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Evaluation of Airfield Pavement Drainage Layers

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Geotechnical and Structures Laboratory

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Geotechnical and Structures Laboratory
U.S. Army Engineer Research and Development Center
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

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139 Barnes Avenue, Suite 1
Tyndall AFB, FL 32403-5319

Abstract: During the period August to November 2008, airfield pavement drainage layers at Elmendorf Air Force Base, Alaska, Tinker Air Force Base, Oklahoma, and Fort Bliss, Texas, were observed for the purpose of evaluating their efficiency and determining if long-term performance justifies the additional cost of installation. Evaluation procedures included the artificial introduction of water into the pavement structure and observation of flow. Pavement performance data were also analyzed. Data from each field testing location were used to determine the effectiveness of the drainage layer. These data were used to provide recommendations for future use of pavement drainage layers on airfields.

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Preface

The project described in this report was sponsored by Headquarters, Air Force Civil Engineer Support Agency, Tyndall Air Force Base, Florida.

Personnel of the U.S. Army Engineer Research and Development Center, Geotechnical and Structures Laboratory (GSL), Vicksburg, MS, prepared this publication. The findings and recommendations presented in this report are based upon evaluations of drainage layers performed at Elmendorf Air Force Base, Alaska; Tinker Air Force Base, Oklahoma; and Fort Bliss, Texas, during the period of August to November 2008. The evaluation team consisted of John F. Rushing, Mariely Mejías-Santiago, Terry V. Jobe, Blake Andrews, and James Rowland, Airfields and Pavements Branch (APB), GSL. Rushing and Mejías, APB, prepared this publication under the supervision of Dr. Gary L. Anderton, Chief, APB; Dr. Larry N. Lynch, Chief, Engineering Systems and Materials Division; Dr. William P. Grogan, Deputy Director, GSL; and Dr. David W. Pittman, Director, GSL.

COL Gary E. Johnston was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

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Unit Conversion Factors

Multiply	By	To Obtain
acres	4046.873	square meters
cubic feet	0.02831685	cubic meters
degrees Fahrenheit	(F-32)/1.8	degrees Celsius
feet	0.3048	meters
gallons (U.S. liquid)	3.785412 E-03	cubic meters
inches	0.0254	meters
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.45359237	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
square feet	0.09290304	square meters

Summary

Personnel of the U.S. Army Engineer Research and Development Center (ERDC), Vicksburg, MS, observed airfield pavement drainage layers during the period August to November 2008 to evaluate their efficiency and to determine if long-term performance justified the additional cost of drainage layer installation. Field tests took place at Elmendorf Air Force Base in Anchorage, Alaska; Tinker Air Force Base in Oklahoma City, Oklahoma; and Biggs Army Airfield, Fort Bliss, Texas. The effectiveness of the airfield pavement drainage layers was studied by measuring the water flow through the drainage system. Data from these tests and other observations at the field test sites were used to provide future guidance for the use of pavement drainage layers on airfields.

The results of the tests and evaluation reveal the following:

- Design and construction both play important roles in the functionality of airfield pavement drainage layers. Improper oversight of either can lead to a poorly performing system. Several pavement areas observed in this study were not functioning properly as a result of poor design or construction.
- Evidence of routine maintenance of pavement drainage systems was not observed on any of the airfields evaluated in this study. A lack of maintenance could inhibit the flow of water and reduce the functionality of the drainage system.
- Permeability rates through the drainage layers meeting the aggregate gradation specifications were within acceptable limits.
- Pavement drainage layers that are daylighted to the edge of the pavement are able to remove water through multiple pathways and are less likely to have flow interrupted by a lack of maintenance.
- Differences in the performance of pavements with and without drainage layers could not be ascertained. Deterioration rates of the pavements evaluated did not demonstrate statistical differences in their condition. Pavements of the same age constructed without drainage layers were in similar condition to those constructed with drainage layers.
- The ground penetrating radar (GPR) provided a useful tool for determining the location of moisture in the drainage layer beneath

asphalt concrete (AC) pavement. The depth of penetration of the GPR was too shallow to locate moisture in the drainage layer beneath thick portland cement concrete (PCC) pavements.

- Flow measurements provided sufficient data for quantifying the functionality of the drainage system.
- The climatic region in which the pavement is located will impact the amount of water that potentially enters the pavement and can be removed through the use of pavement drainage layers.

Until further testing and analyses are conducted, the following recommendations are offered based upon the results of the field testing of airfield pavement drainage layers:

- Construction of pavement drainage layers should be closely monitored to ensure that they would be functional after construction. Specifications should be followed for all material properties and design considerations.
- A routine maintenance program should be implemented for pavement drainage systems on airfields. Maintenance should include clearing all soil and vegetation from the flow path to prevent clogging.
- Alternate designs for pavement drainage systems should be considered. Daylighted drainage layers are an example of a design that may provide acceptable performance.
- An additional evaluation of pavement performance should be considered in the future when enough deterioration has occurred to determine differences in the performance of pavements constructed with and without drainage layers.

1 Introduction

Introduction

Pavement subsurface drainage systems have been studied for years (Allen 1991; Cedergren et al. 1973; Cedergren 1974; Christopher and McGuffey 1997; Hall and Correa 2003; Hall and Crovetti 2007). It has been recognized that water has a detrimental effect on the pavement performance, especially when the pavement is subjected to the heavy loading of military aircraft traffic on airfields. The weakening of the base, subbase, or subgrade when saturated is one of the main causes of flexible pavement failures. In rigid pavement, the main cause of failures is the pumping of the subgrade material to the surface. Pumping occurs when free water, trapped between the bottom of the rigid concrete layer and the impermeable subgrade, moves due to pressure caused by loading. This movement erodes the subsurface material, creating voids under the concrete layer.

In seasonal frost areas, subsurface water can contribute to frost damage by heaving during freezing and weakening the subgrade during thawing. Secondary damages caused by poor drainage include "D" cracking and swelling of subsurface materials.

Water infiltration into the pavement structure cannot be completely stopped. Therefore, the necessity of a subsurface drainage system capable of moving the water away from the pavement at an acceptable rate of time without compromising the strength of the pavement system resulted in the development of drainage layers. The U.S. Army Corps of Engineers developed the Unified Facilities Criteria (UFC) 3-320-06A (Headquarters, Departments of the Army, the Navy, and the Air Force 2004) to provide guidelines for planning, design, construction, sustainment, and restoration of subsurface drainage systems at military installations. The use of drainage layers to improve the subsurface drainage in military airfield pavement systems has become a very common practice. However, as the construction and installation of drainage layers has increased, so have the discussions of their performance. Some of the issues under discussion are constructibility, added cost of installation, and maintenance.

The project presented in this report consisted of an evaluation of the effectiveness, constructibility, and long-term performance of in-place airfield pavement drainage layers to determine if the additional cost of installation is beneficial and justified.

Objective and scope

The purpose of this research was to evaluate the performance of in-service drainage layers in airfield pavements. In order to accomplish this, several airfield pavements constructed with drainage layers were identified and evaluated. The evaluation consisted of simulating rainfall events using a water truck, where the amount of water applied was measured and controlled by a flowmeter to determine the flow through the layers. Then, ground penetrating radar (GPR) was used to determine if moisture was accumulating in the drainage layer. The structural performance of the drainage layers was evaluated by using pavement condition index (PCI) data for similar pavements with and without drainage layers. Design, construction and maintenance issues were also evaluated to determine their influence in the performance of the pavement drainage layers.

The objectives of this investigation were to evaluate the effectiveness, constructibility, and long-term performance of drainage layers and determine if the additional cost of installation is justified. The scope of this project involved:

- Evaluation of the effectiveness of in-place airfield drainage layers.
- Comparison of structural performance of pavements with the same operational age, design, traffic, and climate that contain drainage layers with pavements that do not contain drainage layers.
- Determination of drainage layer construction and maintenance issues.

This report provides field testing procedures, data analysis, and conclusions to address the benefits gained from drainage layers versus the cost of installation and maintenance.

2 Background

In the design of pavement subdrainage systems, water is considered to come from two sources: infiltration and subterranean water. Surface water is the principal source of water. It enters through cracks or joints in the pavements, infiltrates the pavement surface and through shoulders from adjacent areas. Subterranean water can come from a high water table, capillary forces, artesian pressure, and freeze-thaw action.

Subdrainage systems are designed to remove water that enters the pavement system and reduce water movement into subgrades. An example of a subsurface drainage system is shown in Figure 1.

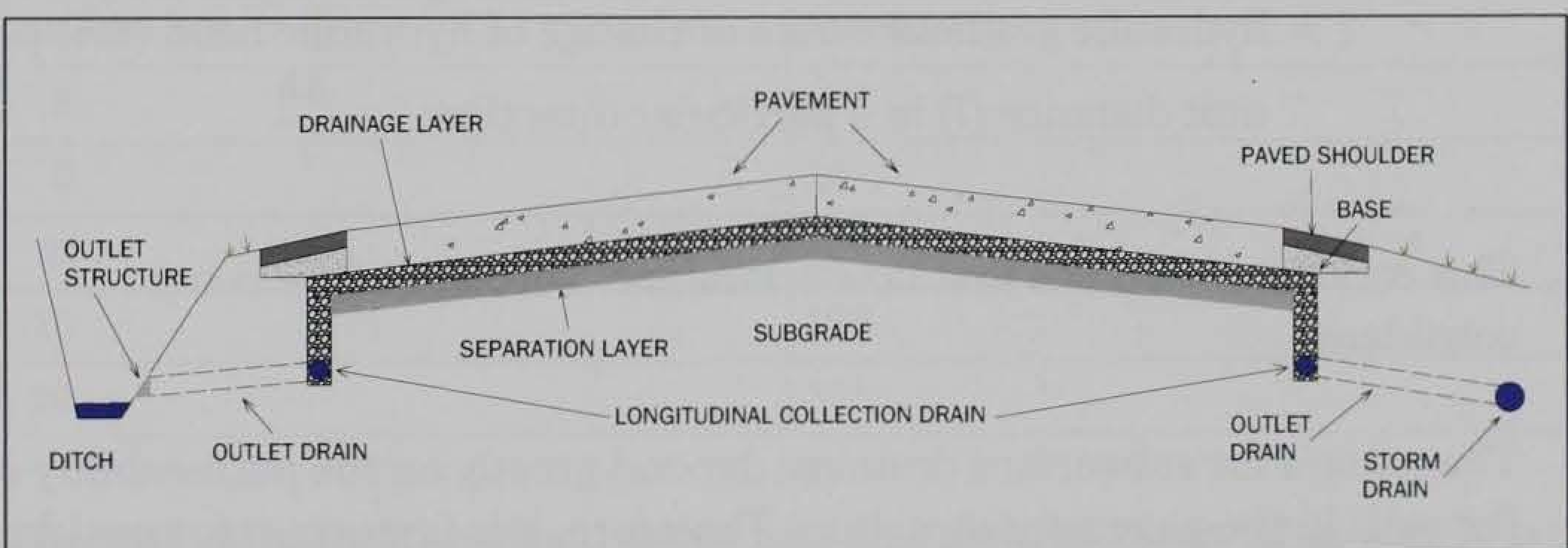


Figure 1. Collector drain used to remove infiltration water from the pavement system.

Placement of a drainage layer beneath the pavement surface is the most commonly used method to remove water from airfield pavements. The drainage layer's permeability converts the vertical inflow from precipitation into horizontal flow, which is moved away from the subgrade material and collected by a longitudinal collection system.

As the use of drainage layers has increased, so have the discussions concerning cost-efficiency and long-term performance of these drainage layers. To evaluate the efficiency of drainage layers, it is fundamental to know about drainage basics, design criteria, and construction procedures. A summary of the fundamentals of water flow and permeability, the design criteria developed by the U.S. Army Corps of Engineers (UFC 3-230-06A; HQs, Departments of the Army, Navy, and Air Force 2004) and the construction procedures most commonly used in practice are presented in the next sections.

Flow of water through soils and permeability

The performance of drainage layers was evaluated in this project by estimating the flow of water that was drained through the layer in a certain period of time. The water flow through soils is expressed by Darcy's empirical law which establishes that the velocity of flow is directly proportional to the hydraulic gradient and the permeability of the drainage media. This relation can be expanded to obtain the rate of flow (Q) through an area of soil (A) as follows:

$$Q = kiA \tag{1}$$

where:

k = coefficient of permeability, ft/min
i = hydraulic gradient = rate of change of hydraulic head (Δh) per unit distance (l) in a particular direction: $i = \frac{\Delta h}{l}$

This condition assumes that flow is laminar. Turbulent flow is not considered.

The criteria for subsurface drainage depend greatly on the permeability of the soils in the pavement structure. Therefore, it is important to consider the factors affecting permeability in the design. The following equation demonstrates the influence of the soil and pore fluid properties on permeability:

$$k = D_s^2 \frac{\gamma}{\mu} \frac{e^3}{(1-e)} C \tag{2}$$

where:

k = coefficient of permeability, ft/min
D_s = effective particle diameter, ft
γ = unit weight of pore fluid, lb/ft^3
μ = viscosity of pore fluid, lb · min/ft^2
e = void ratio
C = shape factor

This equation states that the permeability is directly proportional to the unit weight of water and inversely proportional to the viscosity. The unit weight of water is basically constant, but viscosity of water will vary with temperature.

Equation 2 also states that permeability varies with the square of the particle diameter. The smaller the grain size, the smaller the voids and, thus, the lower the permeability. The coefficient of permeability for sands and gravels used for pavement bases and subbases is estimated based on the percentage by weight of particles passing the No. 200 (0.0029 in.) sieve presented in Table 1.

Table 1. Coefficient of permeability for sand and gravel materials (Headquarters, Departments of the Army, the Navy, and the Air Force 2004).

Percent by Weight Passing No. 200 Sieve (0.0029 in.)	Permeability for Remolded Samples (ft/min)
3	10^{-1}
5	10^{-2}
10	10^{-3}
15	10^{-4}
20	10^{-5}

The void ratio and the effective porosity (n_e) are also essential factors to be considered when determining the permeability of a soil. The more dense the soil the lower the permeability, and hence the amount of water that can be retained in the soil is less. The water that is retained as thin films adhering to the soil particles and held by capillarity does not drain. Therefore, it is important to know the effective porosity of the soil in order to determine the volume of water that can be removed from that soil. Effective porosity can be expressed as:

$$n_e = 1 - \frac{\gamma_d}{G_s \gamma_w}(1 + G_s W_e) \quad (3)$$

where:

γ_d = dry density of soil, lb/ft^3
G_s = specific gravity of solids
γ_w = unit weight of water, lb/ft^3
W_e = effective water content (after soil has drained)

Vertical drainage from a pavement can be interrupted by an impermeable layer. Therefore, pavement structure and soil stratification are also important factors that affect permeability. In pavement construction the subgrade, subbase, and base materials are placed and compacted in layers with a vertical permeability that differs from the horizontal permeability. Typically, in pavements systems, subgrades have a very low permeability compared to the base and subbase materials. In layered pavement systems, the horizontal permeability is the weighted average of the layer permeability. Typically, the permeability of a drainage layer is much greater than the other materials in a pavements section. Consequently, the flow of water from the pavement can be computed based only on the characteristics of the drainage layer.

Drainage layers design criteria

This section summarizes the subsurface drainage design criteria established in UFC 3-230-06A (Headquarters, Departments of the Army, the Navy, and the Air Force 2004). The purpose of the use of drainage layers is to promote horizontal drainage of water away from pavements, prevent the buildup of hydrostatic water pressure, and facilitate the drainage of water generated by cycles of freeze-thaw. In rigid pavements, the drainage layer is generally placed directly beneath the concrete slab to permit rapid drainage of the water entering through cracks and joints. In flexible pavements, the drainage layer is normally placed beneath the base to reduce the stresses on the drainage layer to an acceptable level and to provide drainage for the base course.

The design of a subsurface drainage system (using drainage layers) consists of selecting a material with sufficient permeability to provide rapid drainage and yet provide sufficient stability to withstand load induced stresses. A material with a permeability of 1000 ft/day will provide sufficient drainage for most applications (Headquarters, Departments of the Army, the Navy, and the Air Force 2004). Other important design components (as shown in Figure 1) consist of the base material, a separating filter layer to prevent infiltration of subgrade fines into the base, and a collection and removal system (e.g., edge drains). However, the designer must have an understanding of the environmental conditions (rainfall and frost penetration) and subsurface soil properties (permeability, frost susceptibility and groundwater conditions) to ensure the success of the subsurface drainage system.

Criteria for requiring a subsurface drain system

All pavements not meeting the following criteria are required to have a subsurface drainage system:

- pavements in nonfrost areas and having a subgrade with permeability greater than 20 ft/day;
- flexible pavements in nonfrost areas and having a total thickness of structure above the subgrade of 8 in. or less.

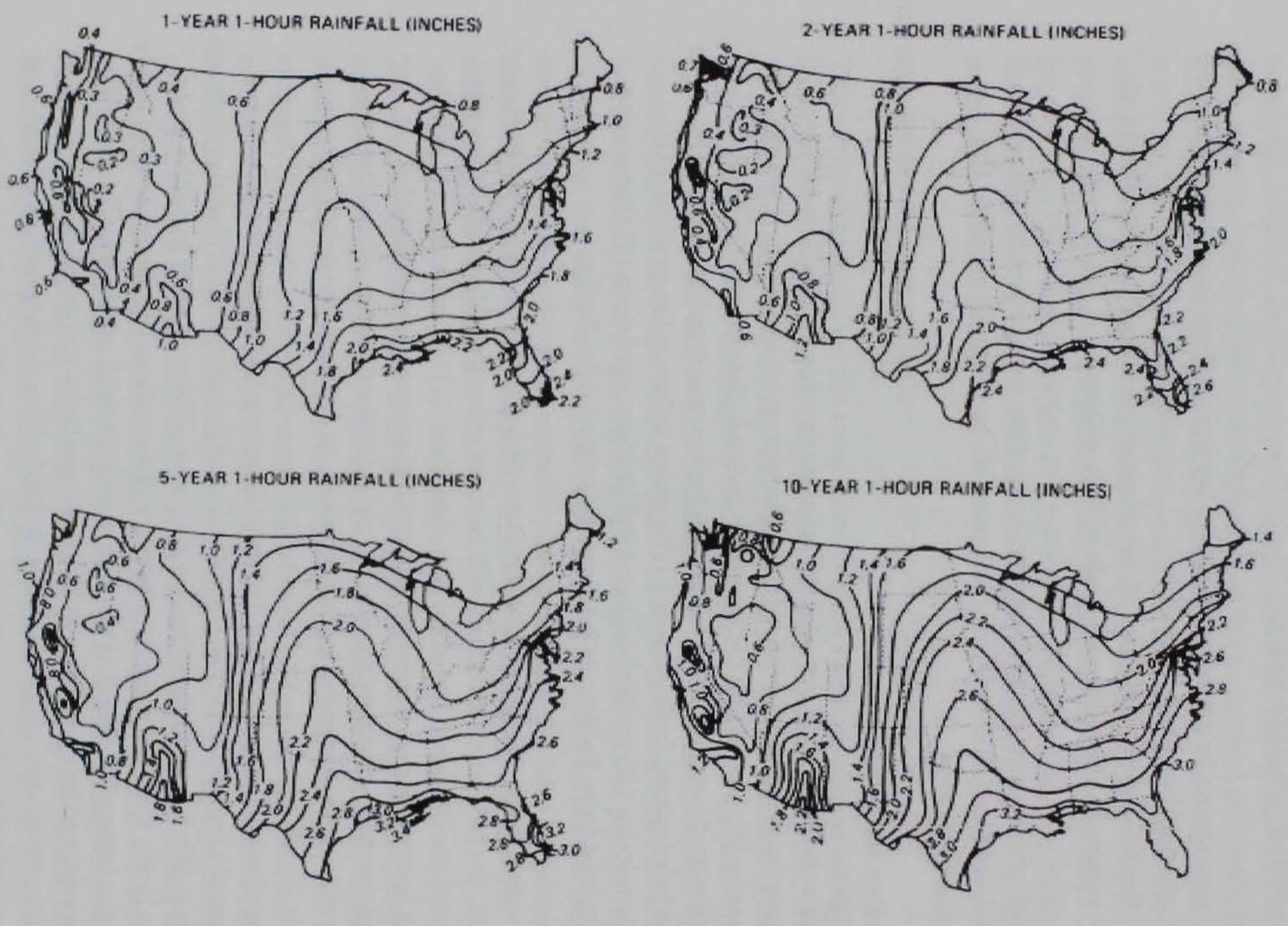
Even when a pavement meets the exemption requirements, a drainage analysis should be conducted for possible benefits of including the drainage system.

Design water inflow

The subsurface drainage system must be capable of handling infiltrated water from a design storm of 1-hr duration at an expected return frequency of 2 years. The design storm index is the standard rainfall intensity-frequency relation, lasting for various durations of supply, and it can be obtained from Figure 2. The water inflow is the product of the storm index (R) multiplied by an infiltration coefficient (F) that can be obtained from Table 2 or be assumed to be 0.5 for design.

If water enters the pavement structure at a greater rate than the discharge rate, the pavement structure will become saturated. Pavement drainage layers are designed based on two capacities: (1) the capacity of the drainage layer to serve as a reservoir for the excess water entering the pavement, or storage capacity (q_s), and (2) the capacity of the drainage layer to drain water during a rain event, or drainage capacity (q_d).

The storage capacity of the drainage layer is a function of the effective porosity (n_e) of the drainage material and the thickness (H) of the drainage layer. If it is considered that not all water will be drained from the drainage layer, then the storage capacity will be reduced by the amount of water in the layer at the start of the rain event. Current criterion calls for 85% of the water to be drained from the drainage layer within 24 hr; therefore, it is conservatively assumed that only 85% of the storage will be available at the beginning of a rain event.



(Chart reproduced from U.S. Weather Bureau, Technical Paper No. 40, Rainfall Frequency Atlas of United States, Washington, DC, May 1961)

Figure 2. Design storm index: 1-hr rainfall intensity-frequency data for continental United States (excluding Alaska).

Table 2. Infiltration rate for generalized soil classifications (uncompacted).

Description	Soil Group Symbol[1]	Infiltration, in./hr
Sand and gravel mixture	GW, GP SW, SP	0.8 - 1.0
Silty gravels and silty sands to inorganic silt, and well-developed loams	GM, SM ML, MH OL	0.3 - 0.6
Silty clay sand to sandy clay	SC, CL	0.2 - 0.3
Clays, inorganic and organic	CH, OH	0.1 - 0.2
Bare rock, not highly fractured	—	0.0 - 0.1

[1] Classified by the Unified Soil Classification System (MIL-STD-619).

The amount of water which will drain from the drainage layer during a rain event is a function of the duration of the rain event (t), the permeability of the drainage material (k), the slope of the drainage layer (i), and the thickness of the drainage layer (H).

The total capacity of the drainage layer is the sum of the storage capacity (q_s) and the drainage capacity (q_d):

$$q = \frac{0.85(n_e)(h) + (t)(k)(i)(H)}{2} \tag{4}$$

where:

H = thickness of the drainage layer
t = length of the design storm, hr
n_e = effective porosity
k = permeability of the drainage layer, ft/hr
i = slope of the drainage path

Length and slope of drainage path

The drainage path is the maximum distance water will travel in the drainage layer and is measured from the crest of the slope to the point where the water will exit the drainage layer. Equation 5 is used to calculate the length of the drainage path.

$$L = \frac{L_t\sqrt{i_t^2 + i_e^2}}{i_t} \tag{5}$$

where:

L_t = length of the transverse slope of the drainage, ft

$\sqrt{i_t^2 + i_e^2}$ = i = slope of the drainage path, %

i_t = transverse slope of the drainage path, %

i_e = longitudinal slope of the drainage path, %

Thickness of drainage layer

The thickness of the drainage layer is computed such that the capacity of the drainage layer will be equal to or greater than the infiltration from the design storm. It is computed with the equation:

$$H = \frac{2 \times F \times R \times L \times t}{(1.7 \times n_e \times L + k \times i \times t)} \quad (6)$$

where:

H = thickness of the drainage layer

F = infiltration coefficient

R = design storm index, ft/hr

t = length of the design storm, hr

n_e = effective porosity

L = length of the drainage path, ft

k = permeability of the drainage layer, ft/hr

i = slope of the drainage path

Time of drainage

As established previously, the drainage layer should reach 85% drainage within 24 hr from the end of the design storm. The time for 85% drainage (T_{85}) is computed by the equation:

$$T_{85} = n_e \times \frac{L}{(i \times k)} \quad (7)$$

where:

T_{85} = time for 85% drainage, days

n_e = effective porosity

L = length of the drainage path, ft
i = slope of the drainage path
k = permeability of the drainage layer, ft/day

This time is controlled by the type of material and the length and slope of the drainage path. Providing a more open drainage material would decrease the time for drainage, but it can also decrease the stability of the layer for construction. Therefore, the drainage material must be as dense as possible to avoid rutting problems. The slope of the drainage path depends on the geometry of the pavement surface, since it is usually placed parallel to the surface. Another way to reduce the time of drainage is to reduce the length of the drainage path by placing longitudinal and transverse collector drains. In summary, the design of the drainage layers involves matching the type of drainage material with drainage path and slope to meet the criteria for the time of drainage.

Placement of the subsurface drainage system

In rigid pavements, the drainage layer should be placed directly beneath the concrete slab as shown in Figure 3a. In flexible pavements the drainage layer should be placed either directly beneath the surface layer or beneath the base course. If the thickness of the base is equal to or greater than the thickness of the drainage layer plus the separation/filtration layer, the drainage layer is placed beneath the base, as shown in Figure 3b. When the total thickness of the pavement structure is less than 12 in., the drainage layer may be placed directly beneath the surface layer and be used as a base (Figure 3a). A separation/filtration layer is used to protect the drainage layer from contamination of fines from the underlying layers.

Material properties

For most drainage layers, the materials should have a minimum permeability of 1,000 ft/day (Headquarters, Departments of the Army, the Navy, and the Air Force 2004). Rapid draining material (RDM) and open graded material (OGM) are two materials that have been used in drainage layers. Their gradations and design properties are given in Tables 3 and 4, respectively. The RDM has sufficient permeability (1,000 to 5,000 ft/day) to serve as a drainage layer and will also have the stability to support construction traffic and the structural strength to serve as a base or subbase. The OGM has a very high permeability (>5,000 ft/day), but normally requires stabilization for construction traffic or structural strength to serve as a base in a flexible pavement.

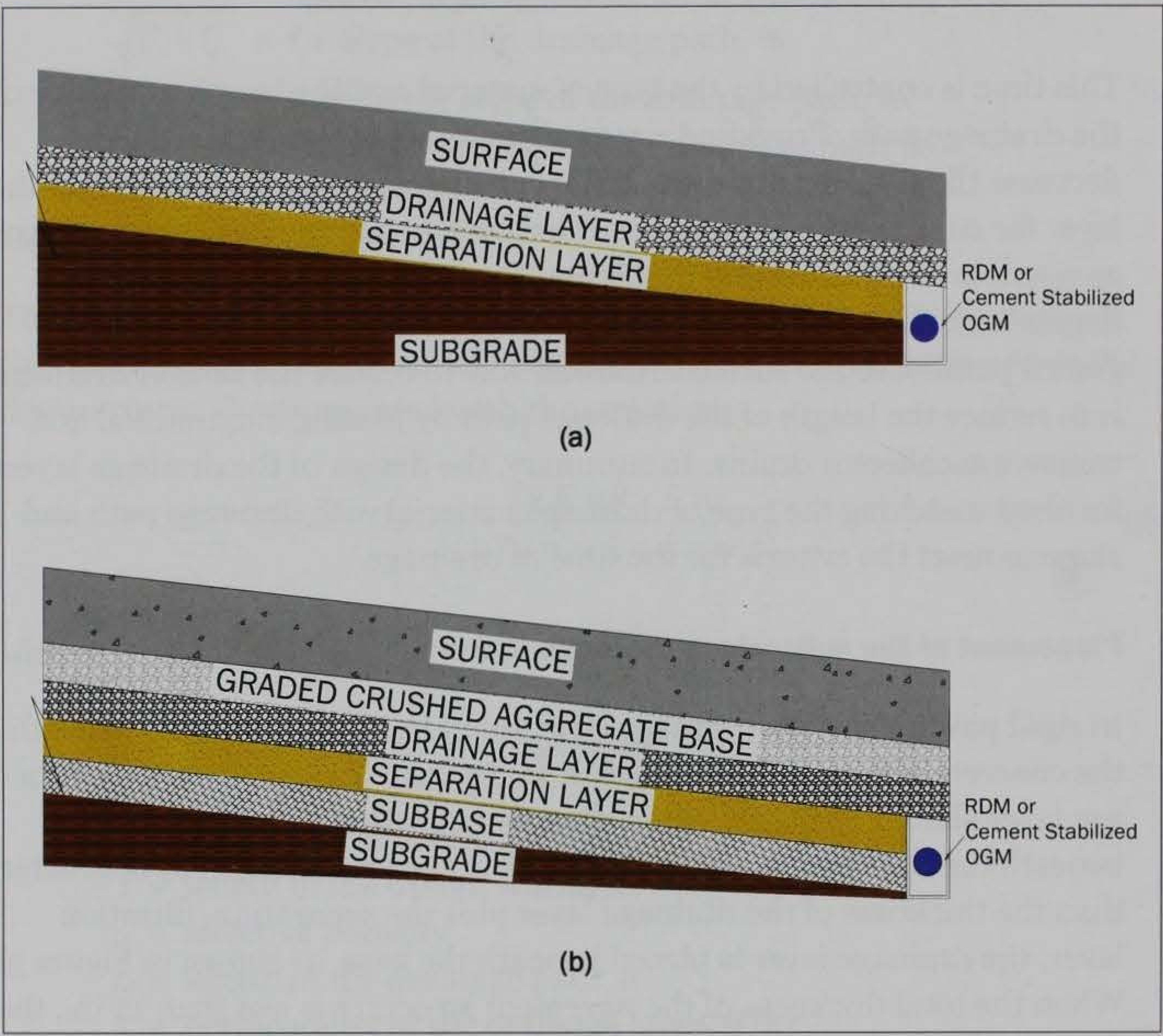


Figure 3. Drainage layer placement in pavement.

Table 3. Gradations of materials used for drainage layers.

Sieve Designation (in.)	RDM	OGM	Choke Stone
1-1/2	100	100	100
1	70-100	95-100	100
3/4	55-100	–	100
1/2	40-80	25-80	100
3/8	30-65	–	80-100
No. 4	10-50	0-10	10-100
No. 8	0-25	0-5	5-40
No. 16	0-5	–	0-10

Table 4. Design properties of materials used for drainage layers.

Property	RDM	OGM
Permeability (ft/day)	1,000-5,000	> 5,000
Effective Porosity	0.25	0.32
Percent Fractured Faces (COE method)	90% for 80 CBR 75% for 50 CBR	90% for 80 CBR 75% for 50 CBR
C_v	> 3.5	–
LA Abrasion	< 40	< 40

The material for the separation/filtration layer should be a graded aggregate meeting the requirements of a 50 CBR subbase, with a maximum percent passing the No. 10 sieve of 50% and a maximum percent passing the No. 200 sieve of 15%. The maximum aggregate size for this material should not be greater than one-fourth (1/4) the thickness of the separation/filtration layer. The permeability of the material to be used as the separation/filtration layer should be greater than the permeability of the subgrade material, but the material should not be so open to allow pumping of fines into the separation/filtration layer (Headquarters, Departments of the Army, the Navy, and the Air Force 2004).

Alternatively, a geotextile can be used to provide filtration and separation. The criteria for selecting a geotextile filter are provided in UFC 3-220-08FA. The important characteristics of the geotextile material are strength for surviving construction and traffic loads, and apparent opening size (AOS) to prevent pumping of fines into the drainage layer.

Stabilization of the drainage layer

Stabilization is often required for stability and strength, and for preventing degradation of the aggregate during construction. Stabilization is accomplished mechanically by the use of choke stone or by the use of a binder, such as asphalt or portland cement.

The choke stone should be a hard, durable, crushed aggregate having 90% fractured faces. The ratio D15 (diameter of the 15% finer) of the coarse aggregate to the D15 of the choke stone must be less than 5, and the ratio of the D50 (diameter of the 50% finer) of the coarse aggregate to the D50 of the choke stone must be greater than 2 (Headquarters, Departments of the Army, the Navy, and the Air Force 2004). The gradation range for acceptable choke stone is given in Table 3.

Stabilization of the drainage material using asphalt is accomplished by using only enough asphalt required to coat the aggregate without filling the voids. For stabilization of OGM, 2% to 2.5% asphalt by weight should be sufficient. Higher rates may be necessary for stabilization of RDM. Asphalt grade should be AC20 or higher (Headquarters, Departments of the Army, the Navy, and the Air Force 2004).

The amount of portland cement required for stabilization of the drainage material should be approximately 2 bags/yd^3, depending on the gradation of the aggregate. The water-cement ratio should be just sufficient to provide a paste which will adequately coat the aggregate (Headquarters, Departments of the Army, the Navy, and the Air Force 2004).

Construction procedures

Construction quality control as well as the training and experience of the construction personnel are important keys for the success of a pavement subsurface drainage system. Drainage layer construction is described in the UFC 3-230-06A (Headquarters, Departments of the Army, the Navy, and the Air Force 2004) as follows:

> The drainage material must be placed in such a way that segregation is prevented and a layer with uniform thickness is maintained. Placement of the RDM and OGM is best accomplished using an asphalt concrete paver. To ensure good compaction, it is important to place the drainage material in relatively thin lifts with a maximum thickness of 6 in. and to have a good foundation beneath the drainage material. If choke stone is used to stabilize the surface of OGM, the choke stone is placed after compaction of the final lift of OGM. The choke stone is spread in a thin layer using a spreader box or paver. The choke stone is worked into the surface of the OGM by the use of a vibrator roller and by wetting. The choke stone remaining on the surface should not migrate into the OGM by the action of water or traffic.
>
> Sufficient compaction can be obtained by six coverages or less of a vibrator roller loaded at approximately 10 tons. Material not being stabilized with asphalt or cement should be kept moist during compaction. Asphalt-stabilized material for drainage layers must be

compacted at a somewhat lower temperature than dense-graded asphalt material. In most cases, it will be necessary to allow an asphalt stabilized material to cool to less than 200°F before beginning compaction.

After compaction, the drainage layer should be protected from contamination by fines from construction traffic or from flow of surface water. The surface layer should be placed as soon as possible after placement of the drainage layer. Precautions must also be taken to protect the drainage layer from disturbance by construction equipment. Only tracked asphalt pavers should be allowed for paving over any RDM or OGM that has not been stabilized. Although curing of cement stabilized drainage layers is not critical, efforts should be made at curing until the surface layer is placed.

Proof rolling the graded crushed aggregate base (even when used over a drainage layer) and the separation layer is important. It is accomplished by using a rubber-tired roller load to provide a minimum tire force of 20,000 lb, and inflated to at least 90 psi. A minimum of six coverages should be applied, where a coverage is the application of one tire print over each point in the surface of the designed area. During proof rolling, action of the separation layer must be monitored for any sign of excessive movement or pumping which would indicate soft spots in the separation layer or the subgrade. All weak spots must be removed and replaced with stable material and the replaced material must be proof rolled as specified above.

Construction problems can significantly affect the performance of a drainage layer. Problems can occur during handling, placement, and compaction when the personnel are not adequately trained or do not have enough experience in the area. Problems can also occur if the drainage material does not have enough stability during the placement of the surface layer above it.

Problems are not only caused by poor construction quality. The design of a drainage layer is also a process that could involve many problems. The next section addresses the main issues around the design and construction of pavement drainage layers.

Design and construction issues

The design and construction criteria were evaluated to identify any potential changes based on the observed performance of the drainage layers. Some of the issues addressed include the following:

- When should drainage layers be required? Which of the following factors are important when determining if a drainage layer is needed
  - Soil type (permeability, swell potential, plasticity)
  - Annual precipitation
  - Freezing potential
- What is the proper permeability rate for the drainage layer? Do the current criteria for material properties provide reasonable results?
- Are the current material property specifications at the appropriate level to achieve acceptable performance
- What is the best mechanism for transport and eventual removal of subsurface water?
- Which pavement features need to have drainage layers?

Incorporating a drainage layer into a new pavement system is costly. It is important to ensure that the life of the pavement will be appropriately extended to justify the initial investment. Not all areas are expected to benefit from drainage layers. Some soils drain sufficiently on their own and do not have moisture-related problems. Other areas are located in climatic regions that are less likely to incur moisture-related damage.

Another issue is the permeability of the drainage layer. The current design procedure uses hydraulic equations to determine appropriate thickness for drainage layers. The input values are the total volume of water entering the system and the permeability of the layer through which it is transported. This approach was evaluated to determine if it is appropriate.

The material criteria for the components of the drainage system were also evaluated. This included all of the allowable types of materials used. Three types of drainage layers are allowed to provide flexibility during design. As these types of pavement systems were first being constructed, problems existed with the instability of rapid draining material. Without fine aggregate, the mixture could move around when construction equipment operated on its surface. This problem hindered placement of asphalt or portland cement concrete on the surface. For this reason, many engineers began designing pavement systems with stabilized drainage layers.

Asphalt stabilized drainage layers are the most commonly used type. This type of drainage layer typically uses an open-graded aggregate mixture with approximately 2% asphalt cement. The asphalt binds the aggregate but does not fill the void spaces in the mixture. A relatively stiff layer is formed that provides support for subsequent construction. The largest negative attribute of this type of drainage layer is the high cost.

The methods used to remove water from the drainage system were evaluated to determine if they were efficient and effective. This included the piping system within the drainage layer as well as the outlet pipes and their supporting structures.

3 Evaluation Procedures

The main objective of the field tests was to evaluate the efficiency and performance of in-service drainage layers by measuring the water flow and the moisture accumulation through the drainage layers. Field test procedures, test sites descriptions, and test results are presented in the following sections.

Field test procedures

Test locations were selected by identifying Air Force and Army airfields where drainage layers were in-place. Once the airfields were selected, construction drawings, airfield pavement condition survey reports, and nondestructive testing evaluation reports were used to identify the specific airfield pavement sections to evaluate. Construction drawings were used to determine the pavement profile and to locate the outlet structures and manholes.

The test equipment is shown in Figures 4 through 8. A core rig with a 6-in.-diam by 15-in.-long core bit was used to drill a hole through the pavement surface down to the top of the drainage layer. Once the hole was drilled, a 4-in.-diam by 5-ft-long PVC pipe was installed in the hole to increase the pressure head. The pipe was sealed to the hole with polyurethane foam, as shown in Figure 5. After the foam set (20 min), a 2-in. hose was placed in the PVC pipe. The hose was connected to a flowmeter that was connected to the water truck (Figures 6 and 7). The initial flowmeter reading was recorded. Water flow was started and allowed to reach the maximum that the drainage layer could accommodate without water spilling out of the top of the PVC pipe. Then the water flow was reduced to a steady-state rate maintaining a water column of about 4 ft inside the PVC pipe. Water volume and time were constantly recorded as water was allowed to flow into the pavement. Once water outflow was observed at the nearest outlet, a tracer dye was added to the inflowing water. The following times were recorded: (1) the time to when outflow was first observed, (2) the time to when tracer dye outflow was observed, (3) the time to when water inflow was stopped, and (4) the time to when water outflow (when observed) ceased.

Figure 4. Coring and flow test equipment.

Figure 5. PVC pipe used to increase pressure head.

Figure 6. Test setup.

Figure 7. Flowmeter.

Figure 8. Ground penetrating radar van.

The GPR shown in Figure 8 was used to establish moisture profiles underneath asphalt concrete pavements.

Once the test was finished, the PVC pipe was removed and material from the drainage layer was collected for characterization. Subgrade material was also collected at each airfield for characterization. Core holes were patched with appropriate patch materials, as shown in Figure 9.

Figure 9. Core hole patching.

A brief description of each test site, specific test procedures, and flow test results are presented in the next section.

Test sites

The field evaluation of in-service drainage layers began in August 2008 at the Elmendorf Air Force Base in Anchorage, AK. A second field evaluation was performed at Tinker Air Force Base in Oklahoma City, OK, in September 2008. Field testing continued in November 2008 at Biggs Army Airfield in Fort Bliss, Texas. Figure 10 shows the geographical locations of the test sites. The three locations represent specific regions of the country. Elmendorf AFB, AK, was chosen as a location where the subgrade is subjected to freezing and thawing cycles of the soil. Frost heave from freezing and weakening during thaw periods are concerns that warrant including drainage layers in this location. Tinker AFB, OK, is located in a moderate semiarid climate, where the natural subgrade materials are expansive soils. In the pavement structure, preventing water to reach this type of subgrade is expected to reduce swelling problems. Finally, Biggs AAF, TX, is located in a desert climate. This type of climate receives minimal rainfall during the year and may not benefit as much from the incorporation of a drainage layer. Table 5 lists the pavement features evaluated at each test site.

Figure 10. Test site locations.

Table 5. Test sites location and description.

Test Site	Airfield Location	Feature ID	Pavement Drainage Layer Description
Elmendorf AFB, AK	Fuel Cell Taxiway	T40C	Permeable asphalt drainage layer
	Hangar 18 Apron	A47B	Permeable asphalt drainage layer
	Weather Shelters Apron	A38B	Aggregate drainage layer
	Fuel Cell Taxiway	T40C	Base (Aggregate)
Tinker AFB, OK	Taxiway Bravo	T9A	Permeable asphalt drainage layer
	ALC Ramp	A13B	Subgrade (Clay)
Biggs AAF, TX	DAACG Ramp	A1B	Aggregate drainage layer

Elmendorf AFB, Anchorage, AK

An evaluation of pavement sections at Elmendorf Air Force Base in Anchorage, AK, was conducted 23–26 July 2008.

Construction history

The following excerpt from the 2007 airfield evaluation describes the recent construction history at Elmendorf AFB, Alaska.

> Alaska's military importance in the Pacific region was further recognized when the F-15E Strike Eagle-equipped 90th Tactical Fighter Sq. was reassigned to Elmendorf Air Force Base from Clark Air Base in the Philippines in May 1991. The increased mission drove the reconstruction of the asphalt portion of Runway 06/24 in 1994. The asphalt was removed and replaced with new asphalt concrete (AC) pavement. Major mill and overlay or complete reconstruction of Taxiways J and K were also accomplished at this time.
>
> In the last several years, there have been several reconstruction projects completed to maintain or improve the existing pavement infrastructure. In 1999, the C-130 Ramp on the north side of the airfield was completed. From 2001 to 2004, several phases of construction were done to replace the deteriorated pavement on the Gold Ramp. In 2005, major repairs were accomplished on Runway 06/24. These repairs included a complete reconstruction of the keel section with asphalt over a granular base and selective slab repairs on both portland cement concrete (PCC) ends of the runway. The last several years have also seen various other small projects, including several mill and overlay projects on the taxiways and a number of projects that

have expanded the original hardstands on the north side of the airfield. This expansion concept was carried out on the C-5 Ramp, the C-17 Ramp, the JMC Apron, and the Hot Cargo Pad. Currently, the base is busy preparing for yet another mission change, with the impending arrival of C-17 and F-22 aircraft and the addition of a squadron of ANG C-130s being reassigned from the local airport to Elmendorf AFB.

Climate

The weather at Elmendorf AFB has two main seasons—summer and winter. Summer lasts from June to early September, with average high temperatures of about 60°F. Winter begins in mid-October and lasts until early April. Average temperatures in the middle of winter are in the teens and twenties, with some daily highs below zero. About 16 in. of precipitation are recorded at the base each year, and about 70 in. of snow is recorded. The ground remains snow-covered throughout the winter and snow removal on the airfield is required to keep the pavements clear (AFCESA 2003).

Soil conditions

According to the 2007 airfield evaluation, the soils at Elmendorf AFB are typically well-drained very cobbly silts, very gravelly silty sands, very gravelly sands, or very gravelly sandy silts. Typically, the surface layer is about 60 in. thick. These soils generally classify as either GM or SM in the Unified Soil Classification System (USCS) and have a liquid limit ranging from 0 to 15% and a plasticity index ranging from non-plastic to 5 (AFCESA 2003).

Figure 11 shows the grain size distribution of the subgrade soil collected during this evaluation. The soil was classified as silty sand (SM) according to the USCS system. This agrees with the description of the soil type above. The subgrade soil was retrieved from an exposed area adjacent to the weather shelter area. It is assumed that this soil is representative of the native soil found in the area. The actual subgrade soil beneath the pavement could vary.

Drainage layers are used in most new construction projects at Elmendorf AFB. These layers are designed based on requirements in UFC 3-230-06a, Chapter 2.

Figure 11. Grain size distribution of subgrade soil at Elmendorf AFB.

Since Elmendorf AFB lies in a frost area, it is not exempt from the drainage requirement. The following excerpt from the 2007 AFCESA pavement evaluation at Elmendorf AFB describes the frost evaluation for the airfield.

> **Frost evaluation** — Pavement systems subjected to extended freezing conditions, such as in Alaska, should be analyzed for the effects of frost. Frozen soils may actually gain strength during the winter freeze. But in some cases, ice lenses form which expand the soil and move the pavement, possibly cracking it. During the spring thaw period, as surface temperatures begin to warm, the ice melts from the top down. Trapped by the frozen soil underneath, the melt water has nowhere to go and over-saturates the soil. This can drastically reduce the load-bearing capacity of the pavement structure until the water drains and the soil returns to its unsaturated state. The following discussion pertains specifically to the frost analysis at Elmendorf AFB.
>
> *Depth of freeze.* The ModBerg program developed by the US Army Corps of Engineers Cold Regions Research & Engineering Laboratory indicated the Design Air Freezing Index for Elmendorf AFB is 2,893 °F-days. Using a Modified Berggren analysis, the design depth of

frost penetration under pavements at Elmendorf AFB is approximately 100 in., depending on pavement thickness, soil type, and snow cover.

Frost susceptibility. Soil samples collected during this evaluation and from previous evaluations indicated that the base course materials were well-graded or poorly graded gravels, some with silt. These thick layers generally have a low susceptibility to freeze-thaw action, with an S1, PFS, or NFS classification. Construction drawings indicated most areas have at least 60 in. of select granular base/subbase material under the pavement.

Water. The soils under the pavements were damp, but not fully saturated. Along the northwest edge of the airfield, the runway, taxiways, and apron hardstands appear to have been cut into the hillside. However, it does not appear that seepage water from this hillside saturates the soils in this area. During this evaluation, several open pits from a POL pipeline construction project were observed. These pits were all approximately 8 to 10 ft deep. The observed soil in all pits was a poorly graded gravel with some silt. One pit was underlain by highly plastic clay. None of the open pits had any water standing in the bottom, even though it had rained for almost two full weeks before and during the evaluation.

Methodology. AGLs and PCNs are typically calculated for the thaw-weakened period using reduced modulus values for the soil layers as established by CRREL according to the soil classification. This approach creates a second thaw-weakened structural model based on the reduced moduli. AGLs are then calculated, as described in the previous section.

Application. The reported and observed materials that underlay the airfield are not frost susceptible. Additionally, no water was observed in open pits throughout the airfield despite heavy precipitation before and during the evaluation period. Therefore, thaw-weakened evaluations were not completed for any features on the airfield. However, all pavement areas should be routinely inspected for damage during the thawing season.

Drainage layer evaluation

The evaluation of pavement drainage layers at Elmendorf AFB was performed at the following locations:

- Fuel cell taxiway,
- C-130 hangar apron,
- Weather shelter apron.

Descriptions of each pavement area are given below.

Fuel cell taxiway. The fuel cell taxiway was constructed in 2004. This pavement provides access to a refueling area on the east side of the airfield. A typical cross section of the pavement is shown in Figure 12. The drainage layer in this location consisted of an asphalt stabilized, open graded material.

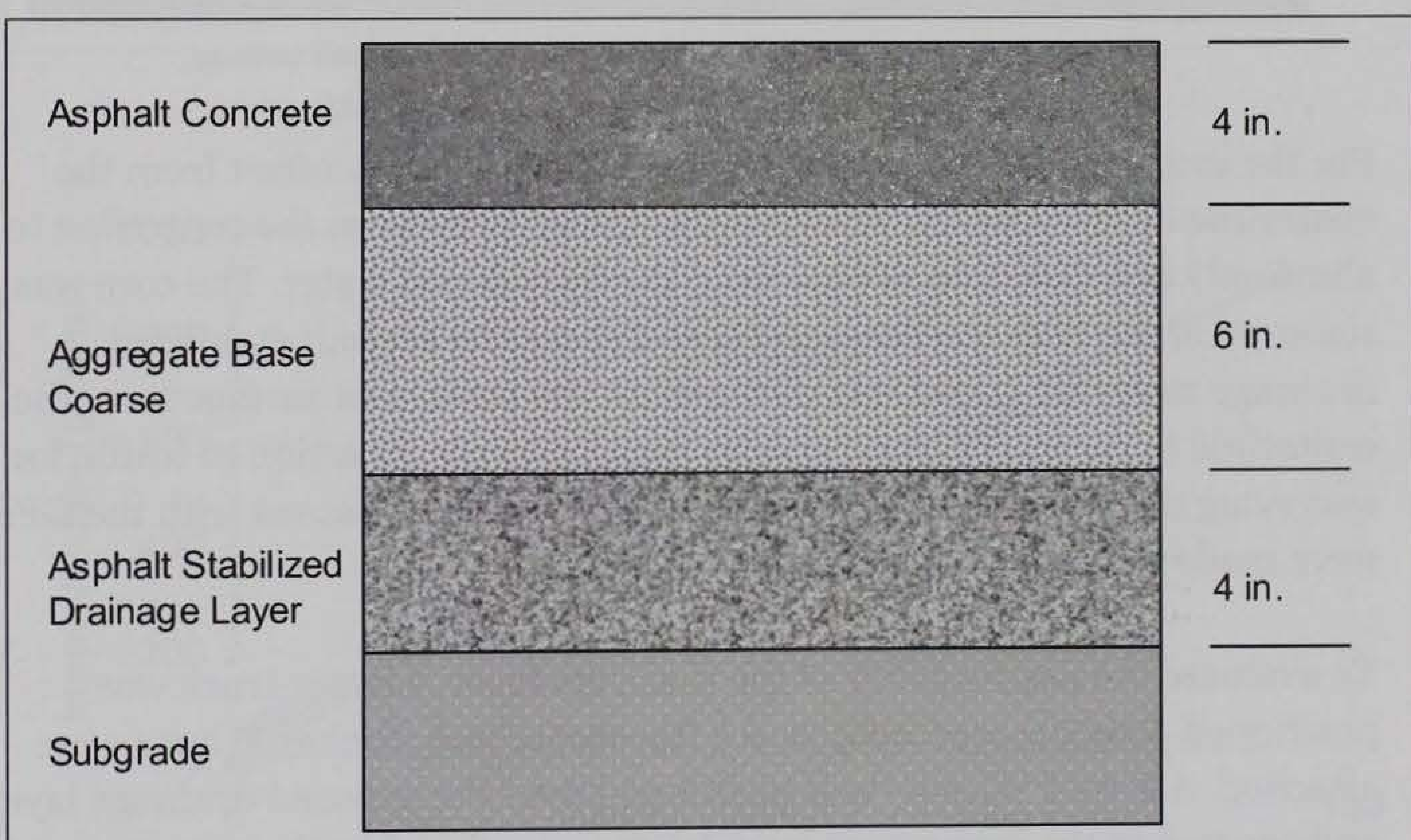


Figure 12. Fuel cell taxiway pavement cross section.

The drainage layer beneath this pavement is "daylighted" to the pavement shoulders (Figure 13). Water is expected to flow transversely to the traffic direction and flow out of the pavement shoulders. This type of drainage design is sometimes used in highway pavements but is not recommended as a drainage system in UFC 3-230-06a.

Figure 13. Photo of daylighted drainage layer on fuel cell taxiway.

For the evaluation, a 6-in. core hole was drilled slightly offset from the centerline of the taxiway. The core was taken away from the centerline to allow only one direction of flow from the introduced water. The core was removed along with the base course to expose the asphalt stabilized drainage material. A grid was outlined on the pavement surface from the centerline to the pavement shoulder parallel to the direction of traffic for surveying the section with the GPR (Figure 14). Initial scans with the GPR were made to provide baseline data.

To evaluate the permeability of the drainage layer, a water truck was positioned near the core hole, and a flowmeter and discharge hose were attached. A 4-in. PVC pipe was positioned over the exposed drainage layer and secured in the larger core hole. Water was placed in the pipe at a constant rate as it flowed into the drainage layer. The discharge rate was adjusted initially to provide a constant 2-ft head of water in the pipe.

Figure 15 shows a plot of gallons of water placed in the pipe over time. Each data point was determined by reading the output gage on the flowmeter and monitoring the time. The trendline in the graph represents the volumetric flow rate in gallons per minute.

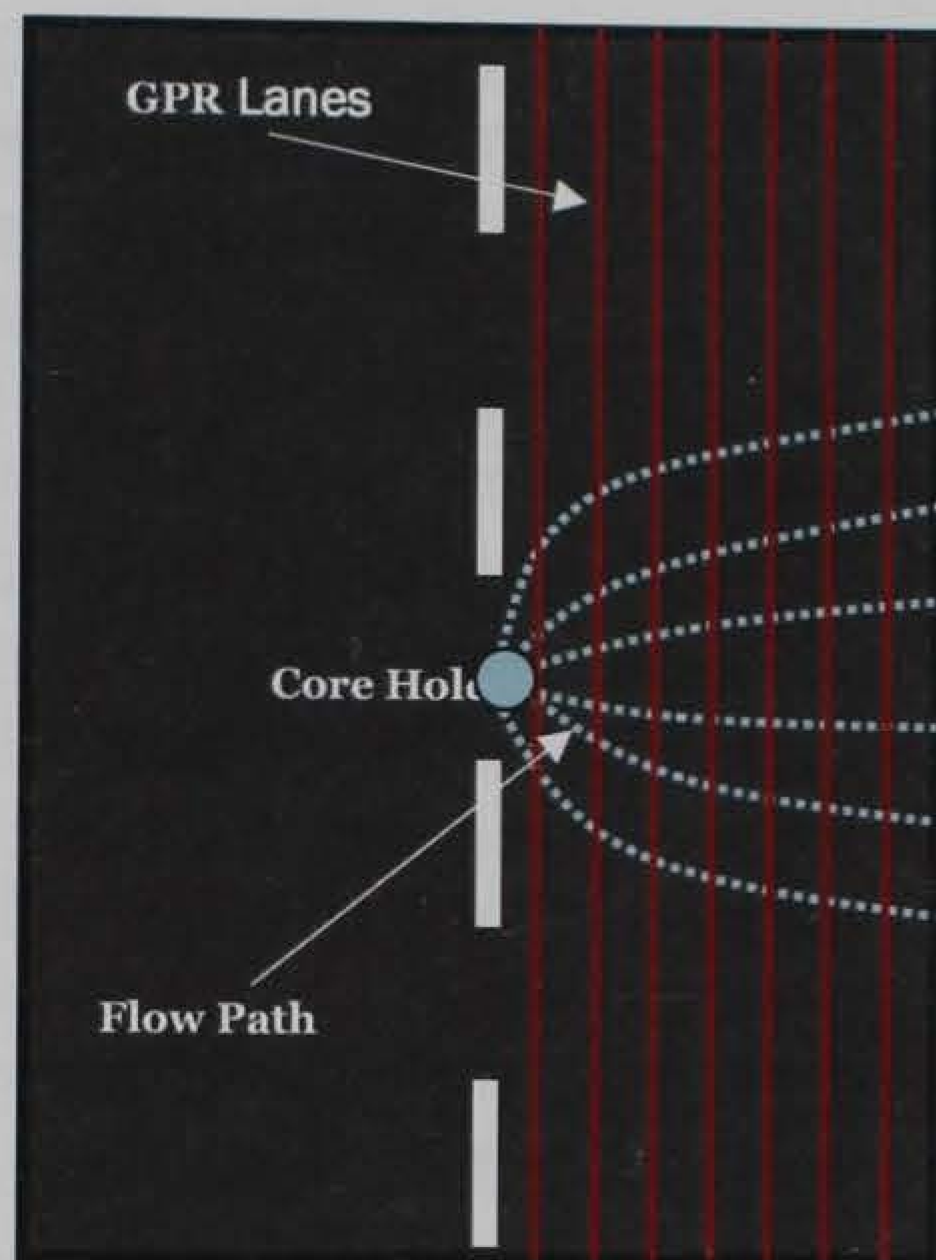


Figure 14. GPR lanes and flow path of subsurface water on fuel cell taxiway.

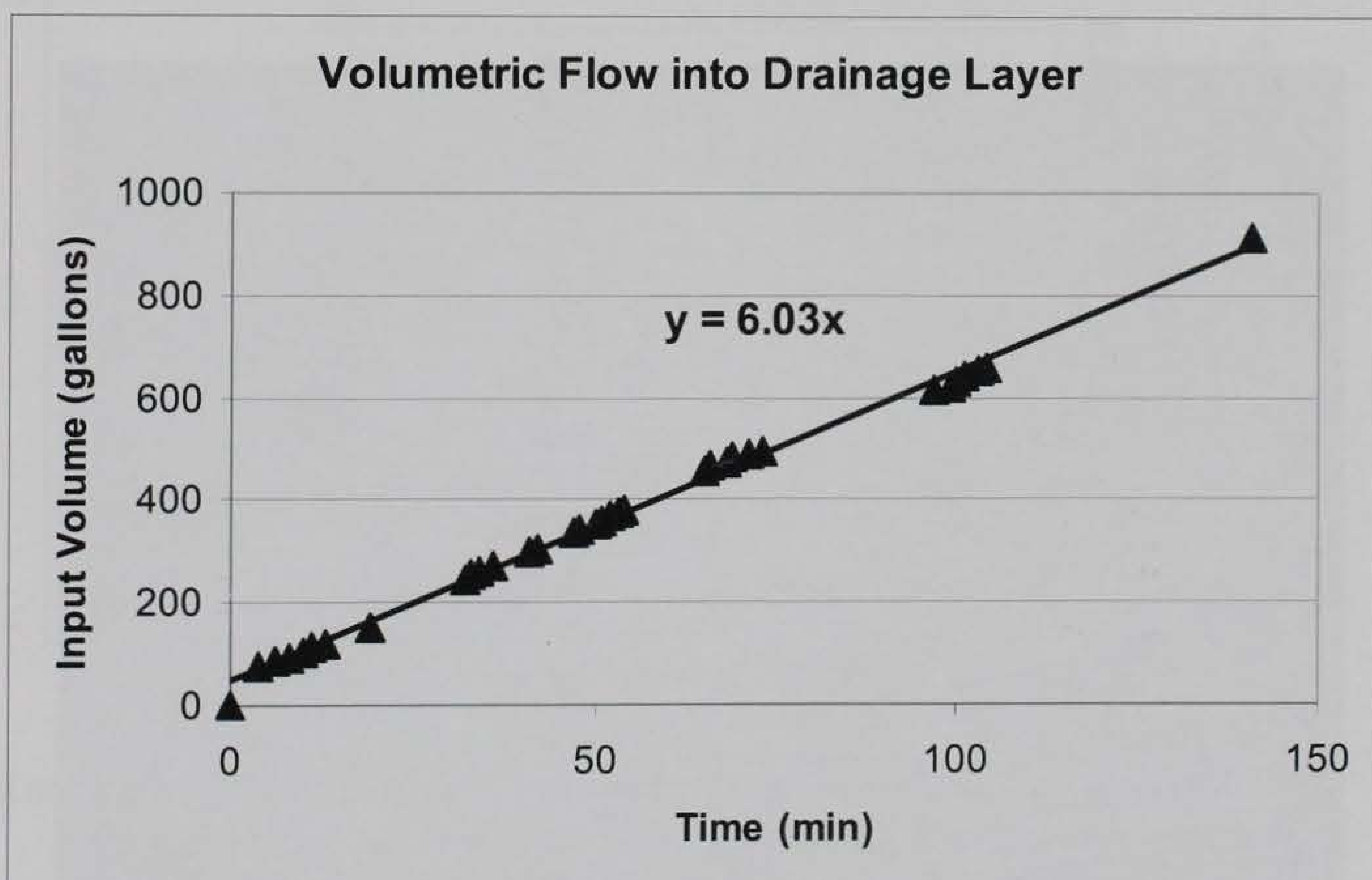


Figure 15. Flow rate of water introduced in fuel cell taxiway drainage layer.

Permeability testing was conducted on the dense graded base course between the AC layer and the stabilized drainage layer to provide data for comparison. A 6-in.-diam core was taken only in the asphalt layer. The 4-in. pipe was placed directly on top of the base coarse. Water was placed in the pipe at a constant rate as it flowed into the base layer. The discharge rate was adjusted initially to provide a constant 2-ft head of water in the pipe. The flowmeter was used to measure the volume of water introduced to the layer over time. The resulting flow rate into the base coarse was 0.5 gal/min.

The GPR was used to visualize the moisture profile in the pavement section during the introduction of water. Data were collected by scanning the parallel lanes at different time intervals. The lanes were spaced 5 ft apart. Data were analyzed to determine the point in the lane where moisture was detected by the GPR. Additionally, the point was recorded where moisture was no longer detected. Recording these points for each lane provided the ability to create a 2-dimensional map of the moisture in the drainage layer. The moisture profiles for the pavement section at different time intervals are shown in Figure 16.

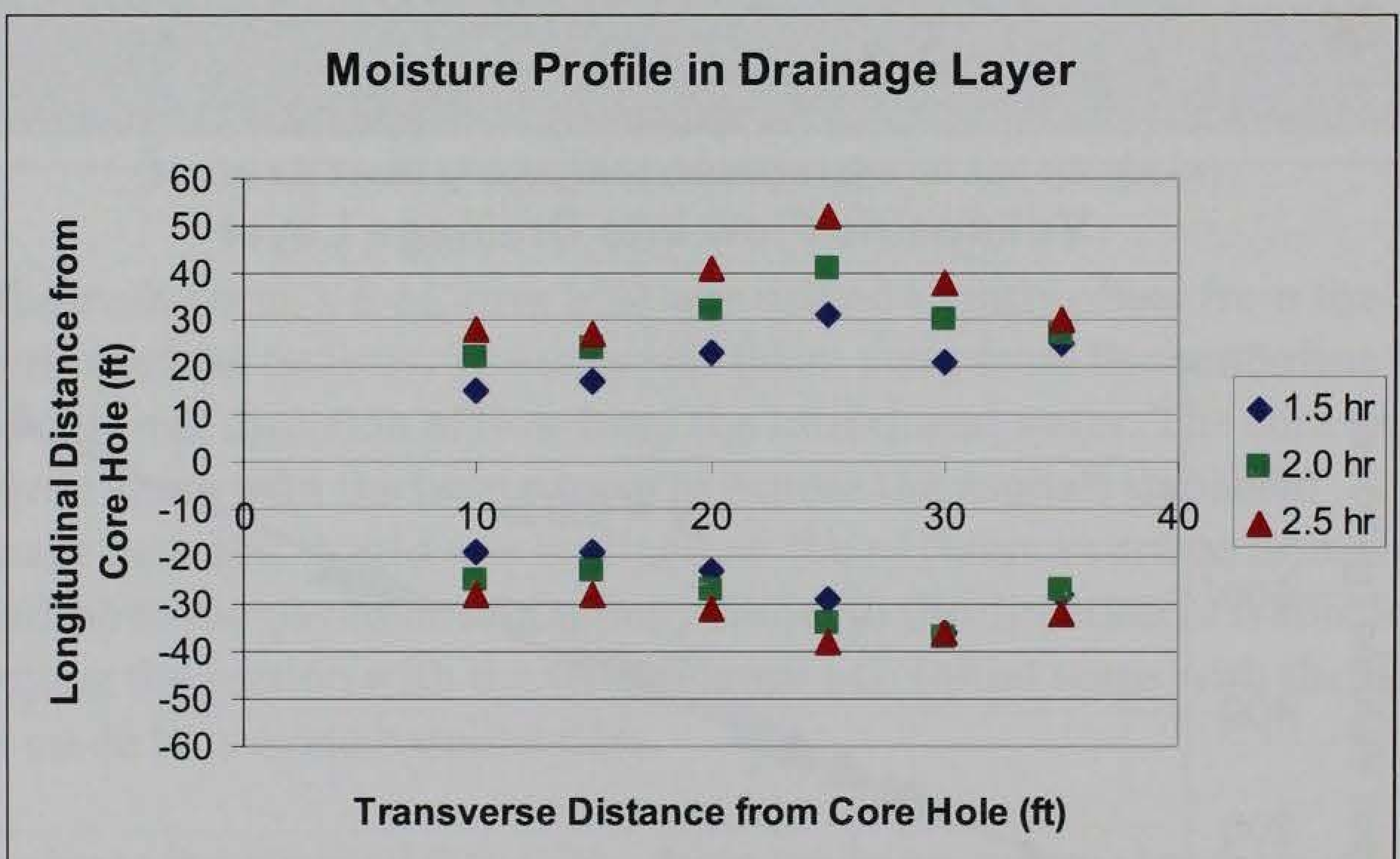


Figure 16. Moisture profile in drainage layer from GPR data.

Only the points along the lane where the appearance and disappearance of moisture are noted, and all of the space between these points contains moisture. These data were used to determine the width of the area through which the water was moving. As the water was introduced into the core hole, it spread uniformly in every direction. As equilibrium in flow was established, the plume of water was fixed as indicated below. Only small changes in the width were observed in subsequent scans. The first scans in the procedure were taken only on the south side of the taxiway. The north side was not used, because the water truck and equipment were stationed on that side. The assumption was made that half of the water would flow to each side of the taxiway, since the core hole was cut at the centerline. However, nearly all of the water was flowing to the north side. The location of the equipment was switched to the south side, but no moisture profiles were obtained on the north side during the initial stages of the work. Baseline data for all of the lanes show no moisture initially present.

The GPR did an excellent job at detecting the location of water moving through the drainage layer. The flow was observed to form a plume of approximately 50 ft nearest the core hole and approximately 90 ft at a distance of 25 ft from the core. A value of 50 ft was selected for use in the permeability calculations since the equations assumes a saturated condition.

During the process of introducing water into the pavement, moisture was noticed on the surface of the pavement (Figure 17). The water was not evenly dispersed. Only some areas of the pavement experienced this phenomenon.

Figure 17. Water bleeding up through pavement on fuel cell taxiway.

After approximately 2.5 hr and the introduction of 900 gal of water, moisture was noticed emerging from the daylighted edge of the drainage layer (Figure 18). However, GPR data indicate that the water had reached the pavement shoulder after 1.5 hr of flow.

Figure 18. Water emerging from drainage layer on fuel cell taxiway.

The permeability of the drainage layer was calculated using Darcy's Law as follows:

$$k = \frac{Q}{i \cdot A}$$

where:

k = estimated hydraulic conductivity (ft/day)
Q = maximum inflow rate (cu ft/day)
i = hydraulic gradient
h = elevation head (ft)
L = flow length (core to pavement edge) (ft)
A = cross sectional area of flow (sq ft)
t = thickness of drainage layer (ft)
w = assumed width of flow plume (ft)

In this scenario, the variables were defined as the following:

Q = 6 gal/min

$$h = 2 \text{ ft}$$
$$L = 60 \text{ ft}$$
$$t = 4 \text{ in.}$$
$$w = 50 \text{ ft}$$
$$i = \frac{h}{L}$$
$$A = t \bullet w$$
$$i = 0.033$$
$$A = 16.667 \text{ ft}^2$$
$$k = 2.079 \times 10^3 \text{ ft/day}$$

Based on these input values, the permeability of the drainage layer was approximately 2,000 ft/day. In addition, the calculated permeability values were compared to theoretical permeability values. The theoretical values were calculated according to the following equation:

$$k = \frac{621400(D_{10})^{1.478} \cdot n^{6.654}}{P_{200}^{0.597}}$$

$$n = 1 - \left(\frac{\gamma_d}{\gamma_w \cdot G} \right)$$

P_{200} = percent passing No. 200 sieve
D_{10} = grain size at 10% passing (mm)
γ_d = dry density of aggregate (lb/cu ft)
γ_w = density of water (lb/cu ft)
G = specific gravity of aggregate

$$P_{200} = 2\%$$
$$D_{10} = 3 \text{ mm}$$
$$\gamma_d = 130 \text{ lb/ft}^3$$
$$\gamma_w = 62.4 \text{ lb/ft}^3$$
$$G = 2.7$$
$$k = 1.76 \times 10^3$$

Using these input values, the theoretical permeability of the drainage layer is approximately 1,800 ft/day, similar to the observed value. The material properties were assumed in the theoretical calculation. The values were selected based on the specifications for construction of drainage layers and typical pavement material properties. The dry density value used in the calculation assumed an air void content of 14% in the asphalt stabilized mixture. Higher air voids would result in much higher permeability.

C-130 hangar apron. The C-130 hangar apron was constructed in 2000. This pavement provides access to a hangar on the northwest side of the airfield. A typical cross section of the pavement is shown in Figure 19. The drainage layer in this location consisted of an asphalt stabilized, open graded material.

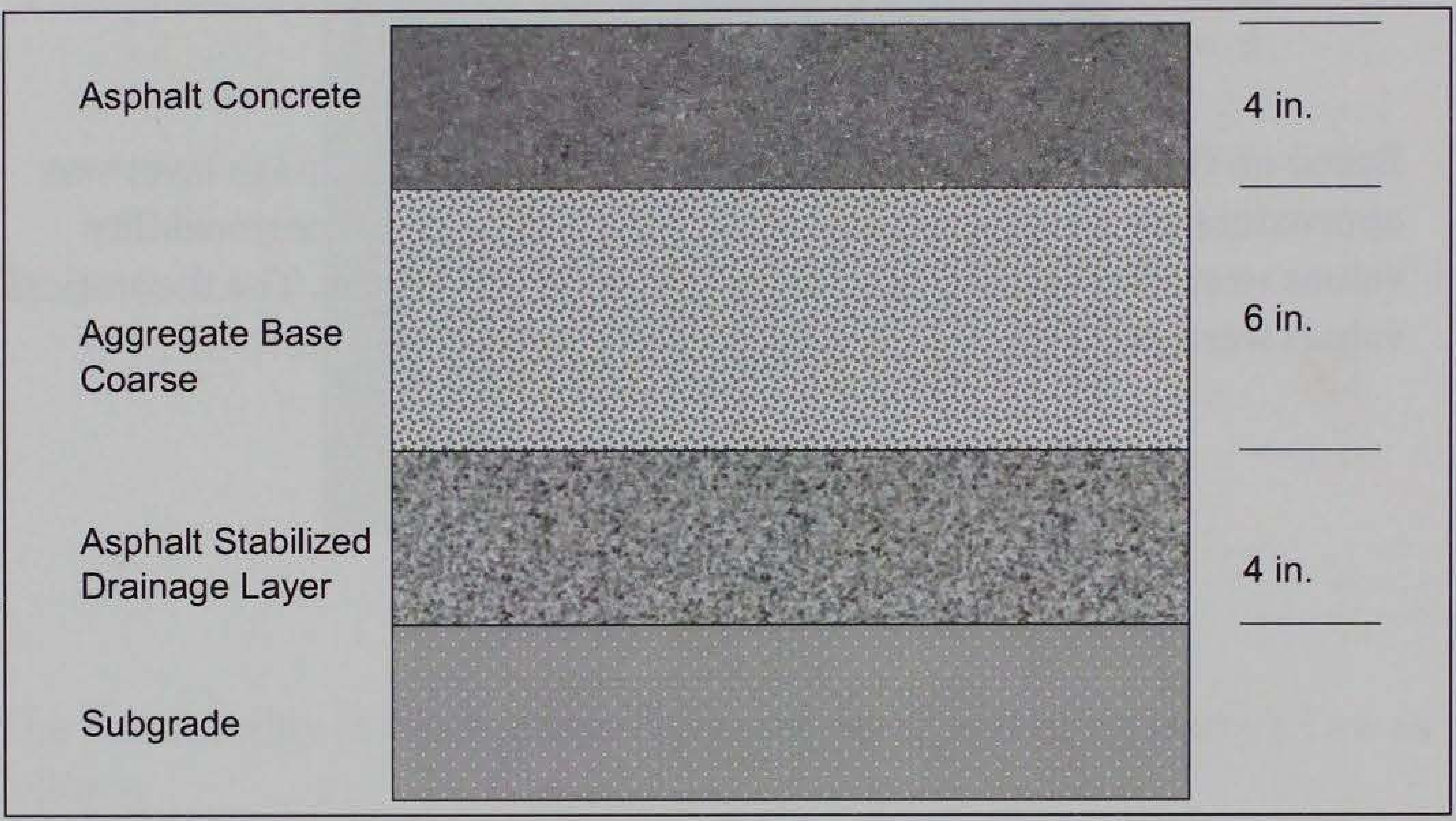


Figure 19. Cross section of pavement at Hangar 18 apron.

The drainage layer beneath this pavement was designed to flow towards a collection pipe and then discharge into a drainage basin adjacent to the apron area. Water was expected to flow underneath the apron pavement until it reaches the drain pipe. A visual inspection of the drainage basin showed no evidence of a discharge pipe. According to UFC 3-230-06a, a headwall should be installed at the discharge pipe to protect it from damage and allow access for maintenance. No such structure was located.

For the evaluation, a 6-in.-diam core hole was drilled 25 ft from the expected location of the drainage pipe. The core was taken in this location to promote flow towards the drainage pipe and at a distance that was considered reasonable for measuring flow in the time allotted. The core was removed along with the base coarse to expose the asphalt stabilized drainage material. Permeability of the layer was determined according to the previously described procedure.

Figure 20 shows the plot of gallons of water placed in the pipe over time. Each data point was determined by reading the output gage on the flowmeter and monitoring the time. The trendline in the graph represents the volumetric flow rate in gallons per minute.

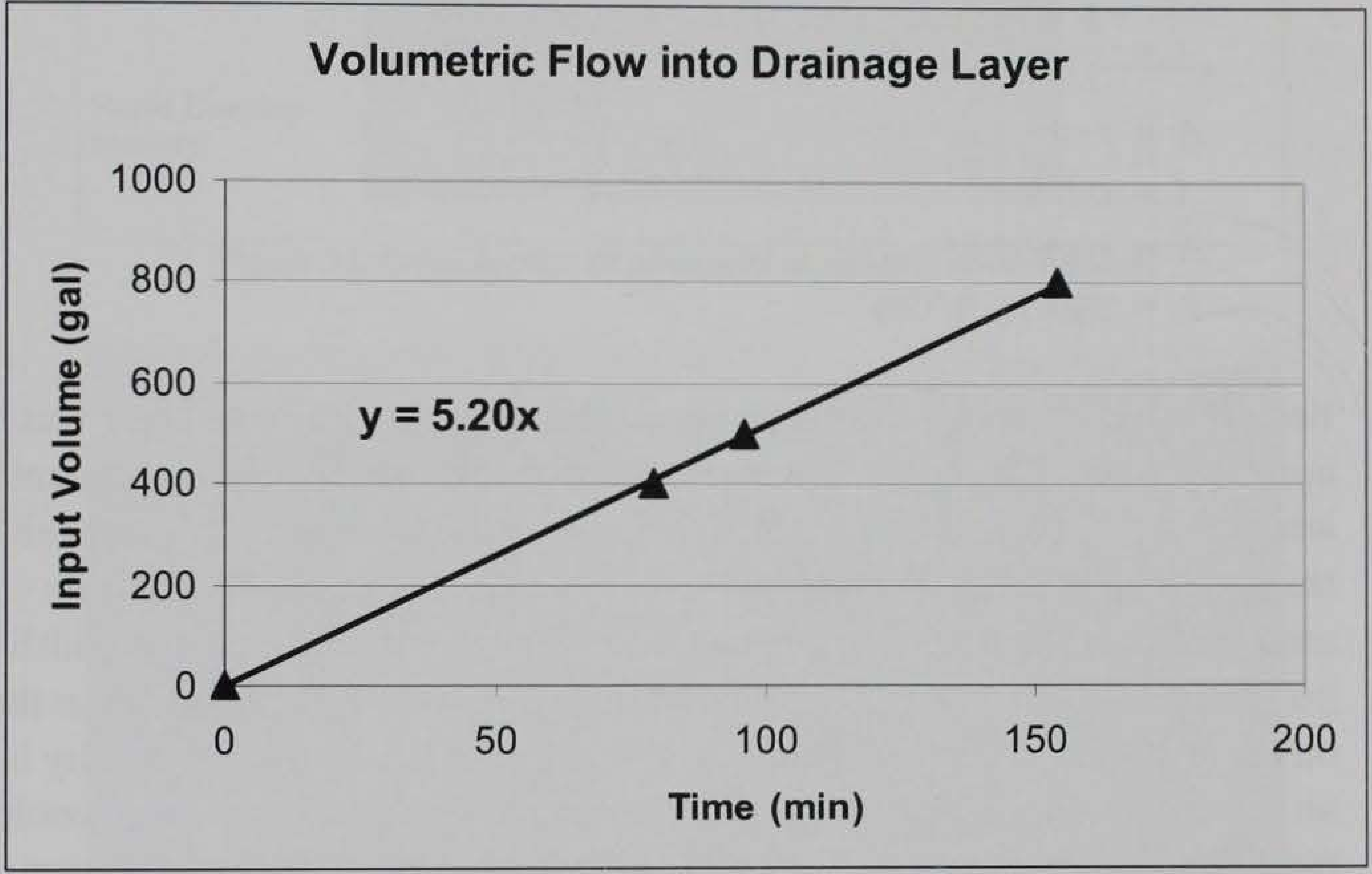


Figure 20. Flow rate of water introduced in drainage layer at Hangar 18 apron.

The GPR was used to attempt to identify the moisture profile of the pavement layers. The GPR data did not show moisture in the pavement distinctive of a flow pattern. After monitoring the area several times with no evidence of flow, the GPR data collection attempts were discontinued.

The area adjacent to the pavement section where the outflow pipe was expected to be located was observed for evidence of discharge. The inflow water was treated with a dye to impart an intense color for observation. After 2.5 hr and the introduction of 800 gal of water, no visual evidence of outflow existed.

The permeability of the drainage layer was calculated according to the following equation:

$$k = \frac{Q}{i \cdot A}$$

In this scenario, the variables were defined as the following:

$$Q = 5.2 \text{ gal/min}$$
$$h = 2 \text{ ft}$$
$$L = 25 \text{ ft}$$
$$t = 4 \text{ in.}$$
$$w = 50 \text{ ft}$$
$$i = \frac{h}{L}$$
$$A = t \cdot w$$
$$i = 0.08$$
$$A = 16.667 \text{ ft}^2$$
$$k = 750.75 \text{ ft/day}$$

Based on these input values, the permeability of the drainage layer was approximately 750 ft/day. The permeability of this layer was significantly less than the permeability of the drainage layer underneath the fuel cell taxiway. The inability to track the flow of water prevented an accurate assessment of the drainage layer. In the equation, the flow plume width of 50 ft was used. However, the slope of this pavement was greater than that on the taxiway, so the width may have been smaller. The permeability is an inverse linear relationship to the plume width. A smaller width would result in a higher permeability. Additionally, the permeability equation assumes an equal input and output volume. Since no output was observed, the applicability of the equation is limited. Water may have been trapped beneath the pavement and traveled laterally saturating the drainage layer.

Weather shelter apron. The weather shelter apron was constructed in 1992. This pavement provides parking and access to a hangar on the north side of the airfield. A typical cross section of the pavement is shown in Figure 21. The drainage layer in this location consisted of an unstabilized rapid draining material.

The drainage layer beneath this pavement was designed to flow toward a collection pipe and then discharge into a drainage basin adjacent to the apron area. Water was expected to flow underneath the apron pavement until it reached the drain pipe. A large drain pipe set in a head wall was identified and observed during the evaluation (Figure 22).

For the evaluation, a 6-in.-diam core hole (Figure 23) was drilled 195 ft from the end of the drainage pipe. The core was taken in this location to promote flow towards the drainage pipe and at a distance that was

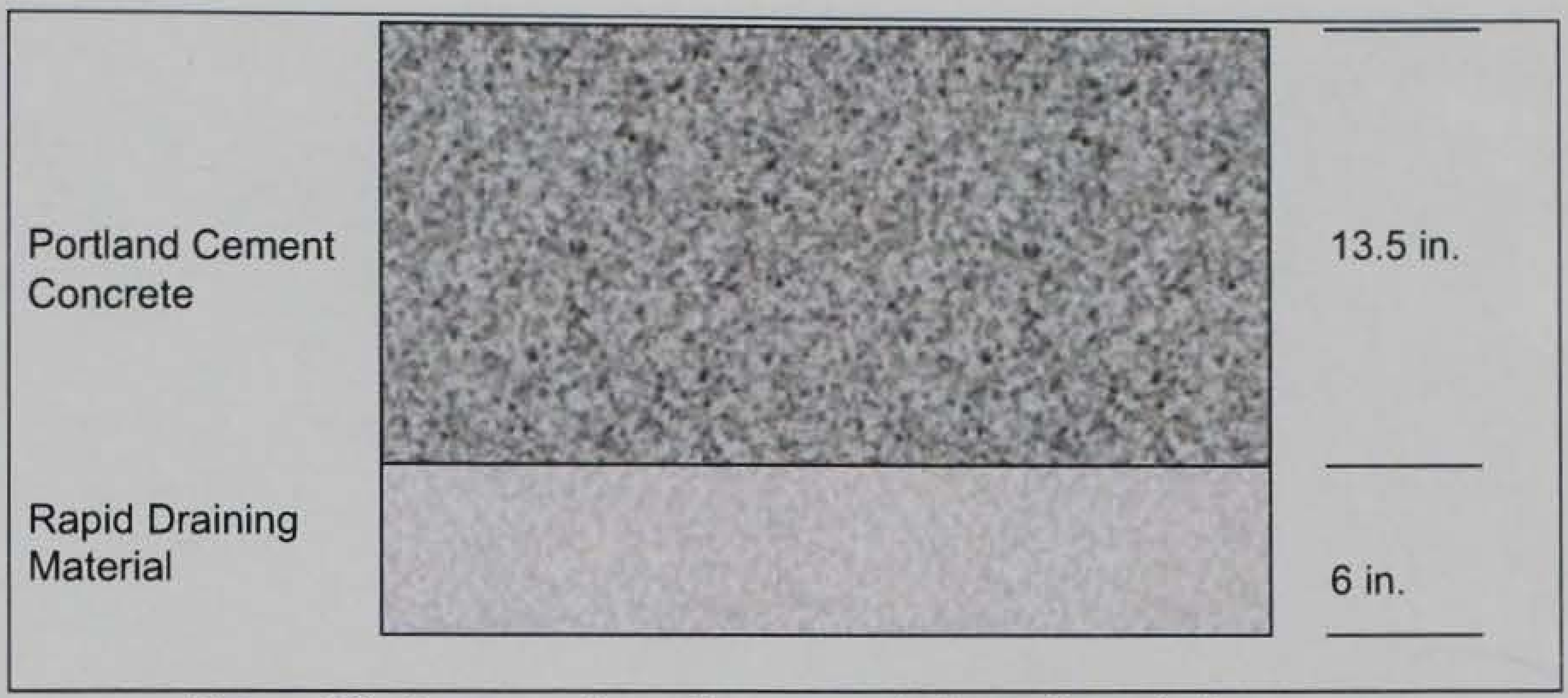


Figure 21. Cross section of pavement at weather shelter apron.

Figure 22. Outlet pipe leading from weather shelter apron.

considered reasonable for measuring flow in the time allotted. The core was removed to expose the rapid draining material.

The permeability of the drainage layer was determined using the previously described procedure. In this case, flow into the drainage layer was limited by the maximum discharge rate of the gravimetric flow from the water truck. Less than a 1-ft head of water was maintained in the pipe.

Figure 24 shows the plot of gallons of water placed in the pipe over time. Each data point was determined by reading the output gage on the flowmeter and monitoring the time. The trendline in the graph represents the volumetric flow rate in gallons per minute.

Figure 23. Rapid draining material beneath PCC at weather shelter apron.

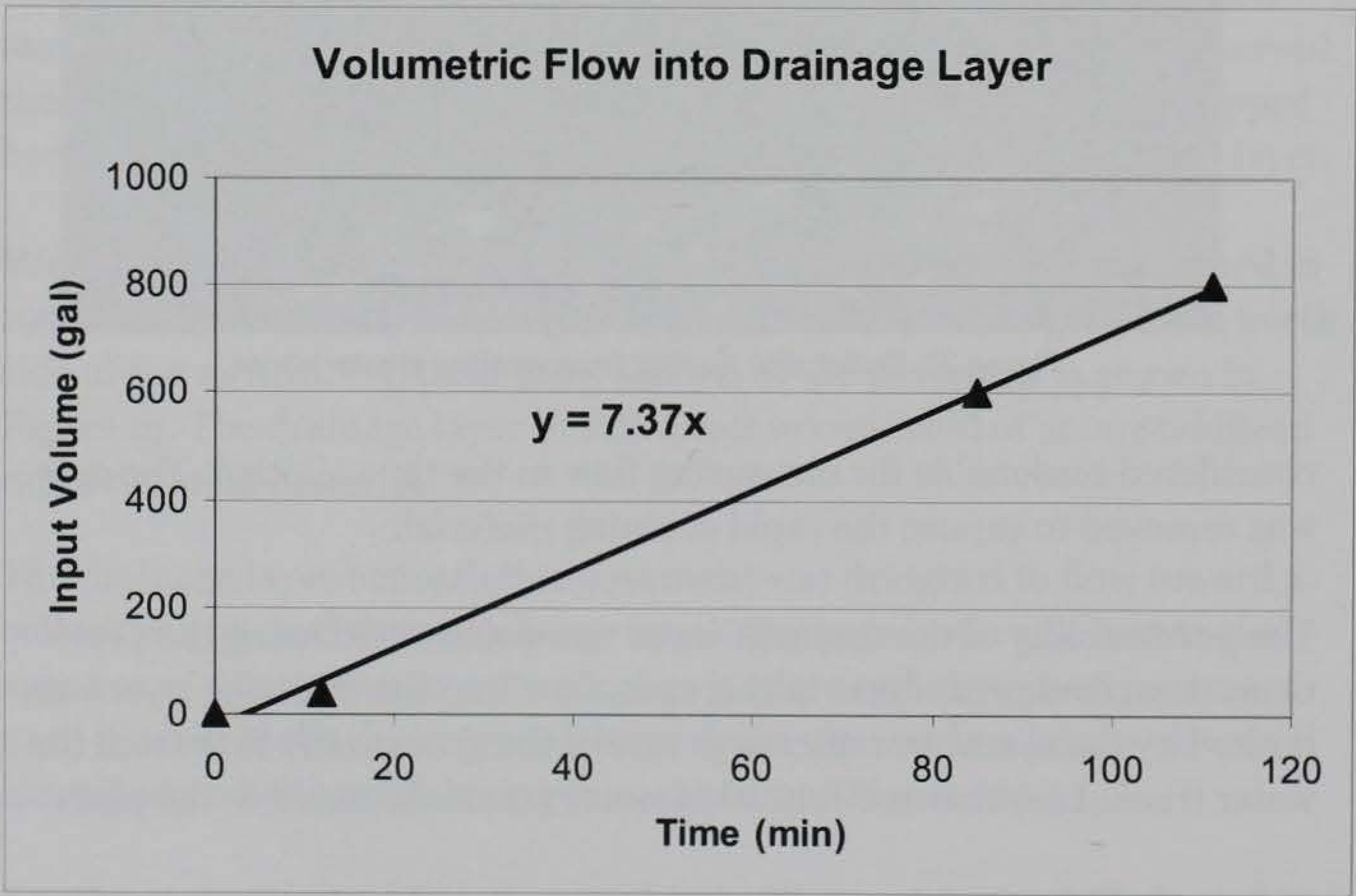


Figure 24. Flow rate of water introduced in drainage layer at weather shelter apron.

At this location the GPR was not used to observe the moisture profile in the drainage layer. The GPR was unable to penetrate sufficiently through thick PCC pavement to provide discernable differences in moisture.

The area adjacent to the pavement section where the outflow pipe was located was observed for evidence of discharge. The inflow water was treated with a dye to impart an intense color for observation. After nearly 2 hr and the introduction of 800 gal of water, no visual evidence of outflow existed.

The parking apron was extended after construction. A new pavement section was added to the apron where the outflow pipe was located. The pipe may have been damaged during the construction of this addition.

The permeability of the drainage layer was calculated according to the following equation:

$$k = \frac{Q}{i \cdot A}$$

In this scenario, the variables were defined as the following:

Q = 7.37 gal/min
h = 2 ft
L = 100 ft
t = 6 in.
w = 50 ft
$i = \frac{h}{L}$
$A = t \cdot w$
i = 0.02
A = 25 ft^2
k = 2.837 × 10^3 ft/day

Based on these input values, the permeability of the drainage layer was nearly 3,000 ft/day. The permeability of this layer was the highest of all pavement areas evaluated. The drainage material was unstabilized and likely had a higher porosity. Higher flow rates would have been achieved if the outflow from the water truck had not been limited. The inability to track the flow of water limited the assessment of the drainage layer. Not all variables in the permeability equation were known. However, rapid flow of water through this type of material was easily achieved. Whether it was effectively removed from the pavement system is unknown.

Performance data evaluation

Data were extracted from Air Force pavement evaluation reports to identify distresses on pavements constructed with and without a drainage layer. Table 6 lists the PCI and distress types for the pavements included in this study and others similar in function. The items listed first were designed with a drainage layer while the items in the lower half of the table were designed without a drainage layer. The PCI data list only the condition rating for the 2006 and 1999 surveys, while the numerical value is given for the 2003 and 1998 surveys. Not all areas were surveyed in at each of the four evaluation periods. Some of the pavements were not constructed until recent years. The age of the pavement was considered when comparing distress levels.

The performance of pavement sections constructed with and without a drainage layer show little differences. The current condition of all pavements selected for this table is good according to the 2006 survey. The pavements constructed with a drainage layer have not been in place long enough to detect differences in performance. The rate of deterioration is slow at this point in the life of the pavement. Comparing old pavements constructed without a drainage layer would not be valid. Figure 25 shows the change in pavement condition over the life of the pavement. The pavements considered in this work are near the beginning of their life and do not reflect large changes in PCI.

Tinker AFB, Oklahoma City, OK

Tinker Air Force Base is located in Oklahoma County, approximately 10 miles southeast of the business district of Oklahoma City, and near the suburb of Midwest City, OK. The base consists of 5,020 acres, with 732 buildings containing 15.5 million ft^2 of floor space, 136 acres of indoor maintenance area, and 254 acres of ramp space. The natural drainage is generally from east to west. The surface runoff water is collected in a series of inlets and carried westerly by storm sewers to a series of natural outfalls that flow into Crutcho and Kuhlman Creeks (AFCESA 2005).

Table 6. Pavement condition survey results at Elmendorf AFB.

Pavement Feature	Year Constructed	Pavement Type (Thickness, in.)	Evaluation Date	PCI	Distress Types
Pavement With Drainage Layers					
Fuel Cell Taxiway	2004	AC (4)	2006	good	no distresses
Hangar 18 Access	2000	AC (4)	2006	good	low severity L/T cracking
			2003	100 PCI	
Weather Shelter	1992	PCC (13.5)	2006	good	low and medium L/T cracking, low shattered slab, low joint spalling, low and medium corner spalling
			2003	97 PCI	joint seal damage
			1999	excellent	low severity L/T cracking
			1998	100 PCI	
Pavements Without Drainage Layers					
JMC Ramp	1996	PCC (14)	2006	good	low severity L/T cracking, low small patch, low joint spalling
			2003	99 PCI	low patching and spalling
			1999	excellent	low L/T cracking, low and medium joint spalling
			1998	90 PCI	low joint spalling, low joint seal damage
C130 Ramp North	1999	PCC (13)	2006	good	low severity corner break, low L/T cracking
			2003	97 PCI	
C130 Ramp South	1999	PCC (13)	2006	good	low and medium severity L/T cracking
			2003	100 PCI	low severity L/T cracking
C130 Ramp Access	1999	AC (4)	2006	good	low severity L/T cracking
			2003	99 PCI	

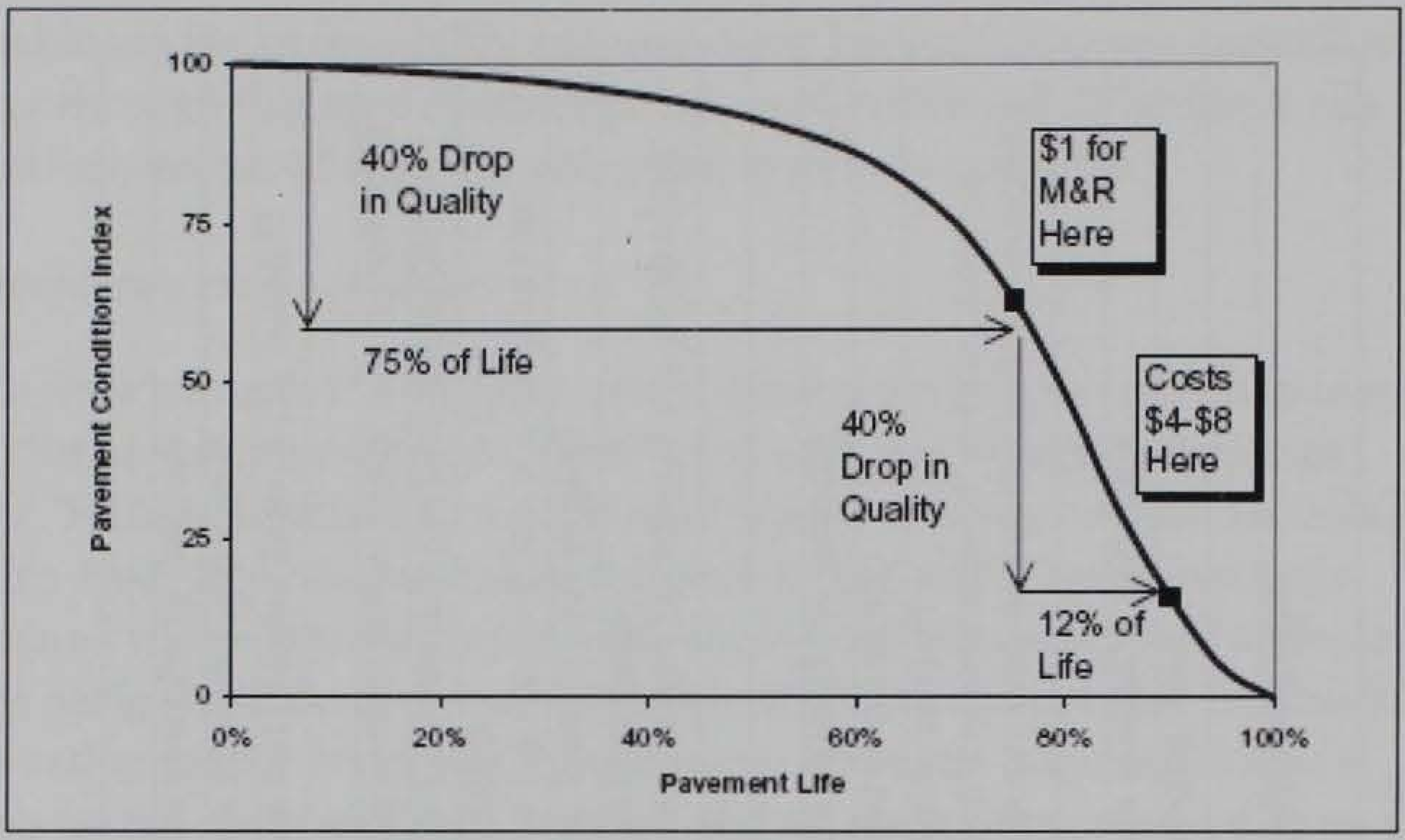


Figure 25. Graph of pavement condition index versus pavement life.

Construction history

The following construction history information was retrieved from the most recent Airfield Pavement Condition Index Survey report (AFCESA 2005):

> The original airfield construction was accomplished between 1941 and 1944. The initial phase consisted of three 150-ft-wide runways—the North-South Runway (Runway 17-35), the Northwest-Southeast Runway (Runway 12-30), and the Northeast-Southwest Runway (now Taxiway H)—each approximately 6,000 ft long. During that time, 18,838 yd^2 of parking aprons and sufficient 50-ft-wide taxiways to connect the runways to the parking aprons were also constructed. Between 1959 and 1960, major portions of the apron system were constructed, including the Reserve Apron (now referred to as the 507 ARW Ramp), the Cargo Apron, the ALC Apron, the Navy Apron, and the Transient Apron. Numerous rehabilitation and reconstruction projects have been performed over the years. Portions of Taxiways A, B, C, D, EE, F, G, and H have also been reconstructed or received major rehabilitation in recent years, as have the ALC Apron, the Cargo Apron, and the 507 ARW Apron.

Climate

The general climate of the Oklahoma City region falls mainly under continental controls characteristic of the Great Plains Region. The dry subhumid to semiarid climate of the region is known to be a prerequisite for swelling problems which are further accentuated by geology and soil conditions. The temperature at Tinker AFB varies from an annual mean daily minimum of 51°F to an annual mean daily maximum of 70°F. The mean temperature varies from 38°F to 82°F over the course of the year. The highest and lowest recorded temperatures are 109°F and -7°F, respectively. On average, the area receives 35 in. of precipitation per year, 73% of which falls between April and October. The Freezing Index is about 235°F-days. Frost penetration into the subgrade is generally not a problem; however, daily freeze-thaw cycles occur frequently during the winter months.

Soil conditions

The surface geology of Oklahoma is fairly consistent. The central region of Oklahoma is underlain by sedimentary deposits of the Permian age, which include shales and reddish brown sandstones. Fine-grained materials, such as low to moderate plasticity clayey silts, silty clays, and clays, formed from the weathering of these Permian shales. These fine-grained materials exhibit low to moderate shrink-swell potential, as shown in Figure 26.

Swell is the expansion of soil that occurs when water infiltrates between and within the clay particles, causing them to separate. Based on the results of a recent soil study performed in May 2008, the Plasticity Index (PI) and Liquid Limit (LL) ranges of the predominant subgrade material at Tinker AFB are 15 to 23 and 33 to 45, respectively. Using these plasticity characteristics and Table 7, the subgrade material at Tinker AFB has a medium potential (5 to 10%) to volume change; which is still significant and requires attention, especially in the design of the airfield pavements.

Aircraft traffic

Traffic data were obtained from the most recent Airfield Pavement Condition Index Survey report, in 2005:

> Nearly 6,000 departures and landings were recorded at Tinker AFB each year. The peak months of operation are May through August. The majority of the aircraft operations are home-stationed, medium-load aircraft such as E-3, E-6, and C-135 variants, with a large number of transient trainers.

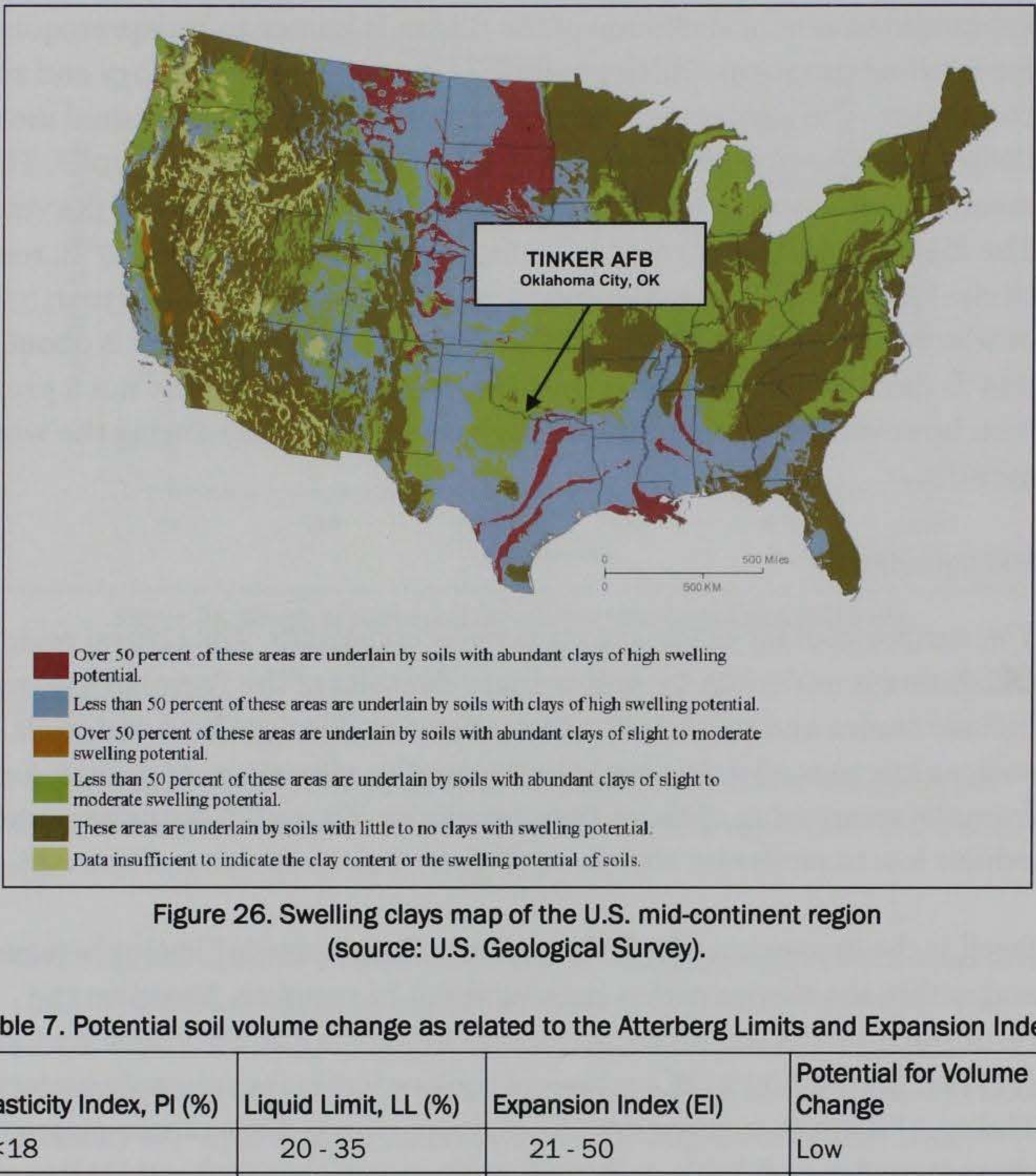


Figure 26. Swelling clays map of the U.S. mid-continent region (source: U.S. Geological Survey).

Table 7. Potential soil volume change as related to the Atterberg Limits and Expansion Index.

Plasticity Index, PI (%)	Liquid Limit, LL (%)	Expansion Index (EI)	Potential for Volume Change
<18	20 - 35	21 - 50	Low
15 - 28	35 - 50	51 - 90	Medium
25 - 41	50 - 70	91 - 130	High
>35	>70	>130	Very high

Source: Bowles (1996).

Use of drainage layers

The natural subgrade soils in the Tinker AFB area are not very permeable and do not drain very well. Pumping (Figure 27) and other problems related to poor drainage were a concern in several areas. Another concern was the swell potential of the expansive subgrade soils. Swelling damage occurs on pavements, due to the little resistance to soil expansion that the small surcharge load of an asphalt pavement or a concrete slab can provide. The use of a subbase provides additional surcharge pressure, but it

Figure 27. Staining from pumping due to inadequate base course material.

can also become a path for additional water to enter the expansive subgrade soil. The addition of a subsurface drainage system is beneficial in these cases.

Considering that the airfield is located in a semiarid climate area underlain by not very permeable soils with a medium swell potential, the newer construction projects at Tinker AFB have included subsurface drainage systems to control these problems.

Drainage layer evaluation

Two areas were evaluated at Tinker AFB: the Air Logistics Center (ALC) Ramp and Taxiway Bravo. The ALC Ramp was tested with the purpose of evaluating the drainage capacity of a pavement structure without any kind of drainage layer. At Taxiway Bravo, the performance of a permeable asphalt drainage layer was evaluated. The next sections are dedicated to describe these tests and discuss the results.

ALC ramp. After coring through the concrete, it was observed that there was no base material. The concrete slab was placed directly on top of the subgrade material as shown in Figure 28. By visual characterization of the soil and by simply observing the accumulation of water inside the core hole for a few minutes after coring, it was determined that the subgrade

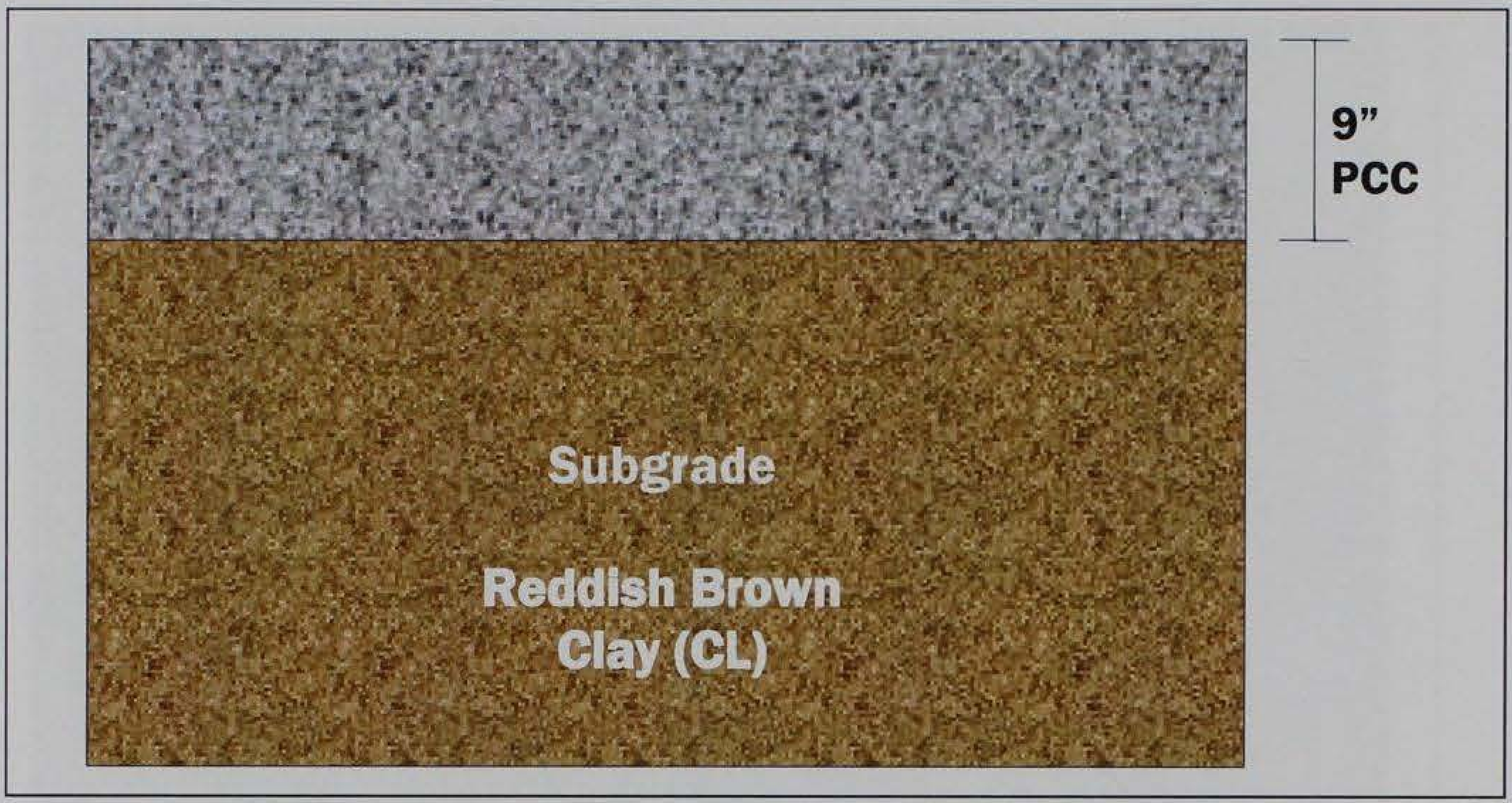


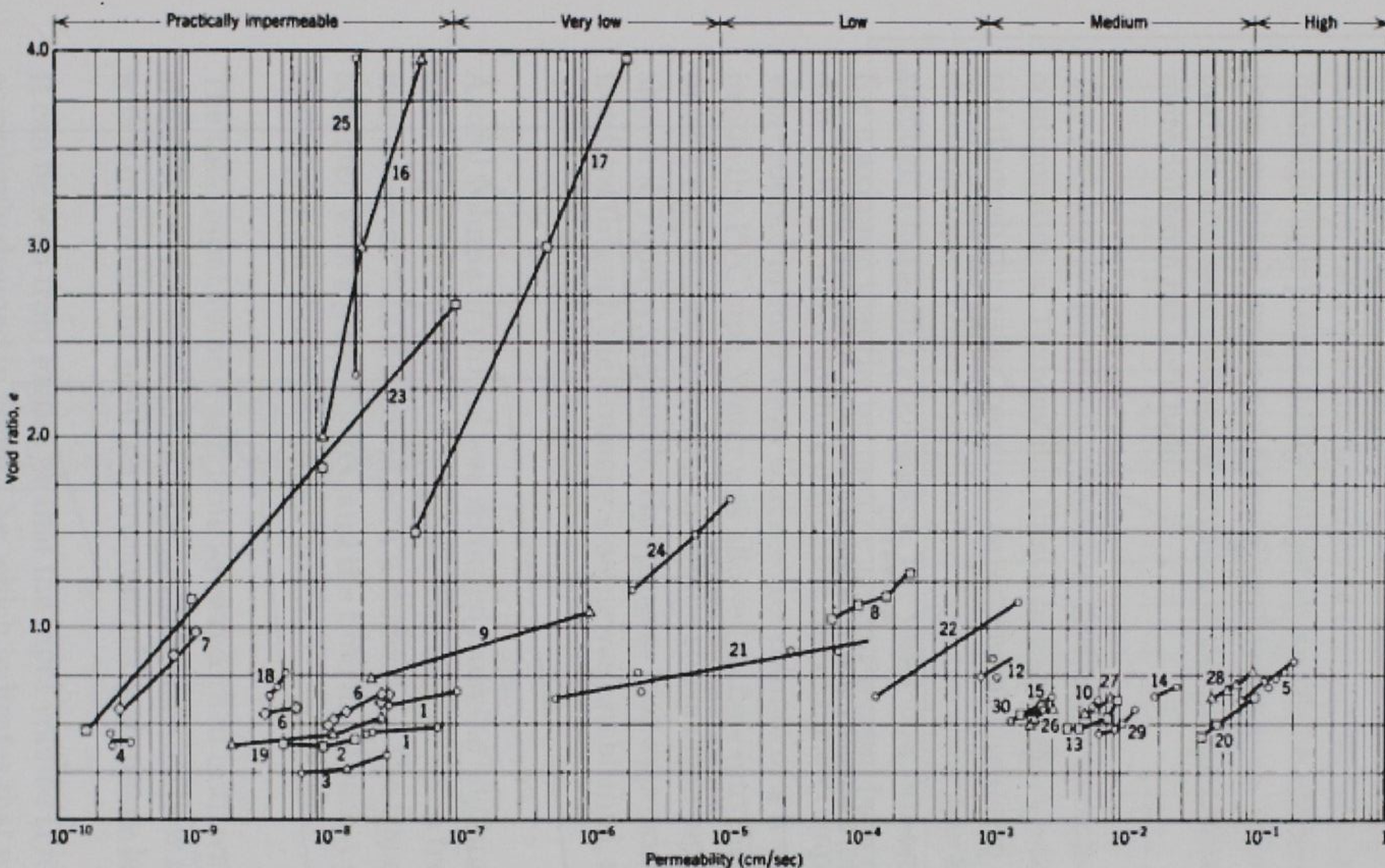
Figure 28. Pavement structure at the ALC Ramp, Tinker AFB, OK.

soil was not very permeable; therefore, no flow measurements could be performed in this area. However, the permeability was estimated using Figure 29. Dry density values were retrieved from the most recent soil study to estimate the void ratio (e). From Figure 29 the estimated permeability of this subgrade soil is 2.3×10^{-5} ft/day.

Soil samples were extracted from the hole for subgrade soil characterization. This subgrade soil was the same reddish brown clay (CL) identified by the soil study mentioned before. The CL was a fine-grained soil with more than 50% finer than the No. 200 sieve, as shown in Figure 30, which, in part, explains the low permeability.

The test at the ALC ramp allowed the characterization the subgrade material at Tinker AFB and confirmed the expected low permeability of the subgrade soil.

Taxiway Bravo. Taxiway Bravo is delineated as a primary airfield section (Type A traffic area), thus it receives channelized traffic and the full design weight of aircraft. It runs East-West, connecting Runway 17-35 with primary Taxiway G. Taxiway Bravo was recently expanded and resurfaced in 2005. The rehabilitation also consisted of the addition of a 4-in.-thick permeable asphalt drainage layer and longitudinal collector pipes. The pavement surface was replaced by 15 in. of portland cement concrete, as shown in Figure 31.



Soil Identification Code

1 Compacted caliche
2 Compacted caliche
3 Silty sand
4 Sandy clay
5 Beach sand
6 Compacted Boston blue clay
7 Vicksburg buckshot clay
8 Sandy clay
9 Silt—Boston
10 Ottawa sand
11 Sand—Gaspee Point
12 Sand—Franklin Falls
13 Sand—Scituate
14 Sand—Plum Island
15 Sand—Fort Peck
16 Silt—Boston
17 Silt—Boston
18 Loess
19 Lean clay
20 Sand—Union Falls
21 Silt—North Carolina
22 Sand from dike
23 Sodium—Boston blue clay
24 Calcium kaolinite
25 Sodium montmorillonite
26-30 Sand (dam filter)

Figure 29. Permeability test data (from Lambe and Whitman).

Figure 30. Gradation curve of subgrade material at Tinker AFB, OK.

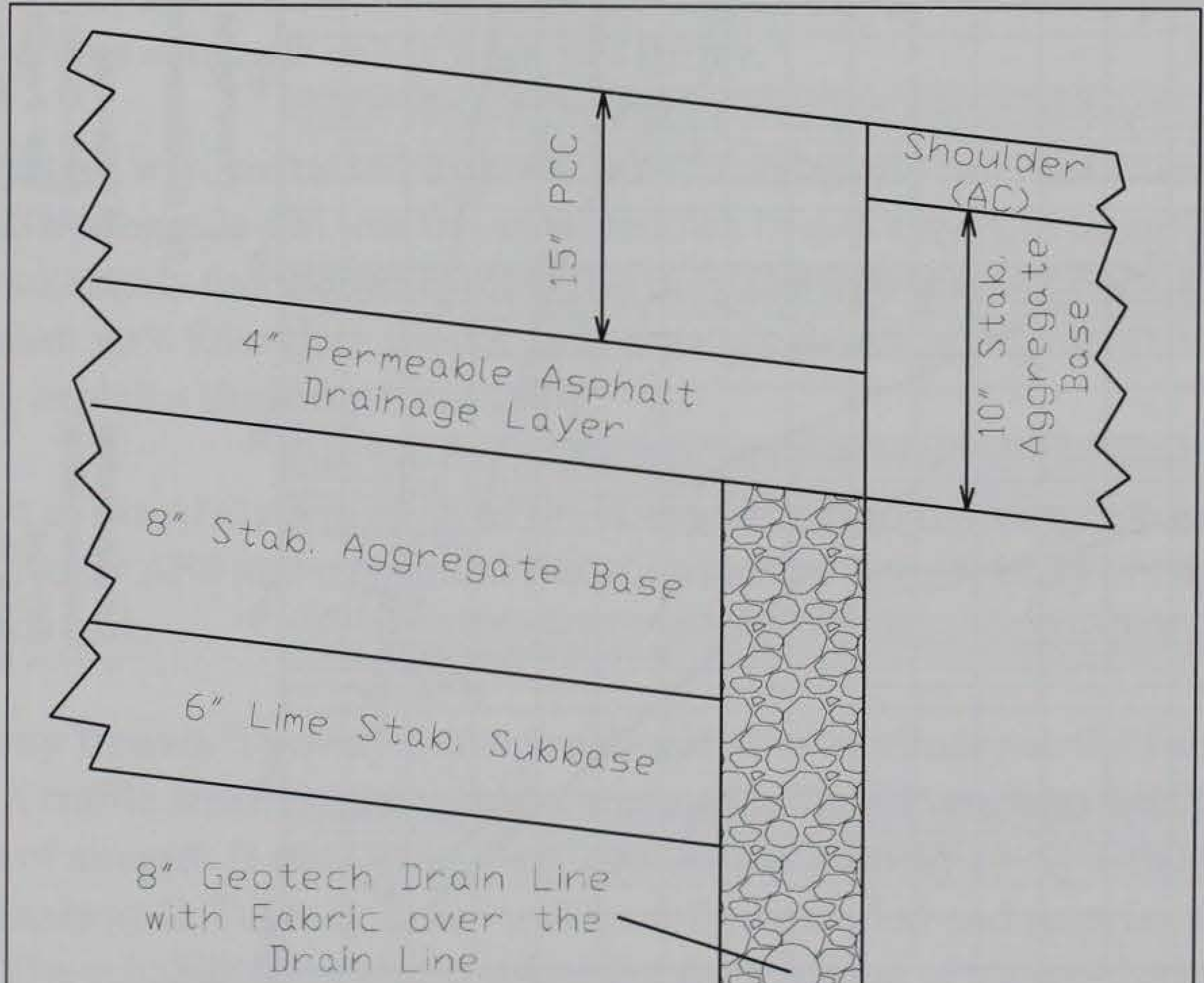


Figure 31. New pavement structure at Taxiway B, Tinker AFB, OK.

The drainage layer in Taxiway B consisted of asphalt stabilized material. The gradation curve for this material is presented in Figure 32. This gradation corresponds to a rapid draining material (RDM) as shown in Table 3, and the expected corresponding permeability (Table 4) for a RDM was between 1,000 and 5,000 ft/day. The gradation curve also shows 23% sand, which falls outside the limits of the specification band. However, drainage layer permeability did not appear to be affected since the field tests results showed a good drainage performance. This could be related to particle breakage during the process of extracting compacted drainage material from the core hole.

The previously described evaluation procedure was followed, and the test configuration at Taxiway B is shown in Figure 33. A 6-in.-diam core was cut through the 15-in. concrete pavement surface down to the top of the drainage layer. Water was pumped from the water truck into the core hole through a hose with a flowmeter. The water flow rate was first adjusted to the maximum that the permeable asphalt-treated base could accommodate without water spilling out of the top of the PVC pipe. Then the flow was reduced in such a way that a water column of 4 ft was maintained inside the PVC pipe. Water was allowed to flow into the drainage layer until it was observed flowing out of the storm drain pipe at inlet box shown in Figures 34a and b. This process took approximately 5 min. Once free flow through the drainage system was established, a tracer dye was added to the inflowing water.

A total volume of 170 gal was recorded from the addition of tracer dye until the water inflow was stopped. The time to when tracer dye outflow was observed was measured with a stopwatch. The tracer dye outflow was collected with a 5-gal bucket, and the time to fill each 5 gal was recorded and is presented in Figure 34c).

The total input volume and time were recorded until water flow was stopped and are shown in Figure 35. The effective flow rate was 23.5 gpm, which indicates the good performance of the drainage layer.

It can be seen from Figure 36 that the input volume (170 gal) was almost completely drained in about 1 hr, which indicates that water retention was minimal or negligible. No flow obstruction was observed.

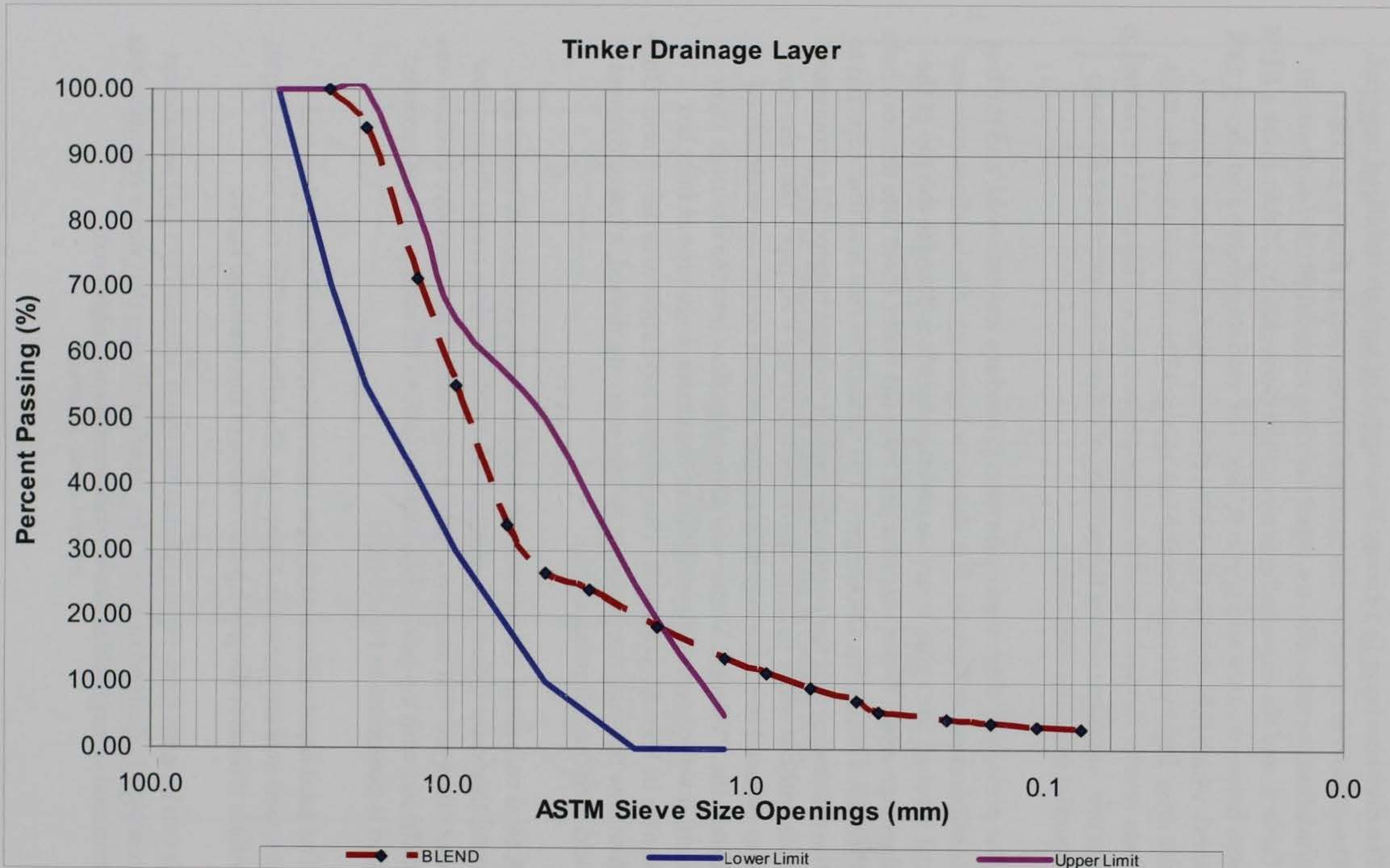


Figure 32. Gradation curve of drainage layer material at Taxiway B, Tinker AFB, OK.

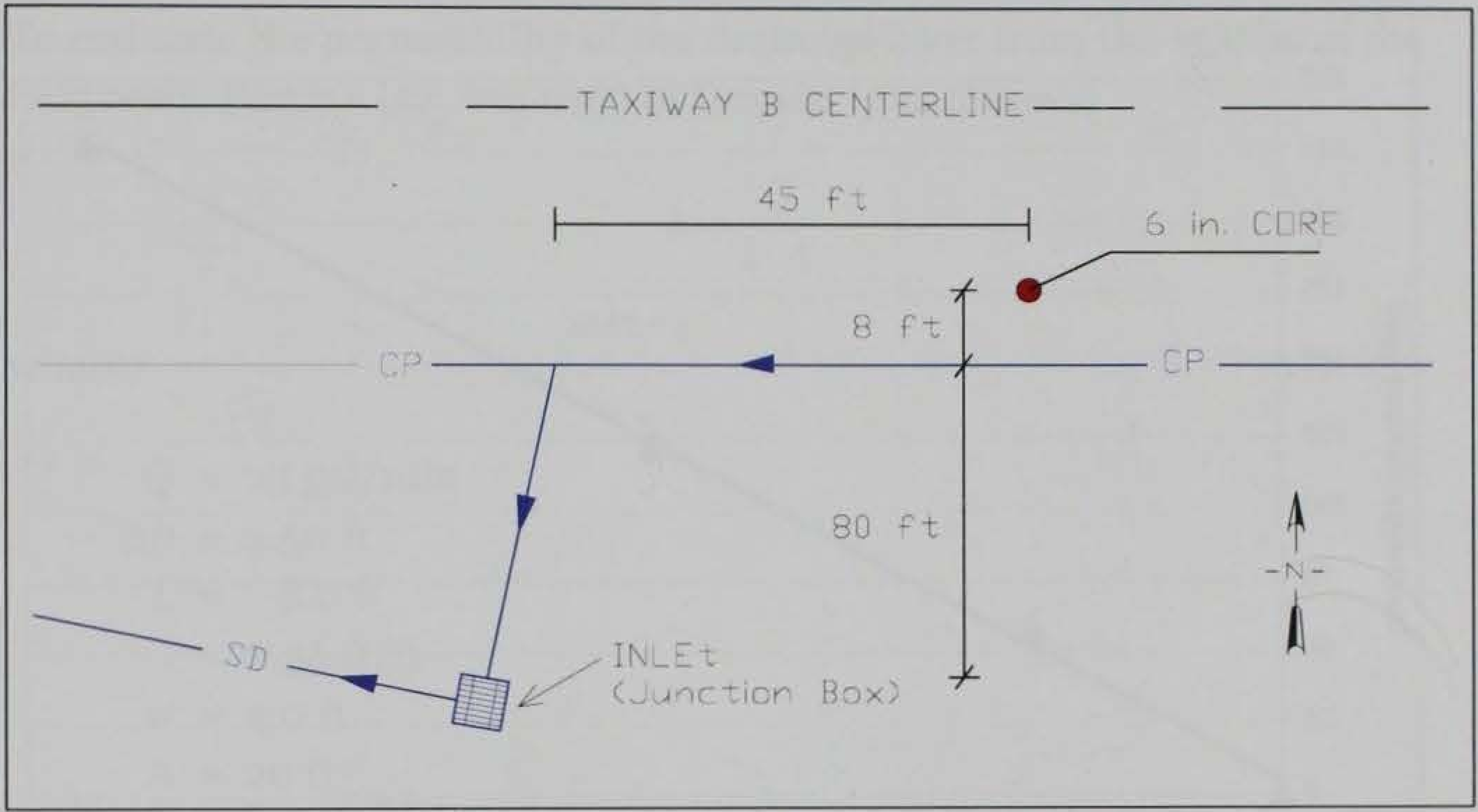


Figure 33. Test configuration at Taxiway B, Tinker AFB, OK.

Figure 34. (a) View from the inlet to the core, (b) lifting the inlet to access the storm drain pipe, and (c) dyed water being collected from the storm drain pipe.

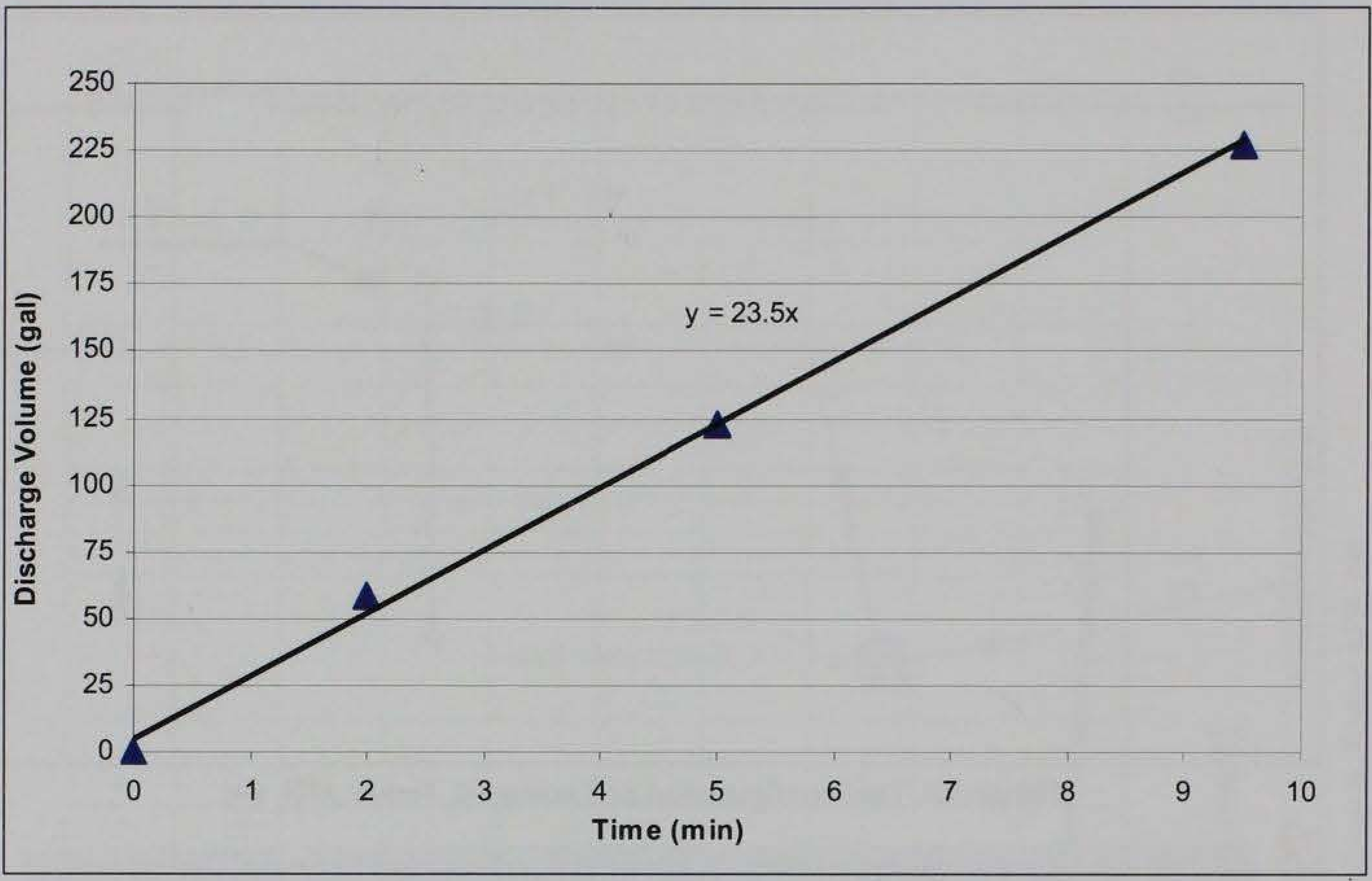


Figure 35. Volumetric flow into drainage layer.

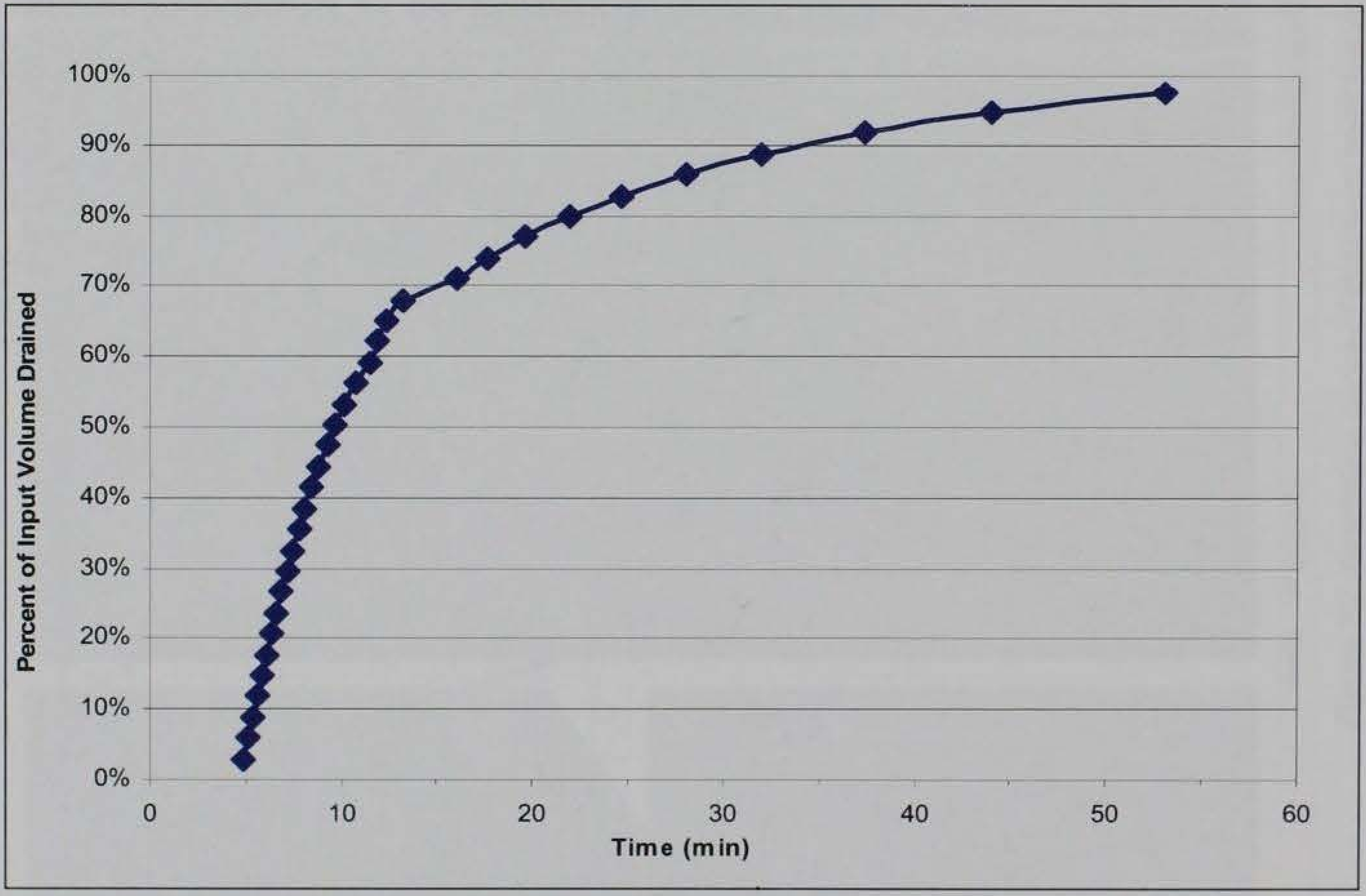


Figure 36. Percent of total input volume collected at the outlet.

To estimate the permeability of the drainage layer from the results of the field tests, Darcy's law was used in this study as follows:

$$k = \frac{Q}{i \cdot A}$$

where:

Q = 23 gal/min
Δh = 4.50 ft
L = 8.0 ft
i = 0.56 ft/ft
w = 4.0 ft
A = 20 ft^2
k = 4,026 ft/day

The results are reasonable and fall within the expected range of permeability for a RDM (1,000 to 5,000 ft/day). However, the calculated permeability is a function of the width of the flow plume shown in Figure 37. Since the surface layer in Taxiway B is a 15-in.-thick PCC pavement it was not possible to use the GPR to measure the moisture profile and determine the width of the flow plume. Therefore, an average width of 4 ft was assumed due to the rapid draining nature of the material that was observed. Although a variation of the assumed width of the flow plume could change *k* by thousands of feet per day, this approach gives a relative indication of the in situ performance of the drainage layers.

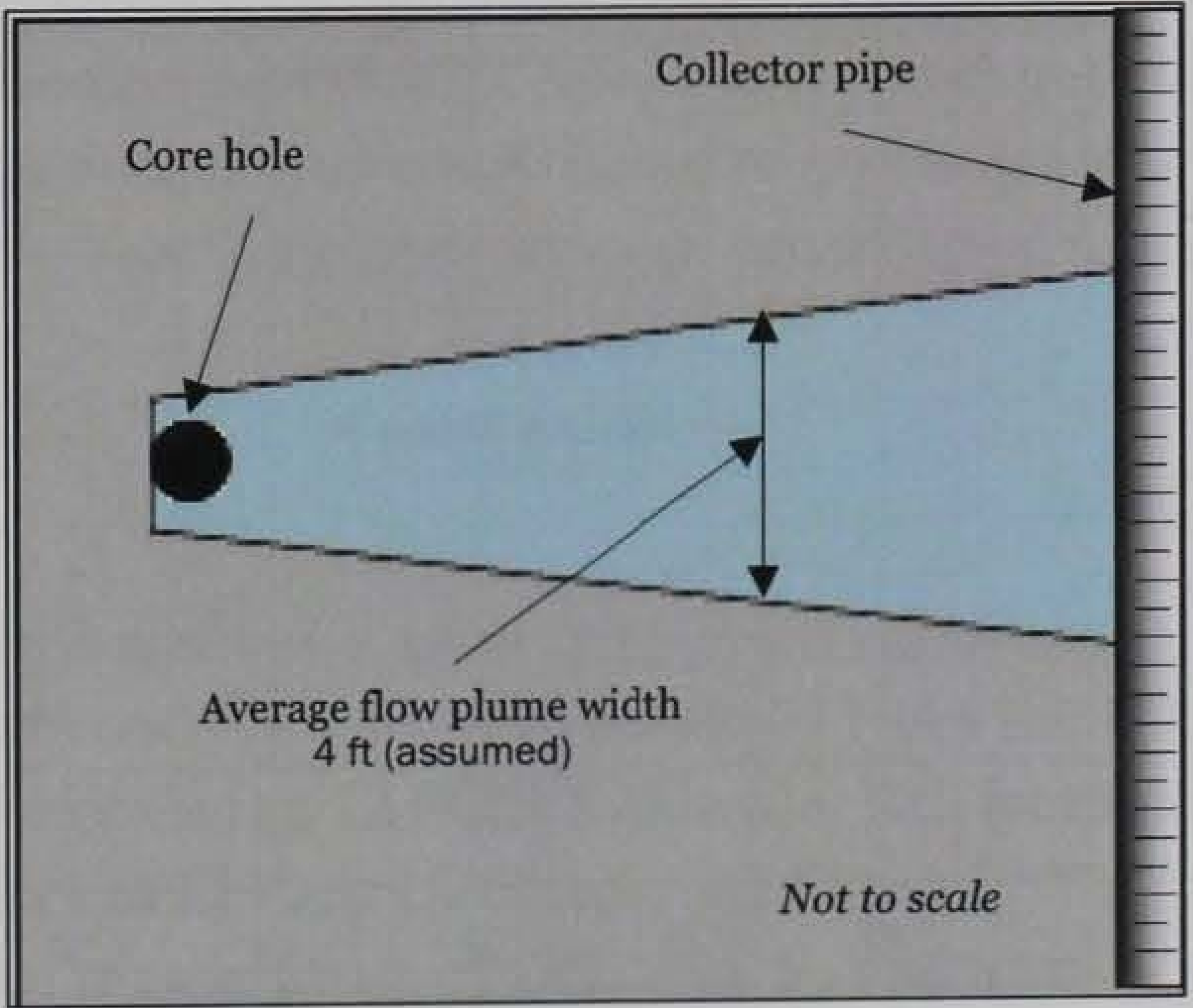


Figure 37. Assumed flow plume width for permeability calculations.

Performance data evaluation

Taxiway B was reconstructed on 2005 after the preparation of the last airfield pavement condition survey and evaluation report. By the time this report was written, no airfield pavement evaluation had been performed on Taxiway B. Therefore, the long-term performance of the drainage layer could not be evaluated. However, PCI records were consulted to determine the performance of the previous pavement structure, without the actual drainage layer. Records from 1986 to 2004 are presented in Table 8 and plotted in Figure 38. The previous pavement structure consisted of a soil

Table 8. PCI records for Taxiway B at Tinker AFB, OK.

Pavement Feature	Year Constructed	Pavement Type (Thickness, in.)	Evaluation Date	PCI	Distress Types
Pavements Without Drainage Layers					
Taxiway B	1969	PCC (14)	2004	51	L/T cracking, joint spalling, map cracking, large patch
			2003	55	
			1992	68	
			1986	82	
Taxiway B Shoulders	1969	AC (2.5)	2004	66	L/T cracking, patching, swelling, weathering
			1986	68	

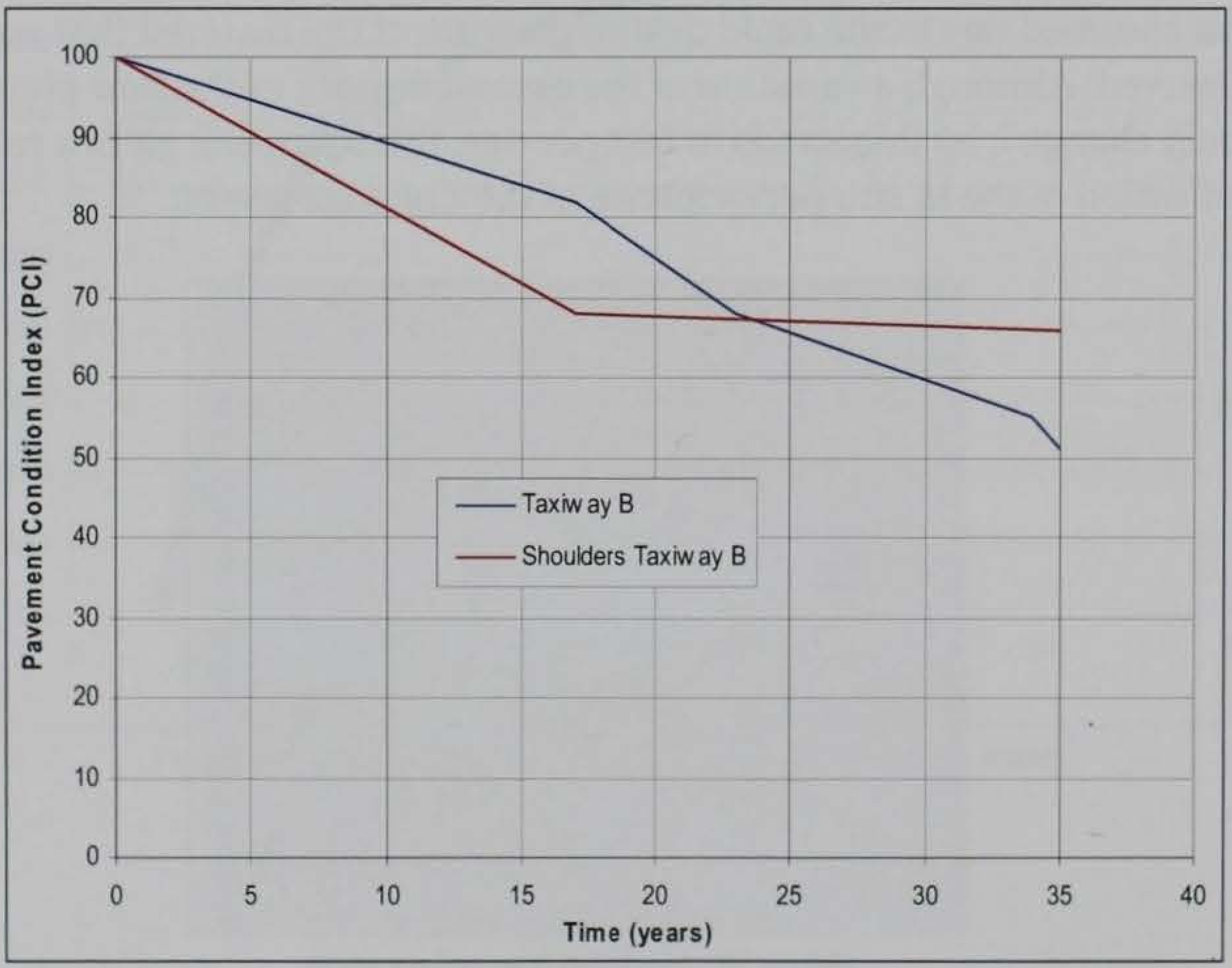


Figure 38. Taxiway B pavement life cycle.

cement base, a silty sand subbase, and the CL subgrade. This structure had a decrease in PCI of about 50% in 35 years, as shown in Figure 38. Recorded PCC pavement distresses include longitudinal cracking, large patch, and joint spalling. None of these distresses is related to poor drainage, but some of them, if not repaired, can become potential paths for additional water to enter the subsoil. Medium-severity swelling problems were recorded in the shoulder sections (AC pavements) of the taxiway during the 2004 survey, decreasing the PCI to 66. These problems are the product of poor surface and subsurface drainage which allows the entrance of water to the expansive subgrade soil, causing it to swell and lose strength.

In 2005, the taxiway was reconstructed to replace the surface, expand the taxiway width, and upgrade the subsurface drainage system.

Biggs Army Airfield, Fort Bliss, Texas

An evaluation of pavement sections at Biggs Army Airfield at Fort Bliss, Texas, was conducted from 17 to 20 November 2008.

Construction history

The original pavements at BAAF were constructed from 1942 to 1945. Upgrading of the pavements, including new construction, reconstruction, or strengthening of the existing facilities was performed at various periods from 1945 to the present. The recent major construction projects at BAAF includes the construction of the DAACG Apron, Taxiway K, and the Hot Cargo Apron in 2002. The DAACG Apron was constructed of 406 mm (16.0 in.) PCC. Taxiway J was constructed of 393 mm (15.5 in.) PCC. Taxiway K and the Hot Cargo Apron were constructed of 368 mm (14.5 in.) PCC.

Climate

The elevation of the airfield is 1202 m (3946 ft) above mean sea level. The climatological data used herein were obtained from the U.S. Air Force Combat Climatology Center (AFCCC) Ashville, NC, from data collected at the weather station at El Paso International Airport, TX. These data reflect an average annual temperature of 18°C (64°F) and an average yearly high of 26°C (78°F). The average annual rainfall in the area is about 220 mm (8.7 in.).

Soil conditions

BAAF is located at Fort Bliss, Texas, in El Paso County, El Paso, TX. The airfield is located physiologically in the Huaco Basin, a feature of the Mexican Highland section of the Basin and Range Province of the Intermontane Plains. The soils in the area are generally reddish, slightly clayey silty sands with caliche at lower depths. Caliche is a sedimentary rock, a hardened deposit of calcium carbonate. This calcium carbonate cements together other materials, including gravel, sand, clay, and silt.

Drainage layers are being used in most new construction projects at Biggs AAF. These layers are designed based on requirement in UFC 3-230-06a given in Chapter 2. Since Biggs AAF lies in a nonfrost area, it would be exempt from the drainage requirement if the subgrade soil were sufficiently permeable.

The subgrade soil (Figure 39) was classified as a silty sand. The large percentage (26%) of material passing the No. 200 sieve suggests that permeability of the subgrade is low. Drainage layers were likely placed beneath the PCC pavement based on the soil gradation.

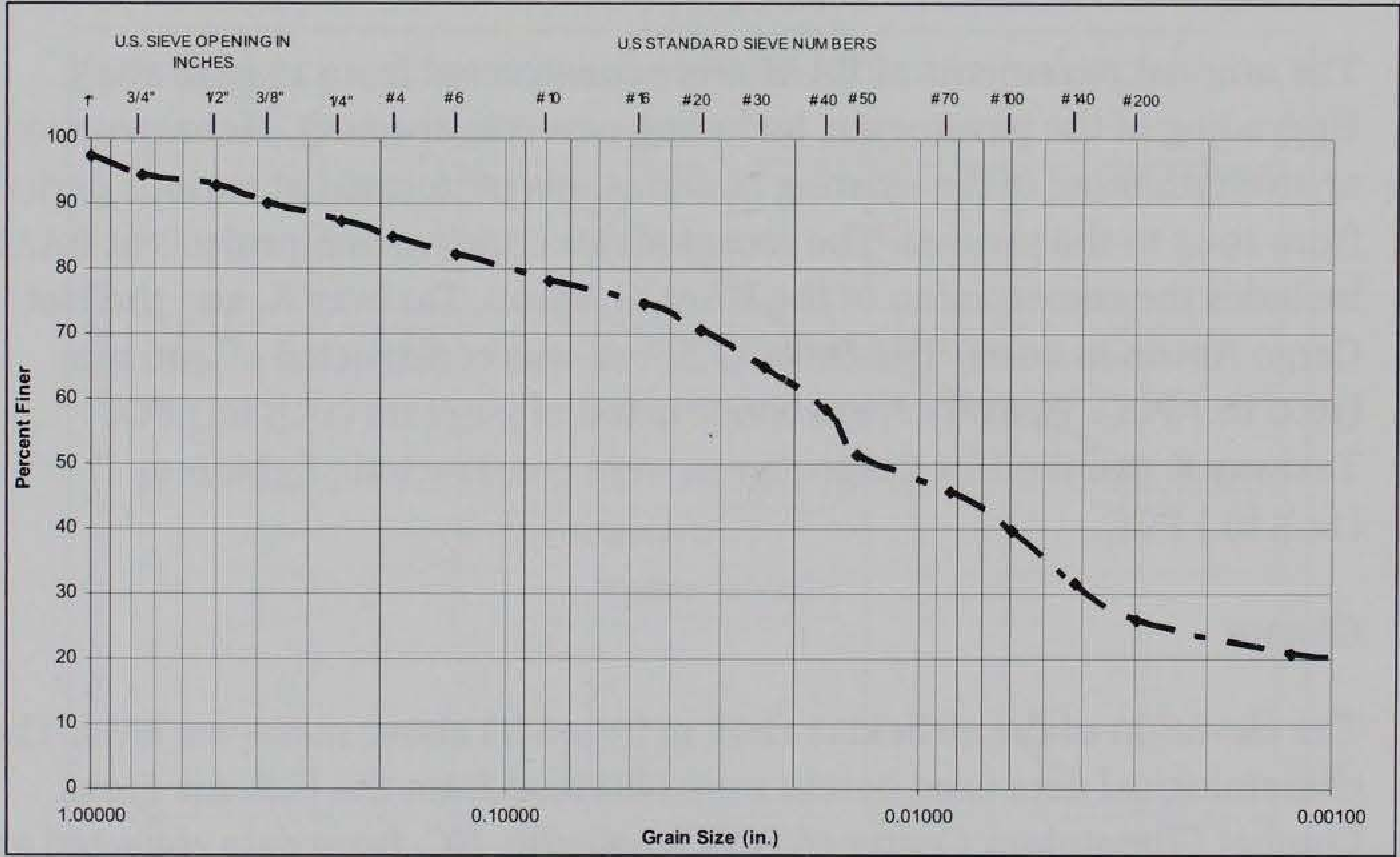


Figure 39. Subgrade gradation at Biggs Army Airfield.

Drainage layer evaluation

Pavement drainage layers have been installed at the following locations at Biggs AAF:

- DAACG ramp,
- Taxiway J,
- Taxiway H,
- Taxiway K,
- Hot cargo ramp.

Only the DAACG ramp was tested during this site visit. The drainage system for this area was interconnected with the drainage system beneath Taxiways J and H. Taxiways J and H were not evaluated because of poor results from the DAACG testing.

The DAACG ramp was constructed in 2002. This pavement provides parking and access to facilities on the west side of the airfield. A typical cross section of the pavement is shown in Figure 40. The drainage layer in this location consisted of an unstabilized rapid draining material.

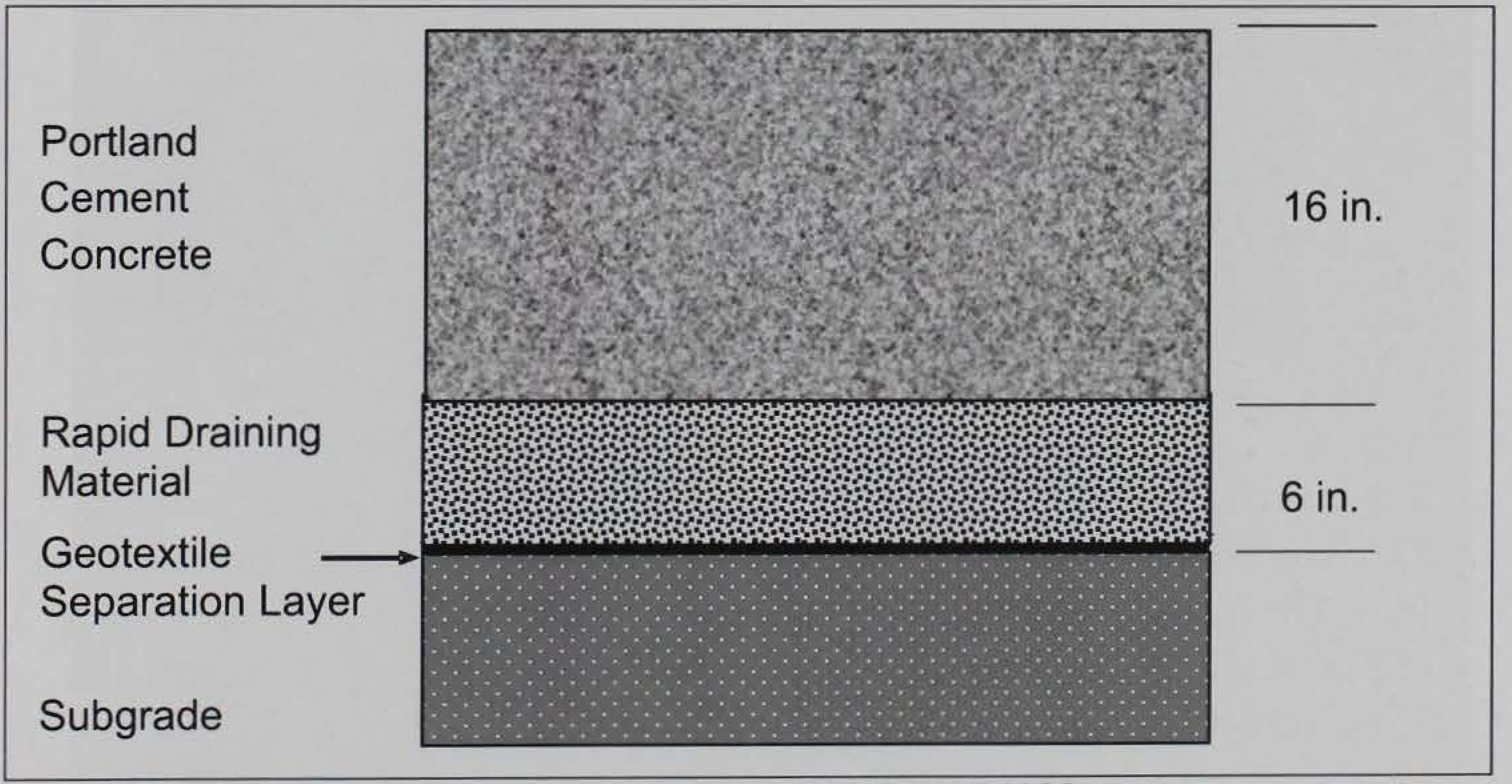


Figure 40. Cross section of pavement at DAACG ramp.

An outflow pipe was located on the northwest corner of the apron (Figure 41). The pipe led from a manhole to a drainage basin. Two pavement drainage pipes were located under the shoulder running parallel to the west and north edges of the apron. These pipes emptied into the manhole (Figure 42).

Figure 41. Poorly maintained outlet pipe leading from DAACG Ramp.

The overall layout of all pipes underneath the pavement was not known. The pavement sloped towards the northwest corner at a slope conducive to flow. The collector drainage pipes were 8 in. in diameter. They were made of PVC and contained uniformly spaced holes for water entry (Figure 43). They were wrapped with a geotextile to prevent clogging.

An area of exposed soil downslope from the outflow pipe was experiencing erosion from water flow. (Figure 44). The erosion was thought to originate from water flowing from the drainage system. However, closer inspection revealed soil erosion on top of the outflow pipe and the observation that all surface runoff travels through the location (Figure 45). The erosion could occur even if no water was flowing through the drainage system.

Figure 42. Drainage pipes converging at manhole on DAACG ramp.

Figure 43. Perforated drainage pipe.

Figure 44. Soil erosion caused by surface water drainage.

Figure 45. Surface drainage following path of drainage outlet pipe.

An initial core was taken in the PCC at the location shown in Figure 46 designated as Core 1. The upper portion of the drainage layer was removed for analysis. A 4-in.-diam PVC pipe was secured on top of the remaining drainage material. Water was introduced into the pipe from the water truck at the maximum rate to sustain a constant 4-ft head of water in the pipe. A plot of the input volume versus time is shown in Figure 47.

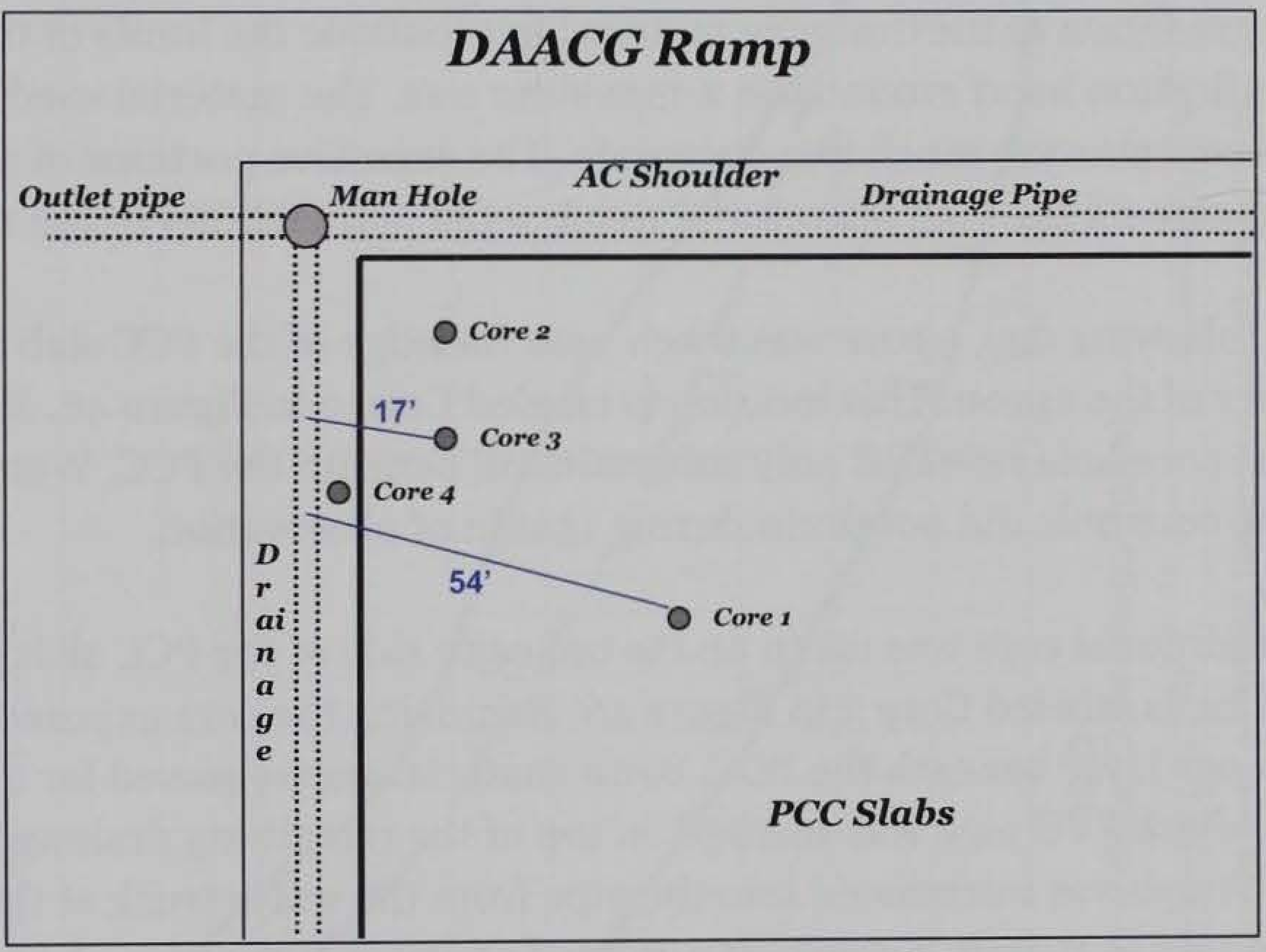


Figure 46. Layout of testing area on DAACG Ramp.

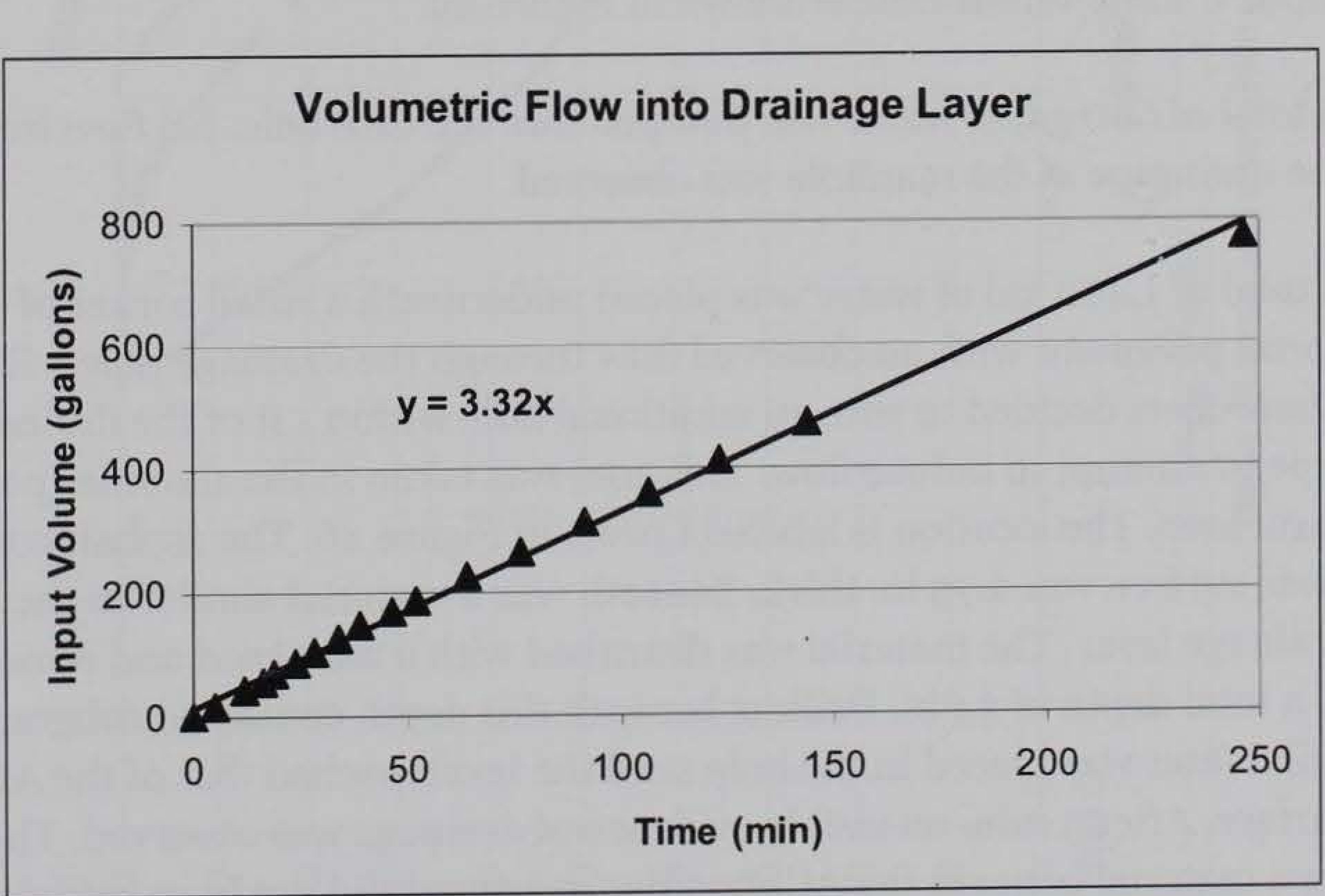


Figure 47. Flow rate of water introduced into drainage layer on DAACG Ramp.

A total of 800 gal of water was pumped into the core hole. No flow from the drainpipe at the manhole was observed.

Figure 48 shows a graphical depiction of the gradation of the drainage material and the limits of the specification given in UFC 3-260-02 (Headquarters, Departments of the Army, the Navy, and the Air Force 2001).

The gradation of the drainage material falls outside the limits of the specification band around the 2-mm sieve size. The material used at Biggs AAF contains too much fine aggregate. The excessive portions of fine aggregate fill the void spaces and reduces the permeability of the material.

The following day, a core was taken near the edge of the PCC slab in the corner of the apron. This location is labeled Core 2 in Figure 46. Removal of the core hole revealed only subgrade soil beneath the PCC. Water placed in the core hole did not drain during 15 min of observation.

An additional core was taken on the opposite side of the PCC slab. This location is labeled Core 3 in Figure 46. Removing the core exposed the drainage layer beneath the PCC. Some material was removed for testing. A 4-in.-diam PVC pipe was secured on top of the remaining drainage material. Water was introduced into the pipe from the water truck at the maximum rate to sustain a constant 4-ft head of water in the pipe. A plot of the input volume versus time is shown in Figure 49.

A total of 600 gal of water was pumped into the core hole. No flow from the drainpipe at the manhole was observed.

A total of 1,400 gal of water was placed underneath a small corner of the apron pavement with no observed flow through the drainage pipes. ERDC researchers decided to core an additional hole within 1 ft of the drainage pipe to attempt to induce flow. This core was taken in the shoulder pavement area. The location is labeled Core 4 in Figure 46. The asphalt concrete surface was 2.75 in. thick. Beneath was a material similar to the drainage layer. The material was disturbed with a metal rod and removed to a total depth of 24 in. Drilling beneath this depth contacted subgrade soil. Water was placed in the hole until the level reached that of the AC surface. After 5 min, no visible evidence of drainage was observed. The base material beneath the AC shoulder pavement did not have sufficient permeability to promote flow.

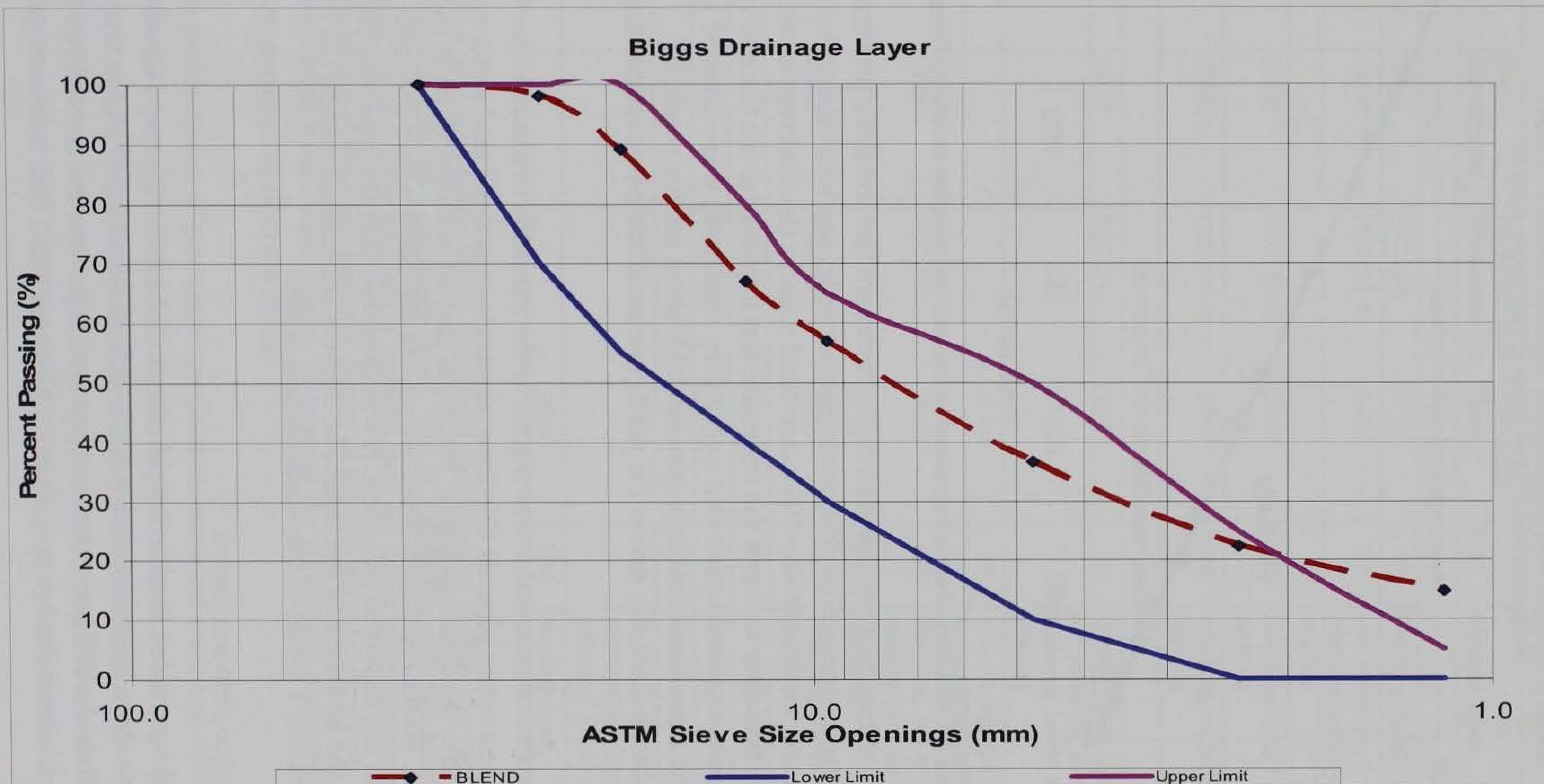


Figure 48. Grain size distribution of drainage layer on DAACG Ramp and specification limits.

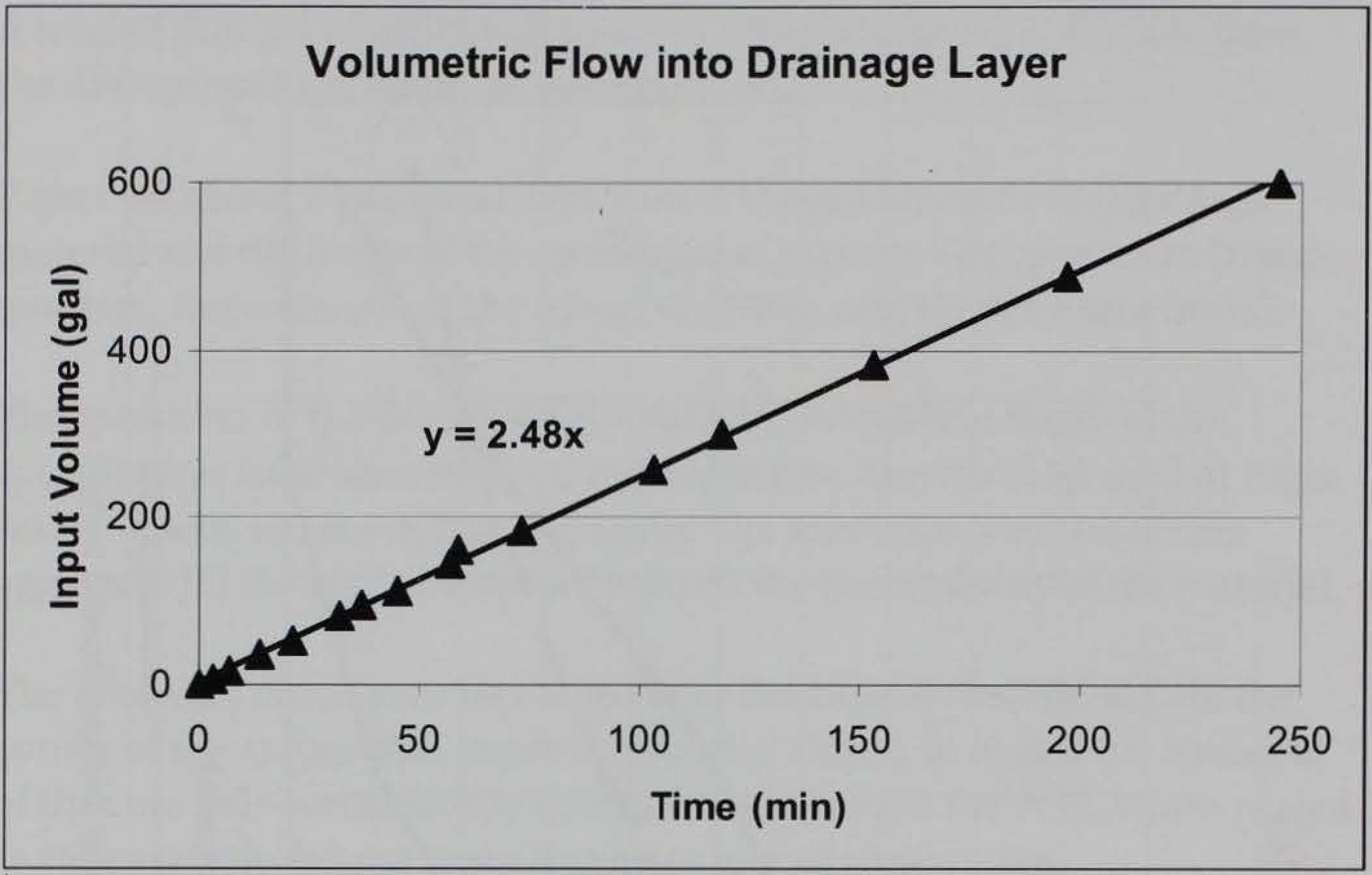


Figure 49. Flow rate of water introduced into second test location on DAACG Ramp.

The low permeability of the material underneath the shoulder prevented flow to the drainage pipe. The water placed underneath the slabs was trapped and likely remained in a saturated state in the drainage layer. Some vertical flow through the subgrade was expected given the soil type. It is important to recognize that only a very small area was observed in respect to the overall drainage system. Additional testing would be required to definitively assess the performance of the pavement drainage system.

After the evaluation, water was placed in the manhole to observe the flow through the outlet pipe. At this time, researchers noticed that water was flowing into the drainage pipes. The pipes had an inverted slope at the manhole. Water eventually filled the pipe to the point where flow proceeded in the expected direction. It was expected that the pipe was raised to meet the elevation of the manhole. Some water cannot escape the drainage system as a result of this construction error.

Performance data evaluation

Data were extracted from Army pavement evaluation reports to identify distresses on pavements constructed with and without a drainage layer. Table 9 lists the PCI and distress types for the pavements included in this study and others similar in function. The items listed first were designed

Table 9. Pavement condition survey results from Biggs Army Airfield.

Pavement Feature	Year Constructed	Pavement Type (Thickness, in.)	Evaluation Date	PCI	Distress Types
Pavement With Drainage Layers					
DAACG Ramp	2002	PCC (14)	2007	96	Patching, spalling
Hot Cargo Ramp	2002	PCC (14.5)	2007	97	Low severity L/T cracking, shrinkage cracking, patching, low severity spalling
Taxiway J	2002	PCC (15.5)	2007	93	Low severity L/T cracking, patching, spalling
Taxiway K	2002	PCC (14.5)	2007	97	Patching, shrinkage cracking, spalling
Pavements Without Drainage Layers					
Taxiway D	2000	PCC (20)	2007	90	Corner break, low to medium severity L/T cracking, patching, shrinkage cracking, low to medium severity spalling
			2002	98	Low severity L/T cracking, patching, low severity spalling
Taxiway E	2000	PCC (20)	2007	95	Patching, low to medium severity spalling, joint seal damage
			2002	100	Patching
Taxiway H	2000	PCC (20)	2007	88	Patching, low to medium severity spalling
			2002	100	No distresses
Taxiway I	2000	PCC (20)	2007	88	Patching, spalling
			2002	100	No distresses
North Warm-up Apron	2000	PCC (20)	2007	90	Corner break, low severity L/T cracking, joint seal damage, patching, low to high severity spalling
			2002	100	No distresses

with a drainage layer, while the items in the lower half of the table were designed without a drainage layer. The PCI data list only the numerical rating for the 2007 and 2002 surveys. Not all areas were surveyed in at each of the evaluation periods. Some of the pavements were not constructed until recent years. The age of the pavement was considered when comparing distress levels.

The performance of pavement sections constructed with and without a drainage layer show little differences. Those without a drainage layer have an average PCI slightly lower than the pavements constructed with a drainage layer. However, the pavements constructed without a drainage layer are older. The current condition of all pavements selected for this table is good according to the 2007 survey. The pavements constructed with a drainage layer have not been in place long enough to detect differences in performance. The rate of deterioration is slow at this point in the life of the pavement. Comparing old pavements constructed without a drainage layer would not be valid. The pavements considered in this work are near the beginning of their life and do not reflect large changes in PCI.

4 Results and Discussion

Field testing was performed to provide data for assessing the current condition of pavement drainage layers on military airfields. The field testing was conducted to determine the functionality of the drainage system and also provided data for relating functionality to design, construction, and specifications. To summarize the field testing results, pertinent observations are given below for each location.

Elmendorf AFB, Alaska

Observations specific to this location are as follows:

- Not all pavement drainage systems were designed and constructed to the UFC criteria. Specific examples include
  - The Fuel Cell taxiway was constructed with a daylighted drainage layer. This type of system is not allowed according to the criteria. Researchers acknowledge that subsequent construction will widen the taxiway and that alternate designs were not feasible.
  - The outlet pipe carrying water from the Hangar 18 apron was not constructed with a protective headwall.
- Maintenance of the drainage layers did not occur. The outlet pipe at the Hangar 18 apron could not be located. Only regular grass cutting was performed at the outlet pipe from the Weather Shelter apron. The Fuel Cell taxiway did not require maintenance since water could escape through the entire length of the taxiway from the daylighted drainage layer.
- During testing, water flowed from the center of the Fuel Cell taxiway to the end of the drainage layer in approximately 4 hr. The total travel distance was 60 ft. No water was observed flowing out of the pavement aprons at Hangar 18 or the Weather Shelter.
- The outlet pipe at the Hangar 18 apron was likely covered with soil and vegetation. Its location could not be determined. Personnel familiar with the construction indicated its supposed location.
- Subsequent construction of the Weather Shelter apron was thought to have damaged the drainage pipes that removed water from the apron.
- The permeability of the drainage layers was acceptable according to the design criteria. The rapid draining material at the Weather Shelter was

more permeable than the asphalt stabilized drainage layer at the Fuel Cell taxiway and the Hangar 18 apron.

- The permeability of the base course material used on the Fuel Cell taxiway was approximately one twelfth as permeable as the asphalt stabilized drainage layer based on the rate at which water was capable of flowing into the pavement.
- The pavements where drainage layers were used were in good condition. However, they are relatively new (less than 12 years).

Tinker AFB, Oklahoma

Observations specific to this location are as follows:

- Pavement drainage systems appeared to have been designed and constructed according to the UFC criteria. The gradation of the drainage layer was within the specifications. Outlet pipes were constructed with protective structures and coverings.
- Maintenance of the drainage system was not evident. The condition of the drainage system on Taxiway B was in excellent condition. However, this pavement was newly constructed. Observation of the drainage system on Taxiway G revealed heavy vegetation covering and blocking drains.
- The rate of flow through the drainage system was observed to be very rapid. The fast rate of flow was evident by the rate at which the water dye was observed at the outlet structure.
- As-built drawings were not accurate. Flow of water was observed at an unexpected location in the outlet structure.

Biggs AAF, Texas

Observations specific to this location are as follows:

- The pavement drainage layer material was not designed and constructed according to the UFC criteria. The gradation of the drainage layer did not fall within the specifications. Too much fine aggregate was present. The excess fine aggregate inhibited permeability. Other portions of the drainage system were built according to UFC criteria. The outlet pipe was constructed with a supporting and protective headwall. Drainage pipes were wrapped with geotextile fabric to prevent clogging. Geotextile fabric was also observed as a filter at the interface of the drainage layer and subgrade.

- Maintenance of the drainage system was not evident. Vegetation and soil surrounded the outlet structure.
- Quality assurance was not successful at achieving satisfactory construction of the drainage layer. At one location, only subgrade was found beneath the PCC. Also, the slope of the drainage pipes was inverted where they met the manhole, allowing some water to flow back into the drainage system. The base course beneath the pavement shoulder was not permeable. This prevented water from traveling through the drainage layer to the drainage pipe. These types of construction flaws could have been prevented with adequate oversight.
- No flow was observed from the drainage pipe after two days of introducing water into the drainage layer. The system was determined to not be operating effectively.

Discussion

The observations made at each of the testing locations were used to determine conclusions and recommendations for this research. Several of the observations were similar at each location. Others specifically address an issue encountered at individual testing locations.

A lack of maintenance of the drainage systems was a common theme during testing. Drainage structures become clogged over time because of a lack of adequate maintenance. Unmaintained drainage systems will likely provide little performance benefit.

Drainage systems were not always designed and constructed according to the UFC criteria. Issues such as improper construction of outlet pipes and improper aggregate gradation were observed. In these cases, drainage was not effective due to restricted flow. Personnel responsible for construction must consider the impacts of altering designs on the overall system. Similarly, quality assurance procedures must be in place and followed to ensure construction practices provide the intended product.

Although daylighted drainage systems are not specified by the UFC criteria, they provide an alternative construction method that requires less maintenance than using drainage pipes. They have a much greater area where water can escape the pavement in case some of the drainage layer becomes clogged. This type of design could be effective, especially on areas such as taxiways where the width of the pavement is small relative to the length.

Determining the benefit of drainage layers on the pavement condition was difficult since the pavements containing drainage layers were relatively new. They have not reached a point in their life cycle to statistically determine if they are performing differently than pavements constructed without drainage layers that perform similar functions.

5 Conclusions and Recommendations

The ERDC was tasked by the AFCESA to perform an evaluation of airfield drainage layers to determine their efficiency and if long-term performance justifies the additional cost of installation. This report addresses the evaluation of drainage layers at military airfields including field testing and data analysis. Conclusions from the investigation and recommendations are provided in the following text.

Conclusions

The following conclusions resulted from the evaluation of airfield pavement drainage layers from August to November 2008:

- Design and construction both play important roles in the functionality of airfield pavement drainage layers. Improper oversight of either can lead to a poorly performing system. Several pavement areas observed in this study were not functioning properly as a result of poor design or construction.
- Evidence of routine maintenance of pavement drainage systems was not observed on any of the airfields evaluated in this study. A lack of maintenance could inhibit the flow of water and reduce the functionality of the drainage system.
- Permeability rates through the drainage layers meeting the aggregate gradation specifications were at acceptable values.
- Pavement drainage layers that are daylighted to the edge of the pavement are able to remove water through multiple pathways and are less likely to have flow interrupted by a lack of maintenance.
- Differences in the performance of pavements with and without drainage layers could not be ascertained. Deterioration rates of the pavements evaluated were not fast enough to determine statistical differences in their condition. Pavements of the same age constructed without drainage layers were in similar condition to those constructed with drainage layers.
- The GPR provided a useful tool for determining the location of moisture in the drainage layer beneath asphalt concrete pavement. The depth of penetration of the GPR was too shallow to locate moisture in the drainage layer beneath thick PCC pavements.

- Flow measurements provided sufficient data for quantifying the functionality of the drainage system.
- The climatic region where the pavement is located will impact the amount of water that potentially enters the pavement and can be removed through the use of pavement drainage layers.

Recommendations

The following recommendations are offered based upon the results of the field testing of airfield pavement drainage layers:

- Construction of pavement drainage layers should be closely monitored to ensure that they will be functional after construction. Specifications should be followed for all material properties and design considerations.
- A routine maintenance program should be implemented for pavement drainage systems on airfields. Maintenance should include clearing all soil and vegetation from the flow path to prevent clogging.
- Alternate designs for pavement drainage systems should be considered. Daylighted drainage is one example of a design that may provide acceptable performance.
- Local precipitation values should be used as criteria for determining when pavement drainage layers are needed. Areas that do not receive frequent rainfall should be exempt from pavement drainage layer requirements even if the underlying soil is not expected to have significant vertical flow. A value of 15 in. of annual rainfall is recommended as the minimum value requiring a drainage layer to be used. Values less than 15 in. should be exempt.
- An additional evaluation of pavement performance should be considered in the future when enough deterioration has occurred to determine differences in the performance of pavements constructed with and without drainage layers.

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14. ABSTRACT

During the period August to November 2008, airfield pavement drainage layers at Elmendorf Air Force Base, Alaska, Tinker Air Force Base, Oklahoma, and Fort Bliss, Texas, were observed for the purpose of evaluating their efficiency and determining if long-term performance justifies the additional cost of installation. Evaluation procedures included the artificial introduction of water into the pavement structure and observation of flow. Pavement performance data were also analyzed. Data from each field testing location were used to determine the effectiveness of the drainage layer. These data were used to provide recommendations for future use of pavement drainage layers on airfields.

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