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Foreword

Thousands of feet of prefabricated geocomposite edge drains installed throughout the United States have given satisfactory performance. Use of geocomposites is generally considered to be a sound engineering solution to pavement drainage problems as well as an economical one. However, methods of evaluating these products and procedures for construction installation are needed. This need has prompted much activity related to development of design parameters for use in the selection of the site-appropriate products. The first eight papers in this Record give information on practical problems that would facilitate efforts to improve installation procedures and would provide improved performance. The last paper, by Hurd and Dunchack, discusses the performance of a transverse underdrain system.

Goddard compares the flow capacity of geocomposite edge drains with that of conventional underdrain pipes of various sizes under different soil types. On the basis of the findings, he gives a guide to systems design that includes information on flow capacity, structural design, geotextile selection, installation, fittings, and inspection.

Frobel considers both the perpendicular and eccentric compressive loadings of the geocomposite core. He describes a method and device that allow the placement of a sample such that the compressive stress can be applied at any required angle to the plane of the geocomposite.

Koerner and Hwu present a brief overview of geocomposites, followed by a specific design methodology. They recommend a comparative evaluation program for use in designing geocomposites. The design parameters included in the program are core strength, core flow capacity, geotextile permittivity, apparent opening size, puncture strength, grab strength, and trapezoidal tear strength.

Stuart et al. describe a test under development to evaluate the core flow capacity of geocomposites. The test is performed using a large triaxial cell, which allows the cores of samples to be placed in a vertical position for testing under different hydraulic gradients and lateral stress.

Steffes et al. report on the use of a 2¾-in. video camera to examine underdrain pipes, geocomposite edge drains, and small-diameter culvert pipes for problems such as buckling, clogging, and compression of the core. The in situ observation has improved the design and construction inspection activities.

Baldwin discusses West Virginia's experience with prefabricated edge drains on a rehabilitated section of Interstate 77. The paper includes information on problems encountered and recommendations that are the result of the study.

Highlands et al. describe the results of a comparative evaluation of construction, performance, and cost of geocomposite edge drains and Pennsylvania Department of Transportation's standard base drain system. The study raised several concerns that the authors believe need to be addressed.

Allen and Fleckenstein report on the effects of different levels of compaction on the performance of geocomposites in Kentucky. They conclude that a postinstallation inspection of core to verify the integrity of geocomposites is essential.

Geocomposite Edge Drain System Design

James B. Goddard

Since their inception and introduction in the early 1980s, geocomposites have received wide acceptance as edge drains, particularly on Interstate highway rehabilitation projects. During that time the industry has learned a great deal about the design requirements of the system to ensure acceptable performance. Through a review of laboratory tests, site investigations, and literature, a guide to systems design with emphasis on controlling factors is offered. Structural, hydraulic, and installation criteria are included.

The need for adequate drainage of highway bases has been known for centuries. Drainage methods have included free-draining bases, french drains, pipe and aggregate subdrains, and, most recently, geocomposite edge drains, the development of which was driven by the design of the Interstate highway system with dense, relatively impervious base-course materials and limited underdrain design. The trapping of water between the pavement and base has led to drastically shortened highway life (*1*).

The design of the geocomposite edge drain is intended to improve response time or rate of the introduction of free water by increasing the surface area in contact with the base-course material and by placing the product in intimate contact with the pavement edge and the base-pavement interface. The products currently in use have been proven to perform this function to a lesser or greater extent. Since the introduction of the first geocomposite edge drain in 1982, a great deal has been learned about the design requirements for these products and the special construction problems related to them.

Although vastly different in design and construction, the currently available geocomposite edge drains are all intended to perform the same function. Each product design, however, is focused on a few design parameters, and specifications promoted by each manufacturer emphasize those areas of focus in efforts to eliminate other products from competition. Further, laboratory research has been done focusing on a single property and has been promoted without regard to actual findings in the field and without addressing the interaction of the various component parts of the design. Actual field problems have been disregarded or downplayed as isolated construction-related problems and have not been addressed. In fact, some products have been modified to make them cheaper in a highly competitive marketplace in a manner that completely disregards field problems and, in fact, aggravates the installation problems—for instance, increased post spacing and flexibility or reduced flow capacity.

A clear focus on the product design requirements with emphasis on actual field performance needs and a complete system design are long overdue. Such a focus on system needs should result in the development of a product performance specification permitting adequate competition and ensuring system constructibility and performance.

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SYSTEM DESIGN

Before development of a product performance specification, it is necessary to establish the performance criteria demanded by the application. Specifically, hydraulic flow capacity, hydraulic inlet capacity, structural capability of the composite, constructibility, geotextile selection, system components, packaging, and outlet configuration and design must all be considered and, where appropriate, limits or minimums set. Uniform construction standards must also be established.

Hydraulic design involves a number of different parameters that must be considered as a whole, with each parameter met by the system design. There is good agreement on in-place flow capacity requirements at this time. The minimum in-plane flow capacity for any edge drain design, based on in-plane transmissivity tests conducted in accordance with ASTM D-4716 with a hydraulic gradient of 0.1 and a pressure of 10 psi for 100 hr on a 12-in. long, full-width sample, should be 15 gal/min/ft of width. The 100-hr time requirement under load should detect product weaknesses due to core or fabric creep. On the basis of tests conducted on geocomposite panels 12 in. wide and 20 ft long at 0 percent slope, this value translates into approximately 700 gal/hr at full flow (12 in.) (*2*).

Work by Dempsey (*3*) recommends that system design be such that a continuous flow capacity of 150 gal/hr at 0 percent slope with a water elevation in the geocomposite at or below the base-subbase interface should be required. This design determines the size of the geocomposite more often than the other parameters. It may also permit a variation in geocomposite height, depending on specific product performance characteristics.

Geocomposite inlet capacity is a subject of some debate. Recent work by Koerner and others (see the third paper in this Record) has revealed that the permeability of the base material and the infiltration rate through the pavement seams and cracks are always less than the inlet capacity of the geocomposite for all commercially available products. Table 1 presents the flow available from a range of base and soil conditions both in inlet flow per foot of 12-in. panel and total flow available to a typical 500-ft length of panel (*4,5*).

From Table 1 it can be seen that in-plane flow capacity is the system hydraulic control for soils that have the permeability of coarse sand or better. For other soils, with reasonable outlet spacing the soil permeability will control system flow. In no case does inlet capacity of the geocomposite control, at least with the designs currently available.

TABLE 1 FREE WATER FLOW THROUGH SOIL-GRAVEL MEDIUM

Flow Medium	Flow/Sq.Ft. of Contact Area (GPM/Ft²)	Flow/500 L.F.- 1 Side GPM/Ft of Height
1½" to 1" Gravel	6.8	3,400
1" to ½" Gravel	2.3	1,150
3/8" to #4 Gravel	0.36	180
Coarse Sand	4.5×10^{-2}	22.5
Fine Sand	4.5×10^{-4}	0.225
Silt	4.5×10^{-7}	2.25×10^{-4}
Clay	4.5×10^{-10}	2.25×10^{-7}

Comparing the in-plane flow of these products with pipe systems and gravel drains provides some insight into how these products perform in moving water from one point to another. Table 2 indicates the cross-sectional area of certain soils or gravels necessary to transport quantities of water equal to those carried by a typical edge-drain product (*4*).

Comparing this flow capacity with that of pipes, a typical smooth-interior, 4-in.-diameter pipe will have 4 to 5 times the flow capacity of a 12-in. edge drain. A 6-in.-diameter pipe will have 12 to 15 times the flow capacity of a 12-in. edge drain.

The structural capacity of these materials is probably the most controversial property of these products. Although in situ tests have indicated very low pressures in the plane of the geocomposite, with a maximum pressure of 12 psi being recorded during compaction of backfill and a duration over 8 psi of only 10 sec, claims as high as 93 psi for required design compression normal to the plane have been made. Unfortunately, tests used to justify this level of loading are made with flat steel plates; only the core load capacity is tested in one plane and the effects of such a load on the geotextile are ignored.

Similar tests using neoprene sheet or fine sand between the geocomposite and the plates provide a somewhat more realistic view of the actual installation condition. Even this test is very "kind" to the geotextile in that it does not include the vibration or pulse loading experienced by these products when installed adjacent to the pavement. Even so, a typical geocomposite with posts or cuspations spaced at 1¼-in. centers experienced ⅜- to ½-in. intrusion of the geotextile into the core after 72 hr at a constant pressure of 5 psi when placed between 1-in.-thick layers of neoprene with a firmness at 25 percent compression of 6 psi (41 kPa) (ASTM D1056). This is still less severe than anticipated soil loadings. Actual excavation of installed panels has shown similar intrusion patterns (Figures 1–3).

A parallel plate test using steel plates against the core with loads normal to the plane of the geocomposite is an index test only and does not reflect actual installed loads. Again, excavation of installed geocomposites in highway edge-drain applications has revealed significant geocomposite deformation, obviously from loads exerted at angles other than perpendicular to the core. These forces may occur during installation and initial backfill and compaction or may occur during soil settlement. The necessity of developing a laboratory test to represent the requirement for geocomposite stability has been clearly shown by site investigations in a number of states. The principal question is the appropriate shear angle to be selected for the test. Frobel, in another paper in this Record, has suggested angled loadings from 10 to 50 degrees, with the requirement that the load-carrying capacity be some percentage of the stiffness normal to the plane.

Two methods of angled loading are being considered: (a) applying the angled load directly through fixed angled loading plates or (b) applying the load through angled sliding teflon blocks. Both methods provide an indication of the stability of the core. Questions still remain as to the selection of the relevant angle for the test. One proposal using the fixed plates suggests that the cores should retain 50 percent of their strength under loads normal to the core at a 50-degree angle. Using the sliding blocks with a 10-degree angle, the allowable reduction in stiffness should be limited to 15 percent.

The selection of geotextile for use in the geocomposite has also become an issue. Generally, manufacturers of geocomposites have standardized on a single geotextile for use with their system. This geotextile-core combination, the geocom-

TABLE 2 FLOW CAPACITY

Flow Medium	Area Needed For Equal Discharge (Ft²)
12" Edgedrain (15 GPM)	0.083
1½" to 1" Gravel	2.2
1" to ½" Gravel	6.5
3/8" to #4 Gravel	46.7
Coarse Sand	333.0
Fine Sand	33,300.0
Silt	3.33×10^7
Clay	3.33×10^9

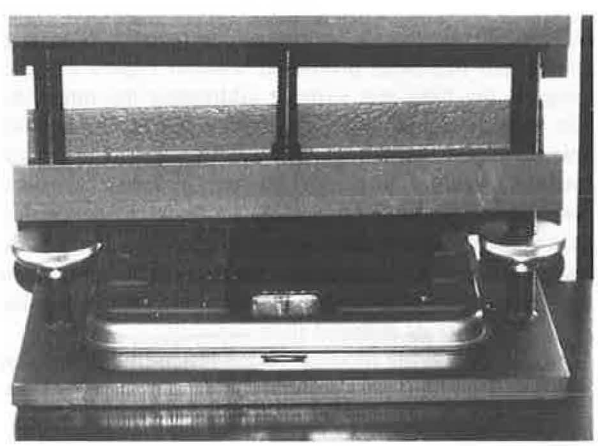

FIGURE 1 Simple compression frame with neoprene sheet over geocomposite face.

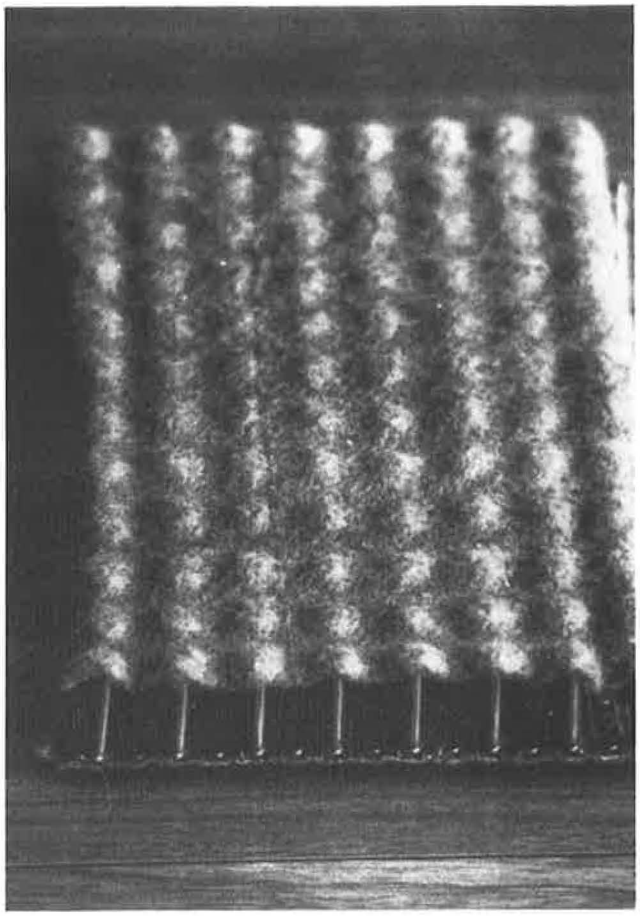

FIGURE 2 Residual geotextile intrusion after loading to 5 psi in the frame shown in Figure 1.

FIGURE 3 Excavated geocomposite showing similar fabric intrusion to that in Figure 2.

posite, has been promoted as a single package, with both core and selected geotextile properties promoted as applicable for all highway edge-drain applications. The initial concern was fabric plugging, and in defense of the selections made by the manufacturers, the author is not aware of a single case in which an edge-drain geotextile used in any geocomposite design became plugged. There have, however, been a number of cases of heavily silted cores—in some cases they were fully closed—with either very fine soils or cementitious fines released in pavement rubblizing (Figure 4). This would indicate that selection of the geotextile should be site specific. Koerner has recommended that a much heavier fabric (i.e., with a smaller apparent opening size) be used as a standard. Such a general change in fabric may, however, simply shift the problem from infiltration of fines into the core to fabric plugging. It should be noted that some reduction in fabric permittivity is acceptable, because the quantity of water available is always substantially less than the ability of the geocomposite to accept it.

Fabric selection criteria beyond A.O.S. and permittivity are largely dependent on core design. In most geocomposite designs, the geotextile serves as the outer boundary—the envelope or filter—and a structural member. In order to be an effective outer boundary, the geotextile must bridge the distance between cuspations or posts with a minimum of intrusion into the core under load. This dictates a high-modulus fabric; the wider the spacing of cuspations or posts, the higher the required modulus. Although no study of this intrusion phenomenon for highway edge-drain geocomposites is available, Koerner studied an 8-oz needle-punched fabric on a geonet and provided some insight into the problem; fabric

FIGURE 4 Geocomposite completely plugged by fines from I-65 in Kentucky.

intrusion into the core or net reduced flow capacity 60 or 70 percent at a soil pressure of 35 psi. This degree of flow restriction on a net with ⅜-in. continuous fabric support spacing should raise some concerns over cores with 1 to 1¼-in. cuspation or post spacing and demonstrates a clear need for compression tests made on the complete panel with some medium around the geocomposite that more closely represents the anticipated soil environment.

Other fabric parameters largely pose survivability issues. Puncture resistance, trapezoidal tear, tensile strength, seam strength, and probably abrasion resistance (due to handling and abrasion by the installation boot) should all be considered. How critical each of these properties is will vary with core design; as a general rule, the larger the spacings between supports for the fabric, the higher the values for each of these items should be.

Manufacturers have argued about assembly of the geocomposite (the attachment of the fabric to the core) for several years. Simply stated, for designs in which the fabric is a structural member, it must be fully attached by gluing or thermal welding to each post or cuspation tip and to the core back. For geocomposite designs in which the fabric acts only as the separator and filter, a tight sleeve around the core is all that is necessary. In both cases, a relatively high-modulus fabric should probably be used.

Complete system design requires a minimum of fittings. All that is necessary is a coupling, a side outlet, an end outlet, and an end cap. The most critical of these is the coupling, which must keep the geocomposite sections connected through the installation process without restricting flow through the system. The past practice of stapling (with box staples) and taping sections together damages the core and reduces flow capacity (Figure 5).

Any coupling method that infringes on the flow channel, blocking or reducing flow or providing sites for collection and buildup of solids, should not be permitted. Any coupling method damaging the core in any way should not be permitted.

Complete system design must include installation practice, particularly geocomposite location, size, and backfill. Generally the geocomposite edge drain is installed in a narrow trench (2 to 5 in. wide) dug directly against the pavement edge at the pavement-shoulder joint. The top of the geocomposite is typically held slightly above the pavement-base interface (1 to 1½ in.). Further, the geocomposite should be sized so that the bottom of the panel is far enough below the lowest point to be drained so that 150 gal/hr can be removed without the water level in the geocomposite being above that level. This may vary with the individual product (Figure 6).

General practice has been to backfill the geocomposite with the material excavated from the trench. This is acceptable as long as that material is compactable, contains no large material that may bridge or wedge between the panel and the trench wall, and is somewhat permeable. There has been good success backfilling with a graded sand, compacted mechanically.

The geocomposite has been placed away from the pavement edge for a number of reasons, particularly because trenching caused voids under the pavement edge. Moving the geocomposite away from the pavement edge is avoided mainly because response time is reduced substantially, and quick response time is the advantage of these systems. The farther away the geocomposite is placed, the more the response time is affected. Further, backfill between the geocomposite and the pavement edge must be highly permeable but cannot permit piping of base-course fines.

RECOMMENDATIONS AND SUMMARY

Geocomposite highway edge drains have been used extensively enough that better material and construction specifications can and should be developed on the basis of the performance experience to date. The following issues need to be covered in any specification.

Hydraulics

Geocomposite flow capacity is the critical hydraulic parameter affecting performance. Specifications should require a mini-

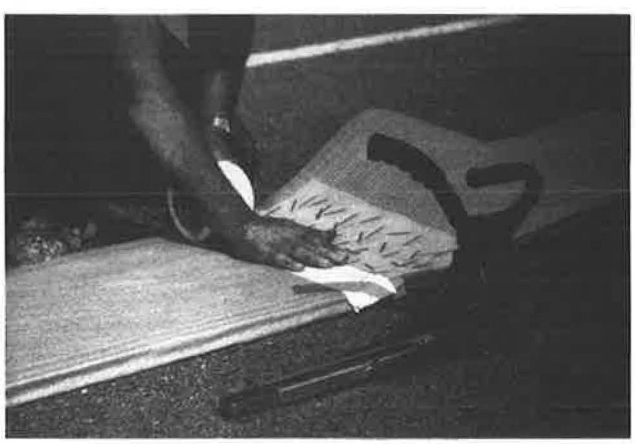

FIGURE 5 Stapled connection in which core has been partially crushed by staple placement.

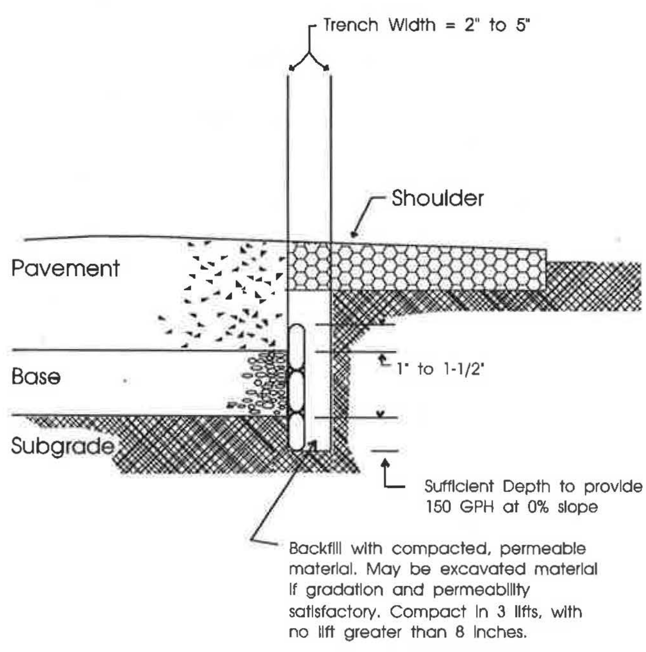

FIGURE 6 Geocomposite edge-drain installation.

mum flow capacity of 15 gal/min/ft of width when tested in accordance with ASTM D4716 at a gradient of 0.1 with a soil pressure of 10 psi for 100 hr on a 12-in.-long, full-width sample.

It is also appropriate that a test be developed to determine the level of flow in a geocomposite at 0 percent slope at 2.5 gal/min (150 gal/hr) with the geocomposite in its normal orientation.

Geocomposite inlet capacity should be greater than the maximum anticipated flow through the surrounding material times some safety factor from 5 to 10 for geotextile plugging over time. Allowance should be made for that part of the geotextile blocked by the core structure (20 to 45 percent depending on core design). For most highway base conditions, a geotextile with a permittivity of 0.2 sec^{-1} will exceed this requirement.

Structural Design

Any structural requirement must be based on the strength of the geocomposite (geotextile and core) and not just on that of the core. Tests must consider the effects of fill around the geocomposite, in which a soft medium is used around the sample during testing. Measurement of both fabric intrusion and core collapse must be made with the worst case governing. Further, the designs must be stable, as shown by a loading test at a shear angle of 50 degrees with a retention of at least 50 percent of the "normal" strength or with a sliding block test at a 10 degree angle with 85 percent of the "normal" strength retained.

A minimum compressive strength value of 3,000 psf (21 psi) for loads normal to the plane and for loads exerted at a 50-degree angle using fixed plates or with sliding blocks at 10 degrees appears appropriate. This exceeds maximum field measured loads by roughly a factor of 2 and equals the worst-case theoretical loading using Boussinesq analysis.

Geotextile

Geotextile selection for geocomposites is both site and core design specific. Individual sites or applications may require specific maximum A.O.S. requirements. For geocomposite designs in which the geotextile acts only as soil filter, Task Force 25, a joint AASHTO–Associated General Contractors–American Road and Transportation Builders' Association committee, considers Class B drainage geotextiles appropriate. For geocomposite designs in which the geotextile functions as a structural component, Class A drainage geotextiles per Task Force 25 should be required. In all cases, the geotextile should be nonwoven polypropylene or polyester.

Where the geotextile is a structural component of the geocomposite, it must be bonded to the core by gluing or heat bonding. If the geotextile is not a structural component, it may be tightly wrapped.

Installation

The geocomposite should be installed directly under the shoulder-pavement joint and in contact with the pavement edge whenever possible. If it must be installed away from this location, careful selection of the backfill material is required.

Couplings and Fittings

Couplings cannot interfere with or reduce flow in any way. Outlets must be designed to carry full panel flow.

Inspection

Installed geocomposites should be inspected for core damage before project acceptance. Borescope inspection at random points seems most practical.

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Eccentric (Angled) Loading of Prefabricated Highway Edge Drains

RONALD K. FROBEL

A new test procedure is described for evaluating the performance of a prefabricated drainage system (PDS) when subjected to loading that is other than normal to the plane of the core. A PDS is an in-plane drain specifically manufactured from polymer materials for subsurface drainage applications, in particular, highway edge drains. A PDS is commonly manufactured from a preformed semirigid polymer core and covered with a geotextile. The major function of the core is to transport water. However, the core must also provide support for the geotextile filter and must resist installation and in-service stress and deformation in order to maintain the design drainage function. One of the factors that can adversely affect the performance of the PDS edge drain is eccentricity of loading or loading occurring at angles other than those normal to the core. Angled loading can adversely affect the compressive failure or collapse of the core, especially a core that is direction dependent. The test apparatus and procedures used in this study are described and results are illustrated for eccentric testing carried out on four popular types of PDS highway edge drains. This type of testing is also suitable for evaluating the eccentric loading that would occur on other types of PDS applications such as sloped walls, vertical or sloped cutoffs, embankments, or landfill side slopes.

One of the newest geosynthetic materials to be applied to civil engineering projects is the geocomposite drain, or prefabricated drainage system (PDS). A PDS is an in-plane drain specially fabricated for subsurface drainage applications such as sloped or vertical walls, embankments, cutoffs, or highway edge drains. A PDS is commonly manufactured from a semirigid formed core (high profile) and typically covered with a geotextile. The major function of the preformed core is to transport water within its plane. However, the core also must provide support for the geotextile and must be structurally designed to resist installation and in-service stress in order to resist crushing or collapse. The geotextile serves two major functions: as a filter between the surrounding soil and the open core and as the outer boundary of the core flow area.

Geocomposite PDSs used for pavement edge drains on highway and airport runway projects have grown from experimental status in 1982 to being widely accepted standard contract items in 1990. These installations have been generally successful, with a few exceptions. Structural problems have occurred where loads have caused large deformations because of core instability. This can be a particular problem on projects in which pavement rubblizing or cracking and seating is done after installation of the edge drain.

The Kentucky Department of Highways has experienced edge-drain structural problems on crack-and-seat pavement rehabilitation projects on the Western Kentucky Parkway, the Mountain Parkway, and Pennyrile Parkway. Borescope investigations found that post or cuspation displacement on each of these projects caused collapse or partial collapse of the geocomposite.

In Michigan, New York, and Kentucky, problems have been experienced with localized crushing of cuspated panels on a number of projects. The cuspations could have been deformed by shipping and installation in Michigan and Kentucky. The New York State Thruway Authority has experienced panel collapse against subgrade voids during backfill installation procedures.

In all of the foregoing examples, it is possible that the failures were the result of installation stress or postinstallation stress induced at angles other than those normal to the plane of the core. The potential effect of confining stress and installation stress and the resultant reduction in core cross-sectional area (and therefore flow performance) are critical to drain design and performance. Obviously, an important characteristic for the PDS is the ability of the core to resist imposed stress without deforming. To date, stress testing on core structures used as highway edge drains has been limited to normal compressive loading, and the manufacturers report crush resistance of their core using various methods and rigid plate sizes. It has been found that small plates (±4.25 in. square) are not recommended in testing of semirigid core structures (*1*). Results of compression tests are useful primarily as an index test for the preliminary comparison or screening of products; however, because of the variability of core structures, larger specimens (±12 in. square) have been recommended (*1*).

Normal compressive strength testing is only one relative measure of the short-term ability to withstand stresses on the drain due to adjacent soil pressures, installation method, backfilling method, or loads from vehicular traffic immediately above and adjacent to the drainage trench.

According to Kraemer and Smith (*1*), factors that can adversely affect the results of compression tests and therefore field performance are small sample size, eccentricity of loading, and the presence of secondary yield phenomena due to the geometry of the core. Eccentricity of loading or loading occurring at angles other than those normal to the core can affect the compressive failure or collapse of a core, especially a core that is highly direction dependent (*1*).

In an effort to study the effects of loading eccentricity on core type in the laboratory, a special load frame device was designed and used to evaluate core structures when subjected to loading other than normal compressive loading. Results of this testing are the subject of this paper.

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ECCENTRIC LOAD FRAME

A special aluminum load frame was designed and built to accommodate testing of 12-in.-wide geocomposite highway edge drains in a standard compression testing machine. The load frame was also designed to be locked at a desired test angle. Figure 1 is a conceptual drawing of the frame and upper loading platen. Both upper loading platen and lower adjustable load frame have removable surface plates so that the surface friction material can be changed when desired. The rotating base table can be locked into position at increments of 10 degrees from 90 (normal compression test loading) to 10 degrees. Figure 2 shows the load frame positioned in a compression test machine and Figure 3, the test device with specimen in place. The upper platen dimensions are 12.5 in. by 13.0 in., allowing the testing of a full width of nominal 12-in.-wide edge drain by a length of 12.5 in. Specimens were cut a minimum of 15 in. long in an effort to avoid possible edge effects.

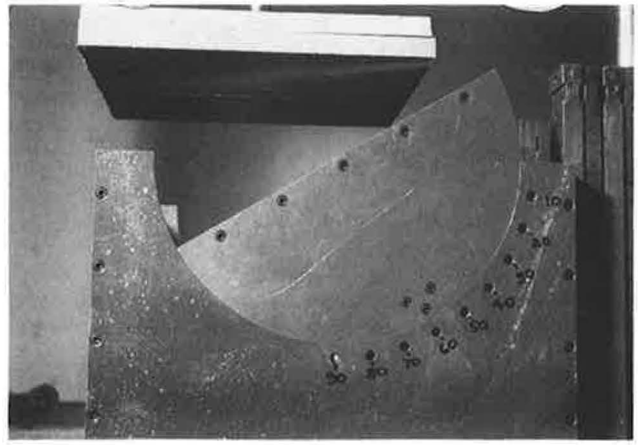

FIGURE 2 Load frame.

TEST METHOD

The test method was designed to evaluate the effect of eccentric (angled) loading other than loading normal to the plane of the drainage core. After numerous preliminary test runs on different types of PDS cores and angles, and at various rates of loading, it was decided to run all tests, including normal compressive loading, at 0.5 in./min as specified in ASTM D2412-87. This speed enabled testing to be carried out efficiently at all test angles, and it more closely approximates "instantaneous" loading such as that found during or immediately after installation. This testing did not address the potential problems associated with creep of polymer cores

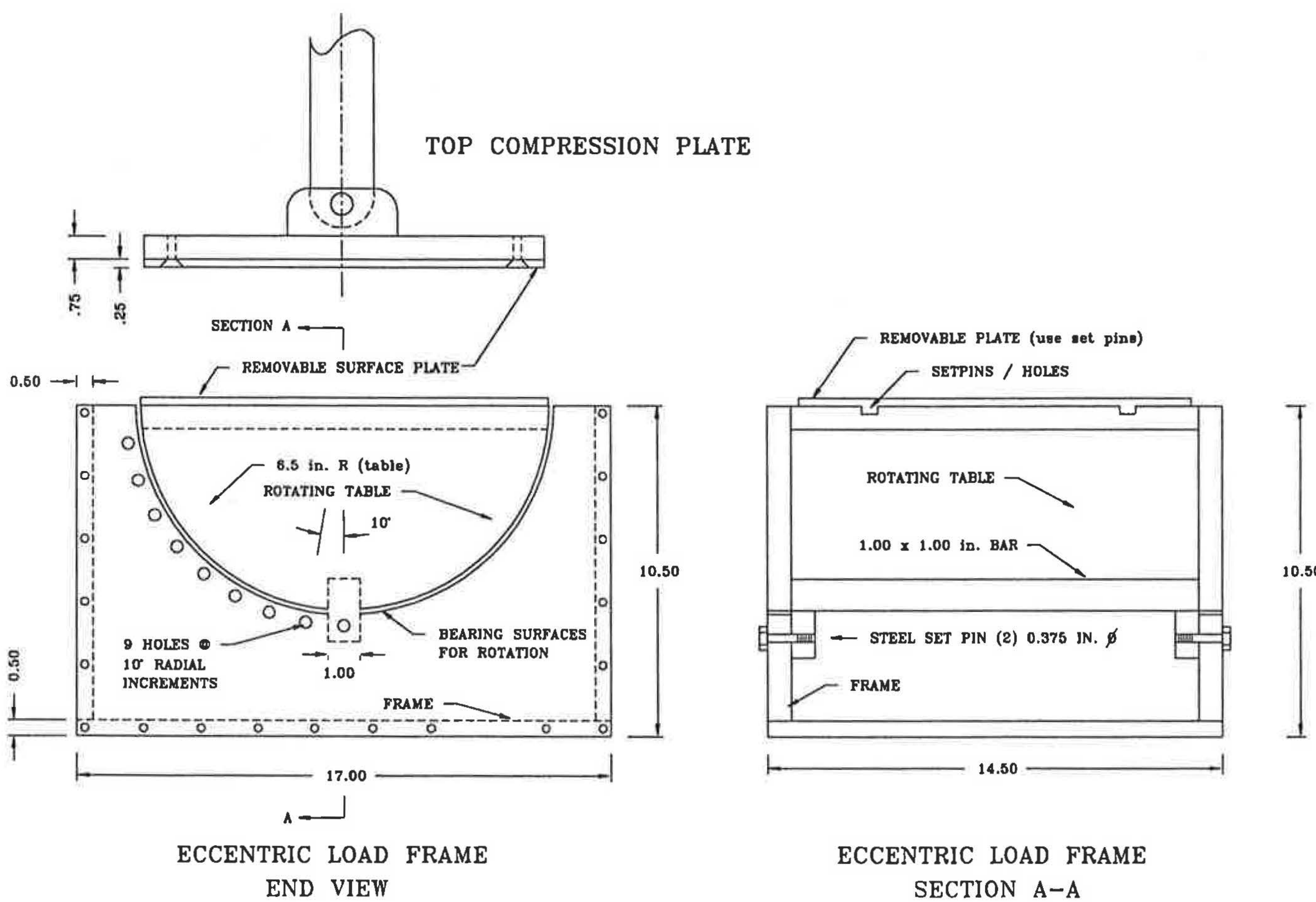

FIGURE 1 Design drawing of load frame device.

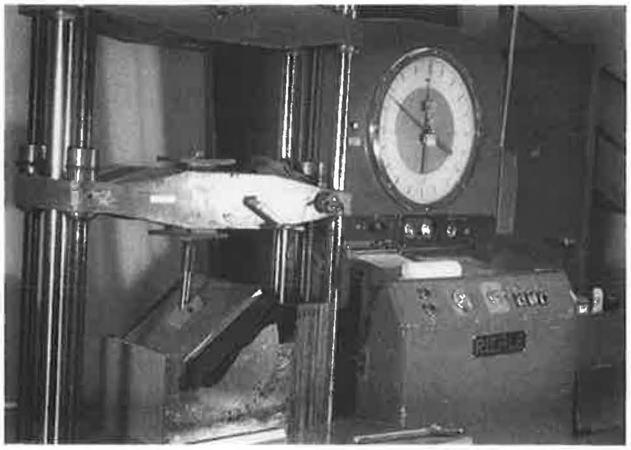

FIGURE 3 Load frame with test specimen in place and positioned in compression test machine.

under normal or eccentric loading. Creep of cores in normal compressive loading has been reported by Smith and Kraemer (*2*). The following is a summary of the test methodology used.

Test Equipment

Test Machine

Any suitable compression test machine capable of operating at a constant rate of traverse and capable of stress-strain measurement by autographic recorder can be used. It is desirable that the load cell be located in the base of the test machine as opposed to on the moving crosshead.

Test Base Frame

The frame is a specially designed rotational frame that allows the specimen to be rotated on a platen between 10 and 90 degrees to the vertical and that can be locked in place at minimum 10-degree increments (see Figure 1).

Upper Loading Platen

The upper loading platen should be attached to the moving crosshead and allowed 2 degrees of freedom in movement. Platens should be covered with a removable high-surface-friction material such as a cellular neoprene or 40-grit commercial-grade abrasive paper.

Load Indicator

The load indicator should have a precision of ±0.1 percent.

Deformation Indicator

The deformation indicator should have a precision of ±0.1 percent.

Test Specimens

Test specimens should be the manufactured width (±12 in.) and 15 in. long. Width dimension should be positioned on the lower platen in the direction of eccentric loading and placed against the stop. The specimen should be allowed to extend beyond the sides of the loading frame to prevent edge effects.

Test Condition

The test should be conducted in standard laboratory atmosphere of 23° ± 2°C (73.4° ± 3.6°F) and 50 to 65 percent relative humidity.

Test Procedure

Platen Alignment

The bottom platen should be set to the desired angle, the upper platen lowered to within 0.5 in. of the bottom, and the test frame adjusted to accommodate upper platen movement with a fully loaded specimen.

Crosshead Motion

The load should be applied so that it is distributed uniformly over the entire loading surface of the specimen. Rate of crosshead movement should be 0.5 in./min (±0.01 in./min).

Crosshead movement should be continued until a yield point or collapse is reached or until the specimen has been compressed to approximately 25 percent of its original thickness. Once a yield point has been reached, movement of the crosshead should be continued another 3 percent.

Calculation

A calibrated *X-Y* plotter or autographic recorder should be used to accurately record load-deflection curves and the estimated 10 and 20 percent and yield point deflection loads.

Report

The report should contain specimen identification, including the thickness and manufactured width, weight, type of geotextile; the angle of the lower platen; the number of specimens tested; the load and deformation values at 10 and 20 percent, yield, and failure; and observations of deflection mode and failure modes.

Compressive Properties

Normal Loading

All specimens should be tested for standard compressive strength characteristics using ASTM D 2412 with the following changes:

Specimen size: width (±12 in.) by 15 in.;
Upper platen size: 12.5 in. by specimen width; and
Number of specimens: minimum of five.

Eccentric (Angled) Loading

All specimens should be tested at the desired inclination, with a minimum of five specimens for each angle.

PDS HIGHWAY EDGE-DRAIN SAMPLES USED

Four types of the most common and geometrically different cores used as highway edge drains were chosen for this testing:

A: double cuspated (Hitek-type);
B: single cuspated, truncated conical cuspates with perforated base;
C: oblong (elongated) corrugated pipe section with slotted perforations; and
D: high-profile columns with perforated base.

All the above products are manufactured from polyethylene base resin. Core Types A and D polyethylene resins are classed as low-density polyethylene (LDPE), whereas Type B and C resins are classed as high-density polyethylene (HDPE). The core profiles used in this testing and their approximate dimensions and weights are shown in Figure 4.

All cores were wrapped with a nonwoven, needlepunched geotextile. Two core types (A and D) were wrapped and adhesively or thermally bonded to all of the cuspates or columns, as well as to the base (in the case of Core D). Bonding of the geotextile will impart greater stiffness to the core structure and give added stability under load, especially loads occurring other than normal to the plane of the drain core. For the cuspated or column-type cores (A, B, and D), the geotextile must act as a part of the structural composite or outer boundary and also as a soil filter. For Core C, the geotextile acts only as a soil filter. The geotextile selected for use with Cores B and C could be varied for given soil conditions without affecting the structural performance. The geotextiles used on the cores tested were as follows:

Core Type	*Geotextile*
A	Nonwoven, needlepunched staple fiber polypropylene, heat set on one side, adhesively bonded to all cuspates; weight, 4 oz/yd²
B	Nonwoven, needlepunched continuous-filament polyester; weight, 4.1 oz/yd²
C	Nonwoven, needlepunched staple fiber polypropylene, heat set on one side; weight, 3.5 oz/yd²
D	Nonwoven, needlepunched staple fiber polypropylene, heat set on one side, thermally bonded to base and all columns; weight, 4.5 oz/yd²

SUMMARY OF TEST RESULTS AND OBSERVATIONS

Throughout the following discussion, test samples will be referred to by structure or core type letter as shown in Figure 4.

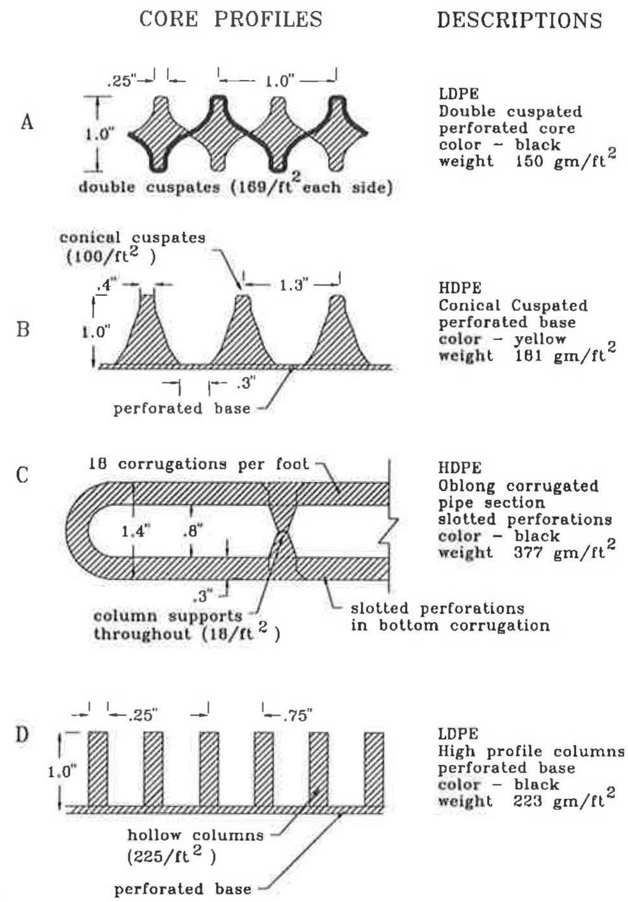

FIGURE 4 Drain core structural profiles.

In general, all the core structures were testable at 90, 70, and 50 degrees. However, the stiffer, heavier-weight Cores B and C were difficult to test at higher angles and resulted in surface slippage and no significant deformations beyond 10 percent. Cores A and D were more prone to deformation and could be tested at all angles. At 10 degrees, Core D deformed easily but could not be accurately measured as to load versus deflection.

Normal compressive testing (90 degrees) was carried out on all core types as a basis for comparison with eccentric load angles. Table 1 shows the normal loads when tested at 0.5 in./min. Figures 5 through 8 are load deflection curves for Cores A through D tested at various angles. As a general method of comparison, loading at approximately 20 percent deflection will be examined in the following paragraphs.

Core A (Figure 5) exhibited only a 17.6 percent drop in load at 70 degrees as compared with normal loading but showed

TABLE 1 AVERAGE NORMAL LOADING OF CORE TYPES

	Deflection (%)		
Core Type	10	20	Failure
A	2,157	5,650	8,832
B	1,498	5,540	11,324
C	2,563	6,307	7,305
D	3,600	15,100	16,000

NOTE: Normal loads are given in pound-force per square foot.

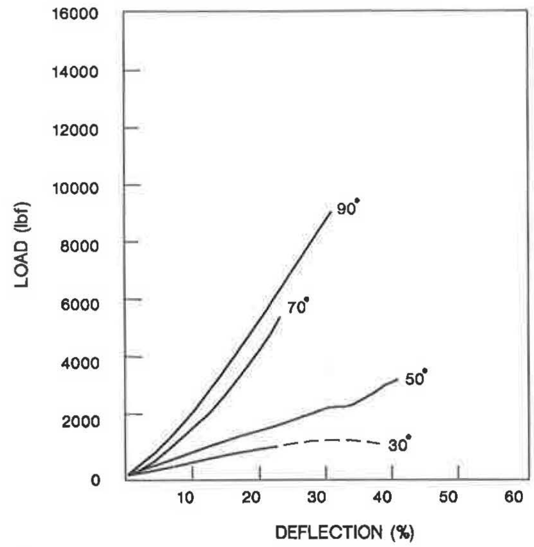

FIGURE 5 Load deflection curves: Core A.

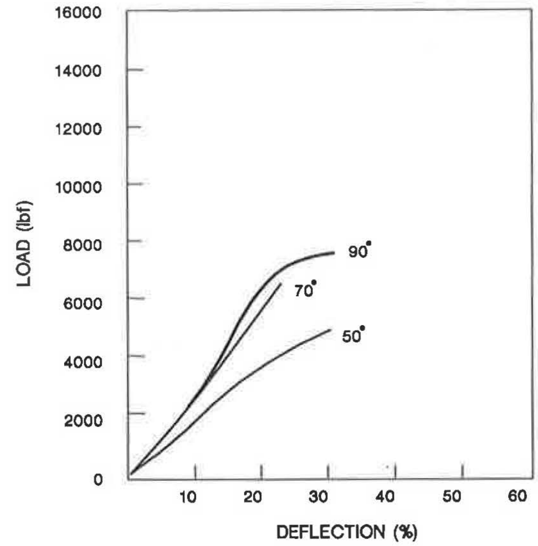

FIGURE 7 Load deflection curves: Core C.

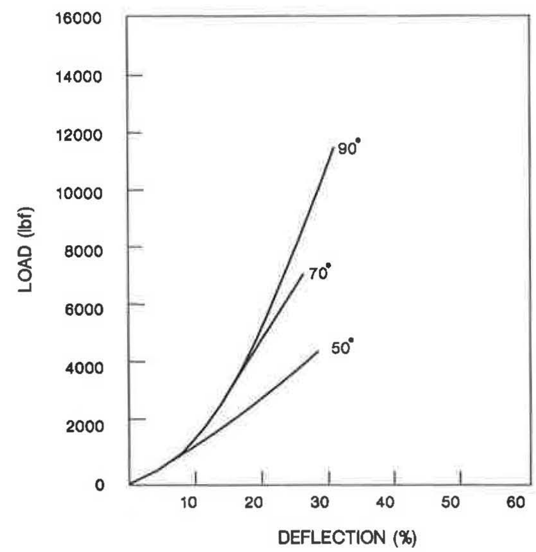

FIGURE 6 Load deflection curves: Core B.

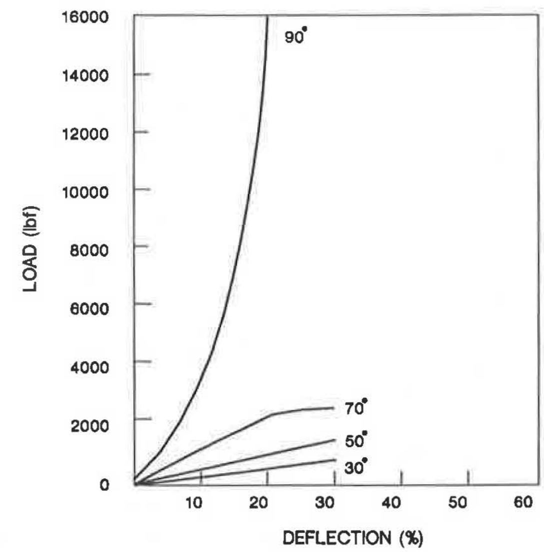

FIGURE 8 Load deflection curves: Core D.

a 71.5 percent loss at 50 degrees and an 83 percent loss at 30 degrees. At the higher angles of 20 and 10 degrees, it was difficult to obtain accurate readings because of the "rolling over" of the core cuspates. Also, the adhesive bonds between geotextile and core were observed to break under stress. Core failure was predominantly due to buckling of the extended part of the cuspates, as shown in Figure 9. The relatively inconsistent quality, low mass, and polymer type were contributing factors in the low failure loads exhibited. Total failure of the core occurred at varying deflections between 22 and 40 percent.

Core B exhibited only a 6.5 percent drop in load at 70 degrees as compared with normal loading and a 49 percent drop at 50 degrees. Because of the relatively stiff structure of the cuspated cones, testing at 30 and 10 degrees resulted in surface slippage and rebounding of the cuspates to their original position. The upper edge of the core tended to roll as the geotextile pulled at high-angle loading, as shown in Figure 10. Failure of the cuspates occurred at the weakest cross section approximately 0.3 in. from the top, as shown in Figure 11. Failure was by collapse or folding of the cuspates in the direction of load. Because of the stiff properties of the core, there was no change in loading at 10 percent deflection for 70 degrees and only an 11.5 percent drop for a 50-degree angle. However, the amount of loading to fail the core at 50 degrees dropped by 62 percent from that at 90 degrees (normal). Ultimate core failure occurs at between 28 and 30 percent total deflection.

Core C was unique in structure in that it is essentially an oblong corrugated pipe section with the corrugations running in the direction of angled loading. The core structure derives its stiffness from the corrugations and interior columns. This product exhibited a relative loss in loading of 12.6 percent at 70 degrees as compared with normal loading and 43.5 percent

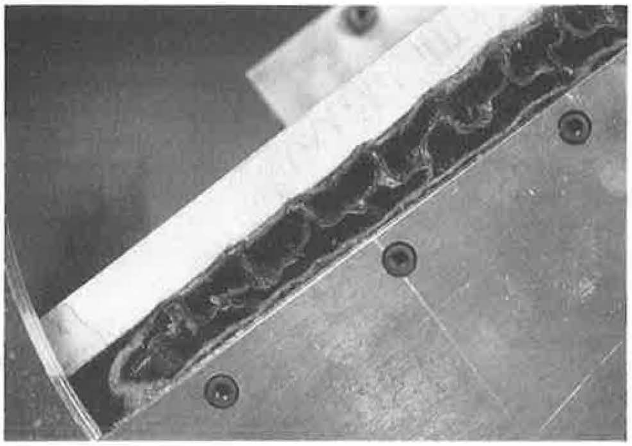

FIGURE 9 Core A: failure mode.

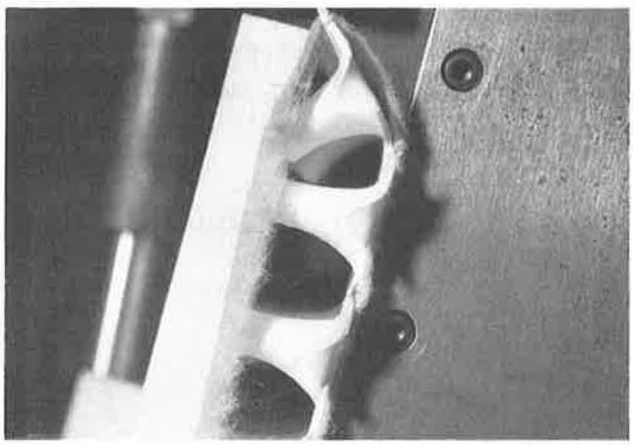

FIGURE 10 Core B: upper edge rollup.

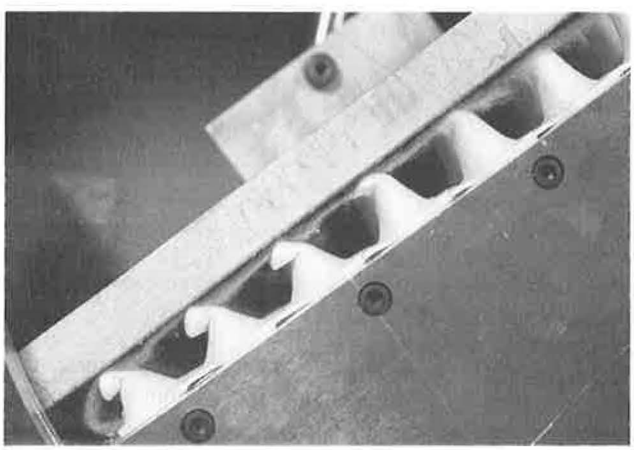

FIGURE 11 Core B: failure mode.

at 50 degrees. As with Core B, accurate load deflection readings were not possible at high angles because of surface slippage and stiff core properties. Failure at 50 degrees required less loading (35 percent drop in load) than at 90 degrees. Ultimate failure always occurred at approximately 25 to 30 percent deflection. Figures 12 and 13 show the core before loading and deformation at 20 percent deflection.

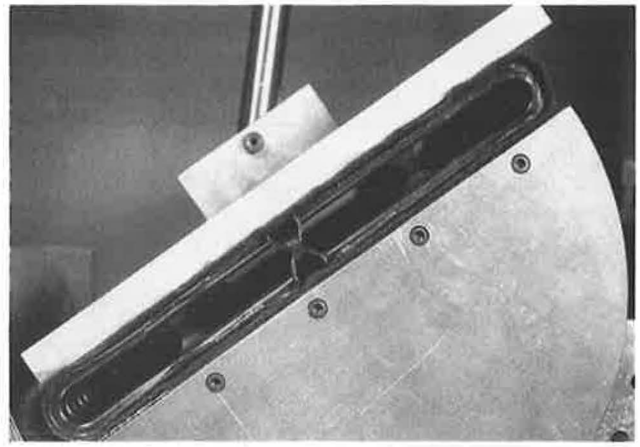

FIGURE 12 Core C: before testing.

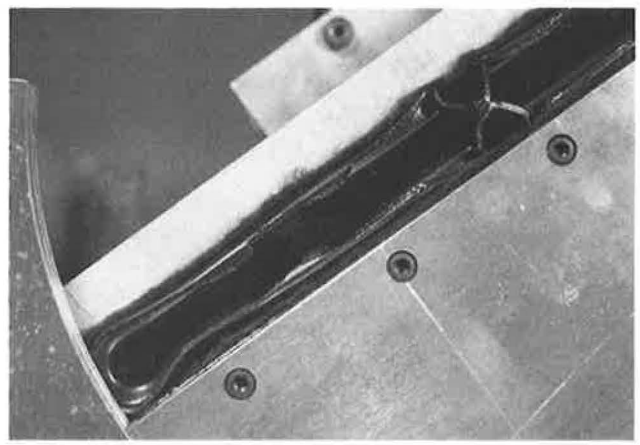

FIGURE 13 Core C: deformation at 20 percent deflection.

Core D, although exhibiting very high normal loading (16,000 lbf/ft^2), showed a distinct disadvantage when subjected to angled loading in that the high-profile hollow tubular columns tended to bend and collapse, sometimes instantaneously. At 20 percent deflection (also the failure deflection), Core D showed a significant change in loading values. At 70 degrees, it exhibited an 83 percent drop in load as compared with normal, a 93 percent drop at 50 degrees, and a 96 percent drop at 30 degrees. Most significant, however, even at a low angle of 50 degrees, the ultimate failure load or crush strength dropped from 16,000 to 2,250 lbf, and 85 percent drop in loading. Failure occurred by column foldover, with ultimate base fracture at the connection of column to base (Figures 14–16 show the column collapse mechanism).

For comparison purposes, Figures 17–20 show all products tested at a given angle of inclination. Again, with the load at 20 percent deflection, one can see from Figure 17 that there is a significant difference among Cores A, B, C, and D. As soon as the angle of loading is changed to only 70 degrees (Figure 18), Cores A, B, and C vary between 9 and 18 percent of each other, whereas Core D drops off significantly, exhibiting less than 50 percent of the loading that the other core types will sustain. At an angle of 50 degrees (Figure 19) there is an even greater difference in load deflection curves; Core C exhibits the best overall performance and the highest com-

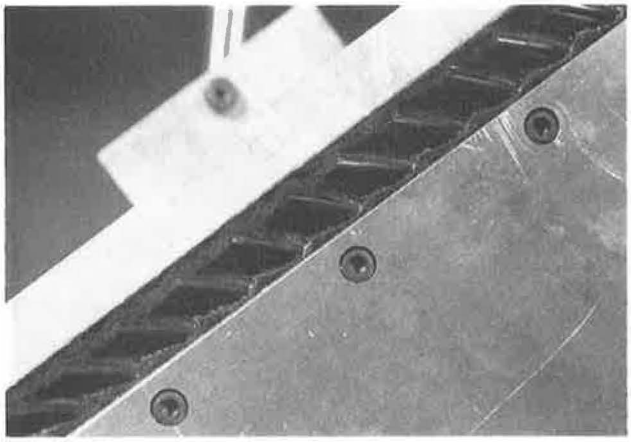

FIGURE 14 Core D: failure mode (column bending).

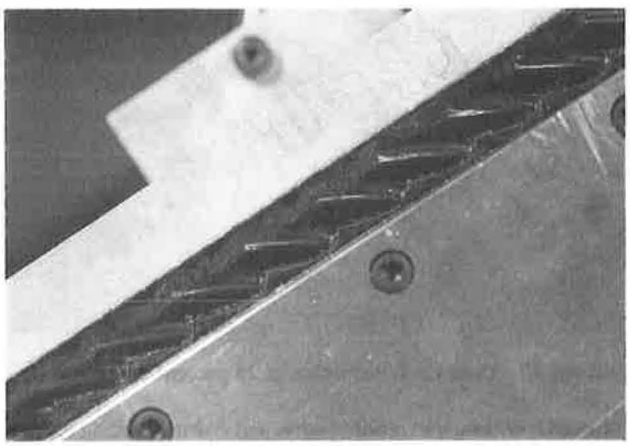

FIGURE 15 Core D: column collapse.

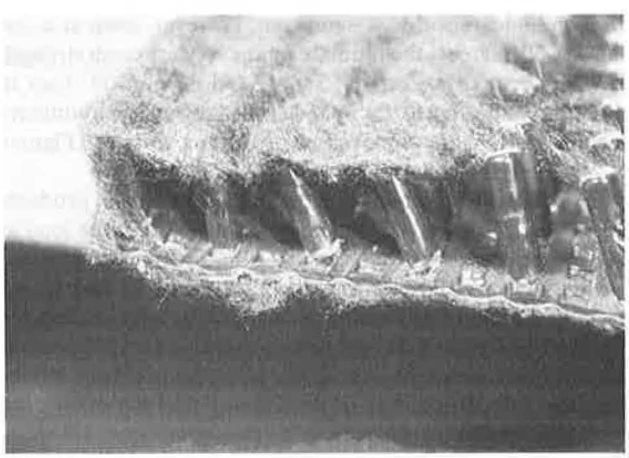

FIGURE 16 Core D: column base fracture.

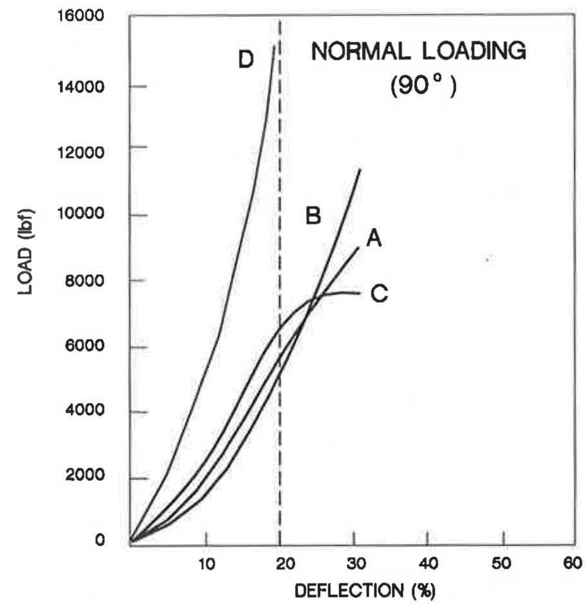

FIGURE 17 Load deflection curves: normal loading.

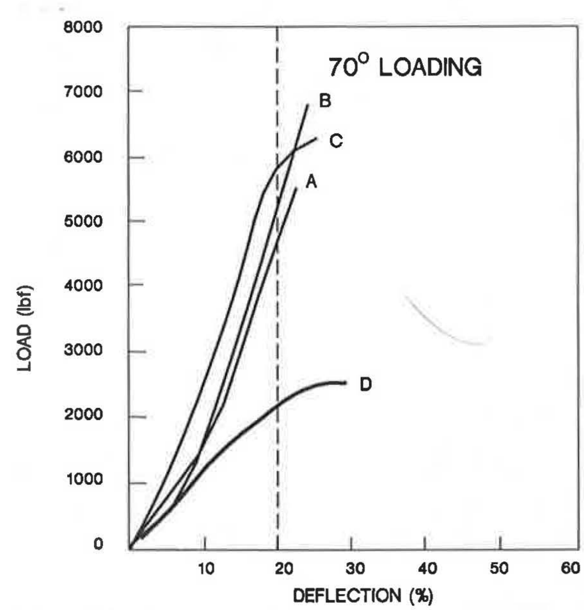

FIGURE 18 Load deflection curves: 70-degree loading.

pressive modulus. Core D exhibits the greatest reduction in initial modulus, loading at 20 percent deflection, and ultimate failure load.

SUMMARY AND CONCLUSIONS

A special laboratory method has been presented designed to evaluate core stability for prefabricated core structures. Although this testing does not represent actual in situ conditions, it does illustrate the significant difference in load deflection properties of PDS cores when subjected to eccentric (angled) loading. Loading of this type can and does occur during in-

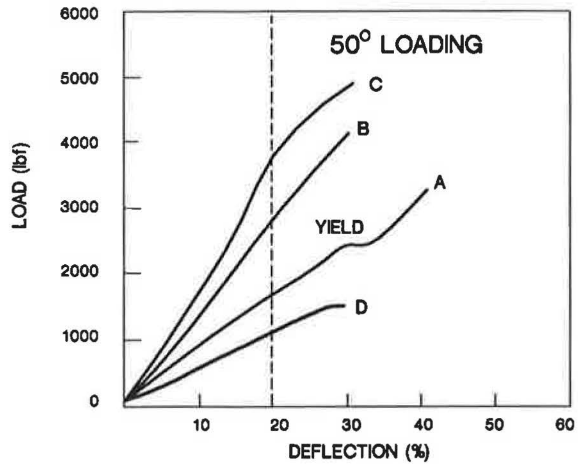

FIGURE 19 Load deflection curves: 50-degree loading.

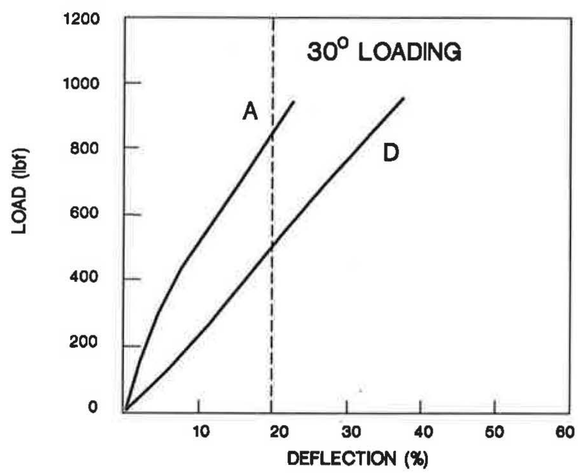

FIGURE 20 Load deflection curves: 30-degree loading.

stallation and compaction and after installation and may account for observed field failures (collapse) of certain core types. This type of testing is relevant as an index test for PDS cores used in sloped or vertical walls, cutoffs, embankments, or highway edge drains.

For this study, four commercially available highway edge-drain products were tested at selected angles of loading. The four products were chosen to represent the extremes in core geometry found in today's PDS highway edge drains. Two of the core structures (Cores B and C) were found to be relatively stable under angled loading, whereas Cores A and D were prone to collapse.

It is obvious that this type of laboratory testing should be examined further as a method to determine PDS core stability when subjected to loads other than those normal to the core. Both normal compressive load tests and testing at a predetermined angle or angles should be accomplished on all types of high-profile prefabricated drain cores. Curves of normal compressive stress versus deformation are virtually useless as a design tool in determination of factors of safety if a product's compressive stress drops by more than 50 percent upon application of load at even a small eccentric angle.

ACKNOWLEDGMENT

The primary sponsor of this study was Advanced Drainage Systems, Inc., Columbus, Ohio.

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Prefabricated Highway Edge Drains

ROBERT M. KOERNER AND BAO-LIN HWU

Prefabricated highway edge drains, made from a polymer drainage core wrapped in a geotextile filter, are being used regularly by transportation engineers for both new and retrofitted roadway systems. Although these drains are acknowledged to be low in cost and rapidly placed, their performance is still being evaluated by long-term monitoring. One reason for this challenge is that their design is essentially empirical. What is intended to be a rational design procedure is presented that can be used in a variety of geographic locations and for a variety of edge-drain products.

The need for proper drainage of paved roadway systems is being rapidly rediscovered by many transportation engineers. Cedergren has long championed this concept and continues to remind engineers of its importance (*1*). Once the pavement has been drained, however, its flow must be intercepted at the edge of the pavement, gravitationally transported parallel to the pavement, and then discharged at intervals of 100 to 500 ft into drainage swales or interceptor pipelines. The edge drains have traditionally been perforated pipes in a gravel envelope, with a sand layer providing a filter transition to the adjacent soil. In Pennsylvania this type of edge drain is currently bid at approximately $6 per linear foot.

In contrast to the edge drains made of perforated pipe and natural soil, prefabricated edge drains are made in the factory and consist of a polymer drainage core wrapped in a geotextile filter. Not only are the basic materials less expensive, but their installation is automated to the point where production rates are hundreds of feet per hour. This is reflected in their current bid prices of less than $2.50 per linear foot. Besides this obvious cost benefit, this edge drain has other desirable features: less excavated soil to be removed, less weight on the subgrade, no need for quarried materials, and potential use of reclaimed or recycled plastic materials, or both.

OVERVIEW

Since the introduction of prefabricated edge drains by the Monsanto Company in the early 1980s, a wide range of products has appeared and is being marketed to the user-owner community. Table 1 is a compilation of seven of these products produced by six different companies. Although the core polymers are currently made from polyethylene, the variations thereafter are considerable. The shape of the built-up cores is seen to be very different, and their thicknesses vary from 0.80 to 1.60 in. These various shapes and thicknesses result in very different compressive strengths and planar flow rates. The only common feature of the geotextile filter is that all appear to be made of nonwoven fabrics. Thereafter, the geotextiles vary as to polymer type (polypropylene or polyester), processing (needlepunched or melt-bonded), and post treatment (some are burnished).

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DESIGN CONSIDERATIONS

The various considerations in the design of a prefabricated edge drain focus separately on the drainage core and on the geotextile filter. From the schematic diagram in Figure 1 it can be seen that (a) the core must be capable of sustaining a certain amount of stress, and (b) it must convey a required flow rate. The geotextile must be capable of (c) passing this flow, (d) retaining the adjacent soil, and (e) sustaining the normal stress between core protrusion locations. These five aspects of design will be addressed sequentially.

Core Compressive Strength

A review of the technical literature (*2*, pp. 73–82) found that the maximum vertical stress due to a vibratory base plate compactor is 12 lbf/in². With a K_0 of 0.43, a horizontal pressure on the edge drain of 743 lbf/ft² is obtained. This pressure value, however, is far less than that in the situation shown in Figure 2. If a truck parks on the shoulder directly over the edge drain (clearly a worst-case situation), a Boussinesq analysis can be performed (*3,4*) that shows the horizontal stress to be 3,020 lbf/ft² (see Figure 2). Note that this value represents a factor of safety equal to 1. Some very compelling reasons for increasing this value are the following:

- Overweight vehicles,
- Impact loads (e.g., bumpy shoulders),
- Long-term creep loads (e.g., overnight parking or truck breakdowns),
- Stresses applied at various angles (see paper by Frobel elsewhere in this Record),
- Variation in edge-drain product strength,
- Effect of moisture on the product's performance,
- Effect of polymer aging on the product's performance, and
- Differences in product strength evaluation from the real-life situation by particular test methods.

The compressive strength test for product evaluation should be on a section of edge drain of full width (usually 18 or 12 in.) by approximately 6 in. long cut so that a reproducible pattern of protrusions exists (*5*). Using this type of test procedure, the cumulative factor of safety for the uncertainties mentioned above should be at least 3. Thus the necessary

TABLE 1 PROPERTIES OF COMMERCIALLY AVAILABLE HIGHWAY EDGE DRAINS

No.	Company	Trademark	Core Characteristics			Geotextile Polymer & Type
			Material	Shape	Thickness (in.)	
1	A.C. F. Inc.	Drain-It	PE	double cuspated	1.00	PP nonwoven needle-punched
2	Advanced Drainage Systems	AdvanEDGE	HDPE	corrugated with columns	1.60	PP nonwoven melt bonded
3	American Wick Drain Co.	Akwadrain 125	HDPE	double cuspated	1.25	PP nonwoven needle-punched
4	Contech Construction Products Inc.	Stripdrain 100	HDPE	tapered column	1.00	PET nonwoven needle-punched
5	Monsanto	Hydraway 2000	PE	straight column	1.00	PP nonwoven needle-punched
6	Pro Drain Systems Inc.	PDS 20	HDPE	double cuspated	0.80	PP nonwoven needle-punched
		PDS 30	HDPE	tapered column	1.20	PP nonwoven needle-punched

From "Product Directory", Geotechnical Fabrics Report, Dec., 1989.

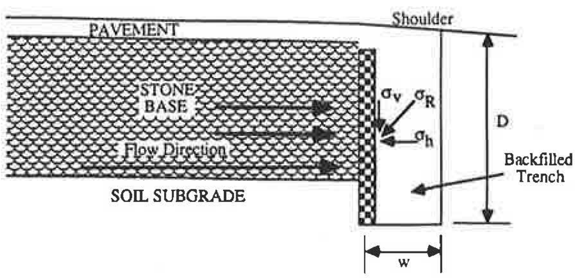

FIGURE 1 Generalized performance concept of a prefabricated highway edge drain.

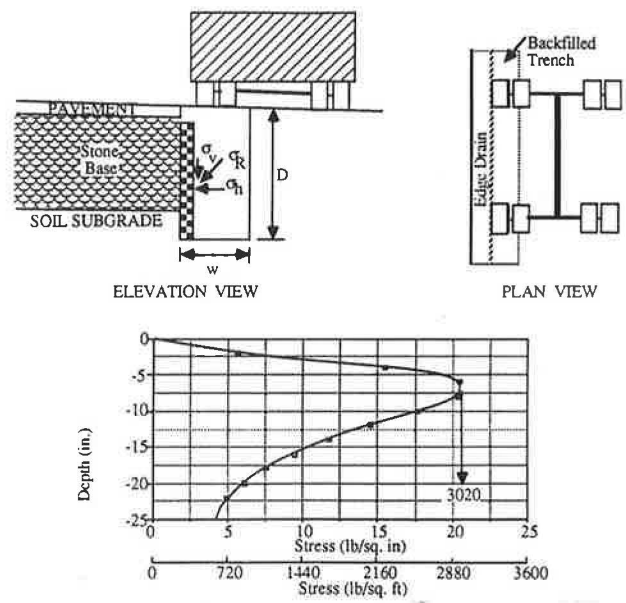

FIGURE 2 Worst-case design loading with resulting Boussinesq analysis for horizontal pressure on edge drain.

ultimate test strength from normally applied loads will be as follows:

$$3.0 = \sigma_{\text{test}}/3{,}215$$

$$\sigma_{\text{test}} = 9{,}600 \text{ lbf/ft}^2 \text{ (67 lbf/in.}^2\text{)}$$

Core Required Flow Rate

To obtain the value of required flow rate for the transport of the water entering into the core, the FHWA (*6*) 1-hr–1-year frequency precipitation rate is used as the design precipitation rate. The design infiltration rate through the pavement used in the guidelines is from one-third to two-thirds of the design precipitation rate, because normally it is expected that less than 100 percent of the water falling on a pavement will enter through the pavement surface. In this regard, the guidelines suggest that the design precipitation rate (1-hr–1-year frequency rate) be multiplied by a coefficient between 0.50 and 0.67 for portland cement concrete pavements and 0.33 to 0.50 for asphalt concrete pavements [see work by Cedergren (*7*, pp. 104–142) for addition details].

The 1-hr–1-year frequency precipitation rates give a maximum value of 1.2 in./hr at a location in the southeast corner of Pennsylvania near Philadelphia. Using this information, one can now calculate the required flow rate to the edge drain. Assumed in the calculation is a two-lane (each 12 ft wide) asphalt concrete pavement that has an infiltration coefficient of 0.42 with an edge drain on one side; W = 24 ft. Also taken in the design are drainage outlets at 300-ft centers with a release factor of water in the stone base course of ⅓.

The required formula is

$$q_{\text{design}} = c i_R (WL) f_R$$

where

q_{design} = design flow rate per foot of pavement length, that is, between drainage outlets;
c = pavement infiltration coefficient;
i_R = rainfall intensity;
W = width of pavement;
L = length between drainage outlets; and
f_R = release factor for water in the stone base.

Substituting the foregoing values into this formula gives

$$q_{\text{design}} = (0.42)(1.2)(W \times L)(1/3)(1/12)(1/60)(7.48)$$
$$= 0.00175WL$$

$q_{design} = 0.00175WL$ gal/min/ft width/ft outlet spacing

and for $W = 24$ ft and $L = 300$ ft, $q_{design} = 12.6$ gal/min for the designed outlet spacing.

In a similar manner, a design table can be generated in which the major variables are the pavement drainage width (W), the spacing between drainage outlets (L), and the release factor (f_R). Table 2 has been generated in accordance with these variables. Here it is seen that as pavement width, outlet spacing, and release factors increase, the design flow rate increases proportionately.

The foregoing analysis is essential in order to properly size edge-drain cores to handle their design flow rates. In the procedure, the major uncertainty is the release factor for the retention time that the water is in the stone base. It is believed that this value varies as follows:

- Well-graded base courses, in service ≥ 5 years: $f_R = 1/4$,
- Well-graded base courses, in service < 5 years: $f_R = 1/3$,
- Open-graded base courses, in service ≥ 5 years: $f_R = 1/3$,
- Open-graded base courses, in service < 5 years: $f_R = 1/2$.

This range of release values was used in preparing Table 2.

As with all design-by-function concepts, a factor-of-safety process is used to arrive at a required flow rate for the edge drain from the factor of safety = 1 design value.

$$FS = q_{test}/q_{design}$$

Using a factor of safety of 1.2 against uncertainties in the design, product variability, intrusion of the geotextile filter, and variations of the test with respect to the actual situation results in a required flow rate for the example problem of the following:

TABLE 2 DESIGN TABLE OF REQUIRED FLOW RATES FOR HIGHWAY EDGE DRAINS UNDER CONDITIONS OF VARYING PAYMENT WIDTH, OUTLET SPACING AND RELEASE FACTORS

Pavement Drainage Width W (ft.)	Outlet Drainage Spacing L (ft.)	Design Flow Rate (gal/min-ft.) for Release Factors of 0.25	0.33	0.50
6	200	1.6	2.1	3.1
6	400	3.1	4.1	6.3
6	600	4.7	6.2	9.4
6	800	6.3	8.3	12.6
6	1000	7.9	10.4	15.7
12	200	3.1	4.1	6.3
12	400	6.3	8.3	12.6
12	600	9.4	12.4	18.8
12	800	12.6	16.6	25.1
12	1000	15.7	20.7	31.4
18	200	4.7	6.2	9.4
18	400	9.4	12.4	18.8
18	600	14.1	18.7	28.3
18	800	18.8	24.9	37.7
18	1000	23.6	31.1	47.1
24	200	6.3	8.3	12.6
24	400	12.6	16.6	25.1
24	600	18.8	24.9	37.7
24	800	25.1	33.2	50.3
24	1000	31.4	41.5	62.8

Note: Table results are based on maximum rainfall intensity in Pennsylvania (1.2 in./hr.) and average asphalt pavement surface conditions (infiltration coefficient of 0.42)

$$1.2 = q_{test}/12.6$$
$$q_{test} = 15 \text{ gal/min-ft}$$

The commonly used laboratory test for highway edge drains is ASTM D-4716 [Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products]. Recommended conditions for performing the test are as follows:

- 1,500 lbf/ft^2 normal pressure,
- Core material placed between solid plates,
- 15 min of dwell time pressure,
- 15 min of test time for flow, and
- Hydraulic gradient of 0.10.

Geotextile Filter Flow Capacity

The geotextile must be able to accept the flow coming from the stone base course beneath the pavement system. The design is precisely the same as for the core except that it is not cumulative along the length of the pavement; that is, each running foot of geotextile is needed for only the associated running foot of edge drain. The other consideration is that some of the geotextile will be blocked from accepting flow because of the bonding or blocking of the core's protrusions. This consideration is easily handled by a geometric factor, f_B, which is product specific.

With the same design example as that used in the foregoing section on core flow rate—a 24-ft-wide road in Pennsylvania—

$$q_{design} = 0.00175W \text{ gal/min-ft}$$

and for $W = 24$ ft,

$$q_{design} = 0.042 \text{ gal/min-ft}$$
$$= 0.00562 \text{ ft}^3\text{/min-ft}$$

Using Darcy's law with an edge drain 1.5 ft high by 1.0 ft long and a core blockage factor f_B as described above,

$$q = kiA$$

$$\frac{q_{design}}{A} = k\frac{\Delta h_{ave}}{t}$$

$$\frac{0.00562}{1 \times 1 \times (1 - f_B)} = k\frac{0.75}{t}$$

$$\frac{k}{t} = \Psi_{design} - \frac{0.00749}{(1 - f_B)}$$

$$\Psi_{design} = \frac{0.00749}{(1 - f_B)} \text{ (min}^{-1}\text{)}$$

where

Ψ = permittivity,
k = coefficient of permeability,
t = thickness, and
f_B = blockage factor (20 to 95 percent).

For $f_B = 0.95$ (worst case),

$$\Psi_{design} = \frac{0.00749}{(1 - 0.95)} = 0.150\ (\mathrm{min}^{-1})$$

and

$$k_{design} = \Psi_{design} t$$

For a geotextile thickness of 0.060 in. (=0.0050 ft),

$$k_{design} = (0.150)(5.0 \times 10^{-3}) = 0.75 \times 10^{-3}\ \mathrm{ft/min}$$

$$k_{design} = 0.38 \times 10^{-3}\ \mathrm{cm/sec}$$

Using a worst-case cumulative factor of safety of 10 for clogging, blinding, and other considerations (*4*), $k_{reqd} \geq 0.0038$ cm/sec. This required permeability is satisfied by many geotextiles. (Values for geotextiles commonly used in filtration applications range from 0.1 to 0.001 cm/sec.) This statement can also be extended for any f_B-value of existing commercial products.

Geotextile Opening Size (0_{95})

The opening size of a geotextile is characterized by its apparent opening size (AOS), but in calculations it is preferable to work with the 0_{95} value corresponding to the nearest AOS sieve size. A review of soil retention criteria in the literature finds that all of them are for retention of fine sands and larger particle sizes (*4*). It is believed that these are not the soils that are troublesome for highway edge drains. Loss of soil through the geotextile and into the drainage core becomes a problem with the fine-soil fraction consisting of fine silts and dispersive clays. For example, the gradation of soil found inside the core of a completely clogged prefabricated edge drain in central Pennsylvania consisted of the following:

Sieve No.	*Percent Fines*
100 (0.25 mm)	99
200 (0.074 mm)	95
400 (0.037 mm)	85
Clay size (0.002 mm)	15

Whenever backfilling against the geotextile is not tight, such soil loss can occur. Thus those geotextiles with low 0_{95} values become very desirable for edge-drain filters. Note, however, that most geotextiles are in the No. 40 to No. 100 AOS ranges, which is illustrated in the following list of types of soil retained on different AOS sieve sizes:

AOS (sieve no.)	*0_{95} (mm)*	*Type of Soil Retained*
40	0.42	Lower medium sand
60	0.25	Upper fine sand
70	0.21	Middle fine sand
100	0.15	Lower fine sand
200	0.074	Upper-range silt
400	0.037	Middle-range silt

Thus, to prevent silts from moving through the geotextile, one must have a No. 200 or No. 400 AOS sieve size. Furthermore, dispersive clays would require still finer opening sizes, which are simply not available.

On the basis of the foregoing discussion and limitations, an AOS sieve size of 100 or higher is recommended, which corresponds to an opening size of 0.15 mm or less. It should be recognized, however, that very few commercially lightweight geotextiles have such a small opening size and a different, or heavier, geotextile than currently being supplied may be required.

Geotextile Strength

The geotextile filter has a secondary, but still very important, role in that it must support the backfill soil from one core protrusion to the next. In other words, it cannot collapse into the core and block the flow. The distance between protrusions is typically 0.5 to 2.0 in., with the core protrusion itself having a diameter of 0.1 to 1.0 in. Note that two strength phenomena are occurring simultaneously. First, the backfill soil is putting stress on the geotextile, thereby pushing it into the core, and the geotextile is going into some type of complicated mode of tension. Second, this action produces a puncture stress in the geotextile around the protruding core tip. Initial laboratory test results of hydrostatically stressing the geotextile against the core show that the puncture mode is more critical than the tensile mode and the problem formulation is developed accordingly. The design puncture strength is simply

$$P_{design} = \sigma_R(A_T - A_S)$$

where

P_{design} = design puncture strength,
σ_R = maximum stress imposed on the geotextile by the backfill,
A_T = geotextile area between centers of adjacent core protrusions, and
A_S = geotextile area over the individual core protrusion supports.

Using a maximum stress value of σ_R = 3,200 lbf/ft^2 (or 22.2 lbf/in.^{2}) from earlier in the paper and a variety of A_T- and A_S-values for the current range of commercially available edge-drain products gives results shown in Table 3. Here design puncture strengths are seen to vary from 4.4 to 83.7 lb, depending on the geometric configuration of the edge drain.

Because many existing edge drains have 1.0-in. protrusion spacings and 0.3-in. protrusion diameters, these values will be used to select the design puncture strength, 19.1 lb (Table 3). Now the required puncture strength value uses a factor of safety for design uncertainties, load variations, product variation, long-term considerations, and so on, as follows:

$$FS = P_{reqd}/P_{design}$$
$$3.0 = P_{reqd}/19.1$$
$$P_{reqd} = 57.3\ \mathrm{lb}\ (\text{use } 60\ \mathrm{lb})$$

P_{reqd} can be taken directly from the results of ASTM D-3787 (Puncture Strength of Geotextiles), which uses a 5⁄16-in. plunger (0.31 in.) and models the edge-drain situation from this paper quite nicely; that is, scale effects are believed to be minimal.

TABLE 3 DESIGN PUNCTURE STRENGTH VALUES FOR GEOTEXTILES ON EDGE DRAINS

Spacing (in.)	Total Area (sq. in.)	Support Diameter (in.)	Support Area (sq. in.)	Design Puncture Strength (lbs.)
0.5	0.25	0.1	0.01	5.0
0.5	0.25	0.2	0.04	4.4
1.0	1.00	0.1	0.01	20.8
1.0	1.00	0.2	0.04	20.1
1.0	1.00	0.3	0.09	19.1
1.0	1.00	0.4	0.16	17.6
1.5	2.25	0.1	0.01	47.0
1.5	2.25	0.2	0.04	46.3
1.5	2.25	0.3	0.09	45.3
1.5	2.25	0.4	0.16	43.8
1.5	2.25	0.5	0.25	41.9
1.5	2.25	0.6	0.36	39.9
1.5	2.25	0.7	0.49	36.9
2.0	4.00	0.1	0.01	83.7
2.0	4.00	0.2	0.04	83.1
2.0	4.00	0.3	0.09	82.0
2.0	4.00	0.4	0.16	80.5
2.0	4.00	0.5	0.25	78.6
2.0	4.00	0.6	0.36	76.3
2.0	4.00	0.7	0.49	73.6
2.0	4.00	0.8	0.64	70.5
2.0	4.00	0.9	0.81	66.9
2.0	4.00	1.0	1.00	62.9

Other geotextile mechanical properties such as grab strength, burst strength, and tear strength are obviously important but not easily determined by a specific design method. Thus values recommended for nonwoven geotextiles are controlled by the 60-lb required puncture strength just calculated. These are taken from the recently (July 1989) adopted survivability table of Task Force 25, Joint Committee of AASHTO, Associated General Contractors (AGC), and American Road and Transportation Builders' Association (ARTBA):

Degree of Surviv-ability	*Puncture Strength (lb)* Woven	*Puncture Strength (lb)* Non-woven	*Grab Strength (lb)* Woven	*Grab Strength (lb)* Non-woven	*Trapezoidal Tear Strength (lb)* Woven	*Trapezoidal Tear Strength (lb)* Non-woven
Medium	70	40	180	115	70	40
High	100	75	270	180	100	75

Thus a nonwoven geotextile with a 60-lb required puncture strength has a high survivability rating, a grab strength requirement of 180 lb, and a trapezoidal tear strength requirement of 75 lb. (Note that the burst value was purposely omitted in this recent version by Task Force 25.)

COMPARATIVE TESTING OF PRODUCTS

Regarding the drainage core, there is no available ASTM method specifically for prefabricated edge drains. Therefore, it was decided to develop a Geosynthetic Research Institute (GRI) test protocol for evaluating edge-drain core strength. The major elements of the test are as follows:

- The test specimen is full width (usually 18 or 12 in.) by 6 in. long and is cut so that a reproducible pattern of protrusions exists.
- The test specimen is evaluated without its geotextile covering.
- The upper and bottom load platens are 18 by 6 in., and the test specimen is placed in the center of the load platens. Note that the stress is calculated on the basis of the specimen size. Thus the imposed stress on the edge-drain core is the force in pounds exerted by the compression test machine divided by the area of the specimen and is then converted to any desired unit, for example, pound-force per square foot, kilograms per square meter or kilopascals.
- The loading rate of the compression testing machine was 0.04 in./min (≈1 mm/min).
- For these tests the compressive load was applied perpendicular to the test specimen and not at an angle.
- The test results for the nine edge-drain cores are given in Figure 3.
- The maximum stress attained by the core is considered to be its strength. It is usually a well-defined peak at which point the protrusions begin to deform noticeably and then to "telescope" or "bulge."
- No particular concern is given to the strain at failure, because these small deformations are not believed to be of major concern in a highway edge-drain application.
- A summary of these findings will be given at the end of the paper.

The flow rate test for edge-drain cores that most manufacturers and testing laboratories use is ASTM D-4716 [Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products]. It is not a particularly good simulation of the edge-drain situation, primarily because the core lies horizontally and flows full instead of being positioned vertically and only flowing in the lower flow zone; recall Figure 1. A more accurate test has been developed by Dempsey (*8*), but it requires more than 30 ft of floor space, which is a major impediment to its widespread adoption. Thus ASTM D-4716 was used.

The salient features of this version of the test method as it applied to edge-drain flow testing are as follows:

- The core size is 18 in. wide by 12 in. long.
- The core is evaluated without its geotextile covering.
- The core is positioned between solid plates on the top and bottom; thus no intrusion occurs.
- The value of normal stress is applied via an air bag at an initial value of 5.0 lbf/in.^{2} and then maintained for 15 min.
- Tap water is sent through the length of the core at different values of hydraulic head, starting at a high value of hydraulic gradient and successively proceeding downward in steps to the limit of the measuring system. The hydraulic gradients evaluated in this project were at the following values: 0.5 (when sufficient water was available), 0.25, 0.125, 0.062, 0.031, 0.016, and 0.0078.
- At each interval of hydraulic gradient the flow rate in gallons per minute is measured and then converted to gallons per minute-foot by multiplying the flow rate by 18/12, since the specimen size is 18 in. wide.
- When this set of data has been obtained, the normal pressure is then increased to 10 lbf/in.^{2} and the entire process is repeated. An entire series of normal pressures is then evaluated. The normal pressures used in this study were as follows: 5, 10, 15, 20, 25, and 30 lbf/in.^{2} (the limit of the air bladder system in this test method).

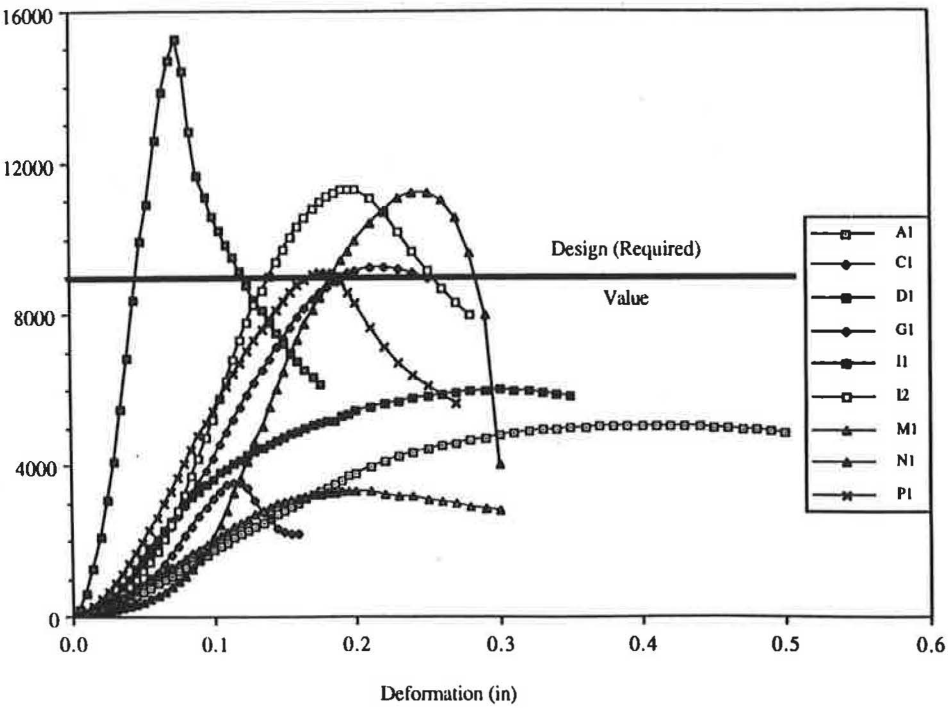

FIGURE 3 Compressive stress versus deformation of various geocomposite highway edge drains.

- The resulting data from all of the foregoing tests are then plotted as normal pressure versus flow per unit width for each hydraulic gradient value.
- The results of this entire sequence of testing have been given by Hwu (*9*) for each of the products. These findings are the essential characteristic curves for flow rate for the various geocomposite edge drains evaluated.
- The flow rate at 1,500 lbf/ft^2 pressure and a hydraulic gradient of 0.10 is then selected for comparison with the required, or design, value (Figure 4).
- A summary of these findings will be given at the end of the paper.

Because the properties of permeability, opening size, and strength of the geotextile filter are readily available from manufacturers' literature, they can be directly compared with the required values as per the previous design recommendations.

SUMMARY AND CONCLUSIONS

A design methodology for prefabricated edge-drain core and geotextile filter properties has been the focus of this paper. The designs have been developed with no specific product in mind but are regionalized to weather conditions in southeastern Pennsylvania. These are as follows:

Requirement	*Method*	*Value*
Core strength	GRI GG4	≥ 9,600 lbf/in.^{2}
Core flow rate	ASTM D4716	≥ 15 gal/min-ft (at 1,500 lbf/ft^2 and 0.10 gradient)
Geotextile permeability	ASTM D4491	≥ 0.001 cm/sec
Geotextile AOS	ASTM D4751	≥ No. 100 sieve (0.15 mm or less)
Geotextile puncture strength	ASTM D3787	≥ 75 lb
Geotextile grab tensile strength	ASTM D4632	≥ 180 lb
Geotextile trapezoidal tear strength	ASTM D4533	≥ 75 lb

Nine commercially available products were evaluated against the above-listed requirements. It was found that five products met the core strength requirement. Seven of the nine cores met the flow rate requirement. Clearly, product testing is required to see if the required design values are being met by the various products.

For the geotextile filters used on the respective cores it was found that all

- Are more than adequate in their permeability,
- Are too open and will allow soil loss into the edge-drain core,
- Have too low a puncture strength,
- Have too low a grab tensile strength, and
- Have too low a trapezoidal tear strength.

Thus they all fail to meet the recommended values on the basis of their large opening sizes and low strength properties. This difficulty, however, is readily overcome by using a somewhat heavier geotextile, which will result in a tighter void

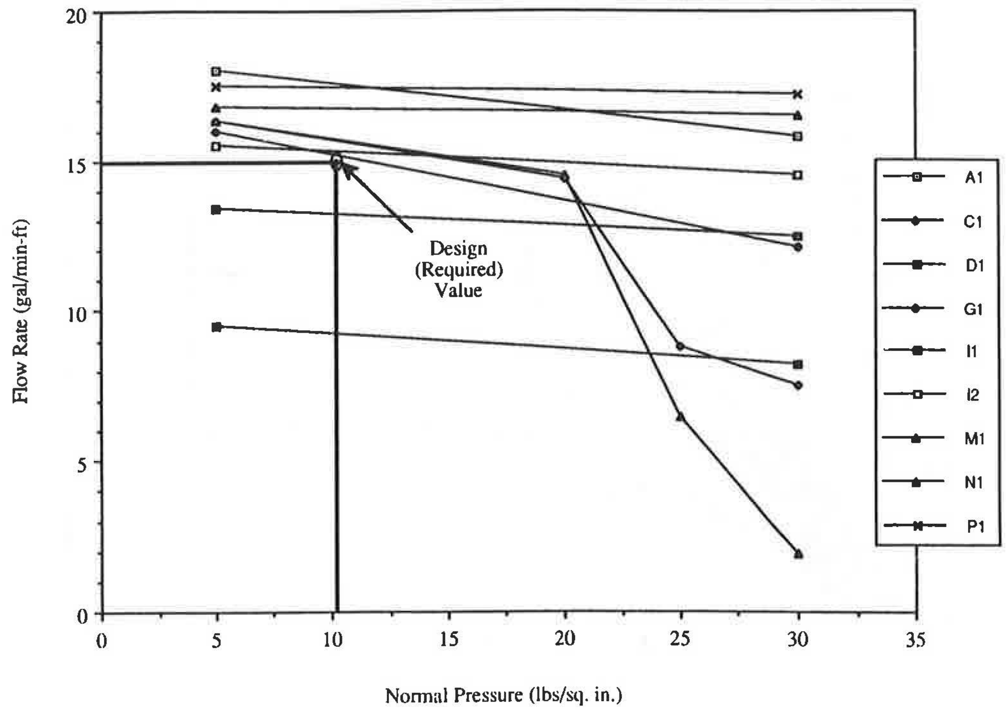

FIGURE 4 Flow rate behavior of geocomposite edge drains at hydraulic gradient of 0.10.

structure with higher strength values yet still adequate permeability.

ACKNOWLEDGMENTS

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Laboratory Procedure for Predicting Geocomposite Drain System Performance in the Field

EDWARD STUART III, KENNETH S. INOUYE, AND JAMES A. MCKEAN

When geocomposite drain systems first came on the market in the early 1980s, no accepted laboratory methods existed for predicting their field performance. Because it appeared that these products had definite applications in USDA Forest Service projects, a test procedure was developed to evaluate them. This test procedure was designed to determine the flow capacity of the geocomposite drain systems subjected to varying lateral loads and hydraulic gradients. The test apparatus consists of a large triaxial chamber and special plumbing. Geocomposite test specimens are placed vertically in a mold 6 in. in diameter by 12 in. high, which is then filled with a compacted silty soil. Changes in flow rates through the specimens are measured as both gradient and lateral pressure are varied.

Originally called fin drains or prefabricated drain systems, geocomposite drain systems consist of a geotextile covering one or both sides of a core material. The geotextile permits water to pass through while retaining the soil, and the core transmits the water to the drain outlet. Currently, at least 15 manufacturers make more than 40 geocomposite drain systems. The primary differences in the products are the cores (which can vary from a dimpled polystyrene or polyethylene sheet to a nylon wire mesh to a polyethylene net) and the geotextile covering (which can be woven or nonwoven).

LABORATORY TEST PROCEDURE

When geocomposite drain systems were introduced, no accepted laboratory test methods existed for predicting their field performance. As a result, the USDA Forest Service developed a test procedure to simulate as closely as possible the conditions in the field to which these products would be subjected. The test was developed to determine the flow capacity of the geocomposite drain systems under varying lateral loads. A preliminary examination indicated two potential factors that could greatly reduce the flow capacity: (a) elongation of the geotextile into the core flow channels as a result of the soil pressures and soil creep and (b) compression or deformation of the core material by the lateral soil loads. Therefore, the test was designed to apply lateral loads to a geocomposite drain sample through a soil medium rather than a stiff platen, which would have compressed the core but would not have caused geotextile elongation.

USDA Forest Service, 2245 Morello Avenue, Pleasant Hill, Calif. 94523.

The developed test involves measuring the flow of water through a 6- by 12-in. sample of the geocomposite placed vertically in a silty soil. The soil (AASHTO Classification A-4) is compacted to a dry density of 100 lb/ft^3 (85 percent of maximum density as determined by AASHTO T99) around the sample in a mold 6 in. in diameter by 12 in. high. The soil-geocomposite test specimen (covered with a latex membrane) is then placed in a large (12-in.-diameter, 40-in.-high) triaxial chamber. A system (Figures 1–3) was built to allow water to flow into the geocomposite sample at the bottom and out the top under varying (but constant for each test run) hydraulic gradients (0.3, 1.0, and 2.0) and confining pressures (5, 10, 15, 20, 25, and 30 psi). The water flow direction was selected so that the sample was always flowing at full capacity. The system was designed to ensure that its flow capacity was greater than the flow capacity of the samples under any confining pressure. The ends of the geocomposite samples were open to permit unrestricted flow of water into the samples. The type of soil used was selected to represent a low strength condition, and the gradients and pressures were selected to represent typical field application conditions.

The test is performed by measuring the flow rate for each sample at various combinations of hydraulic gradients and confining pressures, maintaining the confining pressure until the flow rate stabilizes. This is done to ensure that the effects of soil, fabric, and core creep are included in the results. The period required for this steady-state flow can vary from a few days to many weeks. A minimum of two complete tests is performed on each product evaluated.

Graphs plotting the steady-state flow rate versus confining pressure for each of the tested gradients and flow rate versus time for a given gradient and pressure are developed for each product. Samples of these graphs are shown in Figures 4–7. Consolidated results for typical products are shown in Table 1.

The results of the laboratory testing program show a wide range in performance of the different geocomposite drainage systems. Some products have more rigid cores, and their flow rates showed only slight decreases with increasing confining pressure (PP-1, PP-2, PP-4a, PP-4b). The decrease in flow (around 10 percent) is believed to be due to a reduction in the cross-sectional area of the geocomposite drain system caused by a combination of compression of the core and elongation of the geotextile into the flow channels. However, when the crushing strength of the core was exceeded, the flow rate dropped sharply. Figure 4 shows a dramatic decrease in flow in the PP-2 product when the confining pressure was increased

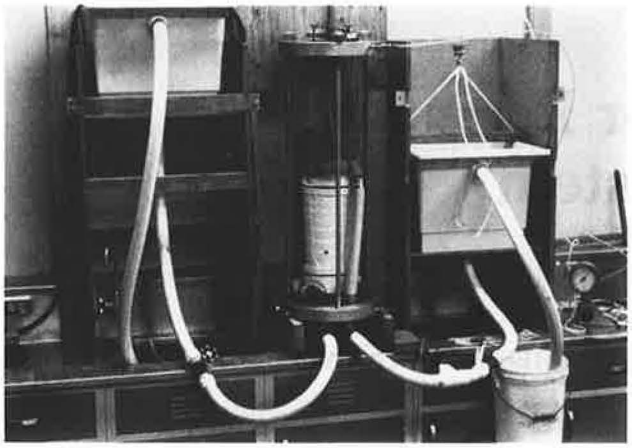

FIGURE 1 Laboratory test system.

from 20 to 25 psi, whereas Figure 5 shows a similar decrease in flow capacity of the PP-4a product when the confining pressure was maintained at 30 psi. Other geocomposite drain systems have more compressible cores. Flow rates for these products dropped markedly with increasing confining pressure. The PP-3a and PP-3b products, which have more compressible mesh cores, were especially susceptible (see Figure 4 for the PP-3b product).

When the confining pressure was increased during the test, the flow rate usually stabilized within a few days. However, the slow crushing process of one product (PP-2) caused the flow rate to gradually decrease over a period of months (Figures 6 and 7).

All products tested—with the exception of PP-2, which partially collapsed at 25 psi—had a minimum equilibrium flow rate of about 1 gal/min per foot of drain width when subjected to a hydraulic gradient of 1.0 and a confining pressure of 30 psi. The PP-2 product exceeded this value at a confining pressure of 25 psi.

FIELD INSTALLATIONS

In conjunction with the laboratory testing, three field installations of geocomposite drain systems were instrumented for future monitoring. These installations were placed behind fills or retaining walls. Instrumentation consists of piezometers placed upslope and downslope of the drains. Results of these installations, it is hoped, will validate the test procedure. However, at this date, sufficient time has not yet passed to allow any substantive data to be obtained.

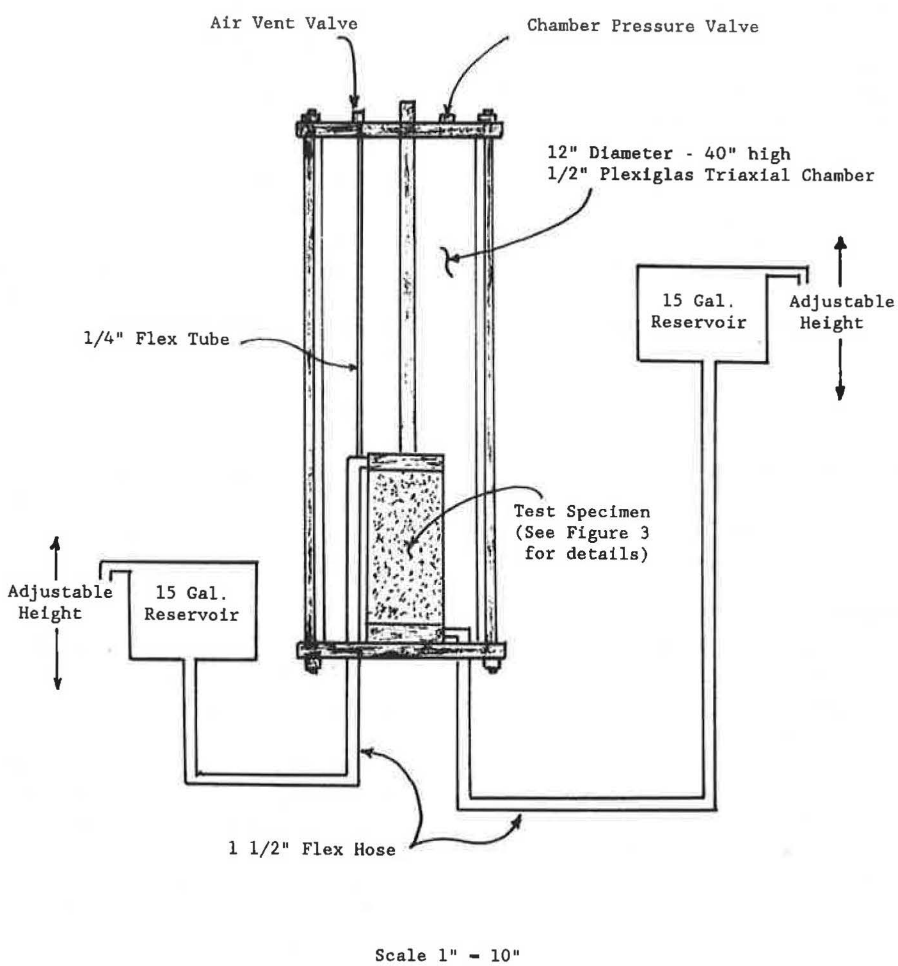

FIGURE 2 Test setup.

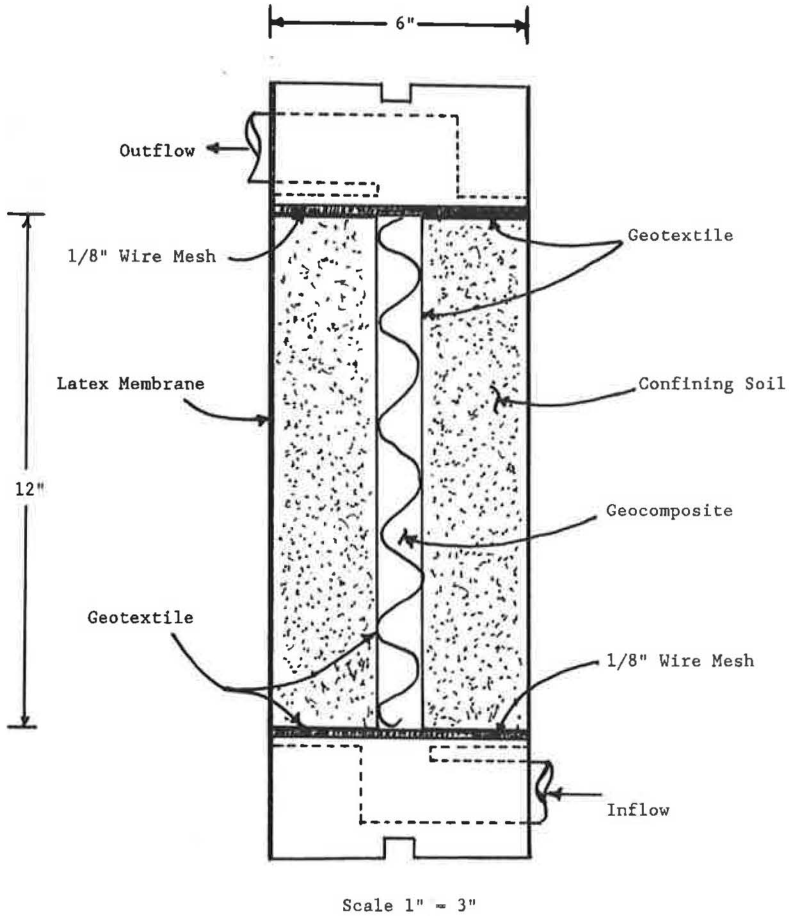

FIGURE 3 Test specimen.

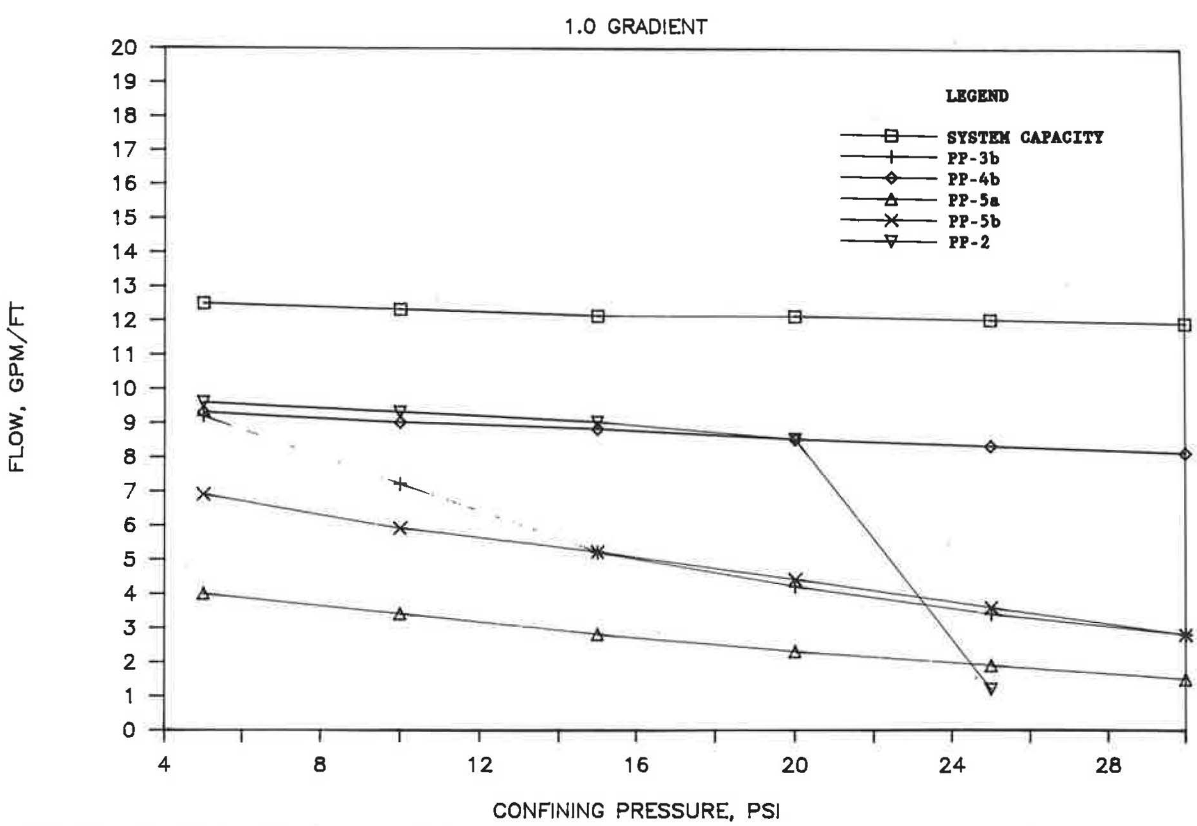

FIGURE 4 Equilibrium flow versus confining pressure.

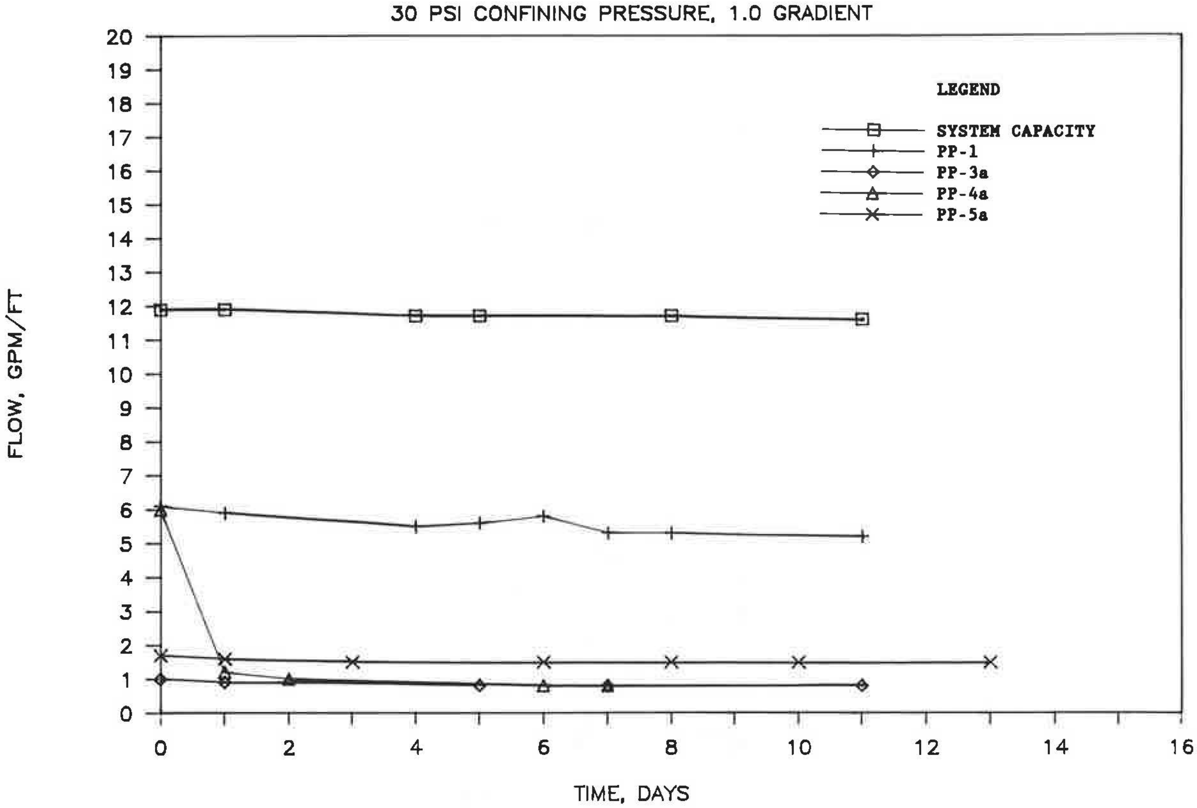

FIGURE 5 Equilibrium flow versus time: confining pressure increased from 20 to 25 psi.

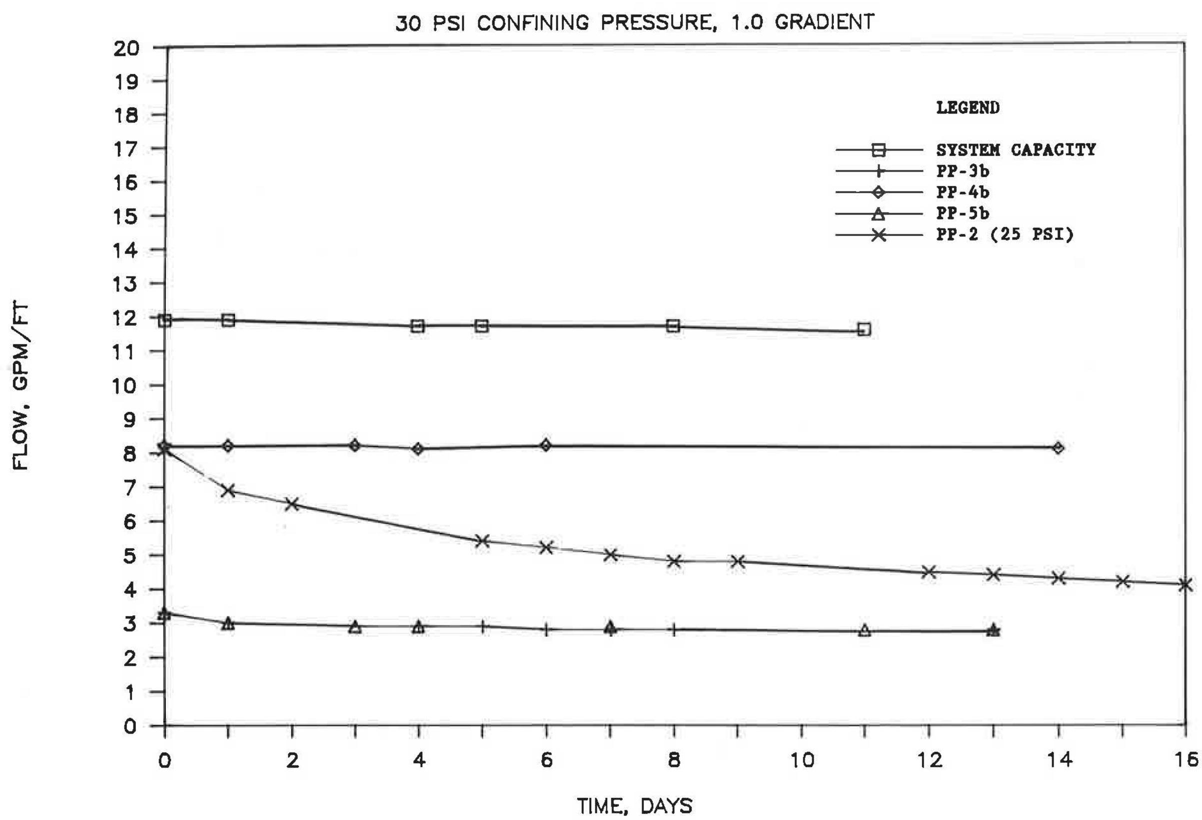

FIGURE 6 Equilibrium flow versus time: crushing of PP-2 over 16 days.

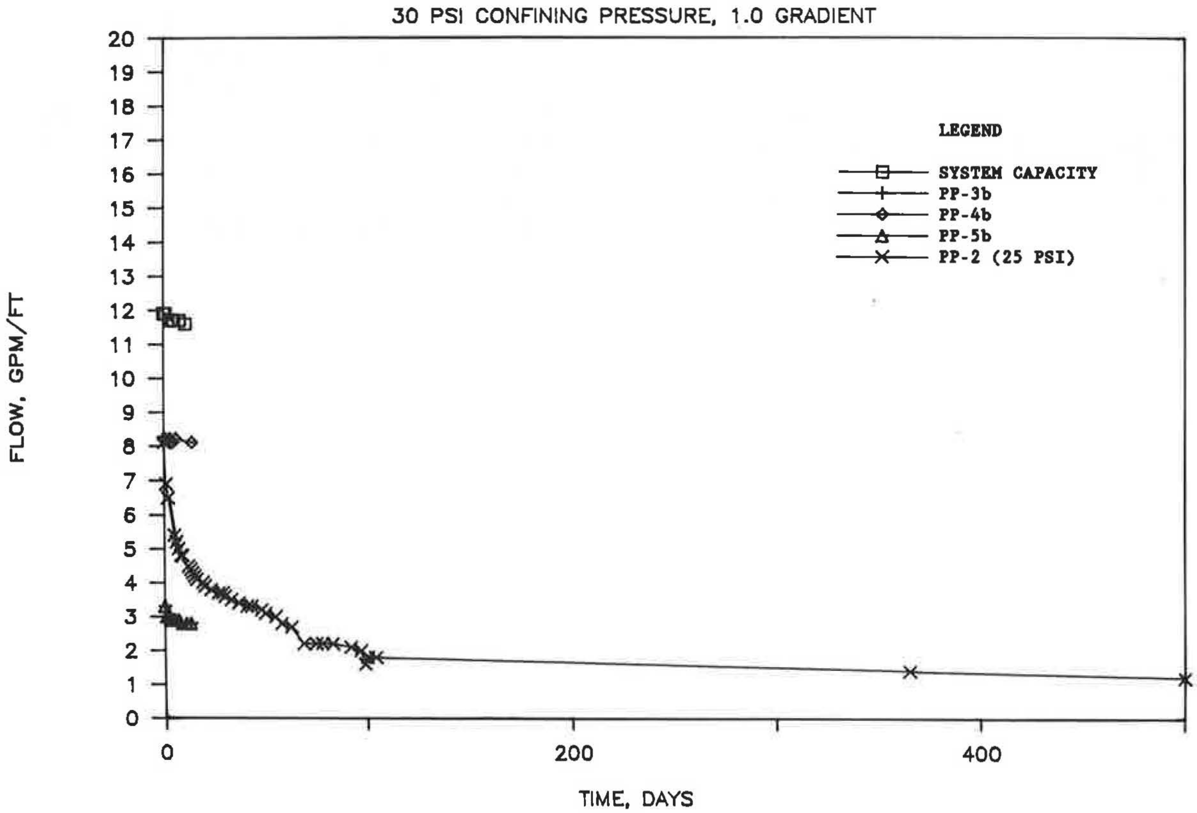

FIGURE 7 Equilibrium flow versus time: crushing of PP-2 over 500 days.

TABLE 1 PERFORMANCE OF GEOCOMPOSITE DRAIN SYSTEMS AT A CONFINING PRESSURE OF 30 psi

	Equilibrium Flow, gpm/ft		
Product Gradient:	0.3	1.0	2.0
PP-1	3.0	5.2	7.5
PP-2*	0.6	1.2	1.8
PP-3a (1 layer of core)	0.4	0.9	1.2
PP-3b (2 layers of core)	1.5	2.8	4.1
PP-4a	0.4	0.9	1.2
PP-4b	4.6	8.1	11.8
PP-5a (1 layer of core)	0.8	1.5	2.2
PP-5b (2 layers of core)	1.6	2.8	4.0
(System capacity	6.8	11.7	16.9)

* 25 psi confining pressure

RESULTS

1. The flow capacity of the geocomposite drain systems tested varies directly with the hydraulic gradient and inversely with the confining pressure.

2. The equilibrium flow of a product at a given confining pressure can usually be determined after a period of several days. However, for some products, the confining pressure must be maintained for longer periods, up to several months.

3. The reduction in flow capacity of the geocomposite drain system may result from crushing or compression of the core material, elongation of the geotextile due to increased soil pressures, or some combination of these effects.

4. All products tested—except for the PP-2 geocomposite, which partially collapsed at 25 psi—had minimum equilibrium flow rates of about 1 gal/min per foot of drain width when subjected to a confining pressure of 30 psi and a hydraulic gradient of 1.0. The PP-2 geocomposite transmitted this volume at a confining pressure of 25 psi.

CONCLUSIONS

The initial impetus for developing this test method was the lack of a known accepted laboratory procedure for predicting field performance of the increasing number of geocomposite drain systems being marketed. ASTM 4716-87 [Standard Test Method for Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products] had not yet been published at the onset of this program, and as a result no comparison between the two test methods was made. Because most earth-retaining structures depend on their drainage systems for stability, it was imperative that a procedure be developed to ascertain the potential field performance of the geocomposites. Two possible conditions that can affect the geocomposites' ability to transmit water are

(a) compression or crushing of the core and (b) stretching and filling of the passageways in the core by the geotextile. Thus, a test using geocomposite samples embedded in soil cylinders was developed. Results of the tests performed on a number of geocomposites showed that as the confining pressure increases, the flow decreases. Visual evaluations of the samples after the test showed that both core compression and geotextile stretch occurred to some extent in most of them.

The test procedure appears to approximate field conditions. However, some considerations could modify or change it. Areas that could be considered for change would include soil type and condition, size of confining soil, size of geocomposite sample, confining pressures, and gradients. Further tests incorporating some of these variables are strongly recommended. However, any test developed in this regard should incorporate a soil-confining medium.

Video Evaluation of Highway Drainage Systems

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Since 1978 the concept of longitudinal edge drains along Iowa primary and Interstate highways has been accepted as a cost-effective way of prolonging pavement life. Edge-drain installations have increased over the years, reaching a total of nearly 3,000 mi by 1989. With so many miles of edge drain installed, the development of a system for inspection and evaluation of the drains became essential. Equipment was purchased to evaluate 4-in.-diameter and geocomposite edge drains. Initial evaluations at various sites supported the need for a postconstruction inspection program to ensure that edge-drain installations were in accord with plans and specifications. Information disclosed by video inspections in edge drains and in culverts was compiled on videotape to be used as an informative tool for personnel in the design, construction, and maintenance departments. Video evaluations have influenced changes in maintenance, design, and construction inspection for highway drainage systems in Iowa.

The Iowa Department of Transportation has determined that longitudinal edge drains are cost-effective in removal of underslab moisture and prevention of premature pavement failures. Before 1978 a minimal number of longitudinal edge drains had been installed in areas with severe moisture problems.

In 1978 approximately 167,000 ft of 4-in.-diameter longitudinal drain was installed along primary and Interstate highways in Iowa. Since then, the annual installation had increased to a peak of approximately 3.5 million ft in 1988 (Figure 1). By 1989 a total of more than 14 million ft of longitudinal edge drain had been installed (Figure 2).

The average cost for installation of edge drains has decreased, in general, since 1987. Some cost fluctuations were due to changes in specifications. The average cost per foot installed over the years is shown in Figure 3, with a current cost of approximately $4.00/foot installed.

Even though a very large number of edge drains was in place by 1989 (Table 1), there was no inspection program or positive method to evaluate the condition of drains other than the visual inspection of the outlets.

OBJECTIVE

The objective of this paper is to describe the benefit of a video evaluation of highway drainage systems and to present the results of the evaluation.

HISTORY OF EDGE DRAINS IN IOWA

An initial 1978 edge-drain installation was placed as a rehabilitation effort for 28 mi of deteriorating 10-in. portland cement concrete (PCC) pavement on I-80 in Poweshiek County. At that time, this roadway carried approximately 6,500 heavy trucks a day, and pavement pumping was a severe problem. The drain design used a 6-in. polyethylene slotted pipe placed at the pavement edge in a trench 24-in. deep measured from the top of the pavement. Slot size and porous backfill were designed according to FHWA implementation package 76-9. Filter criteria assumed a sandy silt AASHTO A-4-3 soil classification. The trench was 12 in. wide and the porous backfill was placed in contact with and 2 in. above the bottom of the pavement. A 3-in. bedding was placed under the pipe, and flow lines were controlled by the grade line of existing pavement to minimize costs. The entire system was designed to be constructed using a "one-pass" mechanical system. Drain outlets at approximately 1,000-ft intervals were constructed using earth backfill and metal pipe aprons.

This drain system rapidly developed problems. Considerable localized plugging of the backfill and drain pipe occurred. During the first winter, a near-disastrous outlet freeze-up occurred; as a result, a substantial amount of water flowed from the top of the drain trench and froze on the pavement. To eliminate that problem, the outlets were reconstructed the following spring by placing full-depth porous backfill so that it would daylight on the foreslope and removing the metal aprons. No further winter freeze-up problems have occurred with this design.

The 1979 designs used a 30-in. trench depth for similar Interstate highways, and the Iowa nondestructive pavement deflection testing (Road Rater) program indicated that there was a small but significant improvement in subgrade strength. Localized backfill plugging also decreased significantly. Of most significance was the discovery that most outflow was now occurring through the porous backfill bedding and that the pipe functioned only during periods of heavy rain. This alleviated many concerns for poor pipe flow line control and failures due to poor construction, which have been verified by excavation.

On the basis of the improvements from early design changes, 1981 designs increased the trench depths to 48 in. and reduced the pipe size to 4 in. and the trench width to 10 in., as shown in Figure 4. It was discovered that subgrade strengths again increased and that localized porous backfill plugging was reduced to areas of complete pavement failure. Subsequently, it was determined by excavation and laboratory testing that the material plugging the backfill consisted primarily of cement dust. It was typical to find less than 10 percent clay in these extracted fines. This meant that permeability in excess of 200 ft per day remained and that the plugging material would flush through the system after the pavement problem had been corrected. It also proved that the system could ac-

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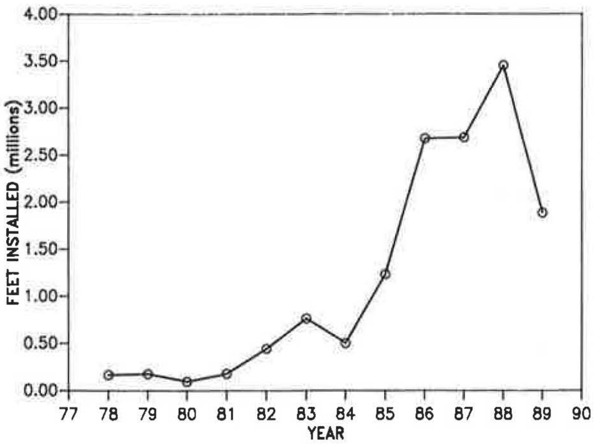


FIGURE 1 Annual 4-in. drain installation.

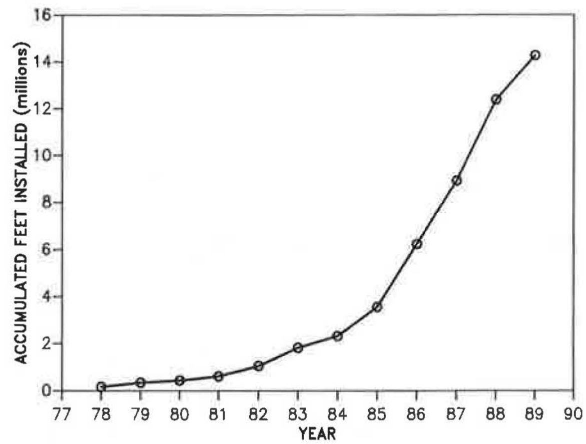


FIGURE 2 Accumulated feet of 4-in. diameter drain installed.

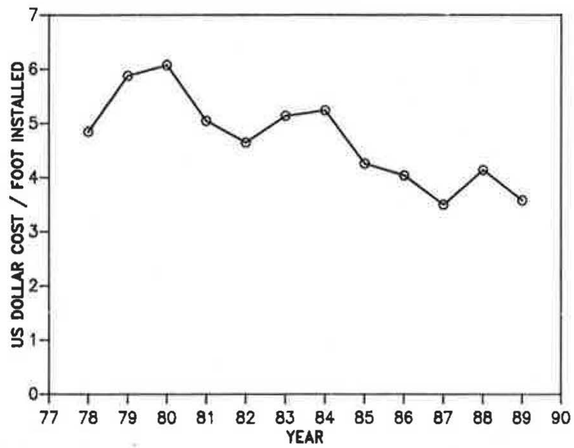


FIGURE 3 Average cost of edge drains.

TABLE 1 SUMMARY OF 4-in.-DIAMETER LONGITUDINAL SUBDRAIN INSTALLATION

Year	Qty. (ft) Installed	Ft. Installed Accumulated	$Cost/Ft Installed	Total $ Cost
1978	167,122	167,122	4.85	810,256
1979	177,273	344,395	5.88	1,043,176
1980	95,289	439,684	6.08	579,119
1981	178,669	618,353	5.05	903,118
1982	441,959	1,060,312	4.65	2,053,779
1983	763,556	1,823,868	5.14	3,924,366
1984	503,126	2,326,994	5.24	2,638,368
1985	1,234,213	3,561,207	4.26	5,263,676
1986	2,676,745	6,237,952	4.04	10,824,118
1987	2,686,218	8,924,170	3.50	9,410,118
1988	3,452,414	12,376,584	4.14	14,294,100
1989	1,884,281	14,260,865	3.58	6,751,087

Total Accumulated Feet Installed	14,260,865.00
Average Cost per Foot	$4.10
	==============
Total Cost	$58,495,281.00

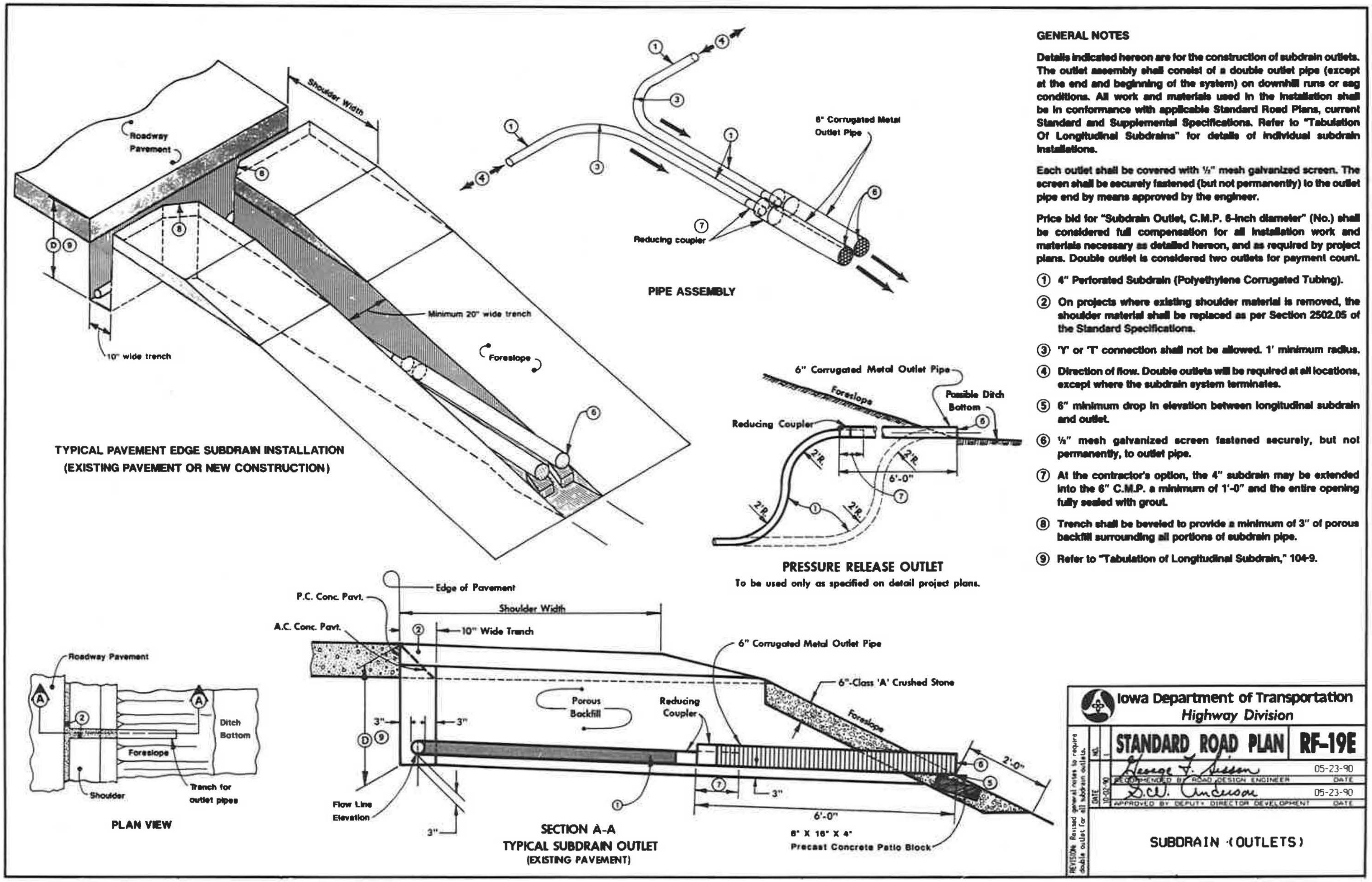


FIGURE 4 Standard road plan for subdrain outlets.

commodate recycled crushed PCC and provided the emphasis for the development of the present drainable base system, which uses crushed recycled PCC almost exclusively.

The deeper drain trench made continual maintenance inspection necessary, and the Maintenance Department responded by establishing an annual inspection policy for all drain outlets. A standard road plan for various types of longitudinal subdrain installations is shown in Figure 5.

During 1985 there were numerous plugging problems on an Interstate project that had been surface corrected by diamond grinding. Investigation revealed that cement fines were again the problem and that they were present in sufficient quantities to plug the pipe as well as the porous backfill. This problem was solved by retrofitting additional outlets at 400- to 500-ft spacing compared with the 1,000-ft maximum used originally. The water would then wash the fines out of the drains as verified by recent video inspections. Design policy was changed to require an outlet spacing of 500 ft for all grades less than 2 percent and again changed during 1988 to require a 500-ft spacing for all outlets.

The 1989 video inspections soon showed that much of the outlet problem was caused by disconnected Y-pipe couplers at the main line outlet junction. It also showed that fines accumulation in the pipe was practically nonexistent even when the pipe was completely ponded, separated, or blocked by porous backfill aggregate. Although numerous sites had been excavated in the past, these conditions had not been readily identifiable until the camera equipment became available. Design changes have been made to eliminate the outlet coupler, and the standard deep drain has been raised to 42 in. to ensure that the outlet occurs above the ditch bottom.

Although numerous changes have been required to improve system performance, the original implementation package design for porous backfill and pipe slot design has performed satisfactorily under all conditions and has provided the porous aggregate alternative drainage necessary for long-term highway edge-drain operation.

VIDEO INSPECTION PROJECT

From 1978 through 1988, the Iowa Department of Transportation installed, under contract, approximately 12 million ft of longitudinal edge drain along primary and Interstate highways. In areas where no subgrade-related problems were present, subdrains were placed on one side of the pavement only. The side of placement was determined by major traffic volume, relative low-side elevation, or primary water source. After the construction inspection, there was no postconstruction evaluation or internal visual inspection of these drains. In 1989 a proposal was presented to the Highway Research Advisory Board for the Iowa Department of Transportation to initiate a research project on evaluation of edge drains.

Information was obtained from 10 suppliers of evaluation equipment. Eight demonstrated their equipment in laboratory or field conditions or both. In addition, product information was obtained through contacts with organizations that were using similar video equipment for other than highway edge-drain purposes. It was determined that two types of video evaluation equipment would be required to inspect the two types of Iowa edge drains. Most edge-drain pipe used in Iowa is 4-in.-diameter corrugated, slotted polyethylene. Three brands of geocomposite edge drain have been used experimentally since 1987, for a total installation of approximately 60,000 ft.

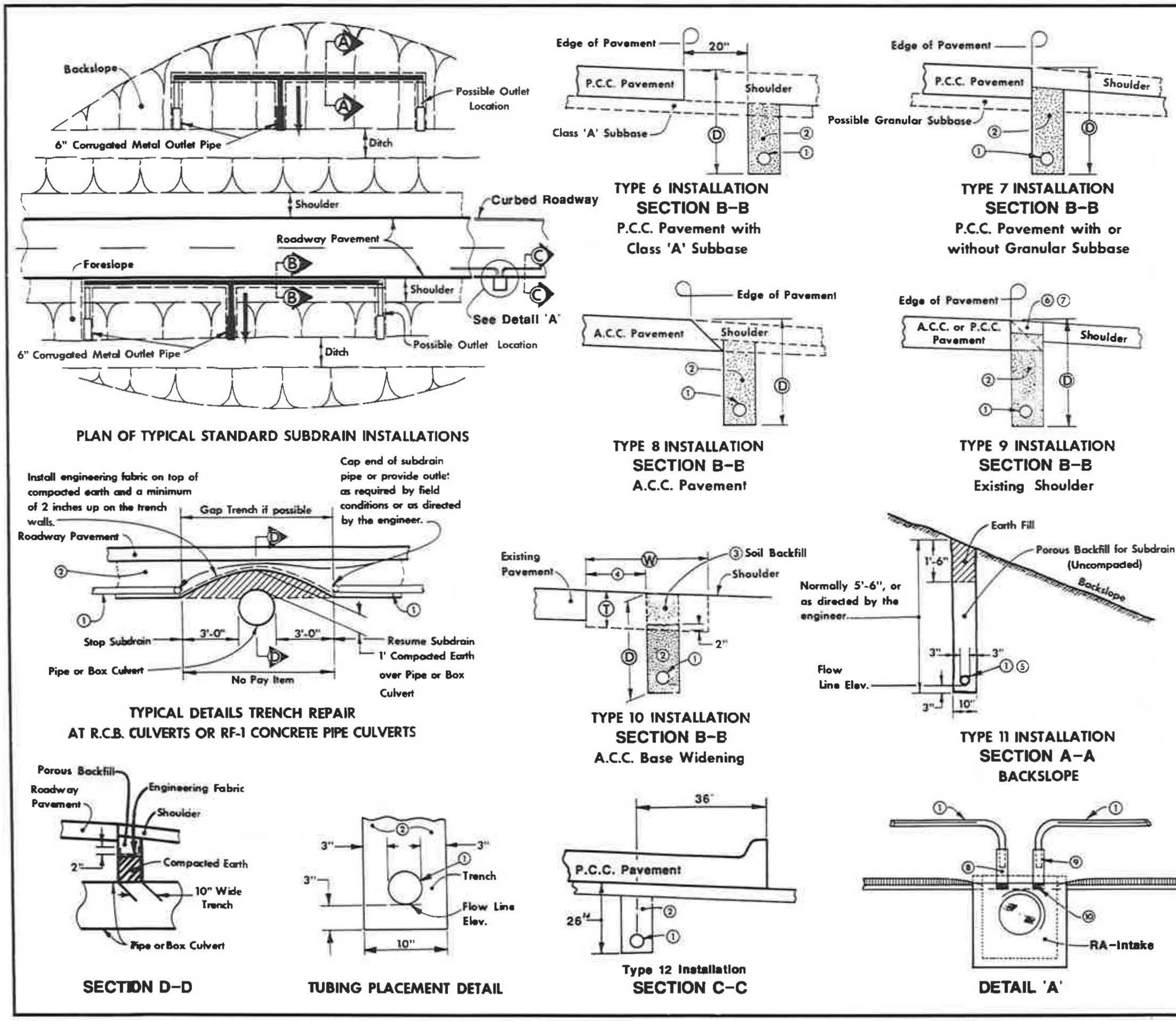
Equipment

For the 4-in.-diameter edge drain, a camera system of 3-in. diameter or less with a cable length of 300 ft was considered desirable. The geocomposite edge drain required a camera probe of maximum ½-in. diameter and minimum 3-ft length. A video recording unit was required to document the inspections and a small portable electric generator was needed for the power supply in the field.

Several product suppliers offered equipment that met the project needs. For the 4-in.-diameter drains, they offered cameras from 2- to 3-in. diameter on a cable that could be pushed to approximately 150 ft. Some systems used a heavy semirigid push-conductor cable to enter the drains. Other systems used a lightweight flexible conductor cable in parallel with a fiberglass push rod. Either of these video camera systems could be adapted to evaluation of small-diameter culverts also. The mini crawler-tractor mobile camera systems offered by some suppliers for deep probes were considered unsuitable for 4-in.-diameter drains. The options for color, black-and-white, or both types of pictures were available. The cost with the color option was considerably higher and the color camera was longer; therefore, the black-and-white option was selected for the larger-diameter camera.

From several suppliers who offered suitable video evaluation equipment for the 4-in.-diameter drain, the Cues, Inc. Mini Scout system was finally selected. This system has a 2¾-in.-diameter camera, including a headlight on a 150-ft semirigid push-conductor cable that connects to a black-and-white 9-in. video monitor. The system was competitively priced and well packaged for field conditions. The equipment cost with some accessories was approximately $12,000. The Cues Mini Scout video camera system and accessories are shown in Figure 6. The cost estimates for other basic video units considered for small drains started around $11,000. As options are added, such as a footage counter, additional cable length, pull system, 35-mm camera accessories, and optional lighting head, the system cost may double.

For geocomposite (1-in. width) edge-drain evaluation, several sets of suitable video probe equipment were considered. For this application, the colored picture and the 50 ft of ½-in.-diameter video probe options were preferred. The probe length is far beyond the 3-ft requirement for geocomposite edge-drain evaluation. However, this probe length and diameter could also be used for entering 4-in.-diameter drains that are partially plugged so that the 2¾-inch Cues camera cannot pass. A 50-ft video probe with an articulating tip was selected so that the equipment would have more potential in adapting to other possible uses within the Iowa Department of Transportation. From several choices of suitable equipment offered for mainly geocomposite edge-drain evaluation, the Welch Allyn VideoProbe 2000 system was selected. The cost of the equipment was approximately $45,000. The Welch Allyn VideoProbe 2000 system and accessories are shown in Figure 7.



GENERAL NOTES:

Details indicated hereon are for the construction of longitudinal subdrains. All work and materials used in the installation shall be in conformance with applicable Standard Road Plans, current Standard and Supplemental Specifications. Refer to "Tabulation Of Longitudinal Subdrains" for details of individual subdrain installations.

Areas of shoulders in project limits not shown in tabulation of longitudinal subdrain and other areas conflicting with subdrain installation will not be trenched.

When RCB culverts or RF-1 concrete pipe culverts which are less than 1 foot below the trench bottom are encountered within a tabulated Subdrain, the trench shall stop 3 feet from the culvert and resume 3 feet beyond the culvert. If the trench is inadvertently carried over the culvert, the trench shall be repaired as detailed on this sheet. Care must be exercised so as not to destroy the tops of culverts with the trenching machine. If obstruction is 1 foot or more below normal trench bottom, carry subdrain line over in continuous alignment.

Subdrain trench shall be located adjacent to edge of roadway pavement. On new construction projects, the subdrain shall be placed after the mainline paving and prior to shoulder placement. On new projects with tied P.C.C. Shoulders, trench location shall be as determined by the engineer. On existing roadways, the trench shall be capped with material per current Standard and Supplemental Specifications.

Price bid for "Longitudinal Subdrain, (Shoulder)" or (Back Slope) (lin. ft.) and "outlets" each, shall be considered full compensation for all installation work and materials necessary as detailed hereon, and as required by project plans.

Porous backfill is considered incidental to "Longitudinal Subdrain".

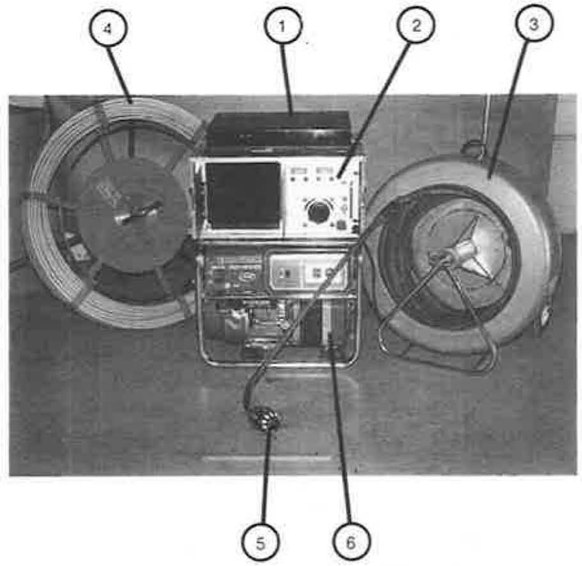
1. 4" Perforated Subdrain (Polyethylene corrugated tubing).
2. Porous Backfill.
3. Backfill of this area is not required if base widening is placed the same day of subdrain construction.
4. Min. 8" to w/2.
5. Subdrain is to be installed as cut proceeds.
6. On existing Granular or Earth Shoulders replace with 4" minimum depth granular shoulder material.
7. On Paved Shoulders refer to specification for finishing shoulder, see Section 2502.
8. 6" CMP 2' long.
9. Perforated P.E. fit into CMP minimum 12" if grout is used. At the contractor's option use reducing coupler or grout.
10. Removable mesh cap ½" hardware cloth.

Iowa Department of Transportation
Highway Division

STANDARD ROAD PLAN	RF-19C
RECOMMENDED [signature] 12/7/87	ROAD DESIGN ENGINEER — DATE
APPROVED [signature] 12/10/87	DEPUTY DIRECTOR — DEVELOPMENT — DATE
LONGITUDINAL SUBDRAINS	

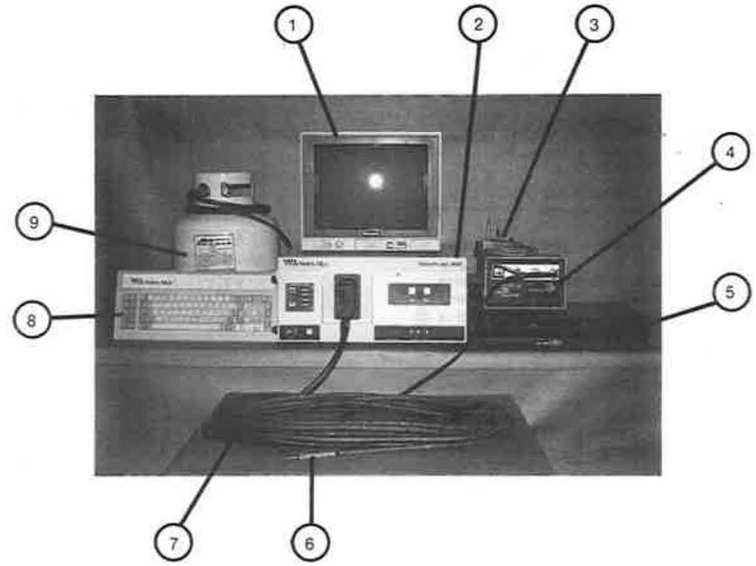
REVISION: Add Detail "A".

FIGURE 5 Standard road plan for longitudinal subdrains.



1. Video Recorder
2. Cues Monitor/Power Control
3. Cues push/conductor cable with camera and storage reel (300′)
4. Fiberglass push rod ⅜″ dia. and storage cage (300′)
5. Cues Camera
6. Portable Generator

FIGURE 6 Cues Mini Scout video camera system and accessories.



1. Monitor
2. Videoprocessor
3. Articulation Control Stick
4. Pneumatic Controller
5. Video Recorder
6. Articulating VideoProbe
7. VideoProbe Cable ½″ Dia. (50′)
8. Data Input Keyboard
9. Air Supply for Camera Head Articulation

FIGURE 7 Welch Allyn VideoProbe 2000 system and accessories.

Some accessories were purchased for the project:

- Small portable electric generator,
- Videotape recorder, and
- Fiberglass push rod (300 ft of ⅜ in.).

The total project expenditure was approximately $60,000.

Modifications

Cues 2¾-in. Mini Scout Video Camera System

The standard Cues Mini Scout system has 150 feet of semirigid push-conductor cable. A modification of cable length to 300 ft was made at the time of purchase. Under normal conditions, the camera could be pushed approximately 125 ft into 4-in.-diameter drain before cable buckling would occur. With the addition of a ⅜-in.-diameter fiberglass push rod, the camera can be pushed 300 ft into a drain.

The option to replace the semirigid push-conductor cable with a flexible conductor cable also exists. That would reduce cable weight from 100 to 30 lb and reduce friction and manpower required to push the camera. With that option, the fiberglass push rod is required.

For small culvert evaluations a skid assembly with battery-powered, waterproof lights is added to the camera. This modification raises the camera off the culvert floor and the extra lights assist in illuminating culvert walls. For evaluations beyond 75 ft, a push rod consisting of 10-ft sections of 1-in.-diameter polyvinylchloride pipes is assembled and used to advance the camera.

For bridge pier evaluation a camera position holder and a guide pole are required.

Welch Allyn VideoProbe 2000 System

To improve visibility of a picture on the video monitor in outdoor sunlight, a sun shield was required.

The addition of a 1/16-in. fiberglass push rod attached parallel to the 50-ft video probe was essential for probe rigidity. The fiberglass rod changed the length that could be utilized in 4-in.-diameter drains from 15 to 50 ft.

VIDEO EVALUATION AND OBSERVATIONS

Initially the sites for video evaluation of edge drains were selected randomly. As the research project and the use of the equipment received more publicity, requests were received for evaluation of specific problems or suspected problem areas.

Both types of equipment were transported to each evaluation site. The 2¾-in.-diameter camera was used in most cases. When a partially buried outlet was encountered, the ½-in.-diameter video probe was used. In some cases, the outlet pipe was found completely plugged or buried. With the porous backfill extending to the outlet, as in a french drain, water can still flow around any plugged or buried outlet pipe.

The random drain inspections did expose some problems:

1. Rodent nests in the drain (Figure 8),

FIGURE 8 Rodent nests in subdrains.

2. Vertical sag from main line to outlet,
3. Polyethylene tubing and connector failures,
4. Break from stretch or puncture, and
5. Geocomposite drain J-buckling.

Rodent Nests

Drought conditions prevailed across Iowa in 1989. With little or no water flow through the 4-in.-diameter edge-drain pipe, the conditions were favorable for rodent nesting in the drains. The rodent guards used were a hanging finger type, and they did not prevent small rodents from entering. The video evaluations in the fall of 1989 showed rodent nests in approximately 50 percent of the drains inspected.

No rodents nests were encountered by video evaluations during the rainy spring of 1990. There was evidence of rodent nest material—grass and fur—around the outlet of the drain. From these observations, it appears that water flows in the drains were sufficiently high or turbulent to flush out the rodent nests. A rodent guard made from ½-in. mesh is more suitable to prevent small rodents from entering.

Vertical Sag (Main Line to Outlet)

Longitudinal edge drains are installed by a trencher-installer that follows the grade of the pavement. Drain outlets are spaced at 500 ft. Occasionally, a vertical sag full of water is observed in the main line when no water is flowing at the outlet.

The outlet section through the shoulder is excavated by a trencher or a backhoe. Even though plans show a continual downgrade, it is common to find the shoulder outlet section high and retaining standing water in the edge drain.

Polyethylene Tubing and Connector Failures

It is often assumed that any time the main line of an edge drain is disrupted by a coupler, Y, T, elbow, or other device, there is an increased risk of failure at that point. Through video evaluations, that assumption can be, to some degree, confirmed. Occasionally, a blockage from porous backfill is found inside the drain at the point of a connection.

Break from Stretch or Puncture

Excessive tension applied to the polyethylene corrugated pipe during installation can, in the worst case, cause it to tear and leave an opening. The opening is likely to allow backfill to enter and a cavity may develop above the opening. Pipe opening can also be caused by an oversized sharp stone, 3-in. diameter or larger, in the backfill, which may puncture the pipe during compaction. The pipe could also be stretched, which reduces its stiffness, resulting in collapse. If a drain is collapsed or plugged completely, the water flow will travel outside the pipe through the porous backfill.

Geocomposite Drain J-Buckling

Some brands of geocomposite drains are designed with one side covered by only filter fabric and therefore quite flexible and weak under vertical load. During installation, the drain is fed downward to the bottom of the trench and is forced to bend in a vertical plane. The force causes the drain to buckle under along its bottom edge, leaving it in a J-configuration as backfill is compacted beside it. Video evaluations have identified J-buckling in soft-sided geocomposite drains.

Summary

The video evaluation equipment has been used as a postconstruction inspection tool in finding stretch breaks and collapsed or damaged drains. The most common video sights of special interest, in descending order of frequency in 4-in.-diameter plastic drain pipes, were

1. Vertical sags,
2. Rodent nests (decreasing after specification change),
3. Collapse from stretch,
4. Connector failures (decreasing after specification change),
5. Break from stretch, and
6. Puncture by oversized, sharp stone.

Two representative views taken from the videotape are shown in Figures 8 and 9.

IMPROVED INSPECTION AND INSTALLATION

The use of the video evaluation equipment for postconstruction inspection can provide valuable information and detect problems. The internal view of an edge drain may show the drain pipe to be parted at a coupler or collapsed from being stretched. These problems could occur in a trench during installation and not be detected by an operator or inspector. Within its limits of travel, the video evaluation equipment can clearly detect some construction or material quality problems. Normally, any water found in an edge drain is quite clear; therefore, a good video picture can be obtained even under water.

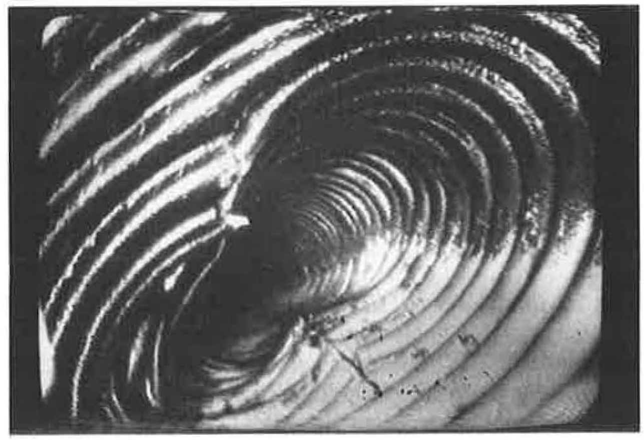


FIGURE 9 Collapsed subdrain.

The exposure of one "buried" edge-drain problem through the use of video evaluation equipment increases the effort to produce quality workmanship. The end result is an overall improvement in quality of edge-drain installation and performance.

Preliminary findings from edge-drain evaluations demonstrated the need for postconstruction inspection immediately following installation for all projects. This program was initiated in Iowa, and any problems found by this spot checking are corrected immediately by the contractor.

BENEFITS FROM RESEARCH

Video evaluation equipment applied to highway drainage systems can provide valuable information for design, construction, and maintenance engineers. Through the visual feedback given by a video evaluation, some design changes have been made to improve drain performance.

The video evaluation equipment used as a postconstruction inspection tool has disclosed a variety of construction problems or damaged drains. The exposure of problems through the use of video evaluations provides information that can assist the construction inspector and the contractor to ensure that the drains are being installed properly and will function as intended.

Maintenance personnel also found a variety of uses for video evaluation equipment. It can provide valuable information on culvert replacement requirements and causes for surface depressions or underground cavities around culverts and drains. The video camera can help find the exact location where a culvert or drain may be plugged or damaged and where excessive corrosion or joint separation has occurred. This information will help the maintenance engineer to make cost-effective, intelligent decisions for repairs based on accurate visual information through the video system.

The use of the video evaluation equipment for underwater inspection of bridge piers is limited. The visibility under water during one trial was encouraging. The water pressure limitation of the camera used (Cues, Inc.) was 15 psi or a depth of approximately 35 ft.

Specific benefits derived from this research cannot be calculated in terms of exact dollars. Information obtained from the video inspections and evaluations has played a part in changes in design and improvements in installation of edge drains. As a result, some improvement is expected in the overall performance and effective life of the edge drains and, in turn, in extended pavement life. Evaluations of culverts 14- to 30-in. in diameter have influenced maintenance and replacement decisions. It can be stated that the research project was cost-effective. The video evaluation equipment has more than paid for itself through internal views and information it provided concerning highway drainage systems. Some of these views were compiled into a 10-min videotape that is being used as an educational tool for design, construction, maintenance, and inspection personnel involved with highway drainage systems.

CONCLUSIONS

The research on video evaluation of highway drainage systems supports the following conclusions:

1. The video evaluation equipment can be used as an effective tool to obtain internal views in 4-in.-diameter edge-drain pipes, geocomposite edge drains, and small-diameter culverts.
2. Information obtained through video inspection of highway drainage systems aids the design, construction, and maintenance engineers with engineering decisions based on visual observations.
3. Video evaluations of edge drains have resulted in design modifications and improved construction inspection.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to the Highway Research Advisory Board for their recommendation of the research and to the Iowa Department of Transportation Highway Division for funding the project. Appreciation is also expressed to Richard Smith, Todde Folkerts, and Gary Harris for their assistance with field evaluations. Kathy Davis and Todde Folkerts were very helpful in preparing the paper.

Evaluation of Pumping Pavement on Interstate 77

JOHN S. BALDWIN

Prefabricated pavement edge drains have rapidly gained widespread acceptance as a workable means of preventing intrusive water accumulation on traditionally designed pavement structures. As found in West Virginia, however, the components of some pavement systems are so dense that effective rapid drainage is not possible without special consideration. This paper documents West Virginia's experience with pumping on a rehabilitated section of Interstate in which prefabricated pavement edge drains had been installed. The investigation of the problem is detailed, and recommendations from the study are outlined.

In an effort to increase the service life of West Virginia pavements, the Department of Transportation has elected to increase attention toward pavement drainage, partly by the use of prefabricated pavement edge drains in conjunction with major primary pavement rehabilitation work.

Although West Virginia has had only 5 years' experience with prefabricated edge drains, all installations until recently appeared to be working satisfactorily.

In March 1989 unusual sporadic staining of the paved shoulders of a rehabilitated section of Interstate 77 just north of Charleston was observed (Figure 1). Closer examination indicated that the staining appeared to emanate from the pavement-shoulder interface and to consist of fine soil particles that had pumped from the pavement substructure. A cursory inspection of the associated outlets indicated relatively good outflow, although similar staining was evident in the outwash.

INTERSTATE 77 REHABILITATION HISTORY

Rehabilitation of portions of Interstate 77 between Charleston and the Ohio River (approximately 80 mi to the north) was begun in 1984. Most of the rehabilitation involved the repair of the badly deteriorated portions of the original portland cement concrete (PCC) pavement, installation of underdrains, breaking and seating of the original pavement slabs, and rcpavment with 4 in. of new bituminous pavement.

Prefabricated edge drains were first used on this section of roadway in 1987. The rapid installation (Figure 2), relative low cost, and alleged superior performance made prefabricated pavement edge drains an attractive product to both the state and the contractors.

By the start of the 1989 construction season, nine separate rehabilitation projects on Interstate 77 had been completed in which prefabricated pavement edge drains were installed. Records indicate that only two brands of prefabricated pavement edge drains were used, namely, Hydraway (Monsanto) and AdvanEdge (Advanced Drainage Systems). Although similar in concept, both products exhibit radically different core designs. The Hydraway core (Figure 3) is basically composed of numerous flexible cylindrical projections or posts from a single flexible base. The AdvanEdge core (Figure 4) resembles a flattened corrugated plastic pipe with slit-type openings in the corrugations on both sides of the panel.

West Virginia Department of Transportation, Division of Highways, 312 Michigan Avenue, Charleston, W.Va. 25311.

FIELD REVIEW

A field review of all rehabilitated Interstate 77 projects north of Charleston revealed that the more obvious pumping was occurring on projects where AdvanEdge was used. Although the pavement sections drained by Hydraway and by aggregated-filled, fabric-wrapped trenches were found later not to be totally free from problem areas, the evidence of pumping was less frequent on those sections and was considerably less severe.

All prefabricated edge drains on Interstate 77, regardless of brand used, were installed 1 ft away from the edge of the pavement in the shoulder, as shown in Figure 5. The edge drain was placed at such a depth that its top 2 in. was on the same plane horizontally as the bottom 2 in. of the pavement slab. With a base course 6 in. deep, the bottom 4 in. (vertically) of edge drain was in the native soil. Consequently, because the material excavated from the trench was used as backfill, one-third of the backfill consisted of soil.

With FHWA an inspection was planned to determine the cause of the pumping problem. The work plan included borescoping of typical problem areas, the excavation of several test pits for a close look at the material involved, and an outlet evaluation.

BORESCOPE INVESTIGATION

Borescoping is essentially a nondestructive test that involves the insertion of a small camera lens and light source on the end of a probe into a small-diameter hole. The use of such an instrument allows a close examination of the flow channel of a pavement edge drain without a major excavation. A total of three problem sites drained by AdvanEdge were chosen for borescope investigation. All borescope observations revealed similar findings:

- The inner flow channel was open and did not appear to have been crushed.

FIGURE 1 Soil-stained shoulders.

FIGURE 2 Edge-drain installation.

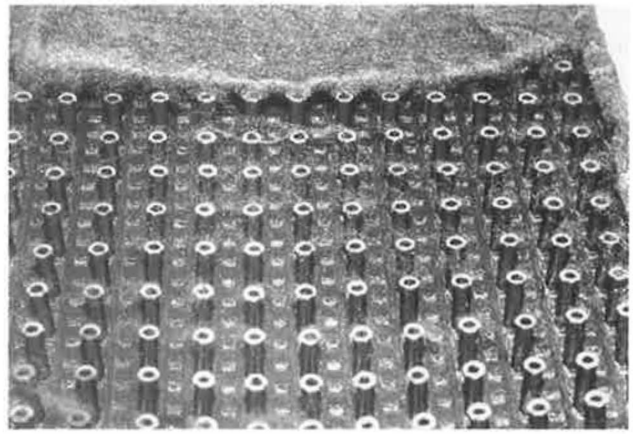


FIGURE 3 Monsanto Hydraway core.

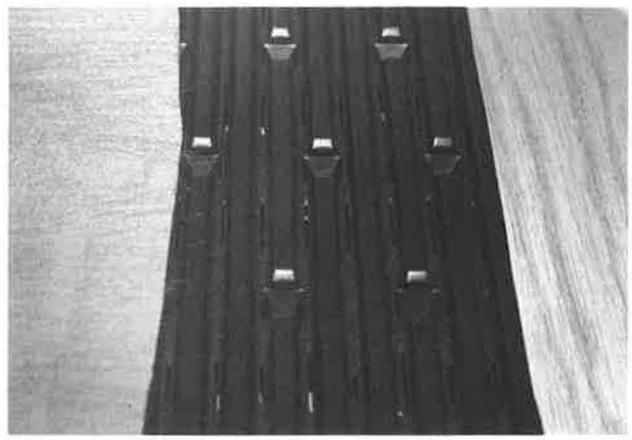


FIGURE 4 Advanced Drainage Systems AdvanEdge core.

- Up to 3 in. of sediment and standing or slowly flowing water was found in the bottom of the flow channel.
- Many of the slit openings, especially in the bottom two rows (of four rows), had been blinded with sediment.

TEST PIT EXCAVATION

Test pits were excavated in the shoulder adjacent to the pavement in three separate sections of Interstate 77:

1. Pumping AdvanEdge pavement,
2. Pumping Hydraway pavement, and
3. Functional (nonpumping) AdvanEdge pavement.

All test pits were excavated in the same manner, using a jackhammer and backhoe as well as manual labor. A trench 3 ft wide was excavated into the shoulder with the power equipment to a depth certain to be below the bottom of the edge drain. The power trenching was not allowed to come closer than 6 horizontal in. to the edge drain itself. Excavation

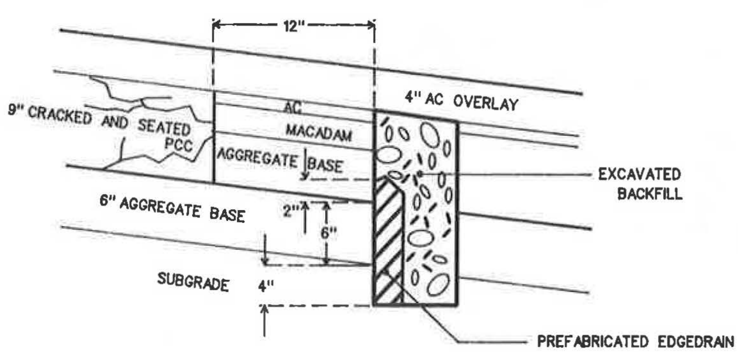


FIGURE 5 Original edge-drain installation design.

by hand was then utilized to expose the edge drain as well as the material between the edge drain and the pavement. Detailed observations, measurements, and samples of the base course, subgrade, backfill, and edge drain were collected from each test pit.

Test Pit 1 (Pumping AdvanEdge)

As shown in Figure 6, Test Pit 1 was excavated in an area that not only showed the most severe signs of pumping, but was still moist from pumping as the excavation was begun, even though it had not rained in several days. Although considerable information was collected, the most significant had to do with the base course and the soil contamination of the edge drain.

The limestone base course used on this project was extremely dense, so much so that it had to be chiseled out of the test pit with a rock hammer (Figure 7). Although the area was moist, no free water was found in the base course on either side of the edge drain. When the base course was removed from the side of the original pavement slab, however, the crack-and-seat operation caused water to begin flowing from the fractures in the pavement (Figure 8). The base course essentially acted as a dam that kept water from reaching the edge drain. Although the original gradation of the base course was not conducive to good permeability, what permeability it had may have been lessened by the addition of pulverized concrete matrix fines from the crack-and-seat operation.

Soil contamination of the edge drain appeared to have a substantial effect on the ability of water to reach the internal flow channel of the AdvanEdge core (Figure 9). It is believed that most of this soil came from the backfill placed during the installation of the edge drain. Examination of the excavated sample revealed that less than 50 percent of the core wall slits remained open. The remainder of the slits were either partially blinded or totally plugged with soil sediment.

Test Pit 2 (Pumping Hydraway)

Earlier field reviews revealed only one small section of pavement (100 to 150 ft) drained by Hydraway that was showing signs of pumping. Although a test pit was excavated in the approximate center of the pumping area, it appeared that the area was located on a slight vertical curve that had not been provided with an outlet. Confirmation of this suspected cause of pumping was made when the test pit was opened. As the

FIGURE 6 Site of Test Pit 1: severe staining of shoulder.

FIGURE 8 Free water from cracked pavement slab, Test Pit 1.

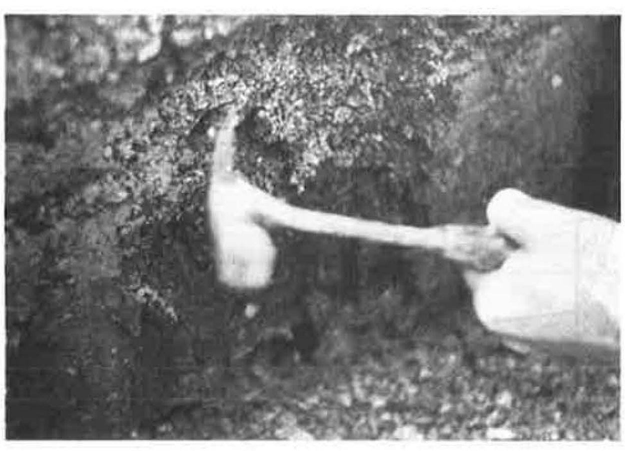


FIGURE 7 Hand excavation of base course, Test Pit 1.

FIGURE 9 Soil-contaminated AdvanEdge core, Test Pit 1.

hand digging got close to the edge drain, water began to flow rapidly from it, washing much of the backfill away from the shoulder side of the drain (Figure 10).

The base course in this section was a mixture of sand and gravel. The gradation appeared to be more conducive to drainage than that encountered in excavation of the pumping AdvanEdge section.

Deformation of the Hydraway core was apparent and is shown in Figure 11. The top row of the support columns was bent downward about 30 degrees and the bottom two rows were bent upward about 45 degrees. In addition, where the edge drain had intersected the base-subgrade interface, there was distortion along a horizontal plane that caused an offset of as much as ½ in. between the top and bottom of the product. Although deformed, it appeared that the edge drain would perform satisfactorily if given a proper outlet. Consequently, before the test pit was closed, a trench was cut into the shoulder that linked the test pit with the side of the embankment. The trench was lined with engineering fabric and backfilled with pea gravel. Recent observations indicate that pumping of the shoulder has subsided.

Test Pit 3 (Functional AdvanEdge)

Excavation of this section revealed a sand-and-gravel base that appeared to be relatively permeable. Although the backfill contained appreciable amounts of soil, there apparently was enough water flow to flush the soil fines from the slit openings into the core's internal flow channel. Evaluation of the core indicated that as much as 93 percent of the slits was completely open.

Outlet Examination

All outlets examined were constructed of 4-in. corrugated plastic pipe. The outlet openings were encased in a concrete headwall as shown in Figure 12. All outlets associated with pumping pavement appeared to be functioning, although soil staining of the outwash was apparent. Occasionally, an outlet was encountered that appeared to have a slight reverse grade, which may have slowed the drainage process. In addition, a calcium carbonate crystalline growth was observed on several of the rodent screens and in the corrugations on the bottom of the outlet pipes.

FIGURE 10 Water flow from hydraway, Test Pit 2.

FIGURE 11 Hydraway deformation, Test Pit 2.

CONCLUSIONS

As shown in Figure 13, the staining of the shoulder is the result of water infiltrating the asphalt overlay into the cracked and seated PCC pavement slab. There it travels through the fractures in the slab to the pavement-shoulder interface. At that point, if anything impedes the flow of water to the surface drainage, it is essentially trapped in the fractures of the pavement. The main flow problem appears to be the very dense base course that surrounds the pavement and the location of the edge drain.

As shown in Figure 14, much of the base course used on Interstate 77 is very dense, almost to the point that it is impermeable. Because the edge drain was placed 1 ft out into this base course, free flow of water to the surface drainage in many locations is not possible. Under the circumstances, free water and suspended fine particles have only one way to go under traffic loadings—up through the asphalt and onto the pavement surface. In an effort to reduce, if not eliminate, the same problem on future rehabilitation projects, the following recommendations have been made:

FIGURE 12 Typical outlet opening.

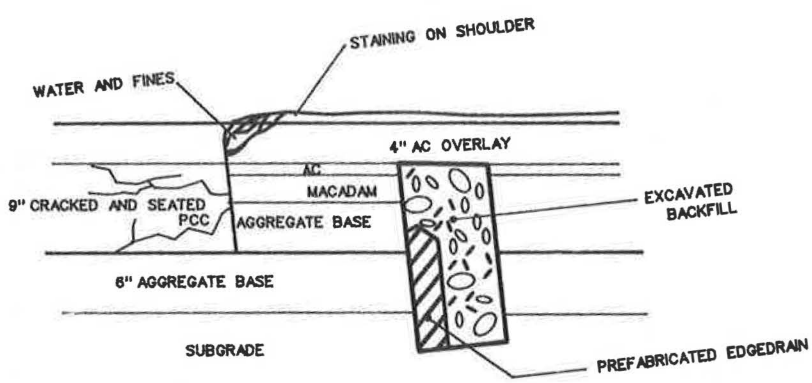


FIGURE 13 Origin of Interstate 77 shoulder staining.

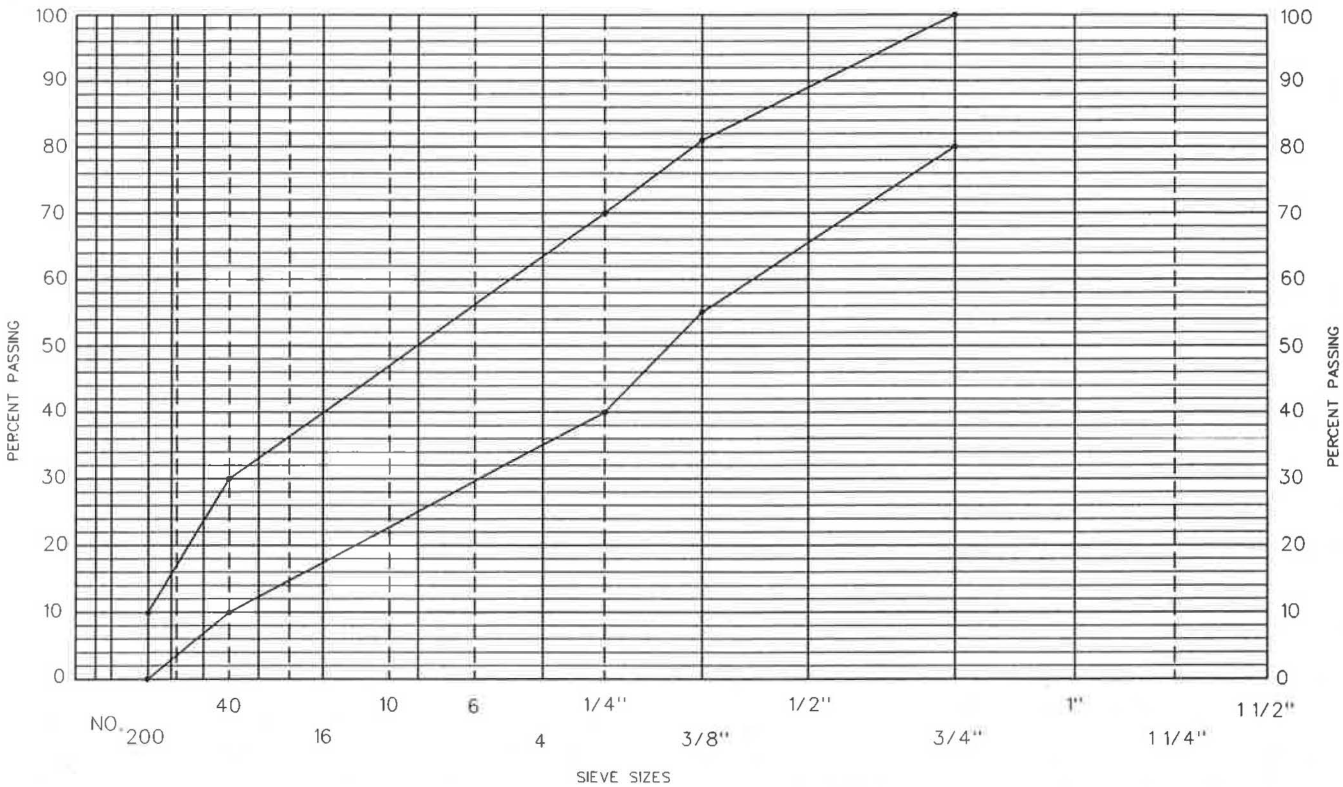


FIGURE 14 Gradation range used for original base course (sieve sizes raised to 0.45 power).

1. Placement of the edge drain should be done in such a way that a large portion of it is in direct contact with the cracked and seated pavement. This can be accomplished by installing the edge drain immediately adjacent to the pavement slab at a depth not to exceed 1 in. into the soil subgrade, as is shown in Figure 15. Elevating the edge drain will also reduce the amount of soil particles available to clog the drain.

2. Outlets should be constructed with smooth-walled rigid pipe to help ensure that the proper outlet grade is maintained.

3. Asphaltic concrete compaction requirements should be tightened to reduce the amount of moisture available to the pavement structure.

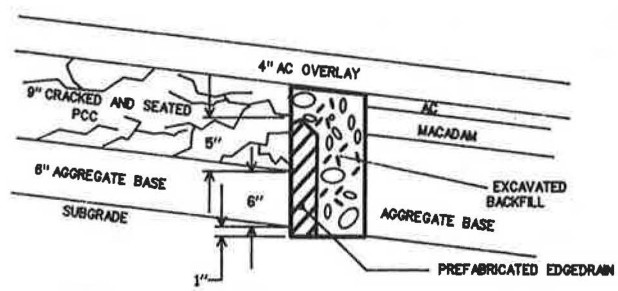


FIGURE 15 Recommended prefabricated edge-drain placement.

Prefabricated Pavement Base Drain

KEITH L. HIGHLANDS, ROBERT TURGEON, AND GARY L. HOFFMAN

The objective of this research project is to evaluate and compare the construction, effectiveness, and cost of a geocomposite prefabricated pavement edge-drain system versus the Pennsylvania Department of Transportation (PennDOT) standard geotextile-wrapped, aggregate and pipe, filled trench pavement base drain system. Generally, PennDOT's standard edge drain appeared to outperform the geocomposite edge drain. Although the flow data collected were inadequate to be conclusive, the flows measured from the geocomposite edge drain were consistently less than the flows measured from PennDOT's standard edge-drain system. Upon investigation by excavation, areas were found where the geocomposite edge-drain core was clogged with fines. A gradation analysis indicated that PennDOT's standard edge-drain system was not significantly infiltrated by fine soil particles from the base or subgrade. The geocomposite properties were inadequate either to prevent fine soil particles from entering the core or to expel the fines from the system after they had entered the core. The conclusion is that the prefabricated geocomposite was inappropriately used on a site where severe dynamic conditions precluded its proper function. Consequently, more restrictive product properties or application criteria need to be developed to ensure future success under similar field conditions.

The objective of this study is to evaluate and compare the construction, effectiveness, and cost of a prefabricated pavement base drain versus the standard pavement base drain system of the Pennsylvania Department of Transportation (PennDOT). There were three experimental installation sites of prefabricated pavement base drain on this research project. This paper details the evaluation at the first site constructed, located on Interstate Highway 70 in Washington County. For approximately 19 mi of roadway, prefabricated pavement base drain was installed along the westbound lanes and PennDOT's standard pavement base drain system was installed along the eastbound lanes. The prefabricated and standard pavement base drain systems were installed as part of a highway rehabilitation project during summer and fall 1984.

CONSTRUCTION

From July to November 1984, 21,481 linear ft (L.F.) of PennDOT's standard pavement base drain and 20,223 L.F. of geocomposite prefabricated pavement base drain were installed at this research site. A detailed description of the sequence of construction operations, as well as construction problems encountered related to the pavement base drain system installation, may be found in the Construction Report (*1*).

The bid price was $5.50/L.F. for PennDOT's standard pavement base drain and $3.70/L.F. for the prefabricated pavement base drain.

Bureau of Bridge and Roadway Technology, Pennsylvania Department of Transportation, Harrisburg, Pa. 17120.

POSTCONSTRUCTION TESTING AND EVALUATION

Following installation, the flow carried by each of the pavement base drain systems was measured during a 3 year period. The project site was also periodically inspected to identify any pavement or shoulder distress related to the construction or performance of the drainage systems.

The flow was monitored along the westbound and eastbound lanes with tipping buckets. The sites selected for installation of the tipping buckets were generally similar. Similar sites were selected to allow a relatively equal direct comparison of the flows measured by the tipping buckets. Approximately 500 ft of prefabricated pavement base drain led to the outlet that emptied into the tipping bucket along the westbound lanes, and 500 ft of PennDOT's standard pavement base drain led to the outlet that emptied into the tipping bucket along the eastbound lanes. The highway grade along the two sections of drainage that led to the tipping buckets also appeared similar.

The flow data collected generally indicated that slightly more than one to three times more water consistently flowed from the standard pavement base drain system along the eastbound lanes than from the prefabricated pavement base drain along the westbound lanes. Exact flow rates cannot be calculated from these data because the flows were not constant or continuous. The data are contradictory to flow comparisons made by a number of other state DOTs whose standard pipe-and-backfill systems use sand as the backfill material. The AASHTO No. 8 aggregate backfill in PennDOT's standard system allows higher flows, but this set of data was gathered during a brief period of time and should not be given too much importance.

The project site was periodically inspected to identify any pavement or shoulder distress related to the construction or performance of the drainage systems. One of the first problems observed was subsidence of the prefabricated pavement base drain. This subsidence was most likely a result of inadequate trench backfill compaction. The importance of properly compacting trench backfill is discussed in the previous section.

Since the 1984 rehabilitation, several pavement areas along this 5-mi roadway section have exhibited distress. By mid-1988, fines on the roadway shoulder indicated that extensive pumping had occurred in several areas of this research project site. Subjectively, it appeared that more pumping had occurred along the westbound lanes. The extensive pumping raised questions as to how well the pavement base drain systems were performing. In July 1987, a borescope inspection into the internal core of the prefabricated pavement base drain installed near the western end of this project had revealed that the core was not clogged and was carrying water in a particular area examined near the western end of the project. However, no pumping was observed along the roadway in the area where the July 1987 borescopings were done, so this inspection gave no real assurance that problems did not exist with the pavement base drain where the pumping was occurring. It was decided that portions of the pavement base drain systems should be excavated and examined to identify how their performance may have influenced the pumping observed in other areas along the research project site.

On June 10, 1988, personnel from the Bureau of Bridge and Roadway Technology (BART) and PennDOT Engineering District 12-0, Design and Maintenance, inspected the site conditions. The Assistant County Maintenance Manager pointed out problem areas and described site conditions. He said that his crews had found wet, muddy material under slabs when they made repairs.

In several areas during the June 10, 1988, inspection, the westbound and eastbound roadway shoulders were stained with fine material pumped from under the pavement. Considering these observations, three locations were selected for excavation of prefabricated pavement base drain and standard pavement base drain materials to determine their possible influence on the observed pumping.

Two locations along the westbound lanes where fines on the shoulder indicated that pumping had occurred were selected for excavation and examination. A third location along the westbound lanes was selected where no pumping or pavement distress was evident.

The first excavation area was approximately at Station 135 + 10, where stains along the shoulder indicated that pumping had occurred. When the trench was excavated, water approximately 1 ft deep was found between the prefabricated pavement base drain and the pavement subgrade. Once a section of prefabricated pavement base drain had been cut free and removed from the trench, its core was found clogged with fine-grained soil throughout the entire height of the drain.

After the prefabricated pavement base drain had been removed, soil samples were taken from both the pavement and shoulder sides of the prefabricated pavement base drain for gradation analysis. Approximately 10 ft of prefabricated pavement base drain was removed and replaced at each of the three excavation locations, even though the adjacent prefabricated pavement base drain was mostly to entirely filled with fines. At all three excavation locations, soil samples were taken adjacent to the prefabricated pavement base drain and the shoulder was rebuilt.

The second excavation location was approximately at Station 134 + 50 westbound. Fines on the shoulder indicated the occurrence of pumping at this location. Once excavated, the prefabricated pavement base drain was also found to be totally filled with fines.

The third excavation location was approximately at Station 107 + 00 westbound. No pumping was noted at this location during the June 1988 research project inspection to select locations for excavation or during July 1988 when the prefabricated pavement base drain was excavated in this area of the research project site. Once excavated, the prefabricated pavement base drain was also found to be mostly clogged. The results of the gradation analyses of the soil samples taken adjacent to the prefabricated pavement base drain at this and the other two excavation locations are shown in Table 1.

TABLE 1 GRADATION ANALYSES OF MATERIAL ADJACENT TO PREFABRICATED PAVEMENT BASE DRAIN

Sieve Size	Percent Passing by Wt
2 in.	100
1–½ in.	100
¾ in.	78–98
⅜ in.	60–77
No. 4	48–61
No. 10	38–49
No. 40	30–40
No. 100	22–32
No. 200	16–24
0.02 mm	11–17
0.002 mm	4–9

The core of the prefabricated pavement base drain at the third excavation location was filled with fine-grained soil approximately 14 in. deep in the roughly 18-in.-high drain. Because no pumping had been evident before excavation, finding the prefabricated pavement base drain clogged at this location on the project raised the question of how much more of the prefabricated pavement base drain was clogged, even in areas where no pumping had yet occurred.

A gradation analysis was done by PennDOT's Materials and Testing Division on nine soil samples taken from the material found clogging the cores of the prefabricated pavement base drain sections excavated. Three samples were taken from the prefabricated pavement base drain excavated at each of three locations. The results of these gradation analyses are shown in Table 2. Six of the samples, which were removed from the prefabricated pavement base drain excavated near Stations 107 + 00 and 134 + 50, were near the fine end of the gradation limits shown in Table 2, with at least 96 percent passing the No. 200 sieve. The three samples taken from material clogging the prefabricated pavement base drain excavated near Station 135 + 10 were near the coarser end of the gradation limits shown in Table 2, with only a minimum of 35 percent passing the No. 200 sieve.

Figure 1 shows two views of the excavated prefabricated pavement base drain without part of the geotextile that wraps the geocomposite core. These photographs illustrate the extent of clogging found in the prefabricated pavement base drain.

The apparent outlet for the first two excavation sites was found approximately at Station 140 + 50 and located at the second inlet down from where the first two clogged sections of prefabricated pavement base drain had been removed. After excavation it was found that the outlet cap of the prefabricated pavement base drain at this location was tilted upward and

TABLE 2 GRADATION ANALYSES OF SOIL FOUND CLOGGING PREFABRICATED PAVEMENT BASE DRAIN

Sieve Size	Percent Passing by Wt
No. 4	100
No. 10	99–100
No. 40	96–100
No. 100	73–100
No. 200	35–100
0.02 mm	13–73
0.002 mm	5–28

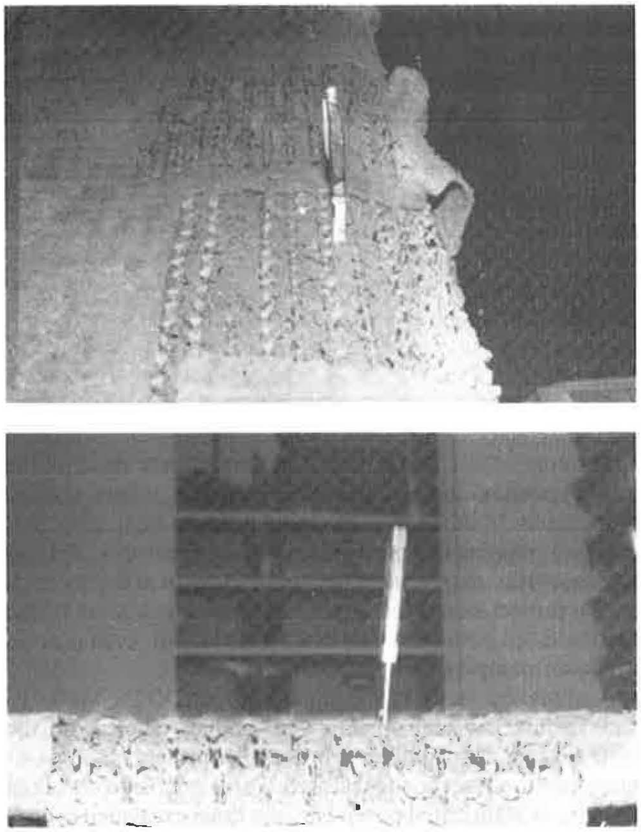


FIGURE 1 Clogged prefabricated pavement base drain (two views).

the cap adapter was partially crushed. This crushing may have been caused by tamping equipment used during trench backfill compaction. The corrugated polyethylene outlet pipe was disconnected from the cap adapter.

The relative elevations of the outlet pipe invert at the outlet cap and outlet end were checked with a lock level and found to be approximately equal. This indicates a flat slope of the outlet pipe, which would not adequately transmit the water away from the pavement. Obviously there were construction-related problems with this particular outlet that could have impeded the flow of water from the pipe.

At this inlet, the prefabricated pavement base drain was clogged within about 1 in. of its top and appeared to run continuously past the inlet. The prefabricated pavement base drain excavated in this area during July 1988 was only clogged approximately 14 in. deep in the core. This possibly indicates that the prefabricated pavement base drain filled with approximately 4 in. of fine-grained soil in that 9-month period. The trench was not excavated immediately adjacent to where the section of prefabricated pavement base drain was removed during July 1988, so it is not possible to determine definitively whether this additional clogging occurred between July 1988 and March 1989.

If the prefabricated pavement base drain became totally filled in this area during this 9-month period, it could explain why fines were found pumped onto the shoulder in March 1989 when no signs of pumping had been found in June 1988. Alternatively, if the prefabricated pavement base drain core was not totally filled, additional fines could still possibly be jetted into the core of the prefabricated pavement base drain. Once the core was filled, additional fines could no longer be jetted into the core and the pore-water pressure buildup might then pump eroded fines onto the roadway shoulder.

The crew then excavated at the inlet near Station 110+00 and found the outlet for the section of prefabricated pavement base drain excavated in 1988. Other than a small amount of soil found near the coupling to the fitting at the end of the prefabricated pavement base drain, the outlet was clear and functional.

The prefabricated pavement base drain itself did not appear clogged near this outlet. This would seem to indicate that an improperly constructed outlet was not the cause of clogging in this area, because water restricted by a nonfunctioning outlet would first back up in the prefabricated pavement base drain near the outlet. If water standing in the prefabricated pavement base drain because of a nonfunctioning outlet caused the core to clog, the core clogging logically would have occurred first near the nonfunctioning outlet.

The maintenance crew excavated the standard pavement base drain trench approximately at Station 103+00 according to site station puddles along the eastbound lanes. Pumping had previously been observed in this area. Two pieces of the Class 1 geotextile wrapping the trench were taken, and the grab tensile strength and percent elongation were determined.

The geotextile test results exceeded PennDOT's specification limits of 90 lb grab tensile strength and 20 percent elongation. No laboratory evaluation was done to determine the amount of clogging in these samples of geotextile, but clogging could have been a problem that contributed to the pumping problem in the area where the geotextile had been removed.

A sample bag of aggregate was taken from the pavement base drain trench for gradation analysis. The sample did not visually appear clogged with an excessive amount of fines. When tested, the AASHTO No. 8 stone removed from the trench was within acceptable gradation limits set forth in PennDOT's specifications (Table 3). Pumped fines had previously been noted on the shoulder in the vicinity of the excavation along the eastbound lanes; however, no pumped fines were clearly evident just before excavation.

As shown in Table 3, the gradation of the AASHTO No. 8 stone removed was not close to the fine end of the gradation limits. This corroborates the subjective judgment that an ex-

TABLE 3 GRADATION LIMITS AND ANALYSIS OF AASHTO NO. 8 COARSE AGGREGATE

	Percent Passing by Weight	
Sieve Size	PennDOT Specified Gradation of No. 8 Coarse Aggregate	Gradation of No. 8 Coarse Aggregate Removed from Project
¾ in.	100	100
½ in.	100	99
⅜ in.	85–100	94
No. 4	10–30	19
No. 8	0–10	3
No. 16	0–5	2
No. 200	2 max.	1

cessive amount of fines had not infiltrated and been retained in the standard pavement base drain system.

OBSERVATIONS

On this project, the prefabricated pavement base drain cost \$1.80 per L.F. less than PennDOT's standard pavement base drain system. This initial cost savings is an incentive to use prefabricated pavement base drains instead of the standard perforated plastic pipe and geotextile-wrapped aggregate trench drainage system. Nevertheless, if prefabricated pavement base drains are not as effective as PennDOT's standard pavement base drain, damage caused to the roadway by standing water could cause damage most costly than the \$1.80 per L.F. saved by using the prefabricated system.

A report by the California Department of Transportation indicated:

> For rigid pavement, the work of Darter, et al. the results of the California edge drain study and the performance of the retrofit edge drain system on the Valencia-Tarrogona Toll Road, suggest a minimum 50% extension of service life of PCC pavements with an efficient, functioning edge drain system. (*2*)

The California study further indicated that for a roadway consisting of a 0.85-ft-thick PCC pavement over a 0.50-ft-thick asphalt-treated permeable base and a 0.70-ft-thick aggregate subbase and edge drains with outlets there is an achievement of "a 35% annual savings in rigid pavement costs over the service life of the pavement due to its drainage features" (*2*).

The initial costs of prefabricated pavement base drain and PennDOT's standard pavement base drain system are substantially different, but it should be kept in mind that the prefabricated pavement base drain system and PennDOT's standard pavement base drain system are two substantially different drainage systems. Both use a geotextile fabric to filter out fines, but they use different designs to collect and transmit water. The aggregate used in the standard pavement base drain trench also may have varying levels of effectiveness as a filter medium depending on the characteristics of the material being filtered. AASHTO No. 8 stone will not be an effective filter for some Pennsylvania soil types.

There is reason to assume that these different systems will also provide some different advantages and disadvantages, a better understanding of which can be obtained by examining some of the characteristics of the materials used in the drainage systems and how these characteristics may affect performance.

The gradation limits of the AASHTO No. 8 stone used in PennDOT's standard pavement base drain are presented in Table 3, as well as the gradation band for the subgrade materials excavated adjacent to the prefabricated pavement base drain.

The geotextiles wrapping the AASHTO No. 8 stone in PennDOT's standard pavement base drain system and in all prefabricated base drain systems are referred to by PennDOT as Class 1 geotextile. PennDOT specifications require Class 1 geotextiles to have an apparent opening size (AOS) ≥ No. 40 sieve or in other words to have an AOS ≤425 mm.

The filter fabric wrapping the prefabricated pavement base drain core has a typical minimum average AOS equal to a No. 70 sieve, which corresponds to a sieve opening of 0.212 mm.

An FHWA publication recommends geotextile design and selection criteria for soil retention or piping resistance (*3*). For projects on soils with <50 percent of material passing the No. 200 sieve; dynamic, pulsating, and cyclic flow; and pumping conditions in which individual soil particles are eroded by dynamic flow and jetted into the geotextile, the FHWA contracting officer's technical representative for their *Geotextile Engineering Manual* indicated that the following criterion is appropriate (*3*, p. 3–29; Jerry DiMaggio, unpublished data): 095 ≤ D15.

The D15 value represents the diameter of soil particles at 15 percent fines by weight on the material gradation curve. DiMaggio also emphasized that laboratory test results obtained by modeling specific field conditions should provide more correct design and selection criteria than the 095 ≤ D15 criterion.

The FHWA's *Geotextile Engineering Manual* provides different criteria for steady-state flow conditions than those presented above, but the movement of rigid pavement slabs under traffic loadings would most likely cause dynamic flow conditions.

The material sampled from the trench backfill on the shoulder side of the prefabricated pavement base drain is probably similar to the special subgrade sampled from under the pavement along the prefabricated pavement base drain, because the trench was backfilled with material that had been excavated to allow installation of the prefabricated pavement base drain.

As shown in the following calculations, neither the core of PennDOT's Class 1 geotextile nor that of the fabric-wrapped prefabricated pavement base drain meets FHWA's soil retention design criteria for dynamic flow conditions when required to filter the finest side of the gradation band of material, which is similar to the special subgrade sampled adjacent to the prefabricated pavement base drain on this project.

As shown in the following calculations, AASHTO No. 8 stone meets these criteria from gradation curves for a filter immediately adjacent to the prefabricated pavement base drain:

D5,#8 = 1.2 mm to 3.8 mm,
D10,#8 = 2.3 mm to 4.8 mm,
D15,#8 = 3.2 mm to 5.3 mm,
D50,#8 = 5.9 mm to 7.3 mm,
D60,#8 = 6.5 mm to 7.9 mm,
D85,#8 = 8.2 mm to 9.6 mm,
D15,ss = 0.012 mm to 0.039 mm,
D50,ss = 2.2 mm to 5.5 mm,
D85,ss = 13 mm to 24 mm.

Prefabricated pavement base drain:
Max. D15, ss = 0.039 mm,
Min. D15, ss = 0.012 mm,
095 ≤ D15, ss to meet criteria,
0.212 mm is not ≤ 0.012 mm.
Class 1 geotextile:
Max. D15, ss = 0.039 mm,
Min. D15, ss = 0.012 mm,

095 $\leq$ D15, ss to meet criteria,
0.212 mm is not $\leq$ 0.012 mm.

Both geotextiles meet the FHWA *Geotextile Engineering Manual* criteria for steady-state flow that the AOS be less than or equal to the D85 (15 to 26 mm) value of the soil to be filtered, but, as discussed earlier, the traffic loadings on rigid slabs would most likely cause dynamic flow conditions. As shown by the pumping, there was quite a bit of slab movement on this project, which would have built pore pressures and have jetted water and fines from under the pavement toward its edge.

FHWA described the following criteria for design of protective granular filters (*4*, p. 98):

D15, No. 8 $\leq$ 5 (D85,ss),
D15, No. 8 $\geq$ 5 (D15,ss),
D50, No. 8 $\leq$ 25 (D50,ss),
D5, No. 8 $\geq$ 0.074 mm,
Cu, No. 8 = D60, No. 8/D10,#8 $\leq$ 20.

Comparing the AASHTO No. 8 stone with the FHWA criteria results in the following:

D15, No. 8 $\leq$ 5 (D85, ss),
4.9 mm $\leq$ 5 (13 mm),
4.9 mm $\leq$ 65 mm meets criteria.

D15, No. 8 $\geq$ 5 (D15, ss),
3.1 mm $\geq$ 5 (0.039 mm),
3.1 mm $\geq$ 0.20 mm meets criteria.

D50, No. 8 $\leq$ 25 (D50, ss),
7.3 mm $\leq$ 25 (2.3 mm),
7.3 mm $\leq$ 57.5 mm meets criteria.

D5, No. 8 $\geq$ 0.074 mm,
1.2 mm $\geq$ 0.074 mm meets criteria.

Cu, No. 8 = D60, No. 8/D10, No. 8 $\leq$ 20,
7.9 mm/2.3 mm = 3.2 $\leq$ 20 meets criteria.

Therefore, by these criteria, the AASHTO No. 8 stone used in PennDOT's standard pavement base drain system is an acceptable filter medium for the gradation of special subgrade material excavated from this research project site. The standard pavement base drain system effectively has a two-filter system. In this particular instance, the Class 1 geotextile wrap around the standard base drain system may not have been necessary.

The facts that the core of the prefabricated pavement base drain was found clogged in the three sections excavated and that the geotextile wrapping the prefabricated pavement base drain was shown not to meet the FHWA *Geotextile Engineering Manual* criteria for soil retention or piping resistance under dynamic flow conditions raise questions as to whether the core clogging caused by the jetting of fines through the geotextile could occur at other locations where extreme pumping conditions exist.

It is not a problem for some fines to move through the prefabricated pavement base drain fabric and into the core if these fines can be flushed out the drainage outlet. However, if there are problems that restrict the flow through the geocomposite or there simply is not enough flow velocity to carry away these fines piped through the fabric, the core will eventually clog.

It is difficult to estimate to what extent the core of the prefabricated pavement base drain near Stations 134 + 50 and 135 + 10 would have been clogged if the outlet serving those areas had been totally functional. A crushed, clogged, or otherwise nonfunctioning outlet could restrict the flow of water through the geocomposite, preventing fines from being carried through the geocomposite and discharged from the outlet. However, the prefabricated pavement base drain core itself was not clogged near the partially crushed outlet for these first two prefabricated pavement base drain sections excavated in July 1988, indicating that the outlet was probably not the only factor influencing the clogging. If restricted flow caused by a nonfunctioning outlet had been the sole problem, it seems probable that there would have been core clogging in the entire prefabricated pavement base drain system upstream from the restriction.

It is also doubtful whether nonfunctioning outlets were the only reason for the clogging of the core, because extensive clogging was found in the prefabricated pavement base drain section excavated near Station 107 + 00. The outlet serving this section appeared functional, and the prefabricated pavement base drain near this outlet did not appear clogged. It appears that there must have been restrictions in the prefabricated pavement base drain core or the invert slope of 3 percent was too low to provide sufficient flow velocities for the system to cleanse itself.

Industry obviously realizes the dynamic flow conditions to which geocomposite edge drains are subjected. In Monsanto's report to PennDOT to describe the performance of prefabricated pavement base drains under dynamic loadings (*5*) a paper entitled "A Dynamic Test to Predict the Field Behavior of Filter Fabrics Used in Pavement Subdrains" (*6*) is provided as an appendix. In this paper, Janssen states that it was "very likely" that fines were pumped through the fabric.

Janssen states:

> It is felt that a graded filter structure was being built up adjacent to the fabric as fines migrated down through the sample. At 300,000 loads this structure collapsed, causing a rapid decrease in permeability. From here the permeability again gradually decreases, possibly caused by the accumulation of fines adjacent to the fabric.
>
> At about 675,000 loads the permeability suddenly increased. Prior to that, at about 650,000 loads, the water again appeared cloudy. It is very likely that the high hydraulic gradient right above the filter along with the stretching of the fabric pores caused piping of the fines through the fabric. This gives the appearance of a "self-cleaning" action. The wide fluctuations in permeability between 675,000 and 700,000 loads may possibly have been caused by soil structure collapse followed by more soil piping. (*6*, p. 11)

The above excerpt from Janssen's paper refers to a graded soil filter. The FHWA *Geotextile Engineering Manual* describes the formation of a soil filter to act in conjunction with a geotextile as follows:

> As fine soil moves through the fabric, larger particles may combine to bridge the apertures of the fabric. Immediately

> behind this bridging zone is another zone consisting of soil particles whose permeability decreases with distance from the geotextile. . . . In the past, this zone has been termed "filter cake" or "soil filter." The zone behind the soil filter is actually the undisturbed existing soil. Once the soil filter zone has been established, no further soil is washed through the system and the system is considered to be in equilibrium. The soil filter zone is, in effect, a reverse granular filter constructed solely from the in situ soil particles. (*7*)

The key phrase in the above discussion is "considered to be in equilibrium." Apparently, in Janssen's testing, the dynamic forces were enough to destroy this equilibrium and cause a breakdown of the soil filter as additional fines were passed through the fabric.

CONCLUSIONS

Although geocomposites show great promise in drainage applications and have been used successfully on many pavements, the problems reported on this project raise concerns that must be addressed. On this project, prefabricated pavement base drains do not appear to perform as well as PennDOT's standard pavement base drain. Although the flow data collected appear to be inadequate to be considered conclusive, the flows measured from the prefabricated pavement base drain were consistently less than the flows measured from PennDOT's standard pavement base drain. It appears that slightly more trench subsidence and pumping took place along the prefabricated pavement base drain than along the standard pavement base drain system.

Core clogging on I-70 was believed to be caused by the AOS of the geotextile, which was too open to retain the high fines percentage under the dynamic loading conditions. Once the fines had entered the core, they may not have been flushed out because of the small amount of water entering the core or blockage of the core downstream, which did not allow for adequate flow velocities.

It should be kept in mind that the crushed outlet pipe, the finer-than-normal subbase material, and the harsh pavement pumping conditions influenced some of the less-than-acceptable performance of prefabricated pavement base drains on this project.

From an economic viewpoint, the initial bid cost of prefabricated pavement base drains is less than that of PennDOT's standard pavement base drain system. However, the cost of roadway damage caused by water left under the roadways could far exceed initial cost savings obtained by using the prefabricated pavement base drain.

When the costs of either type of pavement base drain evaluated in this paper are considered relative to the total cost of a roadway and the damage that water in pavement structures can lead to, it seems that if either type of drainage system does not adequately drain the roadway or does not maintain its effectiveness over the life of a roadway, using that type of pavement base drain system will not be a truly cost-effective alternative. Therefore, from an economic standpoint, cost over performance life rather than initial cost should be stressed the most in evaluating the type of pavement base drain system that should be installed on PennDOT projects. Maintenance and periodic replacement costs for nonfunctioning drains must be factored into the life-cycle cost analysis.

The tendency of the prefabricated drain to clog where pumping dynamic conditions exist along rigid pavements should be studied further. In particular, the fabric opening size characteristics need to be compared with the percentage of fines being retained in the soil.

IMPLEMENTATION

On the basis of the findings and recommendations of this evaluation and on PennDOT Research Project 88-15 performed at Drexel University, generic specifications for prefabricated drains were developed and the outlet pipe specifications were changed (*8*):

1. The core strength load deflection requirement was changed from a maximum 20 percent strain at 20 psi to a crush strength of 40 psi minimum per the Geosynthetic Research Institute at Drexel University's Test GRI-GC4.
2. The core flow rate must be at least 15 gal/min-ft, per ASTM-D4716.
3. The core must permit unobstructed flow through 50 percent of the fabric area on the pavement side of the pavement base drain, and 20 percent of the fabric area on the shoulder side face was added.
4. The AOS specification of the geotextile fabric was changed from a U.S. Sieve No. 70 per CW-02215 to a U.S. standard Sieve No. 70 minimum per ASTM-D4751.
5. The permeability required of the geotextile was changed from 0.2 cm/sec per PTM No. 314 to 0.001 cm/sec per ASTM-D4491.
6. The minimum width of the trench was specified to be the thickness of the pavement base drain plus 1 in.
7. The prefabricated pavement base drain is to be placed on the shoulder side of the trench instead of on the pavement side.
8. Fine aggregate backfill is to be placed on the pacement side of the trench instead of excavated material being placed on the shoulder side of the trench.
9. Outlets are to be solid pipe with a minimum stiffness of 45 psi at 5 percent deflection.
10. Outlets are to be installed within 24 hr after the beginning of trenching for installation of a given section of pavement base drain.

ACKNOWLEDGMENTS

Sincere appreciation is expressed to PennDOT Engineering District 12-0 personnel for their considerable efforts during this evaluation. Washington County maintenance personnel were particularly helpful. Appreciation is also expressed to PennDOT's Materials and Testing Division for their soil analysis testing.

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Evaluation and Performance of Geocomposite Edge Drains in Kentucky

David L. Allen and John Fleckenstein

Longitudinal edge drains have been used in Kentucky for approximately two decades. Most were installed on the Interstate and parkway systems. Currently there are hundreds of lane-miles in place. The first edge drains consisted of 4-in., perforated, polyethylene pipe installed in a 12-in.-wide trench. Various types of backfill were used. Some trenches were fabric wrapped and backfilled with crushed limestone aggregate of uniform size (approximately ¾ in.). Other trenches were not wrapped and were backfilled with a natural sand. In these cases, the pipe was covered with a fabric sock. This paper deals mostly with geocomposite (panel) edge drains, because most of the research and performance monitoring in Kentucky in the past 6 years has been on that type of drain. Much attention should be given to details during installation and construction of the panel drains. Many problems encountered were the result of improper construction practices. Problems included compression of the inner core of the panel drain during compaction of the backfill, damage to flexible outlet pipes during construction, and improper drainage at the outlet. Outlet pipes should be installed at the proper grade. This helps maintain the velocity of the water in the drains, which helps to flush out silt and clay-size particles that enter the drain. The drains and outlet pipes should be well protected during the remainder of construction. Headwall distances should be designed on a project-by-project basis. This prevents exceeding the capacity of the drains. Breaking and seating the rigid concrete slab produces unstable situations in which silt-size particles are set in motion by cyclic loading and by the flow of water. If this silt source is sufficiently great, clogging may occur. Geocomposite fin drains are good alternatives to pipe edge drains because of their ease of construction, narrow trench widths, and more rapid response times. Care should be exercised in the design and construction of the drains to ensure that they perform properly. The core should be inspected after installation to ensure that the integrity of the core has been maintained throughout construction.

Longitudinal edge drains have been used in Kentucky for approximately two decades. Most were installed on the Interstate and parkway systems. Currently there are hundreds of lane-miles in place. The first edge drains consisted of 4-in. perforated polyethylene pipe installed in a 12-in.-wide trench. Various types of backfill were used. Some trenches were fabric wrapped and backfilled with crushed limestone aggregate of uniform size (approximately ¾ in.). Other trenches were not wrapped and were backfilled with a natural sand. In these cases, the pipe was covered with a fabric sock.

This paper deals primarily with geocomposite (panel) drains, because most of the research and performance monitoring in Kentucky in the past 6 years has been on that type of drain.

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INTERSTATE 64, FRANKLIN COUNTY (1985)

The first panel drains installed in Kentucky were on a 5-mi section of Interstate 64 in Franklin County. The pavement was a 10-in. nonreinforced portland cement concrete (PCC) slab. The pavement was being rehabilitated by joint replacement and full-depth and partial-depth patching. The panel drain installed was the Hydraway brand (original design) (Figure 1, Type A). It was placed in a 4-in.-wide trench on the outside shoulder of the eastbound lanes. The westbound lanes were retrofitted with a longitudinal edge drain that was a 4-in. perforated pipe in a 12-in.-wide trench. The trenches for both types of drains were backfilled with a clean coarse sand (Figures 2 and 3). The sand was placed in two lifts in the trench for the panel drain, and each lift was compacted with a vibrating compacting shoe. A single device that automatically records the volume of outflow from the drain outlet was installed on the eastbound and westbound lanes. The sites chosen for the recording devices had the same length of drain and were on the same grade. Results of measurements indicated that, after a rain, the panel drain began flowing much more quickly than did the pipe drain. The panel drains responded within a few minutes after a rain, whereas the pipe drain usually did not respond for approximately 24 to 48 hr (*1*). Because the pipe is located approximately 4 in. above the base of the trench, this area must first become saturated before the pipe begins to drain. The extra storage capacity means that more water is retained before the drain begins to function, which could be detrimental to the life of the pavement.

After the panel drain had been in service for 2 years, approximately 15 ft of the material was dug up and examined. The drain appeared to be in good condition. The fabric was clean and there was no evidence of clogging (Figure 4). The core also was in good condition. The drain was borescoped approximately 2 years later at several locations. Some minor distress was observed in the core because of compaction of the sand backfill. Observations at other sites that used the excavated trench material as backfill showed considerably more damage. It is apparent that the first installation in Kentucky was one of the best. Less effort had to be used to compact the sand backfill, which minimized the potential for damage to the panel drain.

INTERSTATE 64, FAYETTE COUNTY (1987)

The second installation of experimental panel drains was on Interstate 64 in Fayette County. The section was approxi-

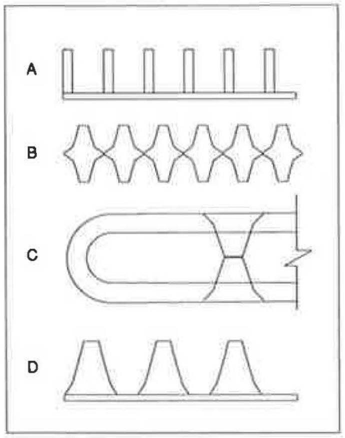

FIGURE 1 Profile of panel drain core types.

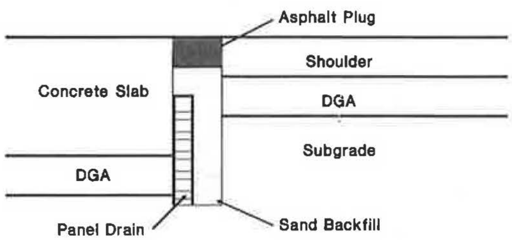

FIGURE 2 Cross section of Type-A drain installation on I-64, Franklin County.

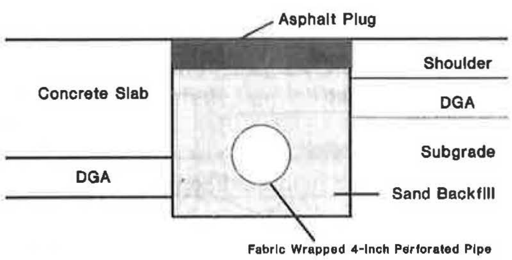

FIGURE 3 Cross section of 4-in. perforated pipe drain on I-64, Franklin County.

mately 5 mi long. The pavement on this section was a 10-in. nonreinforced PCC slab. The pavement was in good condition, and no rehabilitation was being performed. Core Type-A panel drains were installed in the westbound lanes. The eastbound lanes had approximately 4 mi of Core Type-A drains and approximately 1 mi of Akwadrain (original design) (Figure 1, Type B). Outflow-volume monitoring devices were also installed on this project, placed so that each brand of fin drain could be monitored. The devices were placed on equal lengths of drain for each brand and on equal grades. Both brands of panel drains were installed in 4-in. trenches and backfilled with the trench cuttings (Figure 5). The backfill material was compacted by using a vibrating shoe in two lifts (Figure 6).

Outflow-volume data from the drains indicated that Type-A brands drained from two to three times the volume of Type-B brands in equal periods of time. However, the response time appeared to be approximately the same for both brands. One 1,200-ft section of Type A had an outflow volume of more than 50,000 gal in 6 months. Laboratory flow studies conducted at the University of Kentucky indicated that it took Type B on the average of 1.7 times longer to discharge a given volume of water than it took Type A (*2*).

After the drains had been in service for approximately 2 years, a borescope was used to examine and photograph the condition of the core of both brands. The interior of the cores of both brands appeared to be relatively clean, with only trace amounts of silt present. The Type-B core was in good condition (Figure 7). However, the Type-A core appeared to

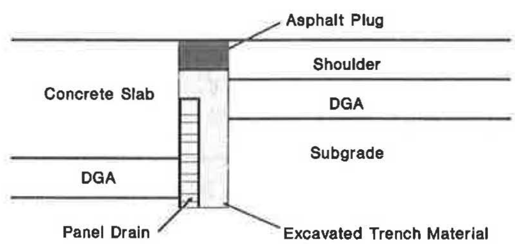

FIGURE 5 Cross section of edge drain installation on I-64, Fayette County.

FIGURE 4 Excavated Type-A drain on I-64, Franklin County.

FIGURE 6 Installation of edge drain on I-64, Fayette County.

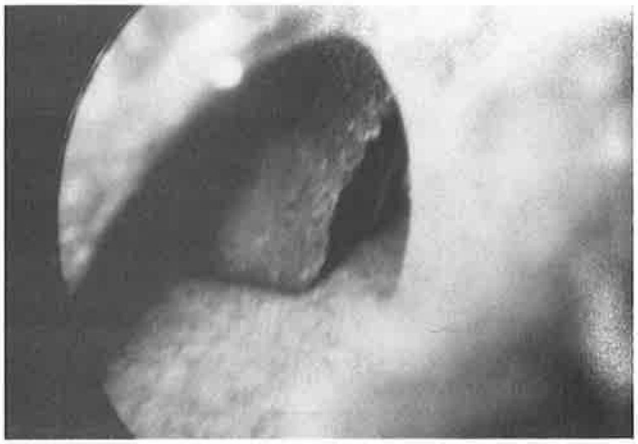

FIGURE 7 Open Type-B core showing no signs of overcompaction.

have been damaged somewhat during compaction. The top three or four rows of columns of the fin drain core were partly crushed. Also, there was evidence that the back part of the core (the shoulder side of the drain) was partly misshapen (Figure 8). It appeared that the compaction process had partly compressed the core vertically. Although the drain was still working, the capacity had been reduced.

Laboratory compression tests conducted on both Type-A and Type-B cores showed that the compressive strengths were similar. The Type-A core tends to test well in compression if the applied force is perpendicular to the support columns. However, if a row of columns starts to bend, this causes adjacent rows of columns to collapse. Both laboratory compression tests and visual inspection in the field indicated that the columns and the backing have a tendency to fold over when compression and shearing forces are placed on the Type-A core during backfilling in the field (*3*). Specifications were rewritten to help prevent core damage during installation. The new specification is discussed in a later section.

FIGURE 8 Partially misshapen inner core of Type-A edge drain.

PENNYRILE PARKWAY (1987)

The Pennyrile Parkway is a 4-lane, limited-access route in western Kentucky. The original pavement was a 9-in. unreinforced PCC slab. An 8-mi section of the pavement was rehabilitated in 1987 by breaking and seating the old slab and overlaying it with 5 in. of asphaltic concrete. Longitudinal panel drains were installed at selected locations throughout the project as part of the rehabilitation work. Type-A drains were used; they were installed in 4-in.-wide trenches, which were backfilled with the trench cuttings. A heavy compaction wheel was used to compact the backfill in two lifts. The edge drain was installed before the breaking and seating operation.

In April 1988, approximately 8 months after the project was completed, an average of five sites per mile showed signs of severe distress. Water and white, silty fines were pumping up through the new asphalt overlay, severe potholes were beginning to form in the overlay, and many of the sites required patches (Figure 9). In addition, water was pumping up through the asphalt overlay at the old joint between the shoulder and the broken slab (Figure 10).

Two of the sites at which severe distress was evident were excavated to determine if the panel drains were working properly. When the 4-in. asphalt overlay was removed, the old broken slab was found to be full of water (Figure 11). After the newly installed panel drain was excavated and pulled away from the side of the old slab, water drained freely from the old slab for almost an hour. It appeared that the drain had actually been acting as a dam (Figure 12). The panel drain was cut and a piece removed for examination. The core had

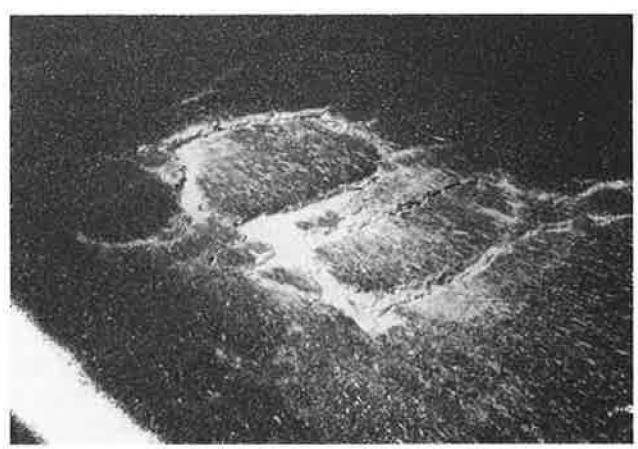

FIGURE 9 Water and silty fines pumping up through pavement on Pennyrile Parkway.

FIGURE 10 **Water pumping up through asphalt overlay directly above edge drain.**

FIGURE 11 **Water ponding in old broken slab.**

been badly crushed during compaction. The rigid backing had been folded at approximately a 90-degree angle (Figure 13). Several rows of the core of the drain had been compressed, and the drain had been deformed into an approximate J-shape, its capacity severely reduced (Figure 14). In addition, the core was almost completely clogged with silt (Figures 15 and 16). Samples of the silt in the core were collected and tested for composition in the laboratory. A high silica content indicated that most of the material was probably concrete debris created by the breaking of the old slab.

FIGURE 12 **Water ponding behind Type-A edge drain.**

FIGURE 13 **Rigid backing of Type-A core folded to approximately 90 degrees.**

FIGURE 14 Severely compressed Type-A drain.

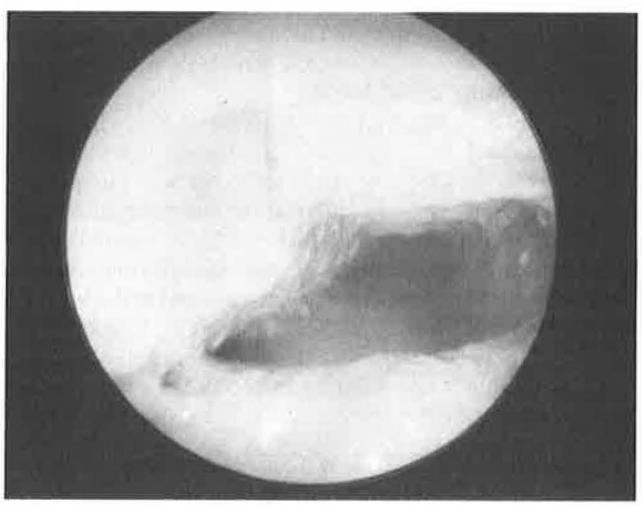

FIGURE 15 Severely silted inner core of Type-A drain.

Improper installation of the outlet pipes and overcompaction of the backfill material in the trench, which caused crushing of the core, appeared to be the principal causes of the failure of the panel drains on this project (*4*). The outlet pipes were made of flexible polyethylene. At both of the sites excavated, the outlet pipes were partly crushed, and one appeared to have a 4-in. hump, which decreased the water velocity and allowed silt to be deposited. The panel drains were completely removed from this project and were replaced by 4-in. pipes in 12-in.-wide trenches, for fear that similar panel edge drains would fail. It was apparent that panel drain and installation techniques needed to be revised.

FIGURE 16 Exposed inner core showing siltation.

In 1988 Type-A drains were being installed on a number of other projects throughout Kentucky. Because of the apparent flexibility of the Type-A drain in the vertical direction and its susceptibility to deformation under compaction, contractors were instructed to use less compactive effort during the backfilling procedure to reduce damage to the panel drain. Several of these projects were examined after installation by using the borescope. It appeared that considerable damage was still occurring to the core, even when less compactive effort was used.

MOUNTAIN PARKWAY (1988)

The Mountain Parkway is a four-lane, limited-access highway in eastern Kentucky. The original pavement was a 9-in. unreinforced PCC slab, which was rehabilitated in June 1988 by breaking (6-in. blocks) and seating and overlaying with 8-in. of asphaltic concrete. Type-A longitudinal panel drains were also installed on most of the project; approximately 5,000 ft of a new drain, manufactured by Advanced Drainage Systems and identified by the brand name Advanedge (Figure 1, Type C), was installed on an experimental basis. Both products were installed in a 4-in.-wide trench, which was backfilled with the cuttings. Lighter compaction was attempted in two lifts using the vibrating shoe. As in all other break-and-seat projects, the panel drains were installed before the breaking and seating of the old slab. To address the question whether the breaking and seating operation may have been the major cause of damage to the drains rather than the compaction procedure, the drains were examined with the borescope immediately after installation and before the breaking and seating operation. Type-A core still showed considerable compression and damage in the vertical direction, leading to the conclusion that compaction was still the major cause of damage (Figure 17). Although Type-C drain is considerably stiffer vertically and showed little or no vertical damage, it showed some horizontal compression (Figure 18). Never-

FIGURE 17 Compressed interior support columns.

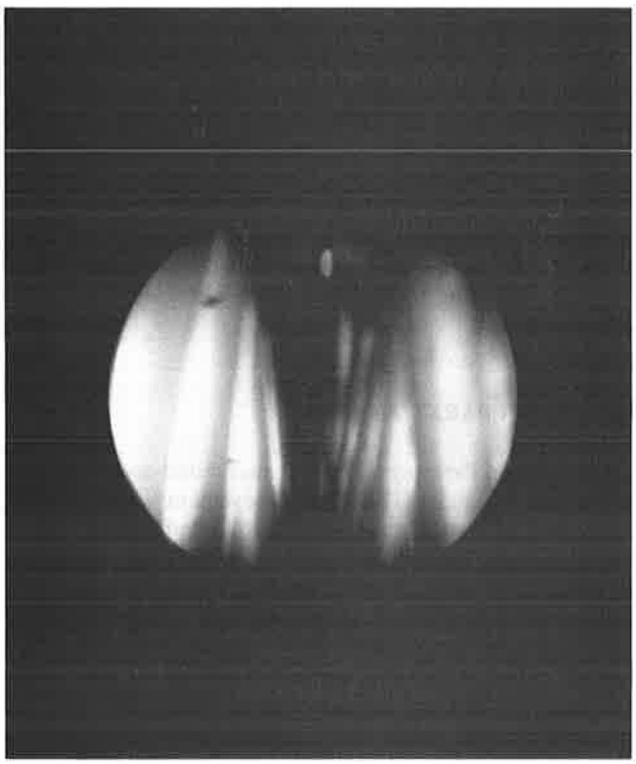

FIGURE 18 Signs of horizontal compression in core of Type-D edge drain.

theless, Type C appeared to be in better condition than Type A (5).

Sections of Type A and Type C were excavated in 1989 after being in service for approximately 1 year. Both products appeared to have some silt in the core, but water was flowing freely through them. The Type-A core was reduced from its original height of 12 in. to approximately 10 in. and many rows of columns were compressed (Figure 19). The Type-C core appeared to be in good condition, although a small amount

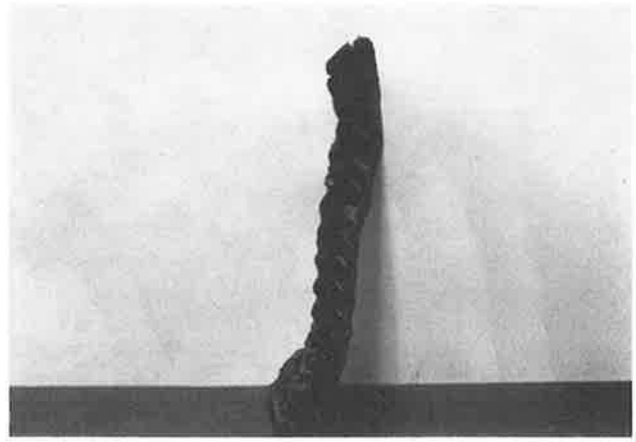

FIGURE 19 Excavated Type-A drain showing compression.

of silting was occurring between the fabric and the core and one of the slits in the core was partly clogged with silt.

Because of the known compaction problem and the possibility of a silting problem, the Kentucky specifications on installation of panel drains were changed in 1989. The new specifications require the panel drain to be installed on the shoulder side of the trench instead of on the pavement side. Backfilling with the trench cuttings is no longer permitted; a clean, coarse sand is specified for backfill material (Figure 20). The sand must also be compacted by flushing the trench with water, thus avoiding the use of heavy compaction over the panel drains. The sand backfill should prevent much of the silt from the broken concrete slab and possibly the dense-graded aggregate from reaching the fabric on the drain core. All projects constructed since the latter part of 1989 have used the new specifications. Later borescope inspections of these projects, after they were completed, showed no damage to the core of the panel drains.

However, in 1989 some spot distresses were beginning to appear in the rehabilitation projects completed on the Mountain Parkway in 1988. At some sites, silt was pumping up through the 8-in. asphalt overlay at the shoulder joint and in the middle of the passing lane (Figure 21). Excavation of two of these sites showed evidence of poor construction practices. At one the outlet pipe had not been connected to the headwall and consequently it was completely blocked. At another the outlet pipe was crushed just before it entered the headwall. Of 14 outlet pipes inspected by using the borescope, 11 had

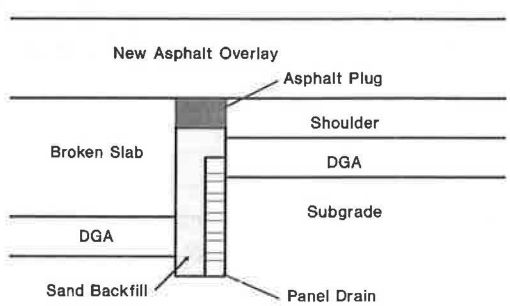

FIGURE 20 Cross section of new installation procedure for panel drains.

FIGURE 21 Silt being pumped up at centerline of Mountain Parkway.

guardrail posts driven through them. All the edge-drain outlet pipes on the inside shoulder were connected to the bottom of the drop inlet boxes located in the median. This allowed silt and trash in the bottom of the box to block the outlet pipe.

A later rehabilitation project, constructed on the Mountain Parkway in 1989, was inspected after the edge-drain installation and the paving operation were completed. The inspection was made during the operation to shape the median. It was noted that many drain outlet pipes were cut by the grading operation, some were simply buried by the grader, and other pipes were crushed (*6*).

WESTERN KENTUCKY PARKWAY (1988)

The Western Kentucky Parkway is a four-lane, limited-access highway. The original pavement was a 9-in. unreinforced PCC slab. Type-A longitudinal panel drains were installed in 1988. The pavement was rehabilitated in July 1989. The old slab was broken (6-in. blocks) and seated.

In July 2 in. of asphaltic concrete base was placed over the old broken pavement and used as a driving surface for a while. During that time, several heavy rains occurred. After the rain it was evident that a large amount of silt had been pumped up through the 2 in. of new base and deposited on the shoulder, but the source of water and silt was not immediately evident. On August 15, 1989, personnel from the Kentucky Transportation Center drilled into the drains and photographed the interior of the drain panel with a borescope. The photographs revealed that the drain panel had been partly crushed from the backfilling operation used during construction. It was estimated that the internal volume of the drain had been reduced by approximately 30 to 40 percent because of crushing (*7*), but the drains were still functioning at this reduced capacity. Although a large quantity of grayish-white silt was present at the headwalls, the drains appeared to be relatively free of silt deposits.

On August 18, the pavement was trenched and sampled at two locations. A trench approximately 8 in. wide was cut from the centerline of the westbound lanes through the outside shoulder and approximately 3 in. into the subgrade. The edge-drain panel was also severed when the trench was cut. There was no free-flowing water in any portion of the pavement, although droplets of water were trapped throughout the newly placed asphaltic concrete base and cracks in the broken PCC slab were damp. The dense-graded aggregate was damp but not excessively wet, and the subgrade was relatively dry.

During the approximately 3 hr the trench remained open, the uphill end of the severed panel drain emitted a stream of water approximately 2 in. in diameter. A water truck was positioned about 100 ft uphill from the open trench, and approximately 500 gal of water was allowed to run onto the asphaltic concrete surface from the spray bar on the back of the truck. Some of the water ran downhill over the surface and into the trench. However, the uphill end of the severed panel drain begin to flow less than 5 min after the water was released. This showed that the newly placed asphaltic concrete base and the old broken slab were very porous and drained freely.

Permeability tests were performed in the laboratory on samples of the newly placed asphaltic base course. A hydraulic and hydrologic analysis of the pavement and drainage system showed that the capacities of the drains were being exceeded by as much as 600 percent, allowing them to overflow. This excess water in the drains was forced upward through the asphalt plug covering the drain and through the 2-in. overlay, depositing concrete debris and limestone fines onto the surface of the shoulder. It was concluded that the distance between headwalls was much too long, in some cases, as much as 2,200 ft. The average was approximately 700 ft. Analysis showed that the maximum distance on grades of over 2 percent should be no more than 450 ft and on grades of less than 2 percent, 200 ft. Additional headwalls were added on this project. As a result of this study, the Kentucky specifications on headwall distances has been changed.

INTERSTATE 75, FAYETTE COUNTY (1989)

The original pavement on this project was a 10-in. unreinforced PCC slab. Longitudinal edge drains were installed in fall 1989. Two panel drain products and a 4-in. perforated pipe edge drain were installed. The panel drains were Type C (the latest design, which was stiffened in the horizontal direction) and Contech (Type D). Both panel drains were installed using the latest specifications on backfilling and headwall distances. The 4-in. perforated pipe was installed in a 1-ft-wide trench. Both the trench and the pipe were wrapped with fabric, and the trench was backfilled with size 57 stone (Figure 22). There were no construction problems during the installation of these products. Borescope inspections after installation showed that all the products were in good condition with no apparent damage to the cores. At present, the pavement has not been broken and seated.

CONCLUSIONS

The conclusion of this study is that greater attention should be given to details during installation and construction of

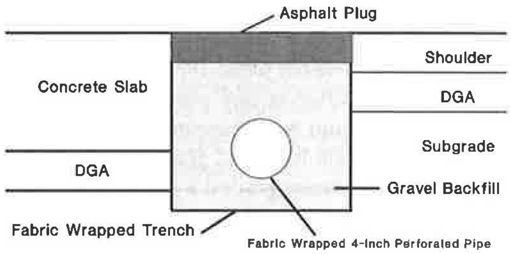

FIGURE 22 Cross section of new installation procedure used on part of I-75.

panel drains. Many of the problems encountered were the result of improper construction practices. Outlet pipes should be installed with the proper grade, which helps maintain the velocity of the water in the drains, flushing out silt and clay-size particles that enter. The drains and outlet pipes must be well protected during the construction operations.

Headwall distances should be designed on a project-by-project basis to prevent exceeding the capacity of the drains.

Breaking and seating of the rigid concrete slab produces the unstable situation in which silt-size particles are set in motion by cyclic loading and by the flow of water. If this silt source is sufficiently great, clogging may occur. At this time, clogging is not a factor under the new specification. The only notable effect of breaking and seating still occurring under the new specification is formation of thin layers of calcium carbonate at the surface of the water inside the core of the drain and at the headwall.

It was concluded that geocomposite fin drains are viable alternatives to pipe edge drains because of their ease of construction, narrow trench widths, and more rapid response times. However, it was also concluded that greater care must be taken in the design and construction of the drains to ensure that they perform properly. To ensure that the integrity of the core has been maintained throughout construction, the authors recommend a specification for inspection of the core after installation is finished.

ACKNOWLEDGMENTS

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Transverse Pipe Underdrains for Highway Groundwater Control: A Case History

JOHN O. HURD AND HERBERT DUNCHACK

Longitudinal pipe underdrains and transverse pipe underdrains spaced at approximately 60-ft intervals were used to provide groundwater control on one section of the Kirtland-Chardon Road highway project in Lake County, Ohio. This area was designated spring area by the geotechnical consultant because of extremely severe groundwater problems in the subgrade. Normal practice is to provide aggregate drains spaced at 50-ft intervals on uncurbed flexible pavements, but the severe groundwater problems on this section warranted additional drainage. Aggregate drains were placed on the remaining uncurbed sections of the project per normal practice. Four years after construction, the pavement in the spring area, where the improved drainage system was provided, is in excellent condition. Severe distress has been observed in the areas with standard drainage that abut the spring area.

Kirtland-Chardon Road is an urban collector that connects the city of Kirtland in Lake County, Ohio, with the village of Chardon in Geauga County, Ohio. The location of highway project LAK-Kirtland/Chardon Road, IX-1A79(1), is shown in Figures 1 and 2. The maximum design year average daily traffic (ADT) for the 4.25-mi section in Lake County is 4,130 vehicles per day. The vehicles are predominantly automobiles and light-duty trucks. Only 1.35 percent of the traffic is heavy-duty vehicles. The current and design year ADTs for the various sections of Kirtland-Chardon Road in Lake County are shown in Figure 3.

THE PROBLEM

The existing 6-in-thick pavement in 1982 was composed of a buildup of seal and chip applications or thin asphaltic concrete overlays, or both. Approximately 75 percent of this pavement rested on a 6-in-thick gravel base and the remainder was on a 8-in-thick portland cement concrete (PCC) base. This information was obtained from 49 borings taken throughout the 4.25-mi highway section. No subsurface drainage had been provided. The typical section of the existing pavement structure is shown in Figure 4.

Despite the fact that the traffic loads on the highway were relatively light, the pavement and base had deteriorated to the point that complete replacement of the pavement structure was required on approximately 75 percent of the roadway length in Lake County (Figure 5). The remaining 25 percent needed salvage construction requiring spot repair, leveling, and a surface overlay. The locations of the replacement and salvage sections are shown in Figure 6. All salvage areas were located outside those sections of highway with the highest traffic volumes.

The subgrade soils in the borings were found to be generally clayey silt, silty clay, and clayey silty sand. At many boring locations, the subgrade soil was moist to wet. Groundwater was observed flowing into the test borings at several locations. One particular steep 1,500-ft section of the highway was designated "spring area" in the foundation consultant's report (*1*). Water had been observed percolating up through the pavement surface at several locations along this section. The spring area is indicated in Figure 6; Figure 7 shows the steep topography of that section.

Additional borings were taken in the spring area and monitor pipes were installed for more detailed groundwater monitoring. Test results of borings in the spring area indicated the presence of shallow bedrock from 3 to 6 ft below the pavement surface. Sandstone bedrock was observed overlying shale bedrock and water was coming from the sandstone-shale interface and cracks in the sandstone.

The severe deterioration of the pavement was attributed in part to the groundwater problems observed (*2*). Groundwater

FIGURE 1 Location of Kirtland-Chardon Road project in Ohio.

Ohio Department of Transportation, Room 620, 25 South Front Street, Columbus, Ohio 43215.

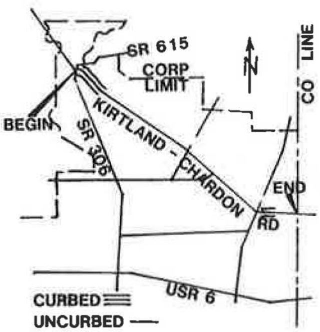

FIGURE 2 Site location map of Kirtland-Chardon Road project.

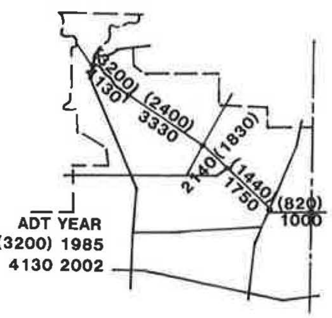

FIGURE 3 Kirtland-Chardon Road average daily traffic.

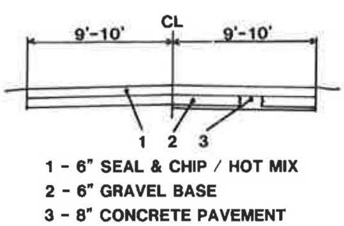

FIGURE 4 Existing pavement buildup.

FIGURE 5 Pavement condition before construction.

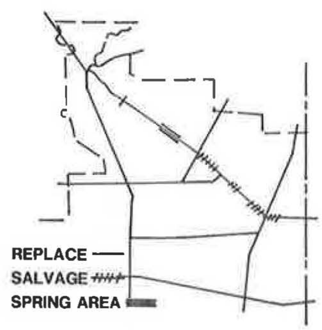

FIGURE 6 Location of salvage sections, replacement sections, and spring area.

near the surface indicates the potential for frost damage within the paving section. In addition, freezing water in cracks or porous seams of the paving section worsens pavement distress. In the warmer months, during periods when the ground is saturated, the subgrade and base are weakened, thereby reducing pavement support.

THE SOLUTION

The new-pavement buildup for the sections in which the pavement was completely replaced was composed of a 3-in asphaltic concrete surface course, a 4-in bituminous aggregate base course, and a 4-in. aggregate base course (Figures 8–10). This relatively light design was all that was required for the design traffic loads. The pavement design for the salvage and overlay sections is shown in Figures 11 and 12.

For groundwater control, 6-in longitudinal pipe underdrains were provided in curbed sections and in the uncurbed spring area (Figures 8, 10, and 11). Aggregate drains were provided on the other uncurbed sections at 50-ft intervals shown in Figures 9 and 12. (Locations of curbed and uncurbed sections are shown in Figure 2.) Normal practice has been to provide longitudinal pipe underdrains on curb sections and aggregate drains spaced at 50-ft intervals on uncurbed sections for this type of highway (*3*). In addition, 4-in transverse pipe underdrains were provided in the spring area (Figure 10). The transverse underdrains were spaced such that the top of the trench for each installation was no lower than the flow-line elevation of the next transverse underdrain upstream. Because of the relatively steep highway grades (approximately 4 to 8 percent), transverse drains were required at approximately 50-ft

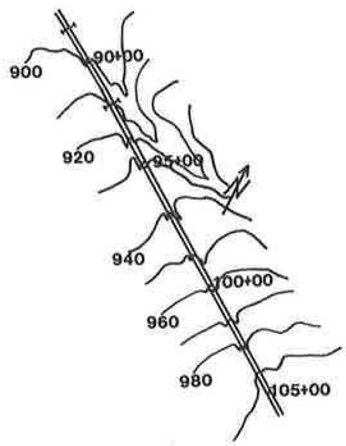

FIGURE 7 Spring area topography.

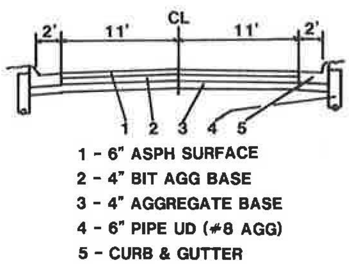

FIGURE 8 Typical section, curbed replacement.

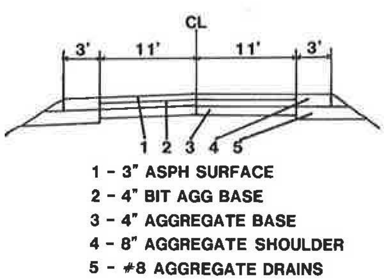

FIGURE 9 Typical section, uncurbed replacement.

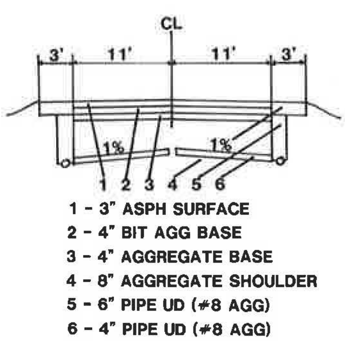

FIGURE 10 Typical section, spring area.

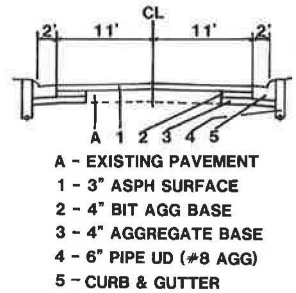

FIGURE 11 Typical section, curbed salvage.

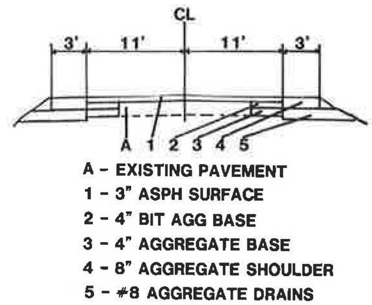

FIGURE 12 Typical section, uncurbed salvage.

intervals. It was anticipated that nearly all seepage lenses would be intercepted in the spring area by this transverse underdrain design.

THE COST

The highest project contract was let in September 1985 and completed in December 1986. The unit prices bid for each type of subsurface drain are shown in Table 1. All bids were within acceptable limits above or below the engineer's estimate. Bidder 1 was the low bidder for the project. The cost of the subsurface drainage in relation to the cost of the pavement is shown in Table 2. The cost of pavement includes the cost of subgrade compaction, subbase, base, surface course, and curb or shoulder. The values shown are based on the successful bidder's unit prices bid for the drainage and pavement items.

It should be noted that the cost of the spring area drainage system per foot of pavement on this project is inflated compared with the same system on a similar project with normal, flatter grades. As a result of the steep slopes in the spring

TABLE 1 UNIT COSTS OF SUBSURFACE DRAINAGE ITEMS

	6-in. Pipe Underdrain			
Bidder	30 in. Deep	50 in. Deep	4-in. Pipe Underdrain	Aggregate Drain
1	4.90	6.75	7.00	4.00
2	4.50	6.00	5.00	3.25
3	5.50	9.00	11.00	4.00
Engineer's estimate	7.00	7.25	7.10	4.50

NOTE: Costs are in U.S. dollars per foot.

TABLE 2 COMPARISON OF SUBSURFACE DRAINAGE SYSTEM COSTS AND PAVEMENT COSTS

	Subdrainage System Cost ($/ft of pavement)	Pavement Cost ($/ft of pavement)	Cost of Pavement for Drainage (%)
Curbed section	9.80	63.60	15
Uncurbed section	0.93	43.90	2
Spring area	17.41	43.90	40

area, very close spacing of the transverse underdrains was required to guarantee complete groundwater interception.

The cost of the pipe underdrain subsurface drainage system compared with that of the pavement would at first glance appear excessive. However, it must be noted that the proposed pavement section is only 22 ft wide and relatively thin. The comparative cost of the pipe underdrain system would be considerably less for a state or federal highway project with a wider and thicker pavement section. The same cannot be said for the comparative cost of the aggregate drain system. Grading requirements on a state highway project would require a greater length of aggregate drain than those specified on this project. Thus, both the aggregate drain cost and the pavement cost would increase in proportion on state or federal highway projects.

PERFORMANCE

The project has been visually monitored periodically for signs of pavement distress since its construction in 1985 and 1986. As of May 1990, the following observations had been made regarding pavement performance.

No significant pavement distress had been observed on the curbed sections. Only a few thin longitudinal cracks or cracked spots were observed (Figure 13). The longitudinal underdrains appeared to have provided adequate subsurface drainage.

Significant distress, including alligator cracking, rutting, or both, had been observed on the typical uncurbed section west of the spring area (Figure 14). Repair of the pavement had been required immediately west of the spring area. This distress was indicative of possible subsurface drainage problems. It was apparent that the aggregate drains had not provided adequate subsurface drainage throughout this area.

FIGURE 14 Condition of pavement on typical uncurbed section west of spring area.

No pavement distress had been observed in the spring area (Figure 15). The inspections indicated (in retrospect) that the spring area subsurface drainage system should have been extended in a somewhat westerly direction outside the delineated spring area. There was little doubt that the spring area drainage system had provided adequate subsurface drainage. Figure 16 indicates subsurface flows through the outlet pipes under relatively dry conditions. Detailed monitoring of subsurface drainage discharge was beyond the scope of this project.

No significant pavement distress had been observed on the typical uncurbed section east of the spring area. Its condition (Figure 17) was similar to that of the typical curbed section. The aggregate drains appeared to have provided adequate drainage. It should be noted that most of this area was composed of salvage sections that were in fair condition before construction.

Whether application of the longitudinal pipe underdrain with or without transverse underdrains on this entire project

FIGURE 13 Condition of pavement on typical curbed section.

FIGURE 15 Condition of pavement in spring area.

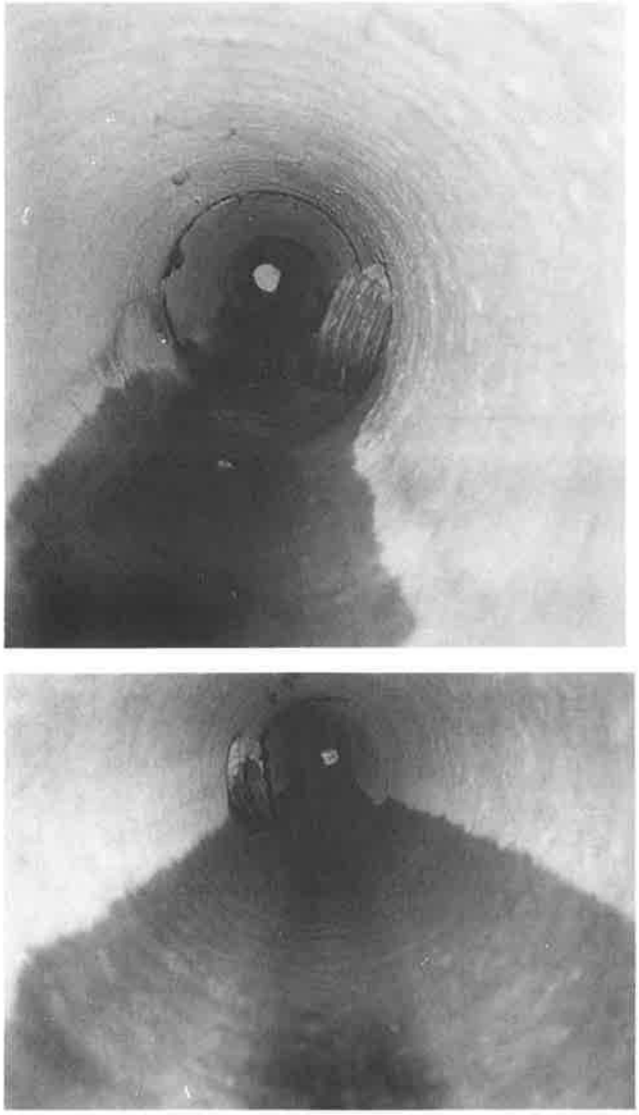

FIGURE 16 Dry-weather flow in outlet pipes, two views.

would have been justified will require long-term investigation and examination of maintenance costs, which are beyond the scope of this paper. There is no doubt that the pipe underdrain system would have been justified on those sections that have required repair to date. The benefit of subsurface drainage would be greater on thicker high strength or wider pavements with higher traffic volumes. In these cases, the comparative cost of the drainage system and the cost of the pavement would be much less.

RECOMMENDATIONS

Where subsurface drainage is required on uncurbed asphalt pavement sections, longitudinal pipe underdrains should be used in lieu of aggregate drains. Where severe subsurface

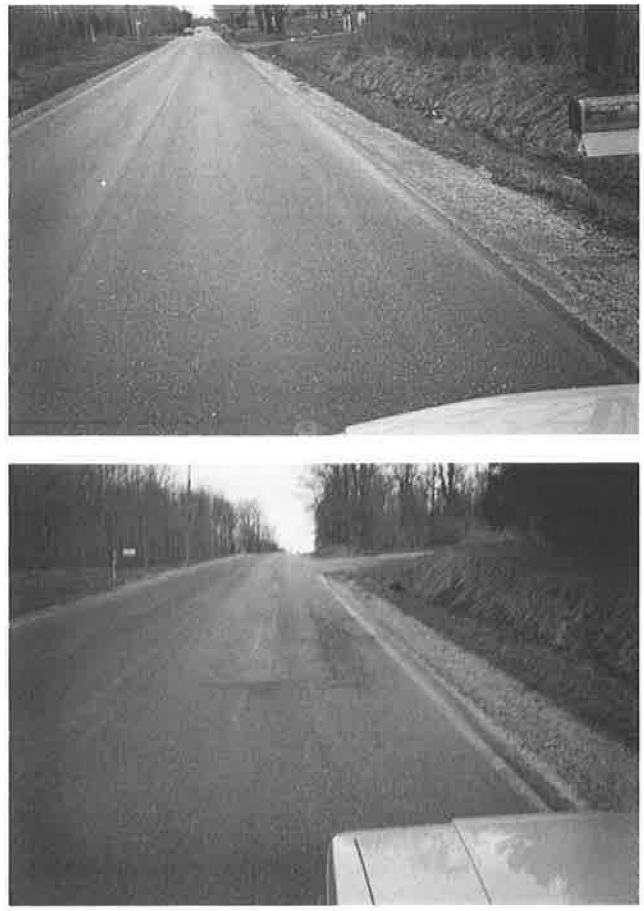

FIGURE 17 Condition of pavement on typical uncurbed section east of spring area, two views.

drainage problems exist, transverse underdrains should be provided in conjunction with longitudinal underdrains to provide adequate and thorough drainage of the subgrade.

ACKNOWLEDGMENTS

This work was funded by the Ohio Department of Transportation.

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The findings and opinions are those of the authors and do not constitute a standard or specification.