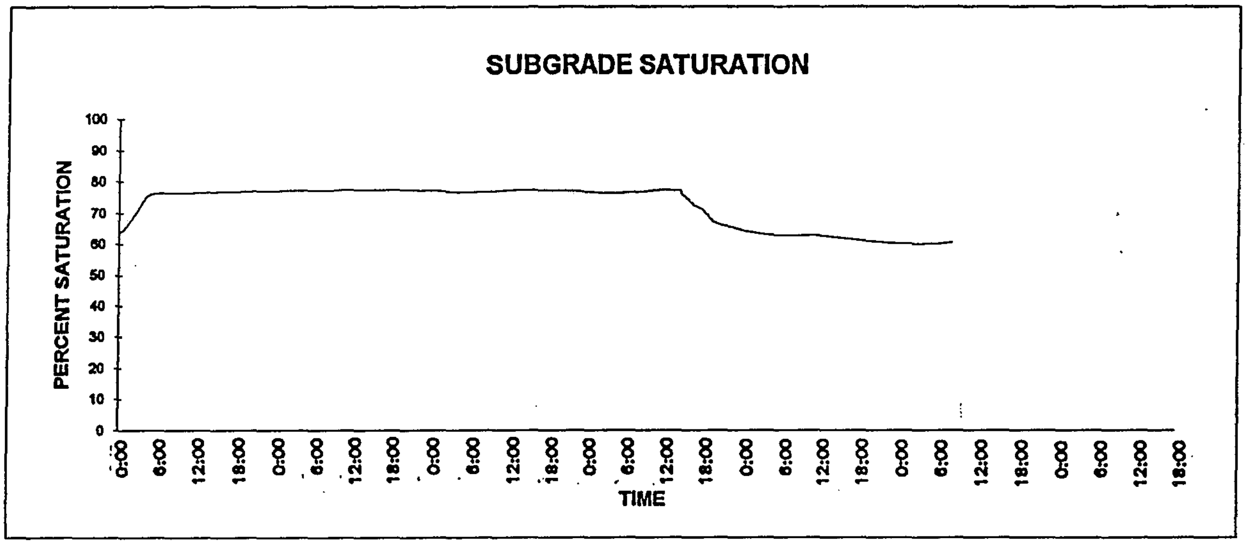


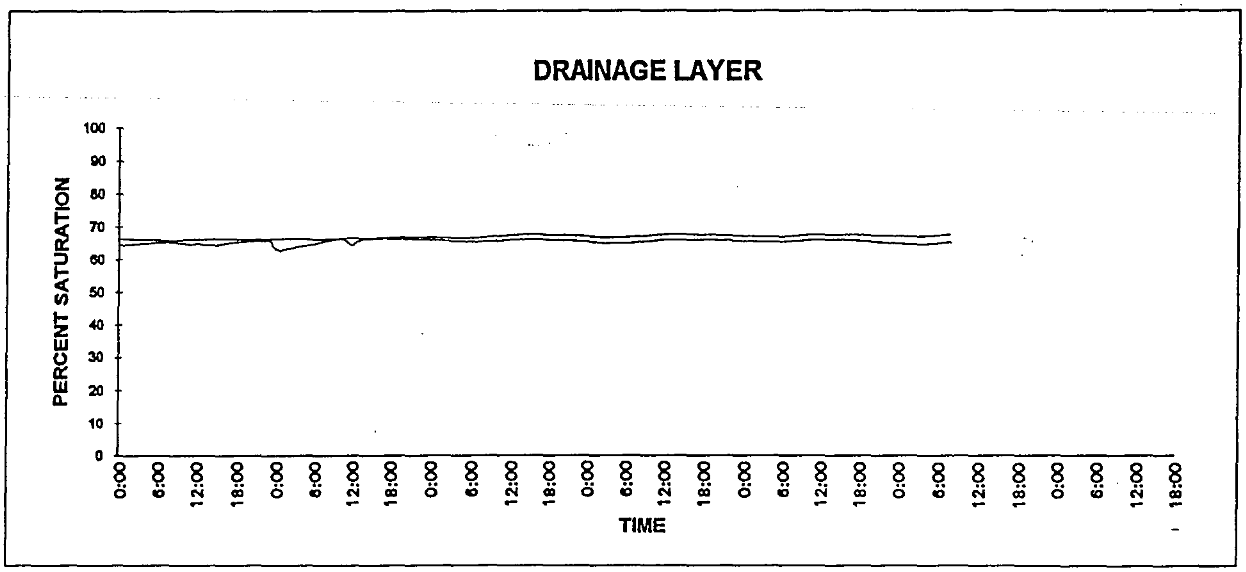




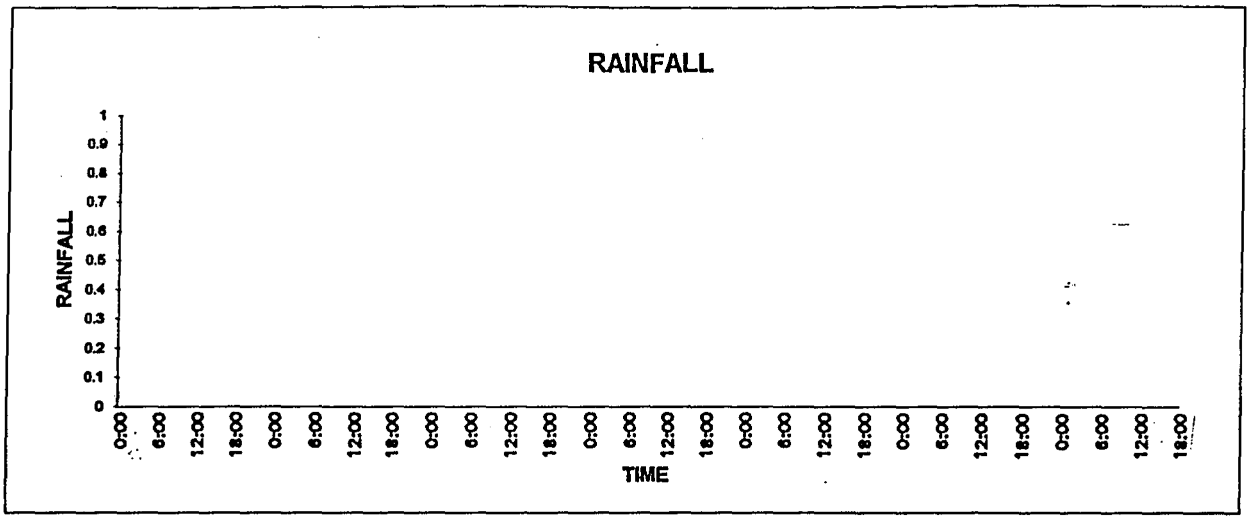
Interstate 459
Good Drainage Location
2/22/94 - 2/28/94

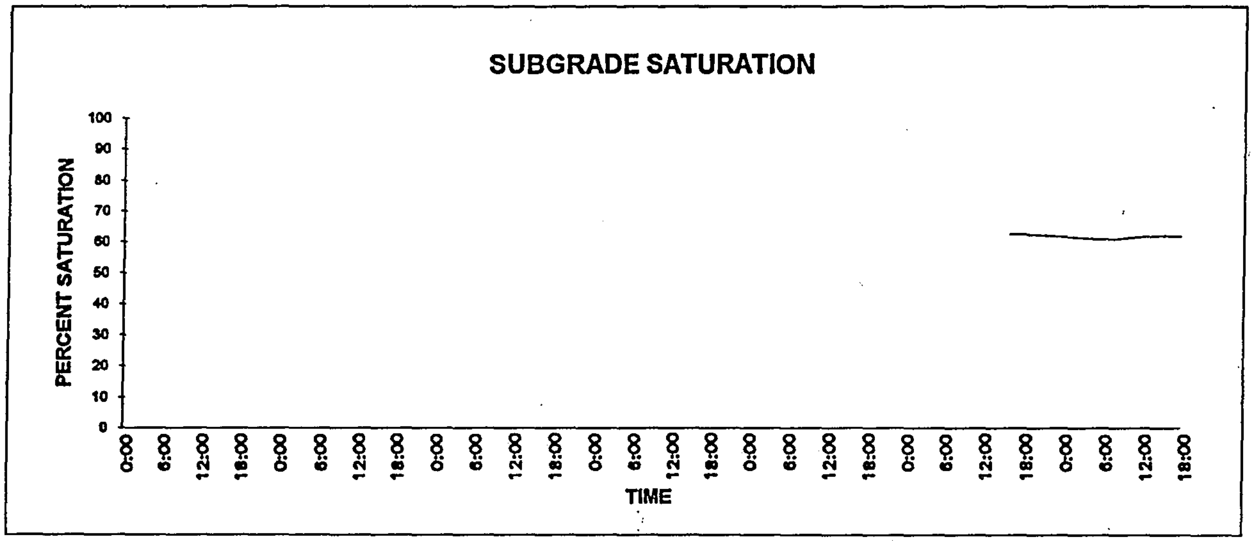


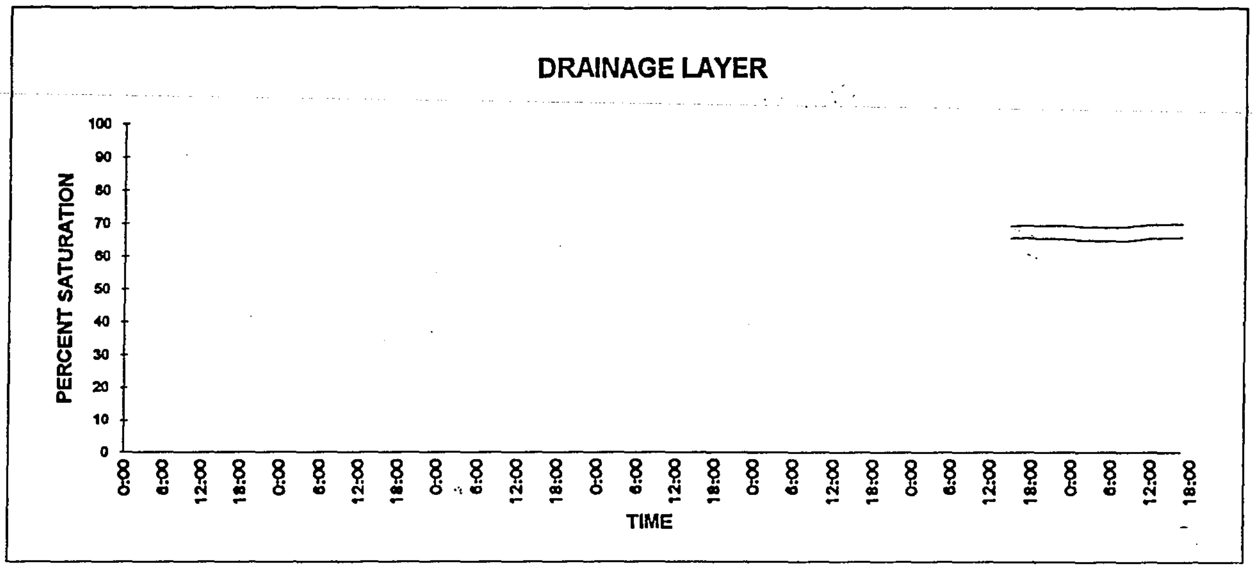




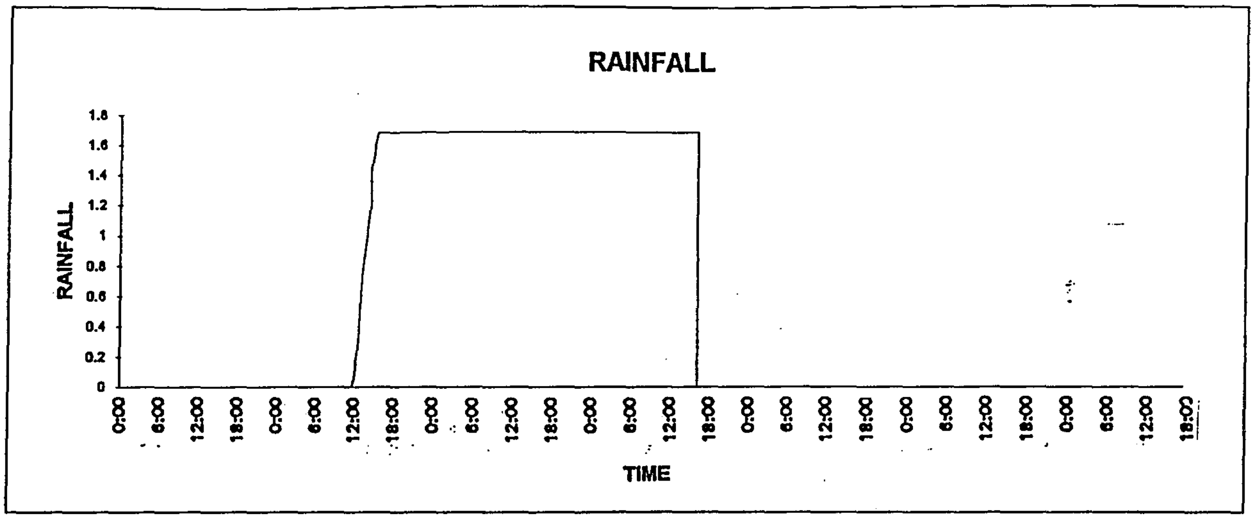
Interstate 459
Good Drainage Location
3/1/94 - 3/7/94

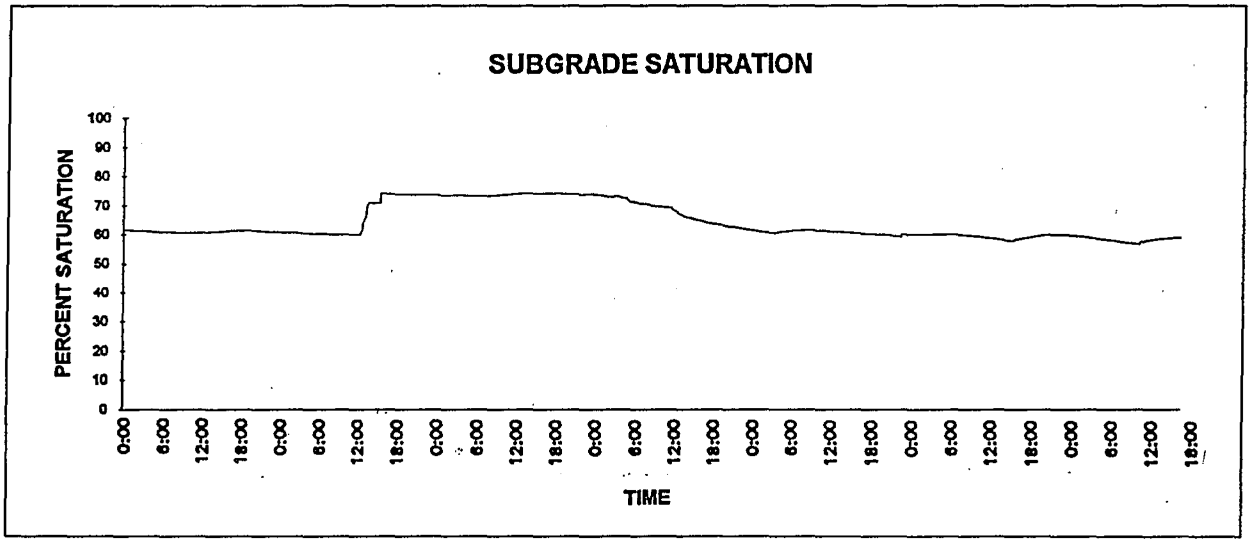


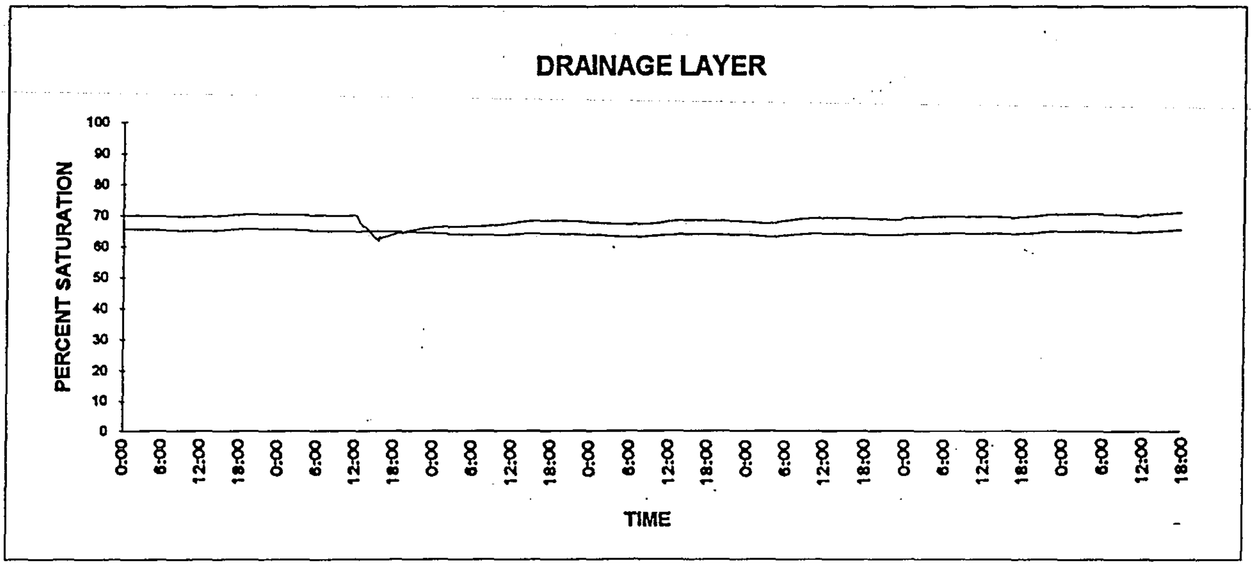




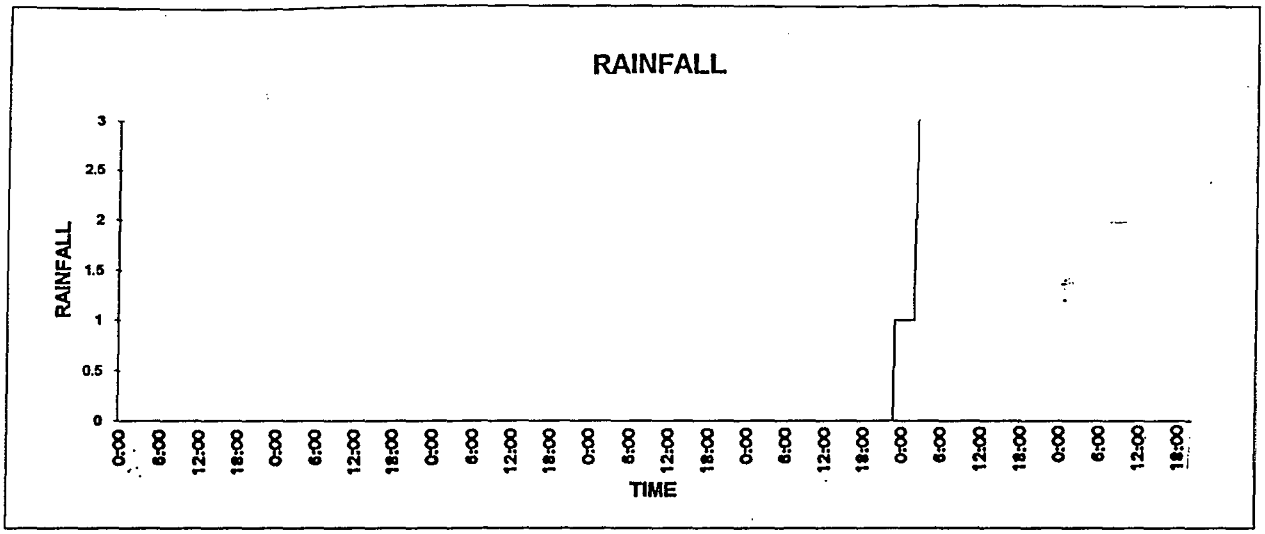
Interstate 459
Good Drainage Location
3/8/94 - 3/14/94

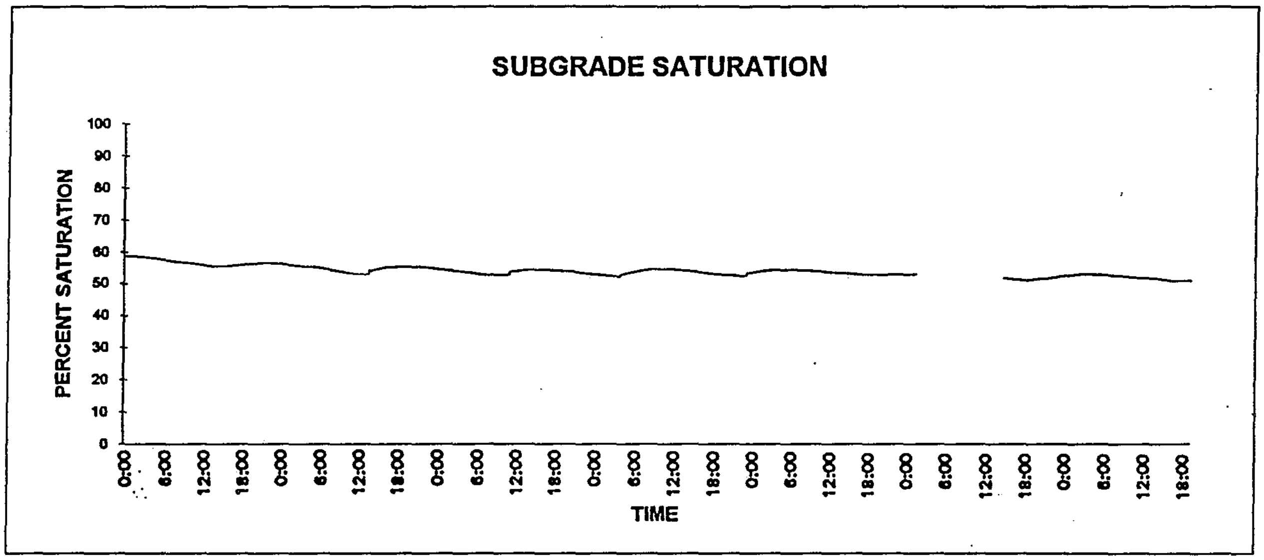


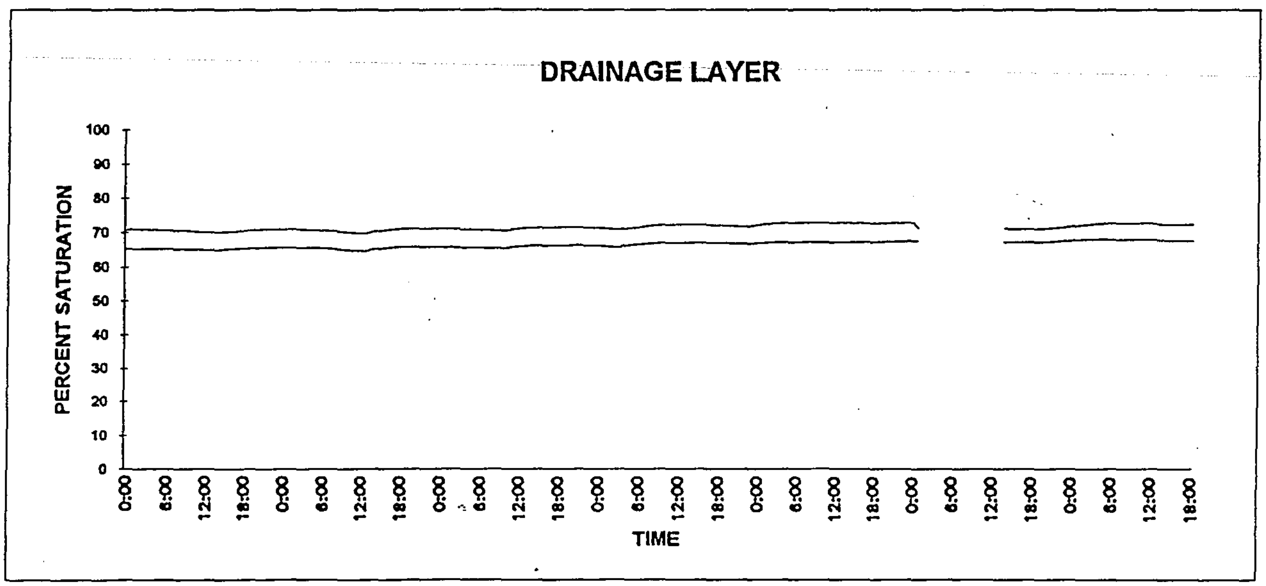




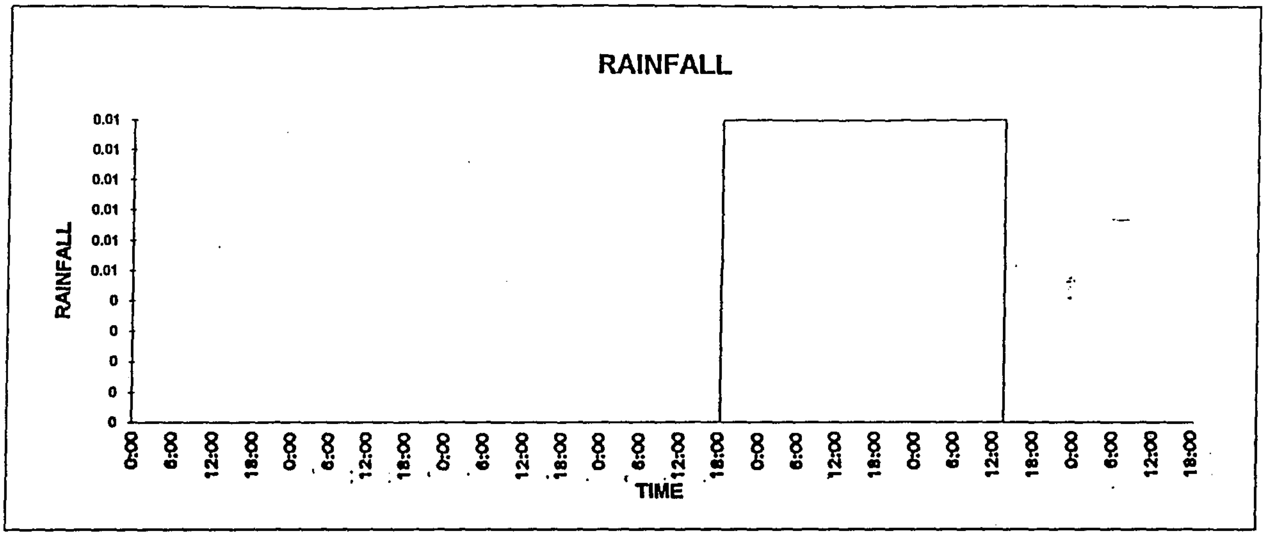
Interstate 459
Good Drainage Location
3/15/94 - 3/21/94

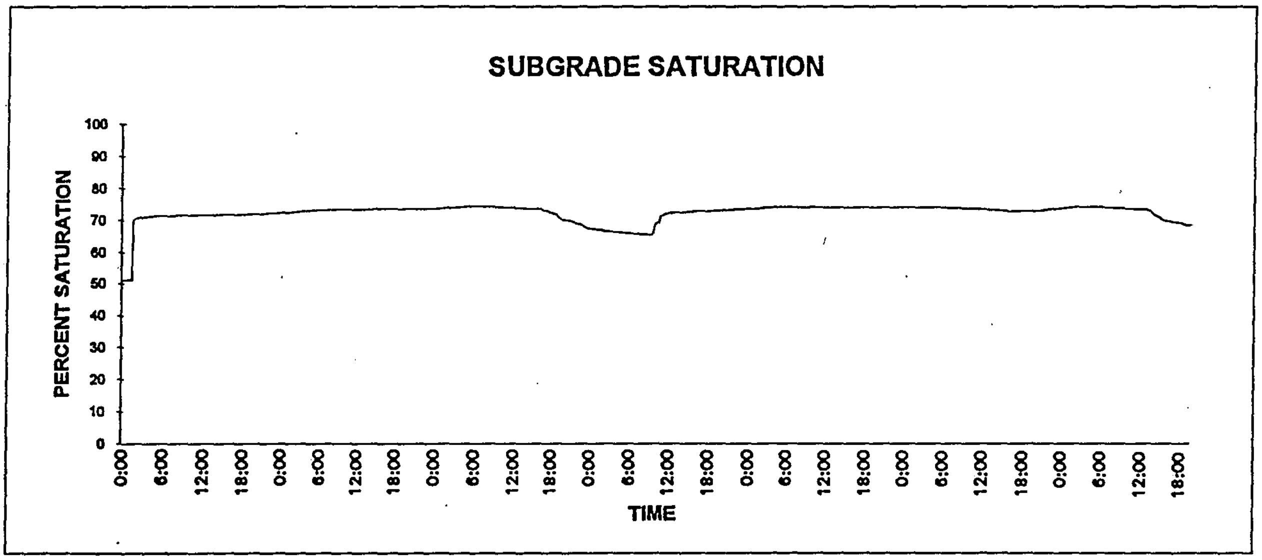


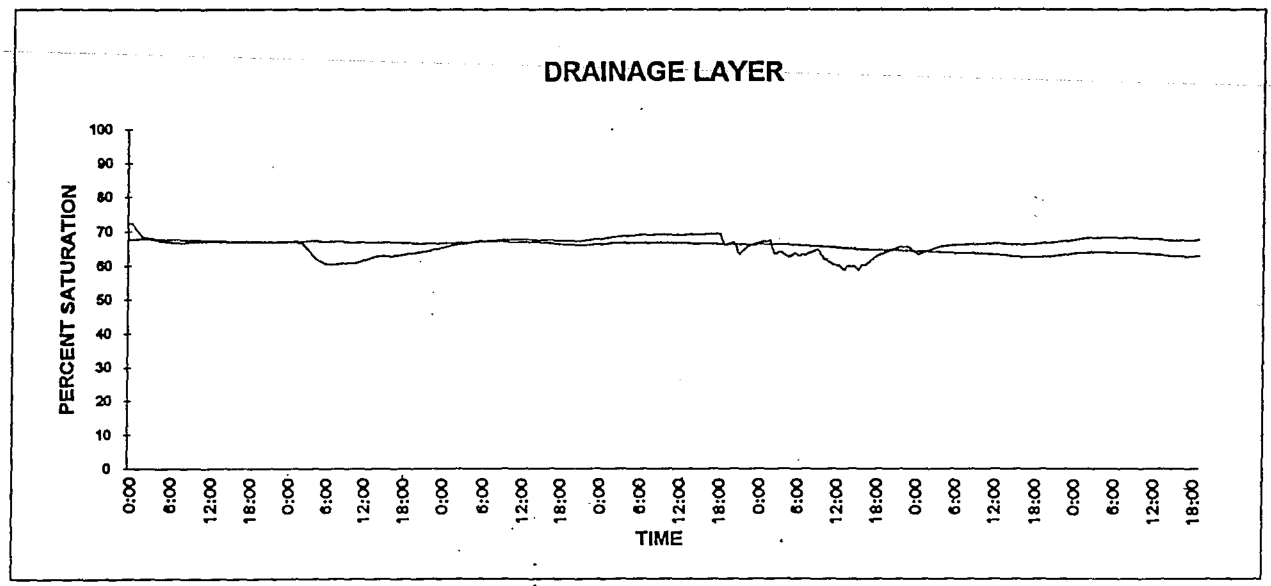




Interstate 459
Good Drainage Location
3/22/94 - 3/28/94







Interstate 459
Good Drainage Location
3/29/94 - 4/4/94