Testing and Performance of Geosynthetics in Subsurface Drainage

L. DAVID SUITS
JAMES B. GODDARD
JOHN S. BALDWIN

Editors

ASTM STP 1390

STP 1390

Testing and Performance of Geosynthetics in Subsurface Drainage

L. David Suits, James B. Goddard, and John S. Baldwin, editors

ASTM Stock Number: STP1390

ASTM
100 Barr Harbor Drive
West Conshohocken, PA 19428-2959

Printed in the U.S.A.

Library of Congress Cataloging-in-Publication Data

Testing and performance of geosynthetics in subsurface drainage / L. David Suits, James B. Goddard, and John S. Baldwin, editors.
p. cm. -- (STP; 1390)
"ASTM stock number: STP1390."
Proceedings of a symposium held in Seattle, Wash., June 29, 1999.
Includes bibliographical references.
ISBN 0-8031-2860-6
1. Road drainage--Congresses. 2. Geosynthetics--Testing--Congresses. 3. Subsurface drainage--Congresses. I. Suits, L. David, 1945- II. Goddard, James B., 1945- III. Baldwin, John S., 1946- IV. ASTM special technical publication; 1390.

TE215 .T47 2000
625.7'34--dc21

00-024658

Peer Review Policy

Each paper published in this volume was evaluated by two peer reviewers and at least one editor. The authors addressed all of the reviewers' comments to the satisfaction of both the technical editor(s) and the ASTM Committee on Publications.

To make technical information available as quickly as possible, the peer-reviewed papers in this publication were prepared "camera-ready" as submitted by the authors.

The quality of the papers in this publication reflects not only the obvious efforts of the authors and the technical editor(s), but also the work of the peer reviewers. In keeping with long-standing publication practices, ASTM maintains the anonymity of the peer reviewers. The ASTM Committee on Publications acknowledges with appreciation their dedication and contribution of time and effort on behalf of ASTM.

Printed in Scranton, PA
March 2000

Foreword

This publication, *Testing and Performance of Geosynthetics in Subsurface Drainage*, contains papers presented at the symposium of the same name held in Seattle, Washington, on 29 June 1999. The symposium was sponsored by ASTM Committee D 35 on Geosynthetics and Committee D 18 on Soil and Rock in cooperation with The National Transportation Research Board (Committees A2K06 and A2K07). L. David Suits, New York State Department of Transportation, John S. Baldwin, West Virginia Department of Transportation, and James B. Goddard, Advanced Drainage Systems, Inc., presided as co-chairmen and are editors of the resulting publication.

Contents

Overview

The effectiveness of subsurface drainage in prolonging the service life of a pavement system has been the subject of discussion for many years across several disciplines involved in the planning, designing, construction, and maintenance of pavement, and other engineered systems. One of the first workshops that I attended on first coming to work for the New York State Department of Transportation over thirty years ago was presented by the Federal Highway Administration in which the benefits of good subsurface drainage in a pavement system were promoted. Even at that time there were many different components of a drainage system that contributed to its overall performance. With the advent of geosynthetics, and their incorporation into subsurface drainage systems, another component has been added that must be understood in order to insure proper performance.

As indicated above, the subject crosses many disciplines. It is with this in mind that four different committees of two different organizations jointly sponsored this symposium. Those co-sponsoring committees and their organizations were: Transportation Research Board (TRB) Committee A2K06 on Subsurface Drainage, TRB Committee A2K07 on Geosynthetics, ASTM Committee D18 on Soil and Rock, and ASTM Committee D35 on Geosynthetics. The purpose of the symposium was to explore the experiences of the authors in the testing and performance of geosynthetics used in subsurface drainage applications. The symposium was divided into three sessions: Session I—Field Performance Studies; Session II—Pavement Design and Drainage; Session III—Testing. This special technical publication (STP) is divided into these three sections.

In Session I, on Field Performance Studies, the authors presented discussions on the performance of three different geocomposite materials. They include a geonet with a geotextile, a geopipe wrapped with a geotextile, and a geocomposite capillary drain barrier.

A study to determine the most effective repair of a shallow slope failure on a racetrack in Singapore showed that an internal drainage system consisting of a geonet and geotextile, placed from depths of 8 to 15 m in the slope, would result in a stable slope. However, with the difficulty of installing a geonet to these depths, an equivalent system consisting of a geopipe wrapped with a geotextile was determined to be more feasible. The paper details the finite element analyses that were performed in relation to the design.

It is pointed out in the paper on the geocomposite capillary barrier drain that drainage of water from soils is generally considered a saturated flow process. It further points out that there are a range of applications where there would be benefit in draining the water prior to saturation. The paper describes the development of a geocomposite consisting of a separator geotextile, a geonet, and a transport geotextile for use in a drainage system that operates under negative pore water conditions associated with unsaturated conditions. The paper describes the study to confirm the geocomposite capillary drain concept.

In Session II, on Pavement Design and Drainage, the papers described the use of geocomposite drainage layers in the base and subgrade of a roadway system, the use of geosynthetics in pavement drainage in cold climates, the development of performance-based specifications for highway edge drains, and some key installation issues in the use of geocomposite edge drain systems.

On a project done in conjunction with the Maine DOT, the University of Maine, and the U.S. Army Cold Regions Research Laboratory, the data from monitoring drainage outlets indicate that a tri-planar geocomposite drainage net placed at or below subgrade was successful in rapidly removing water from beneath the roadway. In addition, the geocomposite facilitated construction in areas where the subgrade was weak, without requiring additional undercuts. In a control section where geosynthetics were not used, an additional 600 mm of stabilization aggregate was required.

Provisions for good highway drainage include surface drainage, ground water lowering, and internal drainage. The focus of the paper on the use of geosynthetics in pavement drainage in cold climates is on the most difficult of these, internal drainage. It reviews the authors' experiences with several types of geosynethetic drainage systems installed in the Canadian province of Ontario. They include pipe edge drains with geotextiles, geocomposite edge drains, and geotextile wrapped aggregate edge drains. Several of these were also used in different types of subgrade. As a result of their experiences, the authors present several recommendations that they feel will result in the effective use of geosynthetic drainage systems in cold climates.

Two problems that arise with any type of drainage system are improper installation and lack of proper maintenance after installation. A study by the Kentucky Transportation Research Center and the Kentucky DOT revealed that at least 50% of the drains investigated were significantly damaged during installation. As a result of further research, a detailed quality control/quality assurance program was established, the intent of which was to decrease the percentage of failures and increase the performance of geosynthetic drainage systems.

In a second paper discussing geosynthetic drainage installation issues, two case histories are reviewed. The first being a site in Virginia, the second being a site in Ohio. The specific issues examined are backfill selection, positioning of the drain within the trench, timely installation of outlets, and selection of outlet piping. The conclusions drawn from the two cases are: (1) proper construction techniques, including verticality, position in the trench, aggregate type, and outlet spacing and installation are critical; (2) proper maintenance, including periodic video inspection of the edge drains, is essential.

In Session III, on Testing, the authors described four different laboratory testing programs that were undertaken to evaluate different aspects of geosynthetic drainage systems. They included the laboratory testing of a toe drain with a geotextile sock, two reports on a modified gradient ratio test system with micro pore pressure transducers inserted into the system, and a discussion on the influence of test conditions on transmissivity test results for geotextile drains.

As the result of the plugging or blinding of 460 and 600-mm-diameter perforated toe drains that had been installed at Lake Alice Dam in Nebraska, the U.S. Bureau of Reclamation undertook a full-scale laboratory test program to determine the best solution to the problem. As a result of the full-scale laboratory test program using a 380-mm perforated pipe with a geotextile sock, several conclusions were drawn regarding the use of geotextile-wrapped toe drains. When used in conjunction with a sand envelope, the socked toe drain's performance was optimized as a result of the absence of any clogging. The socked toe drain allowed the use of a single stage filter that could be installed with trenching equipment at a significant cost savings over the traditional two-stage filter that had been used previously. The use of the socked drain increased flow rates by a factor of 3 to 12.

A study carried out at the National University of Singapore compared the differences of two different transmissivity testing devices. The study was carried out using prefabricated vertical drains and geonets under varying test conditions. The traditional transmissivity device was compared to a newly designed device that has the geosynthetic drain installed in the vertical position encased in a rubber membrane. It was shown that the flexibility of the filter and core material can significantly affect the discharge rate that is attainable in prefabricated vertical drains. Comparing the two test apparatuses showed the ASTM transmissivity device to produce the least conservative results. Thus, knowing the actual site conditions under which to perform transmissivity testing is critical.

A study conducted at Chung Yuan University in Taiwan investigated what the researchers considered to be disadvantages to the current gradient ratio test. Previous research had indicated that the current gradient ratio device was unable to clearly identify geotextile clogging conditions. The test program inserted piezometers at the same locations as the current method, plus an additional one right on top of the geotextile specimen, and inserted 10.0 mm into the test device to eliminate the effects of disturbance.

The installation of the pressure probe directly on top of the geotextile provided a precise understanding of the pressure distribution within the test system. The results also indicated that the current practice of a gradient ratio equal to or less than 3.0 being necessary to avoid system clogging might not be the best criterion to reflect the clogging potential of soil-geotextile systems.

A brief overview of the papers presented in this STP has summarized the basic conclusions reached by the authors and symposium presenters. The papers include summaries of case histories of field experience, field testing, and laboratory testing that has been performed in an effort to better understand the performance of geosynthetic drainage systems. In each instance the importance of providing good subsurface drainage is emphasized. In some instances recommendations are made to improve material specifications, laboratory testing, and the field performance of these systems. It is felt that these recommendations will help to ensure the proper, long-term performance of geosynthetic drainage systems.

L. David Suits
New York Department of Transportation;
Symposium Co-chairman and Editor

Field Performance Studies

Siew-Ann Tan,[1] Soon-Hoe Chew,[2] G.-P. Karunaratne,[1] and Swee-Fong Wong[3]

Performance of Repaired Slope Using a GEONET or GEOPIPE Drain to Lower Ground-Water Table

---

Reference: Tan, S.-A., Chew, S.-H., Karunaratne, G.-P., and Wong, S.-F., **"Performance of Repaired Slope Using a GEONET or GEOPIPE Drain to Lower Ground-Water Table,"** *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390*, L. D. Suits, J. B. Goddard, and J. S. Baldwin, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: A 70 m long by 5 m high slope with gradient of 1(V):2(H) was cut into a medium-stiff residual soil of undrained shear strength better than 60 kPa, with drained strength parameters of about $c' = 10$ kPa, and $\phi' = 22°$, to form the bank for an effluent pond used for irrigation of a racetrack turfing. Both drained and undrained slope stability analysis indicates stable slopes under reasonable ground-water (GW) levels expected in the cut slope. However, after a period of intense rainfall during construction, the slope suffered a shallow slip of about 1 m to 1.5 m depth over a 30m stretch of the slope length with a vertical scarp near the top of the cut slope. This paper examines the causes of slope failure, and the strategy adopted for a permanent repair of the slope by providing internal geosynthetic drains beneath the re-compacted slope, using either a GEONET or closely spaced geo-pipe inclusions in the slope. For design, the GEONET or geo-pipe drains used must have adequate factored transmissivity to conduct expected heavy rainfall infiltration water safely out of the slope mass. Under a steady-state very heavy rainfall condition of 150 mm/h on the racetrack, it is demonstrated by the Finite Element Method (FEM) analysis, that GEONET must be provided to at least as far back as the mid-depth of the slope (about 4 m depth) to produce sufficient GW lowering to give stable slopes. The construction method of the slope repair to avoid further failure is described briefly, and the performance of the sub-soil drains in enhancing slope stability is demonstrated in the field project.

Keywords: GEONET, geo-pipe drains, slope failure, slope stability, ground-water lowering

---

[1]Associate Professor, [2]Assistant Professor, [3]Research Scholar.
Department of Civil Engineering, National University of Singapore, Singapore.

Introduction

A slope was cut into natural ground of an original hill at elevation of 130 mRL (Reduced Level), which was reduced to final elevation of about 110 mRL to form the platform for a 30 m wide racetrack. As part of the landscape, a 70-m-long slope with gradient of 1(V):2(H) was cut into a medium-stiff over-consolidated residual soil of undrained shear strength better than 60 kPa, with drained strength parameters of about $c' = 10$ kPa, and $\phi' = 22°$ (based on consolidated undrained (CU) triaxial test with pore pressure measurements,), to form the bank for an irrigation pond needed for the turf of the track. Both drained and undrained slope stability analysis indicated stable slopes under reasonable ground-water (GW) levels expected in the cut slope. However after a period of unusually intense rainfall, the slope suffered a shallow slip to about 1 m to 1.5 m depth with a vertical scarp near the top of the cut slope, over a 30-m length of slope. Subsequent repair of the slope using dry cut fill soils from the same site also resulted in a similar slip after further exposure to rainfall. Thus, a detailed failure investigation was conducted, with careful site measurements of ground water table (GWT) levels. Soil shear strengths were estimated under different water soaking) conditions for investigation into the causes of slope failure, despite the gentle slope profile.

Possible Causes of Failure

The large overburdened stress relief resulting from the large hill cut to form the embankment slope produced soils at high pore-water suction state. This resulted in higher factors of safety immediately after cutting. These factors of safety would reduce with time since effective stresses decrease from pore-water increases, as soils are exposed to GW rise from rainfall infiltration. Also GWT which was deep in the original hill profile is now brought closer to the ground surface from the removal of overburdened soils. The back analysis using limit equilibrium indicated that failure occurred primarily from inadequate sub-soil drainage. This condition led to: (a) water absorption into the residual soil causing a progressive softening of the soil mass, (b) increased seepage force and mass of water-logged soils thus increasing the driving moment, and (c) rise of water table within the slope mass caused by inadequate internal drainage in the slope.

Site Investigation of Failed Slope

From the site investigation, it was apparent that the slope failure began as a tension crack somewhere at mid-height on the 1:2 cut at between elevations 107 mRL to 105 mRL. The failure mass encompassed an area of about 5 m by 30 m in plan and a depth of about 1.5 m. This constitutes a soil mass of about 225 m^3, which is not a very large mass. Detailed measurements were made of the GW levels from Casagrande-type open standpipes (P1 to P3) installed at three points as shown in Fig.1. These standpipes are the isolated types installed at depths below the slope base to monitor the piezometric levels in the slope body. These standpipes were capped with plastic covers when not in use to prevent rainwater from getting in through the top of the pipes. Measurements were made from May 4 to May 6, 1998 at hourly intervals, the first two days were fine weather, but the May 6, 1998 was rainy conditions. For all intents and purposes, the GW levels were steady and remained unchanged during the three days of monitoring. The data clearly showed that the GW table is very close to the failed ground surface at P3, 106 mRL and P2, 104.6 mRL, and exceeded the ground surface of the failed mass at P1 where GWT is 104.5 mRL and ground surface is 103.8 mRL. This agrees with the field observation that water was seeping out of the slope mass at these lower levels continuously, even during fine dry weather. One obvious contribution to this slope failure is ground water seepage exiting from the slope face. The source of the high GWT could possibly be residual water infiltration from the sand-track bed above the slope, despite the sand track being designed with sub-surface drains for rapid discharge of rainfall out of the track area into edge drains. This has the effect of softening the soils around the potential failure plane; especially after cuts, soils were exposed to swelling from release of the large over-burdened pressure.

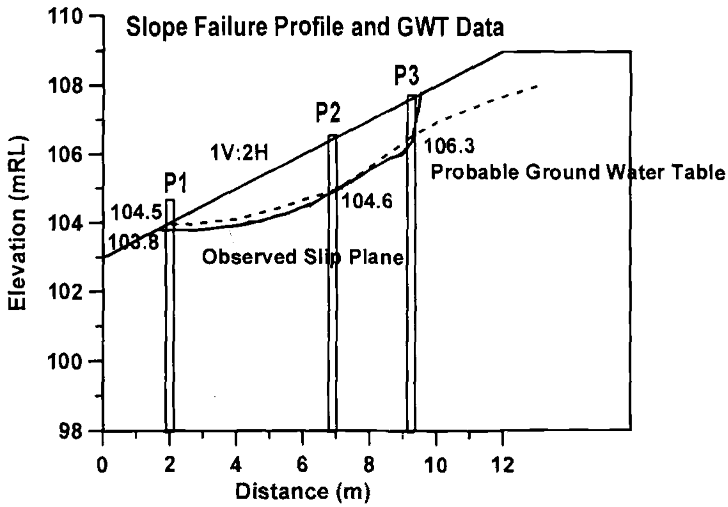


Figure 1 - *Slope failure profile and GWT measurements*

Failure Analysis of Infinite Slope with Seepage

A simple analytical model for analysis of failure is to look at the problem as a shallow slide parallel to the slope face, initiated by tension crack at the scarp level. This model is shown in Fig.2, and several cases were computed to illustrate the progressive nature of the slope failure, as tabulated in Table 1. The factor of safety for an infinite slope failure with parameters given as in Fig.2 (Lambe and Whitman, 1979) is:

$$FS = \frac{c'}{\gamma H \sin\beta \cos\beta} + \frac{(1-(\gamma_\omega h)/(\gamma H))\tan\phi'}{\tan\beta} \tag{1}$$

Where: c' = soil effective cohesive strength (kPa)
ϕ' = soil effective friction angle (degrees)
γ = soil total unit weight (kN/m^3)
γ_w = unit weight of water (kN/m^3)
β = slope angle (degrees)
H = depth to slip surface (m)
h = height of GWT from slip surface (m)

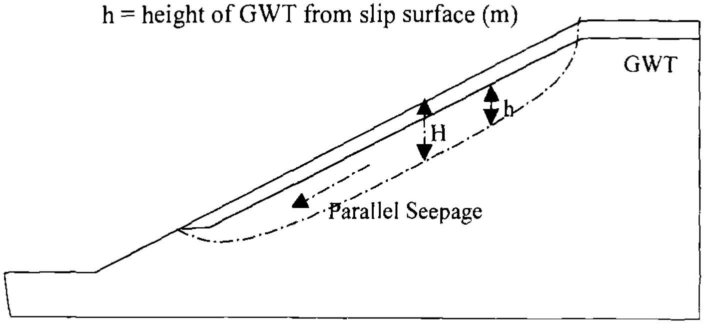


Figure 2 - *Infinite slope failure analysis*

After first cutting the slope to 1:2 gradient, it remained stable at original ground condition based on estimated soil strength from two boreholes (BH16 and BH24) near the site. It is estimated that this residual soil would have c'=10 kPa, ϕ'=22°. At this state, the FS of a dry slope would be 1.74.

The next case is that infiltration will lead to high GWT, which would soften the soil reducing its strength to c'=5 kPa, ϕ'=21°. This would reduce FS to 1.23, provided the slope remains dry. However, judging from the GWT level measurements, the slope was progressively saturated by the downstream discharge flow from the higher landmass behind the slope, beyond the track area. Thus the analysis of cases 4 to 9, showed that if the GWT rises to about 0.5 m below the cut slope surface then FS reduces to 1, leading to shallow slope failures. This is what probably happened at this site. The slope on the opposite bank of the irrigation pond remained dry, as GWT progressively reduced further downstream, causing it to remain stable at its original soil strength.

After the first failure, the softened soil mass became fully soaked, and soil strength is further reduced as in Case 3. Thus re-compaction of dry soil on top of this soaked mass without provision for any internal drains will produce failure. This actually happened at the first attempt by the contractor to repair the slope without any internal drains in the re-compacted soil mass.

Table 1 - *Results of infinite slope stability analysis*

Case	c' kPa	ϕ' deg	γ kn/m^3	β deg	H m	h m	FS	State of Soil
1	10	22	18	26.5	1.5	0	1.74	Dry
2	5	21	18	26.5	1.5	0	1.23	Softened
3	3	20	18	26.5	1.5	0	1.01	Soaked
4	5	21	18	26.5	1.5	0.1	1.20	Seepage
5	5	21	18	26.5	1.5	0.2	1.18	Seepage
6	5	21	18	26.5	1.5	0.4	1.12	Seepage
7	5	21	18	26.5	1.5	0.6	1.06	Seepage
8	5	21	18	26.5	1.5	0.8	1.00	Seepage
9	5	21	18	26.5	1.5	0.9	0.98	Seepage

Design for Permanent Stable Slopes

The average maximum rainfall accumulated in an one-hour period for Changi Airport in Singapore is 78 mm. The extreme maximum ever measured is 147 mm for one hour, which corresponds to a 10-year return storm event. For drainage design, the steady state rainfall used is 150 mm/h. The seepage condition for the slope under very heavy rainfall condition of 150 mm/h can be obtained by FEM analysis using SEEP/W (see Fredlund and Rahardjo, 1993), and the result is shown in Fig.3. Most of the rainfall is conducted into the surface drain through the 0.5 m sand track (about 3.3×10^{-4} m^3/s per m). However, the remaining infiltration (about 3.30×10^{-8} m^3/s per m) will still lead to high GWT rising to the sand subgrade interface level of between 109.5 to 110 mRL. About 1.9×10^{-8} m^3/s per m of this flow will be conducted through the re-compacted residual soil of the repaired slope. This will mean a high GWT in the slope, with seepage exiting on the slope face as shown in the flownet in the Figure 3.

This GWT condition is close to what was observed at the actual slope failure. At high GWT, the phreatic surface would intersect the slope above the pond water level at 104.6 mRL, and exit through the slope face. This would result in softening of the soil, which may lead to eventual slope failure. Using the modified Bishop analysis in SLOPE/W, the computed factor of safety for the slope is 0.923 as shown in Fig.4. The re-compacted soil strength parameters were based on expected soil strengths in the fully soak condition, as in Table 1. Thus, without internal drains, shallow slip failure will occur under long-term drained condition with heavy rainfall of sustained durations.

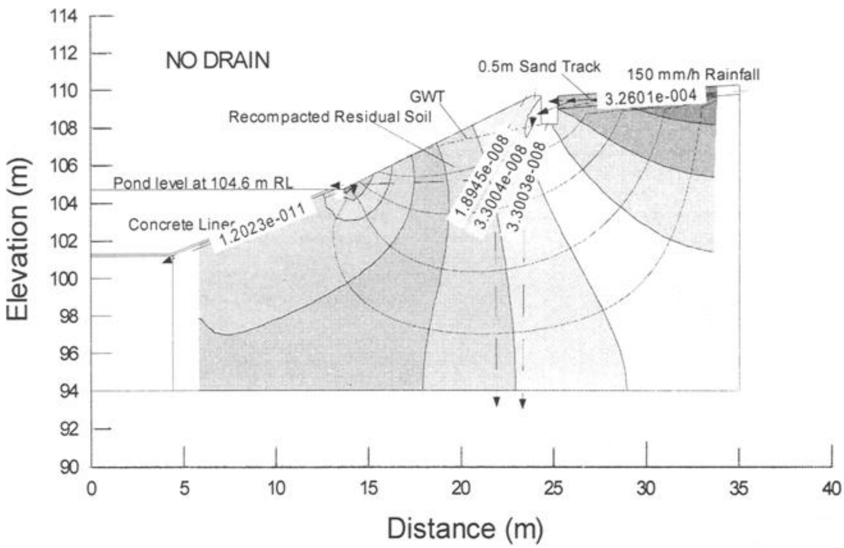


Figure 3 - *Steady seepage under heavy rainfall without internal drain*

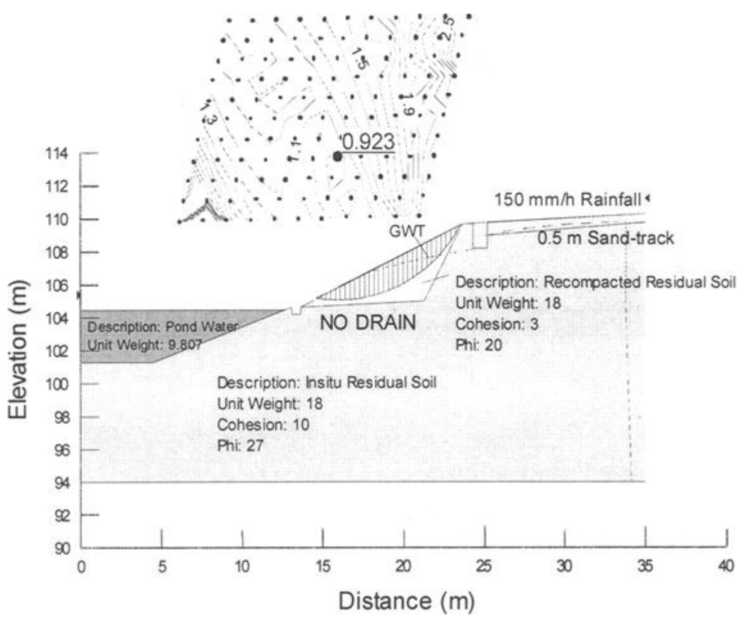


Figure 4 - *Slope analysis for heavy rainfall condition without internal drain*

The permanent solution for a safe slope design is the provision of internal drainage to intercept the GW and conduct it safely below the slope into the concrete lined irrigation pond. Since the estimated seepage into the subgrade below the 0.5 m sand track is of the order of 10^{-7} m^3/s per m, a safe design can be achieved by use of a GEONET drain (about 5 mm thick). Typical GEONETs have transmissivity of about 4.0×10^{-4} m^3/s per m, at a pressure of 100 kPa, tested in accordance with ASTM D4716-95: "Standard Test method for Constant Head Hydraulic Transmissivity In-plane Flow of Geotextiles and Related products." Tests of three different commercial GEONETs at 100 kPa pressure with clay packing at NUS showed transmissivities ranging from 2.2×10^{-4} m^3/s per m to 6.7×10^{-4} m^3/s per m.

Seepage analysis for a GEONET that was laid to a depth of 4 m beneath the re-compacted soil to form the repaired slope is shown in Fig. 5. The GEONET is modeled by soil elements of 100 mm thickness, with transmissivity of the GEONET in use (about 4.0×10^{-4} m^3/s per m). Most of the infiltration (about 3.1×10^{-4} m^3/s per m) from a steady state of 150 mm/h rainfall goes through the 0.5 m sand track and out through the surface drain at the crest of the slope. The remaining infiltration of about 5.21×10^{-8} m^3/s per m, forms the seepage into the subgrade soils and out through the slope. Of this quantity, only 2.18×10^{-12} m^3/s per m flows through the re-compacted fill soils. Thus most of the infiltration into the subgrade is safely conducted out of the slope through the GEONET installed.

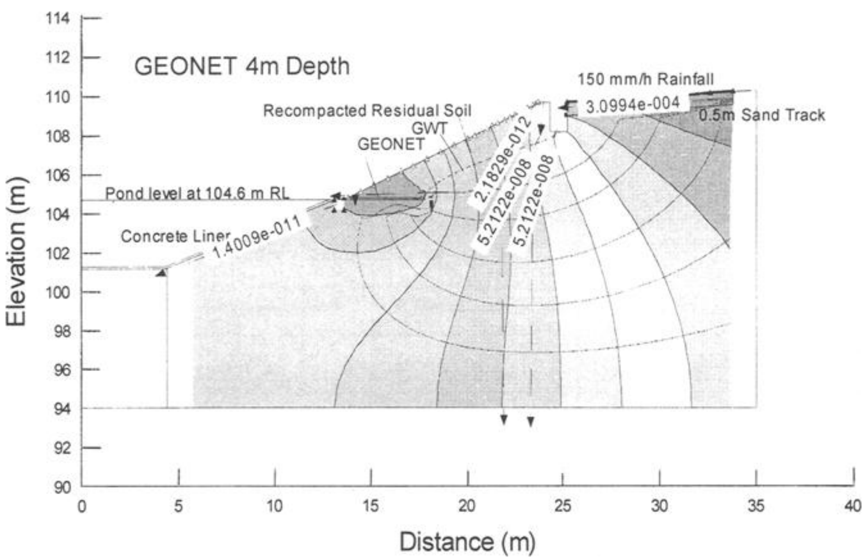


Figure 5 *Steady seepage under heavy rainfall with 4m deep GEONET*

Slope stability analysis of the re-designed slope is shown in Fig.6. It is shown that with the internal drainage provision, even assuming the re-compacted soil is fully soaked, the long-term drained FS is now increased to 1.3, at very high GWT condition. Thus a permanent safe slope is achieved.

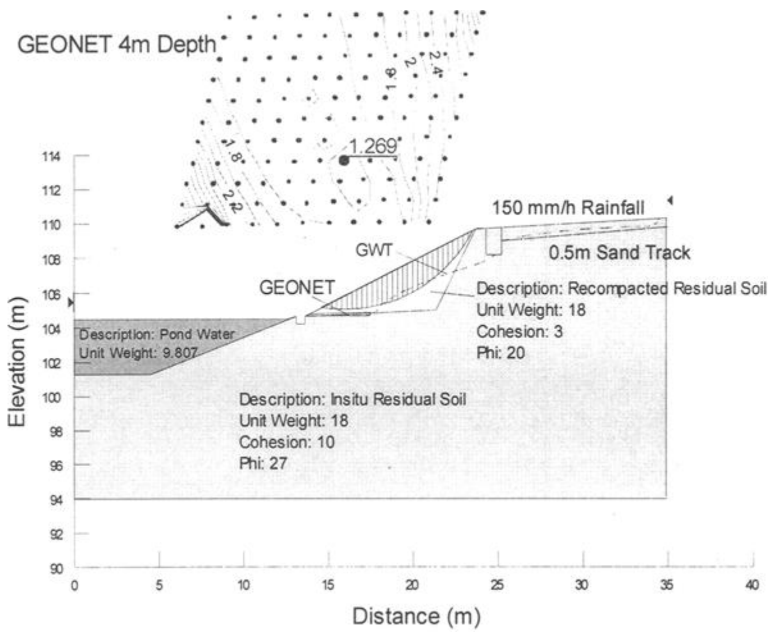


Figure 6 - *Slope analysis for heavy rainfall condition with 4m deep GEONET*

The selected GEONET drain must have drainage capacity of at least 4 times the computed drainage flow from the seepage analysis. This means a discharge capacity of 4 times $5x10^{-8}$ m^3/s per m run of slope, which is $2x10^{-7}$ m^3/s discharge over long-term condition. This can be achieved easily with most commercially available GEONET, which has a transmissivity of about $1.0x10^{-4}$ m^3/s at 200 kPa compression pressure. The GEONET comes with two layers of Geotextile filters, sandwiching the plastic NET drainage layer. This will ensure adequate filtration and prevent soil piping into the GEONET from the silty clay backfill soils.

Figure 7 showed the seepage in the same repaired slope with the GEONET installed to 8 m depth. For this design, a heavy rainfall with high GWT at subgrade level below the sand track, would produce a much lower ground water table within the repaired slope. Nearly all the infiltration into the subgrade is safely conducted out of the slope through the GEONET installed. Thus, we can assume that the re-compacted silty clay backfill will remain relatively drier than the previous case of 4m GEONET depth. For this case, assuming that the re-compacted backfill attained the softened strength of c'= 5 kPa, and ϕ'=21°, a long-term drained FS of 1.6 will be obtained from a slope stability analysis as shown in Fig.8 below.

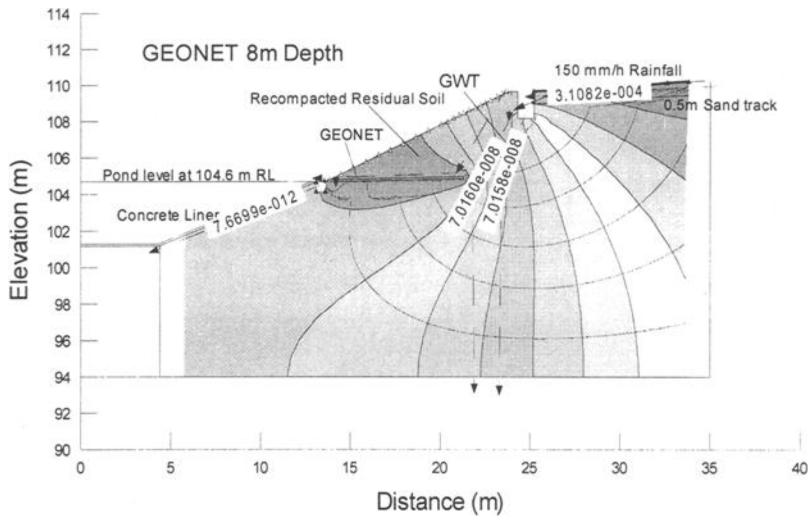


Figure 7 - *Steady seepage under heavy rainfall with 8m deep GEONET*

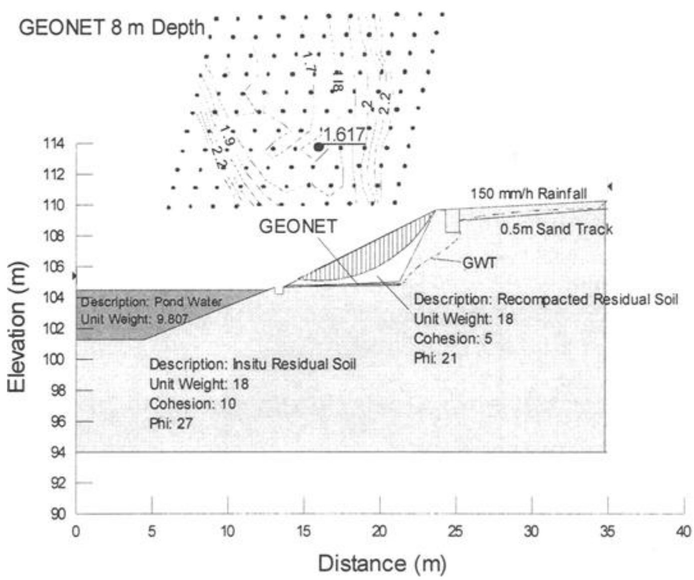


Figure 8 - *Slope analysis for heavy rainfall condition with 8 m deep GEONET*

Parametric Study of Influence of GEONET Installation Depth

A summary of the parametric study of the influence of depth of GEONET installation on the GWT levels in the re-compacted backfill soils, and the long-term drained FS of slope under steady-state heavy rainfall condition of 150 mm/h are presented in Table 2.

Table 2 - *Influence of GEONET depth on GWT and FS of repaired slope*

GEONET Depth (m)	0	1	2	4	8	12	15
GWT at Slope Crest (m RL)	108.1	108.0	107.9	107.6	106.8	104.7	104.7
GWT at Mid-Slope (m RL)	107.1	106.9	106.4	105.7	104.7	104.7	104.7
Seepage into Slope (m^3/s /m)	1.89 x 10^{-8}	1.72 x 10^{-9}	9.80 x 10^{-12}	2.18 x 10^{-12}	< 1.0 x 10^{-12}	< 1.0 x 10^{-12}	< 1.0 x 10^{-12}
Soil State in Slope	Fully Soak	Fully Soak	Fully Soak	Fully Soak	Soften	Comp-acted	Comp-acted
Drained cohesion c' (kPa)	3	3	3	3	5	10	10
Drained friction angle, ϕ' deg	20	20	20	20	21	22	22
Drained FS	0.923	0.968	1.137	1.269	1.617	1.780	1.808

The results showed that internal drains at the base of the repaired slope installed to depths of 8m to 15m would result in dry slope conditions despite the heavy rainfall. This would ensure a slope stability factor of safety of more than 1.5, which is the long-term drained design condition adopted in Singapore. To achieve such depth of drain penetration with GEONET would be quite difficult, especially as the racetrack above has already been constructed before the occurrence of slope failure. Thus the proposed solution is to achieve equivalent internal drainage capacity through the installation of 15 m deep perforated GEO-PIPE subsoil drains protected from clogging by tightly fitted geotextile filter wrappings. The selected geotextile filter must satisfy both the soil retention, as well as the permeability criteria as shown in Koerner's (1998). The geotextile wrapped around the geopipe, and is secured against the geopipe by nylon strands at 250 mm intervals.

The modified Manning's equation for discharge of pipe flow is given in Koerner, (1998) as:

$$Q = 1.137 A R_H^{0.66} S^{0.5} \qquad (2)$$

where,

Q = flow rate (m^3/s)

A = flow cross section area (m^2)

R_H = hydraulic radius = R/2 for full flow (m)

S = slope or gradient (m/m)

For a 75-mm internal diameter smooth wall geo-pipe on a 1:25 gradient, the estimated maximum discharge capacity is 1.15×10^{-4} m^3/s, which is the approximate transmissivity of an equivalent GEONET, when these pipes are spaced at 1 m intervals.

Construction of Repaired Slope

To re-construct the slope without inducing further failure, the repair job must be done panel by panel and not by stripping the entire failed slope all at once. The fully soaked residual soil that has slid was removed completely to base level at Reduced Level of 104 m, until fresh residual soil in its original in-situ state was exposed. Next, the exposed soil was compacted to produce a firm stable base that a GEONET (Polyfelt DC 4514-2 or its equivalent) can be placed on the clean-cut bench at the base of the excavated slope. The residual soil fill can then be re-compacted back layer by layer (each lift about 300 mm) until the top of the slope is reached. Final trimming and turfing should be done once the whole slope has been re-constructed.

For the actual re-construction, the contractor chose to repair the slope with GEO-PIPES protected with geotextile filters, instead of GEONETs. This is a more economical solution as compared to the use of GEONETs. As the race-track was already completed, it was too risky to cut-back the slope for installation of GEONETS under the base of the repaired slope. The contractor preferred to repair the slope with minimal cutting back, and instead create a flatter stable slope in place of the failed slope. Next smooth wall perforated GEOPIPES of 75-mm internal diameter, wrapped with geotextile filter layer around its outer perimeter, were installed into sub-horizontal holes drilled to 15 m depth at 1.5 m intervals, after the slope has been repaired. The installation was made by first boring a 15m depth uncased hole of 100mm diameter in the repaired slope, with a machine auger under dry weather condition. Immediately after drilling, the GEOPIPE with the bottom end closed is placed into the hole. With time, the soil would move in to fill the annular gap between the GEOPIPE and the borehole wall. Once internal drainage is established, the repaired slope was trimmed back to the design slope of 1(V):2(H) gradient.

Conclusions

A slope failure has occurred in relatively competent cut residual soil despite a relatively gentle 1(V):2(H) slope profile. Upon further investigation, it is shown that high water table in the cut slope as well as large stress relief from removal of overburden has resulted in rapid soil softening by the introduction of water via rainfall infiltration into soils which negated high suction. A repair strategy using geosynthetic internal drains beneath the repaired slope would be a cost-effective solution to the problem. FEM seepage analysis together with slope analysis showed that the GEONET drain or its equivalent would provide an effective interceptor drain to the high GWT and conduct the water safely out of the slope below the re-compacted soil zone. This would ensure that the re-compacted soil would not soften from the GWT intrusion from the back of the slope. Thus an in-expensive method of slope repair for a long-term permanent safe slope can be obtained.

References

Fredlund,D., and Rahardjo,H. (1993), *"Soil Mechanics for Unsaturated Soils"*, John Wiley & Sons.

Koerner,R.M. (1998), *"Designing With Geosynthetics"*, Prentice Hall.

Lambe, TW, and Whitman, RV (1979), "Soil Mechanics", John Wiley & Sons.

SEEP/W Users' Manual (1997), by GEO-SLOPE International, Calgary, Canada

SLOPE/W Users' Manual (1997), by GEO-SLOPE International, Calgary, Canada

John C. Stormont[1] and Tomas B. Stockton[2]

Preventing Positive Pore Water Pressures with a Geocomposite Capillary Barrier Drain

---

Reference: Stormont, J. C. and Stockton, T. B., **"Preventing Positive Pore Water Pressures with a Geocomposite Capillary Barrier Drain,"** *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390*, L. D. Suits, J. B. Goddard, and J. S. Baldwin, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: The Geocomposite Capillary Barrier Drain (GCBD) has been developed and tested to prevent positive pore water pressures from developing by laterally draining water while it is still in tension. The GCBD consists of two key layers that function as long as the water pressures in the system remain negative: (1) a transport layer that laterally drains water and (2) a capillary barrier layer that prevents water from moving downward. Prototype GCBD systems have been tested in a 3 m long lateral drainage test apparatus. For most test conditions, the GCBD systems drained water under negative pressures at a rate sufficient to prevent any positive water pressures from developing in the overlying soil. Further, the drain system served as a barrier as it prevented downward flowing water from moving into the underlying soil.

Keywords: unsaturated flow, capillary barrier, lateral drainage, geocomposites

Introduction

Drainage of water from soils is typically considered to be a saturated flow process. There is, however, a wide range of applications where it would be beneficial if water could be drained prior to saturation, that is, while the soil pore pressures remain negative. For example, positive water pressures in the base course layer within a pavement section can reduce its strength and lead to rutting, heaving, and pavement failure. Even open-

---

[1] Associate Professor, Department of Civil Engineering, University of New Mexico, Albuquerque, NM, 87131.
[2] Graduate Research Assistant, Department of Civil Engineering, University of New Mexico, Albuquerque, NM, 87131.

graded materials, which are very permeable under saturated conditions and minimize the build-up of positive pore water pressures in the base course, do not prevent the sub-grade materials from becoming moist. Moisture content changes in the sub-grade soil can result in heave, shrinkage, and change in strength, all of which can affect pavement performance. Drainage of water from pavement layers prior to saturation will probably improve the pavement's performance and longevity. Another application of drainage prior to saturation is where stability is of concern. Design procedures for earth structures such as embankments and retaining walls often include provision for drainage but not until the soil has become saturated. Stability will be enhanced if drainage can maintain negative water pressures in the soil, resulting in an effective cohesive strength component due to soil suction, while also preventing positive water pressures. Waste site covers in dry climates provide another application example. Capillary barriers are simple, low-cost surface cover systems that limit percolation into underlying wastes. These finer-over-coarser soil systems function as barriers to downward water flow as long as the finer layer does not approach saturation. Lateral drainage of water above the finer-coarser soil interface prior to saturation will preserve the capillary barrier and prevent percolation.

A system to drain water while soil pore pressures remain negative has been developed from geosynthetic materials, and is referred to as a Geocomposite Capillary Barrier Drain (GCBD) [1]. In contrast to conventional drainage systems, this drainage system is designed to operate under negative water pressures associated with unsaturated conditions. The GCBD concept evolved from investigations of unsaturated water movement in near-surface soils. Lateral water movement (drainage) in unsaturated soils can occur when downward moving water encounters dipping layers with different properties. This process is enhanced if the overlying soil is finer than the underlying soil and a capillary barrier is formed. In this case water accumulates near the fine-coarse interface, and because hydraulic conductivity of an unsaturated soil increases with water content, lateral drainage will be concentrated in this region. The soil moisture content will increase in the downdip direction due to the lateral diversion of the downward moving water at the interface. A distance termed the drainage or diversion length is commonly used to describe the length along the fine-coarse interface which water is diverted before the soil moisture content increases to the point where appreciable breakthrough into the underlying soil occurs as shown in Figure 1a.

The effectiveness of this approach depends on the unsaturated hydraulic properties of the finer and coarser-grained layers, the slope of their contact, and the infiltration rate. In general, the lateral diversion lengths of these finer-over-coarser systems are relatively short (less than 10 m) when typical near surface soils such as loams and silts are used as the finer layer [2]. The relatively low hydraulic conductivities of these soils limit the amount of water that can be transported under unsaturated conditions, and thus limits lateral drainage capabilities.

The unsaturated drainage of soils can be increased substantially by placing an intermediate transport layer such as a fine-grained sand between the overlying soil and the underlying coarse material (Figure 1b). The intermediate material should be conductive enough to laterally divert or drain downward moving water, yet remain unsaturated so as to preserve the capillary break with the underlying coarse material.

Experimental and numerical investigations indicate that for specific materials and conditions, unsaturated soil drainage using fine-sands as the transport layer and gravels as the capillary break layer can be effective [3]. However, these systems have shortcomings, including the soils for the transport layer and capillary barrier layer may not be readily available at the site and thus can be costly, and the materials can be difficult to place on many slopes and locations.

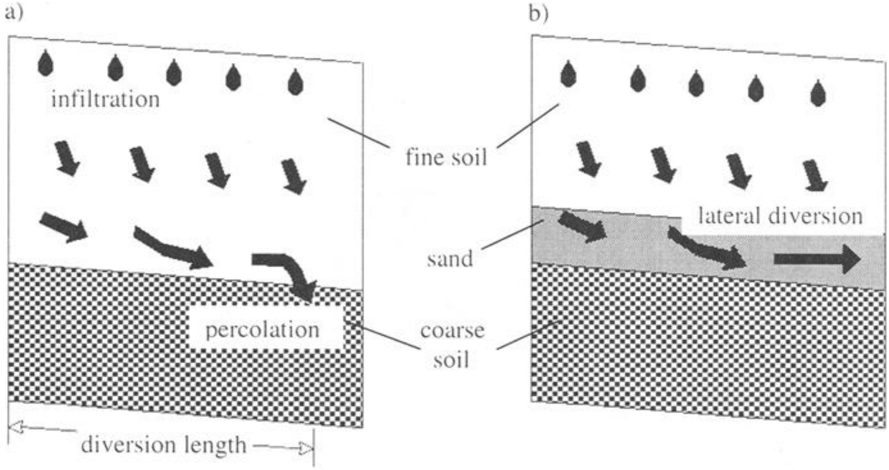


FIG. 1 – *Lateral drainage in unsaturated soil due to (a) simple capillary barrier, and (b) inclusion of transport layer.*

An unsaturated soil drainage system fabricated from geosynthetics has a number of advantages compared to a soil-based system, including:

1. desirable properties can be optimized by design and controlled by manufacture,
2. drainage functions can be combined with other functions such as reinforcement and soil retention,
3. a geosynthetic system will be thinner (on the order of only a few cm), minimizing its impact on the overall project design, and
4. geosynthetics can be readily delivered throughout much of the world.

A schematic of a geosynthetic-based unsaturated drainage, referred to as a Geocomposite Capillary Barrier Drain (GCBD), is shown in Figure 2. The GCBD system is comprised of three layers that are, from top to bottom: a *transport* layer, a *capillary barrier* layer, and a *separator* layer. Some non-woven geotextiles can be used as transport layers, while a geonet with relatively large, open pores can function as a capillary break. The separator layer simply prevents underlying soil from intruding into the pore spaces of the capillary barrier layer. A non-woven geotextile is envisioned for this function. This configuration can also laterally drain upward moving water. In this case, the lower layer would serve as the transport layer.

Although this geocomposite outwardly resembles a conventional geocomposite drain, a GCBD is designed to drain water in the geotextile (not the geonet) under negative water pressures (not positive water pressures). Further, it does not require the underlying impermeable layer that a conventional drain requires. In the GCBD configuration, it is the unsaturated hydraulic properties of the geosynthetic materials that are of principal importance. In this paper, unsaturated hydraulic properties of geosynthetic materials used in GCBD systems are given. The results from a series of drainage tests on prototype GCBD systems are presented that demonstrate the performance and potential of these drainage systems.

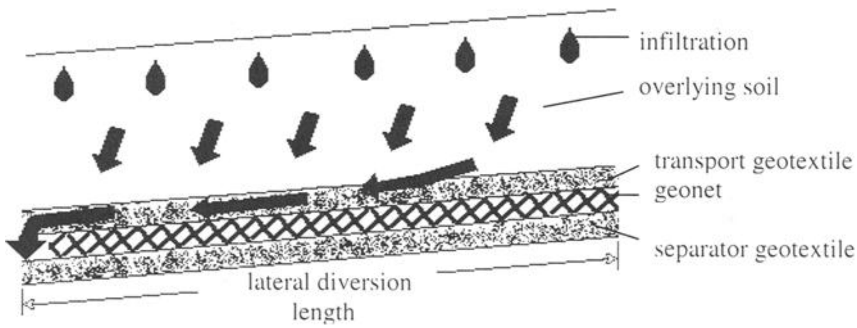


FIG. 2 - *Schematic of Geocomposite Capillary Barrier Drain (GCBD) configuration.*

Unsaturated Hydraulic Properties of Geotextiles

An important consideration is whether certain geosynthetic materials possess properties consistent with the functions of the transport and capillary barrier layers. The capillary barrier layer should have large, open pores to prevent water moving into the layer until the water pressures are nearly positive, similar to a coarse, uniform sand or gravel. Geonets have been demonstrated to effectively serve as capillary barriers to upward unsaturated water movement and consequently frost heave [4].

The transport layer should be conductive under low to moderate values of suction head in order to drain water under negative water pressures. Because of their large saturated transmissivities and applications in saturated drainage systems, non-woven geotextiles are a likely candidate material to serve as the transport layer. Only a few studies have provided insight into the unsaturated transport properties of non-woven geotextiles. Measurement of the contact angle of polymer fibers with water reveal that common polymers used to fabricate non-woven geotextiles are only slightly wetting with respect to water [5]. Capillary rise tests in geotextile strips indicate that water will rise above a free water surface in some non-woven geotextiles, while other geotextiles are hydrophobic and require a positive pressure before they will wet [6]. Once saturated, some geotextiles have the capacity to "siphon" water [7]. These studies suggest certain geotextiles will retain substantial hydraulic conductivity even under suction heads of 10 cm or more.

To more fully characterize the unsaturated hydraulic properties of non-woven geotextiles, test methods have recently been developed to measure the water retention function and unsaturated transmissivity of non-woven geotextiles. These methods are briefly described and results are given below for two non-woven geotextiles that were subsequently used as transport layers in GCBD systems. Some basic properties of these geotextiles are given in Table 1.

Table 1 - *Properties of GCBD components.*

Geosynthetic	Polymer type	Thickness, mm	Saturated transmissivity, mm^2/s
Geotextile A	Polypropylene	5.9	39.0
Geotextile B	Polyester	1.8	6.8
geonet	High-density polyethylene	5.9	N/A

The water retention function describes the relationship between water content and negative water pressure (suction head). The water retention function can be obtained by testing geotextile specimens in a hanging column apparatus [8]. The hanging column is appropriate for suction heads of about 200 cm or less, which is the range of interest for non-woven geotextiles. A water retention function is obtained by systematically adjusting the suction head in the specimen, waiting for equilibrium, and weighing the sample to obtain the water content. From the water retention function, the breakthrough head or water entry head can be determined. The breakthrough head is the suction head at which an initially dry medium will first permit water to form a continuous network through the medium and consequently become conductive. The water entry suction head represents the transition of a material from a hydraulically nonconductive to a conductive state. Thus, if water in contact with the geotextile is at a suction head in excess of the geotextile's water entry suction head, water will not flow into the geotextile. For transport layers, the greater the breakthrough head, the greater the suction heads at which the adjacent soil will be drained.

The water retention functions are given in Figure 3 for the two geotextiles. The results are given for the range of 0 to 30 cm, as this is the region over which most of the water content changes occur. The specimens were first tested under a wetting path: beginning air-dry and progressively decreasing the suction head to zero. The specimens were then dried by progressively increasing the suction head.

The sharp uptake of water during specimen wetting suggests that these geotextiles have a water entry suction head between 3 and 5 cm. The geotextiles did not fully saturate even at suctions near zero. The water retention functions exhibit hysteresis: the specimens contained more water during drying than wetting at comparable values of suction head. The water content did not decrease significantly during the initial portions of the drying path, indicating that once wetted, some geotextiles may remain substantially wetted under small suction heads.

The transmissivity of geotextiles under suction has been measured in the permeameter shown in Figure 4. The body of the permeameter consists of two reservoirs of water connected by a platform. The geotextile lies on the platform, and extends into the reservoirs on both ends. The water level in the reservoirs will be at or below the platform level. When placed in the permeameter, water will rise in the geotextile due to capillary action. The water that moves into the geotextile will be under a suction head

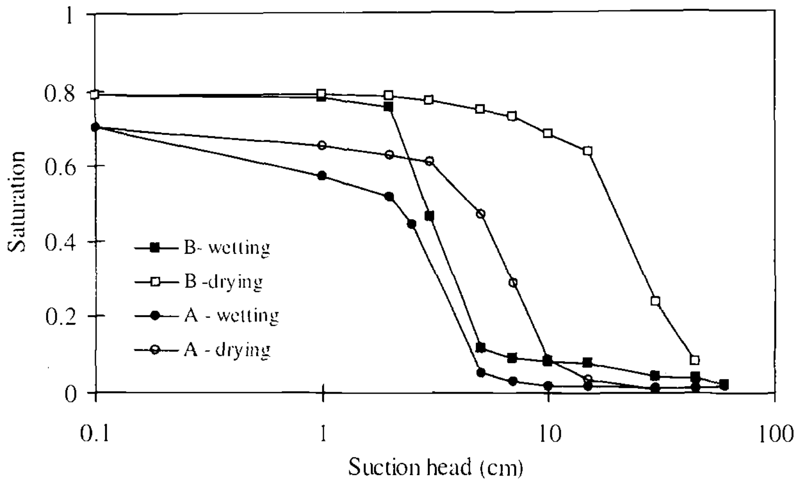


FIG. 3 – *Water retention functions for geotextiles A and B.*

equivalent to the distance it is above the water level in the reservoir. Tensiometers are built into the bottom of the platform to monitor the suctions within the geotextile specimen. A pressurized bladder system permits normal pressures from 0 to 100 kPa to be imposed on the geotextile to simulate overburden pressure. To induce flow, a total head difference is created between the ends of the geotextile by raising one reservoir relative to the other. Transmissivities were calculated using the steady-state solution used to calculate transmissivity under positive pressures given in ASTM Test Method for Determining the (In-Plane) Flow Rate per Unit Width and Hydraulic Transmissivity of a Geosynthetic Using a Constant Head (D 4716-95). The suction head in the geotextile is assumed to be constant along its length during flow, although it does vary a small amount.

Transmissivities are given in Figure 5 as a function of suction head for geotextiles A and B. These measurements were made under a nominal normal pressure of <1 kPa, and with a gradient of 0.25. The test duration varied from 5 to 75 min. During initial wetting, the geotextiles were non-conductive until the suction was reduced to 2.5 cm for geotextile A and 3.5 cm for geotextile B. The transmissivity of both specimens increased by about an order of magnitude as they were wetted to near zero suction heads. The geotextiles remain transmissive to beyond 10 cm, well beyond the suction at which they initially became transmissive. Geotextile A is more transmissive than geotextile B under all values of suction, consistent with their reported saturated transmissivity values.

The water retention and transmissivity data are consistent. During wetting, the water retention function indicates that the geotextiles do not accept much water until the suction head was reduced to less than 5 cm; the geotextiles had an immeasurably low transmissivity until the suction head was reduced to about 3 cm. The geotextile rapidly takes up water as the suction head is reduced to zero, and coincides with the transmissivity increasing by an order of magnitude. During drying, both the water content and transmissivity of the geotextile remain at greater values compared to the values obtained during wetting. These results suggest that these non-woven geotextiles will wet and become transmissive under suction and thus function as a transport layer, but not until the suction in the adjacent soil is reduced to about 3 cm.

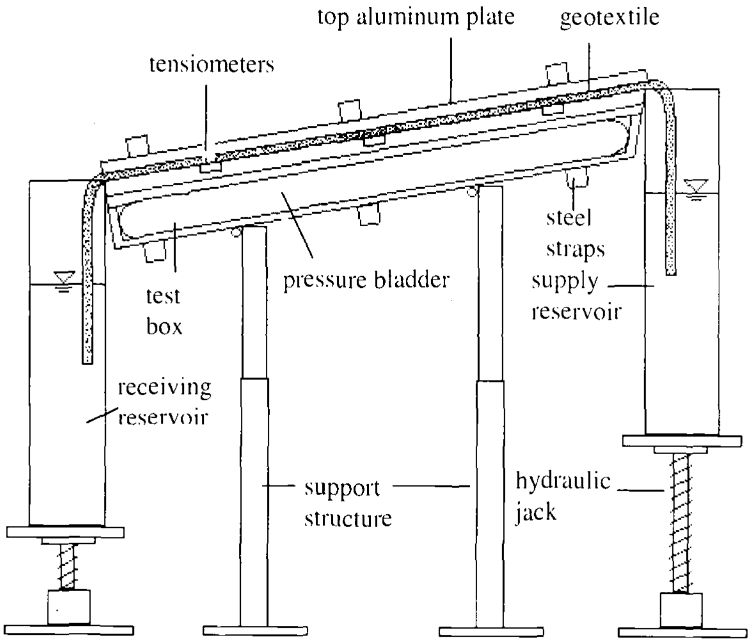


FIG. 4 - *Permeameter for measuring transmissivity of geotextiles under suction.*

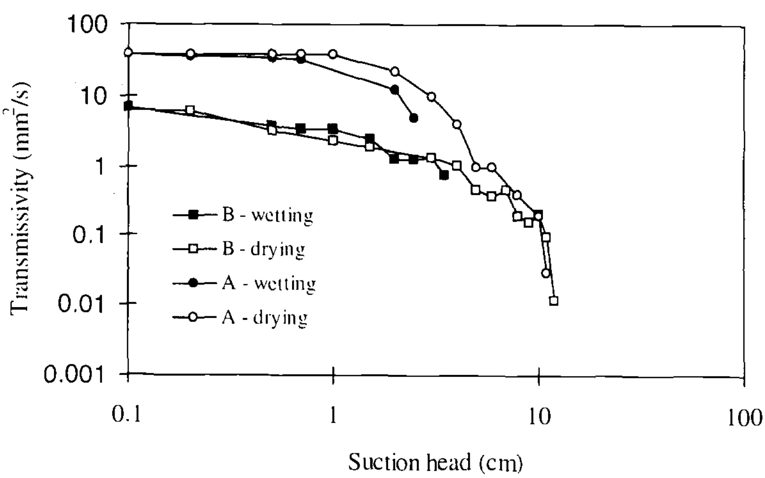


FIG. 5 – *Transmissivities under suction for geotextiles A and B.*

Methods and Materials

Lateral DrainageTest Apparatus

The drainage capacity of GCBD systems was tested in a 3 m long soil box (Figure 6). The profile tested consisted of 10 cm of an underlying soil, the GCBD, and 5 cm of an overlying soil. The underlying and overlying soil layers are also referred to as the sub-grade and base course soils, respectively, in reference to the possible location of a GCBD within a pavement section. Measurements were made of water infiltrated onto the top of the soil profile, water drained out of the GCBD, water laterally drained in the overlying soil and water produced out of the bottom of the sub-grade soil. Measurements were also made of soil suction above and below the GCBD. Post-test water content measurements were made in the sub-grade soil.

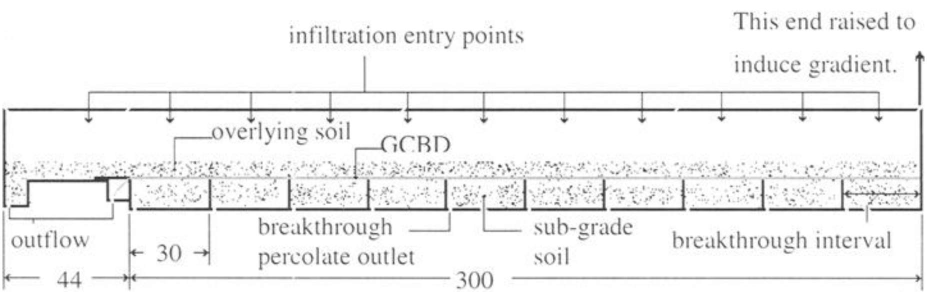


FIG. 6 – *Lateral diversion test apparatus (dimensions in cm).*

The clear acrylic box used to contain the GCBD - soil system was 3 m long, 30 cm wide, and 45 cm high. The soil box was supported by a wood frame that enabled it to be propped at a range of angles between 0 and 30 degrees to induce different hydraulic gradients. Lids to limit evaporation were placed on top of the box.

Interval separators in the bottom of the box discretized the sub-grade soil into ten 10 cm high by 30 cm long intervals. Drains were cut into the bottom of each interval so water percolating into the interval could be collected. These breakthrough intervals served as a means to deduce the location of breakthrough through the GCBD.

The lower end of the soil box was configured to collect water that drained laterally in the overlying soil as well as the transport layer. The GCBD terminated in a collection interval into which the transport layer draped. As water moved to the lower end of the transport layer, it saturated the transport layer and was drained into a container that was used to measure outflow. The collection interval for the overlying soil was located some distance past the end of the GCBD. As water moved into the collection interval, the saturated soil drained into a container.

The sub-grade soil contained in the intervals below the GCBD was packed into each separate interval at a dry density of 1.6 g/cc and a water content of about 8%. The soil was graded to be level with the top of each interval separator to confine breakthroughs to a particular interval. The GCBD was installed in the soil box directly on top of the sub-grade soil. The overlying soil was then placed and compacted on top of the GCBD to a dry density of 1.6 g/cc and a water content of 8%.

The suction measurement system consisted of 20 tensiometers connected to a data logging and control system. The tensiometers were constructed from porous cups that were buried in the soil. These porous cups were connected to water-filled tubes. Pressure transducers were connected to the tubes where it exited the soil box.

Ten tensiometers were installed above the GCBD spaced equally along the soil box, and 10 tensiometers were installed below the drain system in the breakthrough intervals. The tensiometers above the drain system were installed with the porous cups buried in the soil 0.5 cm above the soil-drain system interface. The tensiometers below the drain system were installed with the porous cups buried in the sub-grade soil 1 cm below the separator geotextile. The tensiometers were regularly inspected and de-aired as necessary.

Water was added to the top of the soil profile with a manifold-type distribution system. Ten adjustable drip irrigation emitters with control valves to adjust the delivery rate were installed in the manifold. The manifold was hooked directly into a 50 cm tall constant head supply bottle. Ten tubes directed the water droplets from each emitter through the lid and onto the top of the soil profile. The manifold was mounted to the wall so it was level and the head at each emitter was the same. Inflow rates were determined from the change in water level in the supply bottle. The infiltration rate from the system was regularly monitored and the emitters were adjusted to maintain uniform infiltration across the top of the soil box. The infiltration range used with this system varied from a minimum flow rate of 1.0 cc/min (a flux of 1.9×10^{-6} cm/sec) to 30 cc/min (a flux of 5.6×10^{-5} cm/sec). The soil located past the end of the GCBD collection interval was infiltrated with water to minimize the influence of this soil on suction gradients and subsequent flow in the GCBD.

Infiltration was continued until the rate of laterally drained water was steady and was greater than 90% of the infiltration rate. Tests were sometimes interrupted and continued the next day if steady-state was not reached after approximately 12 hours. Collection of drained water continued after infiltration was stopped until the production approached zero. At the conclusion of many of the tests, the apparatus was disassembled to obtain soil samples for determination of water contents to confirm or refute breakthroughs indicated by the tensiometers.

Materials

Two GCBD systems were tested using the lateral diversion apparatus. The first GCBD system, designated GCBD-A, consisted of a geonet sandwiched between two polypropylene geotextiles (Geotextile A). The second GCBD system, designated GCBD-B, used the same geonet sandwiched between two polyester geotextiles (Geotextile B). Two soils were used in the lateral diversion apparatus. The underlying soil was a clayey sand (designated SC by the USCS classification method). This soil, which is representative of much of the near-surface soils in New Mexico, had 35% fines, a plasticity index of 8, and a saturated hydraulic conductivity of 1.4×10^{-4} cm/sec. The overlying soil was either the SC soil or a poorly-graded silty gravel (designated GP-GW). The GP-GW is commonly used as a base course material in New Mexico soil and was obtained from a local sand and gravel supplier. The GP-GW soil had 7% fines, no measurable plasticity, and a saturated hydraulic conductivity of 1.3 x 10^{-2} cm/sec.

Results and Discussion

Summary of Tests

The drainage tests conducted with the GCBD systems are summarized in Table 2. The tests on GCBD-A included two slopes, two different overlying soil types, and various infiltration rates. A test was conducted with only a geotextile (A) placed in the soil profile in place of the complete GCBD system. Two additional tests were performed using the GCBD-B system. If breakthrough did not occur, the diversion length in Table 2 is reported as greater than the apparatus length for a particular test. For tests in which there was breakthrough into the sub-grade soil, the diversion length is given as the distance to the beginning of the interval in which breakthrough was detected.

Test data are presented in Figure 7 as the infiltration rate, the drainage rate from the GCBD and the drainage rate from the overlying soil. These data were selected from the portion of the test in which lateral drainage was at its greatest measured value. Differences between the infiltration rate and the rate of collected water are principally attributed to water storage changes within the soils, which were not measured directly. Although the apparatus was covered, limited evaporation of infiltrating water represents another water balance component that was not measured.

Table 2 - *Summary of drainage tests.*

Test no.	Slope, %	GCBD System	Overlying soil type (USCS)	Peak infiltration flux, cm/sec	Breakthrough ?	Measured diversion length, cm
1	9.0	A	SC	7.9 x 10^{-6}	No	> 300
2	9.0	A	SC	1.1 x 10^{-5}	No	> 300
3	9.0	A	SC	2.5 x 10^{-5}	No	> 300
4	2.5	A	SC	7.7 x 10^{-6}	No	> 270
5	2.5	A	SC	4.7 x 10^{-5}	Yes	240
6	2.5	A	GP-GM	1.1 x 10^{-5}	No	> 300
7	2.5	B	GP-GM	7.9 x 10^{-6}	Yes	210
8	2.5	B	GP-GM	4.7 x 10^{-6}	No	>300
9	2.5	GT (A)	SC	4.7 x 10^{-6}	Yes	30

Tests 1 through 3, conducted on a 9% slope, demonstrated that the GCBD-A could drain infiltrating water without breakthrough. The slope was reduced to 2.5% for the remaining tests. Test 4 laterally diverted all of the infiltrating water, but an experimental problem reduced the apparatus length to 270 cm. It was not until the infiltration rate was increased to 4.7×10^{-5} cm/s during Test 5 that the capacity of GCBD-A was exceeded and breakthrough occurred at a diversion length of 240 cm. Test 6 was conducted with the GP-GW soil overlying the GCBD system. Comparison of these results with those of Test 4 indicates more water was laterally drained within this soil above the GCBD compared to the SC soil.

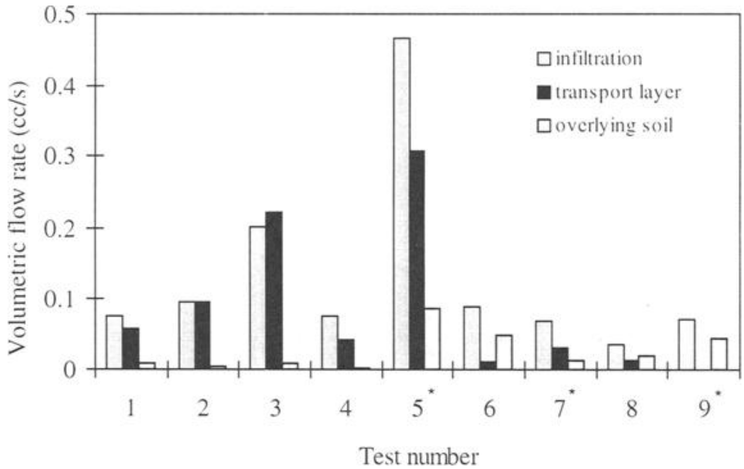


FIG. 7 – *Summary of measured volumetric flow rates during drainage tests. Asterisks denote tests in which there was breakthrough into the sub-grade soil.*

The GCBD-B systems were tested with the GP-GM as the overlying soil. With an infiltration rate of 7.9×10^{-6} cm/s, there was breakthrough into the eighth breakthrough interval to yield a breakthrough length of 210 cm for Test 7. After this test was stopped, the lids covering the apparatus were removed and the system was allowed to dry for 9 days. The configuration was re-tested (Test 8) at a somewhat slower infiltration rate. There was no indication of breakthrough, demonstrating that the functional capacity of GCBD can be "restored" after breakthrough.

Test 9 utilized just Geotextile A rather than a complete GCBD-A system. This test resulted in a diversion length of 30 cm, with water produced into all but the first breakthrough interval. No measurable water was laterally drained in the geotextile. This result is in sharp contrast to comparable tests with the GCBD-A system (e.g., Test 4), and demonstrates that lateral drainage in unsaturated soils requires the combined transport layer-capillary barrier layer configuration.

Tests with No Breakthrough

The tests in which the reported diversion length was greater than the test apparatus length demonstrated the ability of the GCBD systems to divert water without breakthrough. During these tests, water did not move downward through the GCBD system into the underlying soil. Suctions measured in the soil immediately below the GCBD remained nearly constant during the tests, typically at values of between 100 and

400 cm. Further, the water contents of the sub-grade soil after the tests were essentially identical to the as-placed water contents prior to infiltration.

The suctions in the overlying soil indicated the soil above the GCBD remained in tension as water was laterally drained. Typical suction histories for Test 1 are given in Figure 8 along with water balance data. Only three of the 10 tensiometers above the GCBD are reported in the figure for clarity as they all had similar responses. The suctions were reduced to 2 to 5 cm in response to infiltration. These values are consistent with the expected water entry heads of the geotextiles that served as the transport layer (see Figures 3 and 5). The suctions remained at these values as water is produced from the GCBD's transport layer. The overlying soil also laterally drained some water while the water pressures remained negative. Once infiltration was stopped, the suctions increased to about 10 cm and the lateral drainage from the GCBD and the overlying soil slowed and eventually stopped.

Tests with Breakthrough

Two of the tests involving GCBD systems experienced breakthrough into the sub-grade soil. Breakthrough was first indicated from the response of the tensiometers below the GCBD; however, air in the tensiometers could induce a response similar to breakthrough. When one or more of the tensiometers below the GCBD noticeably dropped, the tensiometer was de-aired and closely monitored. An example of a tensiometer response due to breakthrough is given in Figure 9.

Because the tests with breakthrough were not of sufficient duration to produce percolate from their underlying drains, failure was confirmed by post-test water content measurements in the sub-grade soil. The samples for these measurements were obtained immediately adjacent to the tensiometer location at the center of the breakthrough interval. The post-test water contents for Tests 5 and 7 are given in Figure 10. With the exception of the measurement at 255 cm for Test 5 and 225 cm for Test 7, all of the water contents are within 1% of the pre-infiltration values. The single elevated measurement in each test coincides with the location that the tensiometer indicated breakthrough. Downdip of the breakthrough location the sub-grade soil did not have an elevated water content, indicating breakthrough occurred at a discrete location rather than over a continuous area. This result is presumably because breakthrough reduced the amount of water in the transport layer downdip of the breakthrough location to an amount that was within the drainage capacity of the transport layer.

Drainage Capacity Models

The drainage capacity of the GCBD is limited by the transmissivity of the transport layer. The maximum transmissivities of the transport layer during drainage testing can be estimated from

$$\theta = \frac{J}{i} \qquad (1)$$

where J is the flux per unit width of the geotextile and i is the hydraulic gradient. The measured drainage rate from the transport layer divided by its width is taken as the maximum flux. Because the suctions were nearly constant in the overlying soil along the length of the apparatus, the gradient was assumed to be due solely to gravity and equivalent to the slope. The maximum calculated transmissivity during the drainage tests is given in Table 3, along with the saturated transmissivity value for the geotextile. The maximum transmissivity approached the saturated value for the tests that experienced breakthrough (Tests 5 and 7), consistent with the hypothesis that the GCBD will function as long as the transport layer remains at negative water pressures associated with unsaturated conditions.

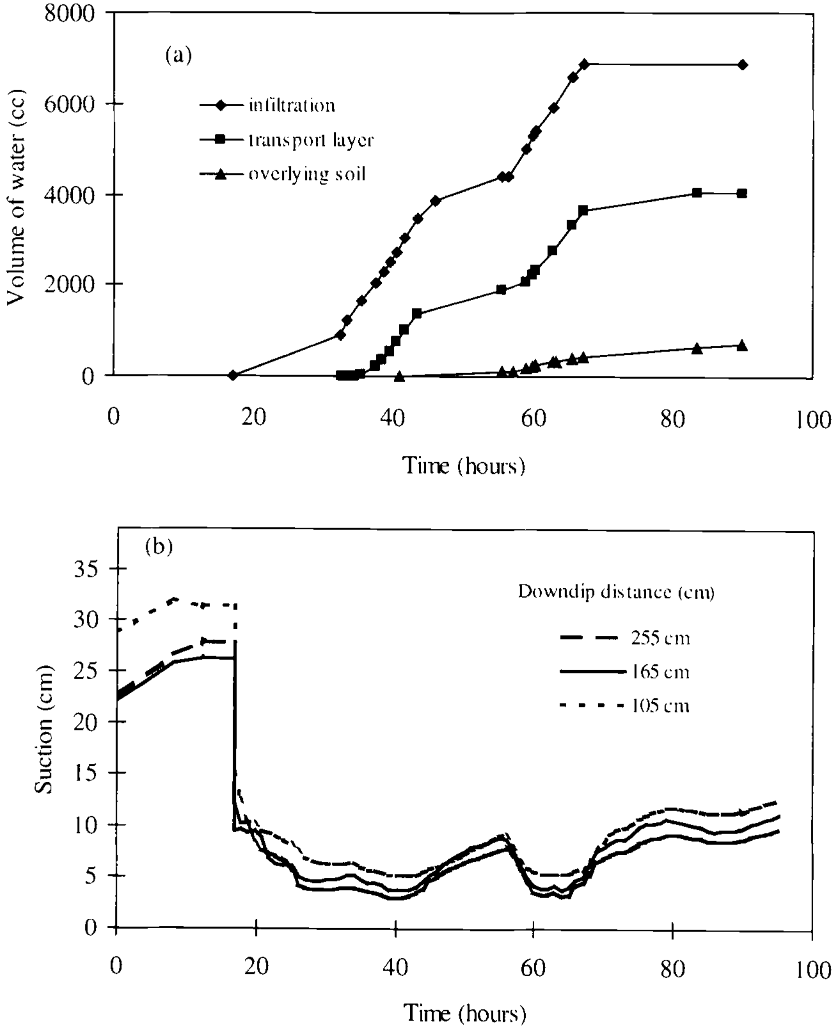


FIG. 8 - *Results from Test 1. (a) History of infiltration, drainage from transport layer and drainage from overlying soil, and (b) suction histories at three locations above GCBD.*

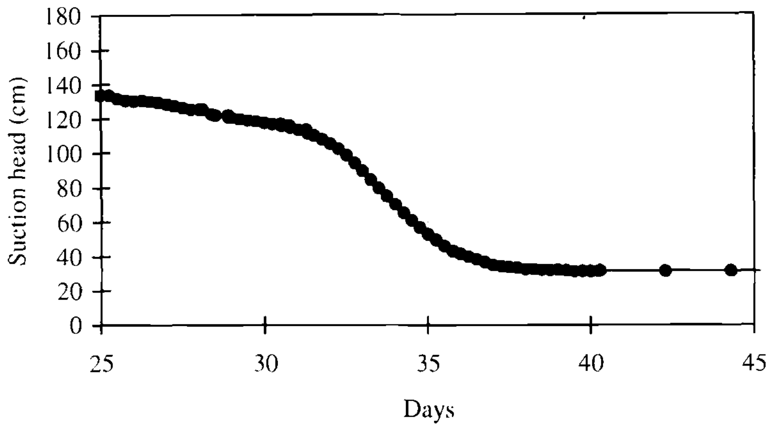


FIG 9 - *Tensiometer response indicating breakthrough during Test 5. Tensiometer is located 1 cm beneath GCBD at a downdip distance of 255 cm.*

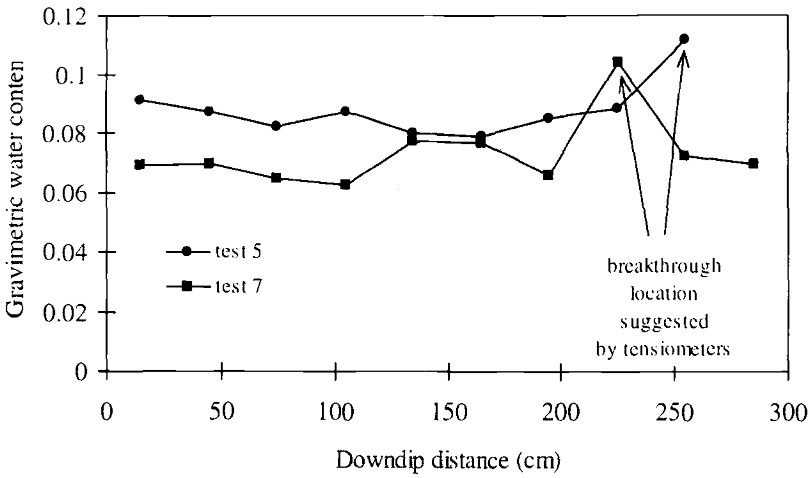


FIG. 10 - *Post-test water contents in sub-grade soil.*

Equation (1) can be arranged to yield an expression for the maximum diversion length of a GCBD system

$$L = \frac{\theta_s}{q} i \qquad (2)$$

where L is the diversion length, θ_s is the saturated transmissivity, and q is the infiltratior flux rate. This expression is applicable to a relatively thin layer (such as a geotextile), steady-state conditions, and constant and uniform infiltration. Comparison between measured diversion lengths and those calculated using Equation (2) above are given in Table 3. The predicted diversion lengths are consistent with the test results, although a direct comparison between measured and calculated diversion lengths is possible only for those tests that experienced breakthrough.

A simple finite difference water balance model was developed to more completely describe the behavior of the GCBD systems during the drainage tests, including the drainage in the overlying soil [9]. The transient model included representations of the unsaturated hydraulic properties of the GCBD components and the overlying soil. The model was configured with the dimensions of the lateral drainage apparatus, and provided estimates of the water drained in the GCBD and overlying soil, breakthrough into the sub-grade soil, and suction histories for a test. Reasonable agreement between the model and the measured values was achieved in light of the simplifications and assumptions inherent in the model and the uncertainties in the measured values. The predicted diversion lengths from this model, given in Table 3, are consistent with the experimental results.

Table 3 - *Comparison of measured and estimated transmissivities and diversion lengths.*

	Transmissivity, mm^2/s		Diversion length, cm		
Test Number	Calculated from drainage data (Eqn. 1)	Measured in permeameter at 0 cm suction	Measured	Calculated (Eqn. 2)	Finite difference model
1	2.2	39.0	> 300	4530	>300
2	3.6	39.0	> 300	3220	>300
3	28.5	39.0	> 300	1420	>300
4	5.4	39.0	> 270	1290	>300
5	39.6	39.0	240	209	190
6	1.3	39.0	> 300	980	>300
7	4.1	6.8	210	220	190
8	1.7	6.8	>300	410	>300

Conclusions

The drainage tests described here have confirmed the GCBD concept; that is, water can be drained from a soil while under suction. For most test conditions, the GCBD systems drained water under negative pressures at a rate sufficient to prevent any positive water pressures from developing in the overlying soil. Further, the drain system served as a barrier as it prevented water from moving beneath it into the underlying soil. A test conducted with just the transport layer (without the capillary barrier layer) revealed that lateral drainage does not occur without the composite system.

The drainage capacity of a GCBD system is a function of the unsaturated hydraulic properties of the geosynthetic materials. In particular, the properties of the transport layer define in large measure how much water can be drained with a GCBD system. Geotextile A was a more effective transport layer than geotextile B in these tests because of its substantially greater transmissivity, attributable to both its greater thickness and its larger pore sizes. The materials used for the transport layer were readily available, stock materials. These materials did in fact drain water under negative water pressures, but not until the suction was reduced to only a few centimeters. The potential of the GCBD concept will be enhanced if water can be drained from soils at greater suctions - this will require a transport layer that accepts water and becomes transmissive at a greater suctions. Ongoing work is focusing on this area.

The diversion or drainage length of a GCBD depends not only on the properties of the GCBD components, but also on the infiltration rate, the slope, and the properties of the adjacent soil. The test results were consistent with estimates of diversion length from both a simple analytical expression and a finite difference water balance model. The simple expression indicates that the diversion length should be a linear function of slope, the saturated transmissivity of the transport layer, and the inverse of the infiltration rate. The role of the adjacent soil layer, hysteresis of material properties, and non-constant infiltration are more difficult to define, and require further study.

Acknowledgments

The authors acknowledge the collaboration and contributions of Dr. Karen Henry and Dr. Ric Morris. Mr. Chandradip Ray conducted the transmissivity measurements and assisted with the laboratory work. Support from the State of New Mexico and the Waste-Management Education and Research Consortium is gratefully acknowledged.

References

[1] Henry, K. S. and Stormont, J. C., "Geocomposite capillary barrier drain," COE-470, U.S. Army Corps of Engineers, patent application, 1998, 20 p.

[2] Nyhan, J. W., Hakonson, T. E. and Drennon, B. J., "A Water Balance Study of Two Landfill Cover Designs for Semiarid Regions," *Journal of Environmental Quality*, Vol. 19, 1998, pp. 281-288.

[3] Stormont, J. C. and Morris, C. E., "Unsaturated Drainage Layers for Diversion of Infiltrating Water," *Journal of Irrigation and Drainage Engineering*, Vol. 123, No. 5, 1997, pp.364-366.

[4] Henry, K. S., "The Use of Geosynthetics to Mitigate Frost Heave in Soils," Ph. D. Dissertation, Civil Engineering Department, University of Washington, Seattle, WA, 1998.

[5] Henry, K. S. and Patton, S., "Measurement of the Contact Angle of Water on Geotextile Fibers," *Geotechnical Testing Journal*, Vol. 21, No. 1, March, 1998, pp. 11-17.

[6] Henry, K. S., and Holtz, R. D., "Capillary Rise of Water in Geotextiles," In S. Knutsson (Ed.), Proceedings, *International Symposium on Ground Freezing and Frost Action in Soils*, Luleå, Sweden, 15–17 April, 1997, pp. 227–233.

[7] Zerfass, K.-Ch., "Syphoning Effect of Geotextiles," *Third International Conference of Geotextiles*, Vienna, Austria, 1986, pp. 1197-1202.

[8] Stormont, J. C., Henry, K. S. and Evans, T. M, "Water Retention Functions of Four Non-woven Polypropylene Geotextiles," *Geosynthetics International*, Vol. 4, No. 6, 1997, pp. 661-672.

[9] Stockton, T. B., "Experimental Testing and Prediction of Diversion Lengths for a Geocomposite Unsaturated Drainage System," M.S. Thesis, Civil Engineering Department, University of New Mexico, Albuquerque, NM, 1999.

Pavement Design and Drainage

Barry R. Christopher,[1] Scott A. Hayden,[2] and Aigen Zhao[3]

Roadway Base and Subgrade Geocomposite Drainage Layers

Reference: Christopher, B. R., Hayden, S. A., and Zhao, A., **"Roadway Base and Subgrade Geocomposite Drainage Layers,"** *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390*, L. D. Suits, J. B. Goddard, and J. S. Baldwin, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: The Maine Department of Transportation (DOT) in conjunction with the University of Maine and the U.S. Army Cold Regions Research Laboratory evaluated the use of a special geocomposite drainage net as a drainage layer and capillary barrier (to mitigate frost heave) on a section of road plagued with weak, frost-susceptible subgrade soils and poor pavement performance. The special geocomposite drainage net that is being used has a higher flow capacity than conventional geonets and, based on tests performed by the University of Illinois, does not deform significantly under heavy traffic loading. For the 425-m-long test section, the geonet drainage geocomposite was placed horizontally across the entire roadway but varied in vertical location to form three separate subsections for evaluating drainage of 1) the base coarse aggregate, 2) the asphaltic concrete pavement, and 3) the subgrade to allow for a capillary break in order to reduce frost action. An integral drainage collection system was installed to collect the water flowing in the geonet. This paper includes a project description, material and construction specifications, installation procedures, instrumentation, and test results based upon two seasons of monitoring. Laboratory characterization and performance testing initially used to evaluate the geocomposite are compared with the monitored results.

Keywords: drainage, drain, frost heave, geocomposite, geonet, instrumentation, pavement, roadway

Introduction

In order to evaluate the potential application of geosynthetics as a roadway drainage layer, the Maine Department of Transportation (DOT) constructed a test section with a

[1]Geotechnical Consultant, Christopher Consultants, 210 Boxelder Lane, Roswell, GA 30076.
[2]Certified Geologist, Maine Department of Transportation, Geotechnical Section, 219 Hogan Road, Bangor, ME 04401.
[3]Technical Director, Tenax Inc., 4800 East Monument Street, Baltimore, MD 21205.

special geocomposite drainage net placed horizontally within the pavement section of a road plagued with weak, frost-susceptible subgrade soils and poor pavement performance. Several drainage schemes were evaluated including use of the geocomposite as a roadway aggregate-base drainage layer, a surface pavement drainage layer and a drainage layer to provide a subgrade capillary break to mitigate frost problems. A special geocomposite drainage net was used which had sufficient flow capacity to drain the roadway section and adequate compression stiffness to withstand the anticipated traffic conditions without significant deformation over the life of the pavement. The test section was constructed during the 1997 construction season. This paper provides a project description, material and construction specifications, installation procedures, instrumentation, and test results based upon two seasons of monitoring. Laboratory characterization and performance testing initially used to evaluate the geocomposite are compared to the monitored results.

Drainage in Pavement Systems

Water in pavement systems is one of the principal causes of pavement distress. It is well known that improved roadway drainage extends the life of a roadway system. The Romans found very quickly that drainage was essential for their roads to last (remnants of which still remain!). In the 19th century, MacAdam recognized that it was necessary to have good drainage if adequate support was to be maintained and the road was to last. Adequate drainage is predicted to extend the life of a pavement system up to 2 to 3 times [*1,2*] over that of undrained pavement sections.

Another drainage issue relates to frost heave and subsequent thaw which causes significant weakening in soils resulting in extensive damage to roadway systems. Frost heave occurs due to the formation of ice lenses in soil which can grow up to several centimeters in thickness and cause expansion of the soil. During thaw, either a void or very soft wet soil replaces the ice lenses resulting in a very weak support condition. Conditions necessary for frost heave include: freezing temperatures, the presence of frost-susceptible soil and availability of water to the freezing front. If water is available, it can migrate through capillary action towards the freezing front and form ice lenses even where the water table is a meter or more below the depth of frost penetration [*3*]. In Maine and many other cold regions traffic weight restrictions must be posted during the spring thaw.

Conventional Solutions

Incorporating free draining base aggregate into the design provides a good solution to the drainage problem and is the current trend in long life roadway design as documented by NCHRP synthesis 239 [*4*]. However, free draining aggregate typically requires an asphaltic or cement stabilization binder to facilitate construction and either a graded granular or geotextile filter to prevent migration of subgrade fines into the open graded base, adding significantly to the cost of the roadway. The geotechnical solution to the frost heave problem usually is to remove the frost susceptible soils down to frost depth and replace the soil with non-frost susceptible material. This may require excavation and replacement of over a meter or more of material. Because of the expense of over excavation and the non-frost susceptible select granular material, this solution is often not performed to the extent necessary. Because of the increasing cost of clean granular material, often the backfill contains significant fines and is still somewhat frost susceptible. Another solution to this problem is to use deep drainage trenches to lower the water table and provide transition zones to limit over excavation; however, it is often difficult to lower the water table to a satisfactory depth.

A layer of granular soil has also been placed above the water table as a capillary break and backfilled with frost-susceptible soils to minimize frost heave and related damage in pavements [*5*]. This concept used geotextile filters above and below the granular layer to prevent intrusion of the adjacent soils. However, high construction costs have deterred the use of this alternative. Thick nonwoven geotextiles have also been evaluated for their potential to provide a capillary break [*3,6*]. Although the use of a nonwoven geotextile seemed promising, recent work by Henry [*7*] suggest that they are unlikely to act as a capillary barrier for long term field conditions.

Potential Geocomposite Drainage Layer Solutions

A potential alternative for both improved drainage and reduction in frost heave would be to incorporate a low compression, geocomposite drainage layer tied into roadway edge drains as shown in Figure 1. The geocomposite drain could be placed between the base and the subgrade, dramatically shortening the drainage path for the base (i.e., to just the thickness of the base versus the width of the road) thus allowing for less select base materials with a higher fines content as shown in Figure 1a). As the pavement ages and cracks are formed, a majority of the water will enter through the pavement surface. Thus it may be more advantageous to locate a drainage layer directly beneath the pavement surface to collect any infiltration before it enters the base and provide more rapid removal as shown in Figure 1b.

For deep frost penetration, the geocomposite net could be placed at a lower depth as a capillary break, replacing the granular layer shown in Figure 1c. Frost-susceptible backfill could then be placed directly over the geocomposite to the pavement base grade level. In this case, the system could be tied into drainage outlets to maintain the groundwater table at or below that depth. This may potentially eliminate the development of ice lenses which in turn could result in the removal of posting traffic weight restrictions during the spring thaw in cold regions.

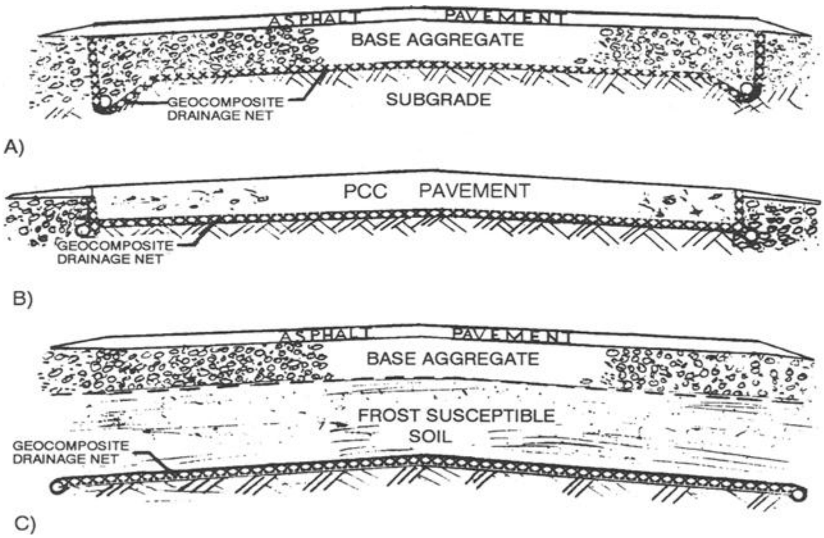


Figure 1. *Potential Use of Horizontal Geocomposite Drainage Layers Including a) Drainage of Roadway Base or Subbase Aggregate, b) Drainage of Surface Asphalt or Concrete Pavement, and c) Drainage of Subgrade to Form a Capillary Break*

Geocomposite Property Requirement

The geocomposite must have the stiffness required to support traffic without significant deformation under cyclic traffic loading. At the same time, the geocomposite must have a flow capacity to rapidly drain the pavement section and prevent saturation of the base. Outflow capacity in relation to the requirements for a roadway system typically require complete drainage within 2 hours. Conventionally a 100 mm thick open-graded base layer has proven adequate to meet the flow requirement [*4*]. This layer has a minimum permeability of 300 m/day and preferably 600 to 900 m/day. For comparison with a geocomposite, this layer would have a transmissivity of 0.00035 to 0.001 m^2/sec. With a typical roadway gradient of 0.02 (for a 2% grade), this layer provides a flow capacity ranging from 0.6 to 2 m^3/day per meter length of road.

With regard to traffic loads and tolerable deformation, the anticipated stress level on the geocomposite in a high use roadway is on the order of 80 to 600 kPa depending on the location of the geocomposite within the pavement section. Although many geocomposite drainage materials have a crush resistance greater than these values, most materials would deform significantly under the upper load range. In addition, dynamic traffic loading could induce significant creep deformation and potential collapse. Considering the high cost of pavement replacement, it is prudent to only consider high modulus, high compressive resistance geocomposites such as geonet drainage composites.

Unfortunately at the required gradient and load levels, most commercially available geonet drainage composites do not have a sufficient flow capacity to match the gravel layer drainage level. Typical in-soil transmissivity values of geonet with two 270 g/m^2 needle-punched nonwoven geotextiles laminated to both sides is on the order 1×10^{-4} m^2/sec to 5×10^{-4} m^2/sec [*8,9*] or even lower [*10*]. Further reduction in these transmissivity values due to long-term compressive creep of the geonet must also be taken into account. This may be why geonet drainage composites have not reportedly been used in this application. Although lower flow rates may be acceptable for some projects, considering this was a first trial, an equivalent flow rate to the gravel layer was desired. In addition, to provide a capillary break, it is critical that an air void exist within the geocomposite [*7*]. Geotextile intrusion on typical, relatively thin geonets is often sufficient to allow the geotextile filters on opposite sides of the geonet to touch, thus eliminating the air void. This was especially a concern for the lower drainage layer, which would be placed between soft clayey soils and have an increased geotextile intrusion potential. On the basis of these two considerations a thicker, higher flow capacity geonet drainage composite than typically available was desirable.

At the time of the project, a high flow geonet drainage composite (Tendrain 100-2 by the Tenax Corporation) had recently been introduced that met the flow requirements and did not allow the geotextile layers to touch. This new geocomposite consists of three extruded net layers to form a tri-planar geonet inner core with a needlepunched nonwoven geotextile laminated to either side. The composite has a transmissivity of 0.0022 m^2/sec under a normal load of 720 kPa and a gradient of 0.1, with corresponding flow capacity of 19 m^3/day/m at a gradient of 0.1 and 3.8 m^3/day/m at a gradient of 0.02, based on ASTM Test Method for Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products (D 4716). Typical transmissivity data indicate that the transmissivity of a geonet decreases with increasing gradients. Consequently, the use of a laboratory-transmissivity value at a gradient higher than the actual field value (i.e., 0.1 gradient for 2% grade) will be conservative for evaluating flow capacity. The hydraulic characteristic thus corresponds very well with the horizontal drainage layer requirements for roadway drainable base. In addition, long term compressive creep tests on the tri-planar geonet core indicated that the material retained over 60% thickness after 10,000 hours under sustained normal load of 1200 kPa.

To further evaluate the performance of this geocomposite in a roadway system, cyclic loading tests were performed at the University of Illinois, Advanced Transportation Research and Engineering Laboratory [*11*]. Cyclic fatigue testing was performed on a concrete beam supported by the geocomposite overlying a clay subgrade and compared to results from a beam supported by the subgrade alone. The tests were performed at stress ratios of 0.76 and 0.83. The test setup along with representative results are shown in Figure 2. The results found insignificant additional deformation in the concrete when the geocomposite was used. The test at the 0.83 stress ratio showed some improvement in fatigue life (visually cracked beam) and the test at 0.76 stress ratio showed some reduction in fatigue life. Although the test results were inconclusive in relation to fatigue life, the geocomposite improved post-cracking behavior of the beam at both stress levels (i.e., minimized continued widening of the crack after break). This improvement in beam performance was attributed to an improvement in the uniformity of support under the beam after cracking and/or frictional improvement at the bottom of the beam which reduced the post-cracking deflection.

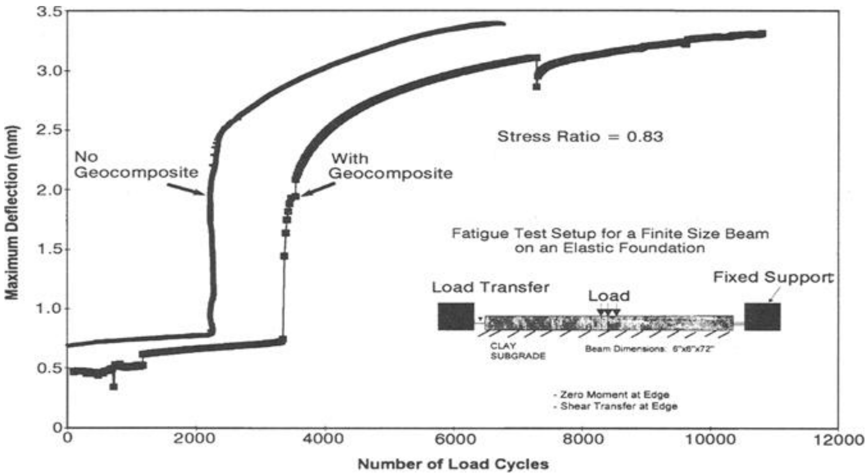


Figure 2. *Fatigue Test Setup and Results the University of Illinois, Advanced Transportation Research and Engineering Laboratory [11]*

Field Test Project Description

The project where field testing of the geocomposite drainage concepts was performed involved reconstruction of a 3.0 km portion of U.S. Route 1A in Frankfort and Winterport, Maine. The existing pavement along this project had been plagued by cracking, rutting, and potholes. The highway required frequent maintenance to maintain a trafficable pavement surface. The conditions prompted the reconstruction project.

A subsurface investigation [12] encountered moist clay soils (locally known as the Presumpscot Formation) along the entire length of the project. These soils are plastic and moist with water contents greater than 20%. Based on soil conditions and past roadway construction experiences, designers initially recommended that the subgrade soils be undercut by 150 mm and replaced with granular soil to create a stable working surface prior to placing the overlying subbase course. It was anticipated that a greater depth of undercut would be required in some areas. However, with the use of geosynthetics (including the geocomposite drainage layers), the designers felt that undercutting would be unnecessary.

Prior to reconstruction water was observed seeping out of pavement sections, even though this was the second driest summer on record in the state of Maine. Water in the pavement section was obviously one of the existing pavement section failure mechanisms. Thus 425 m long drainage test sections were incorporated into a broader study on the effectiveness of geosynthetics in roadway construction [*13,14*]. In order to evaluate the three drainage schemes discussed in the Drainage of Pavement section, the test section was divided into 3 smaller subsections (labeled D-1, D-2 and D-3) and a control section as shown in Figure 3. The geonet drainage composite was placed at 460 mm below subgrade (subsection D-1), at subgrade (subsection D-2), and both directly beneath the pavement and at the subgrade (subsection D-3). In subsection D-1, the undercut subgrade material consisted of a mixture of native clay soils and old base course material. The geocomposite was covered with the material originating from the undercut. In each of the drainage test sections an internal drainage collection system was installed on both sides of the road directly beneath the shoulder break to collect water captured in the geonet drainage composite (Figure 4).

The high flow capacity geonet drainage composite discussed in the previous section was used in each of the test sections. The selection of the geotextile laminated to surfaces of the geonet was based on the FHWA filtration design criteria [*15*] considering the gradation and flow requirements for both the base course aggregate and the subgrade. The properties of the geonet core, geotextile and geocomposite are shown in Table 1.

Construction began May 1997 and extended into November 1997. Even though extremely favorable climatic conditions existed soil problems were still encountered. One of the control sections, which was built without geosynthetics and a stabilization lift, failed during construction (June, 1997). Subsequently, the clay soils in this area were undercut 600 mm and replaced with gravel. A 820 mm pavement section was then constructed over the undercut. In addition to the soil problem in the control section, other undesirable soil locations were identified during construction in the drainage section as well as other test sections. The contractor requested that these areas be undercut. However, since these locations were in areas where geosynthetics were to be utilized, the request for undercutting was denied. Subsequently, construction equipment and traffic were able to operate along these areas in the test sections without incident.

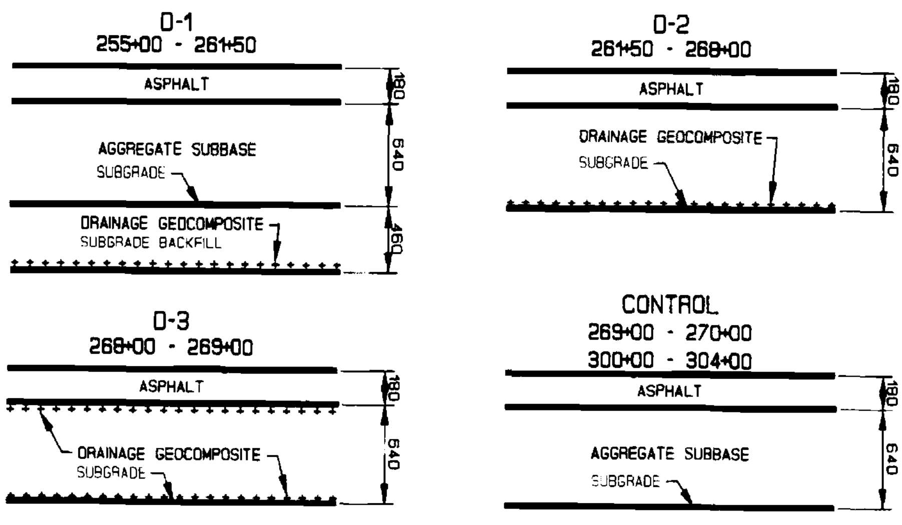


Figure 3. *Test Section Schematics (after [13])*

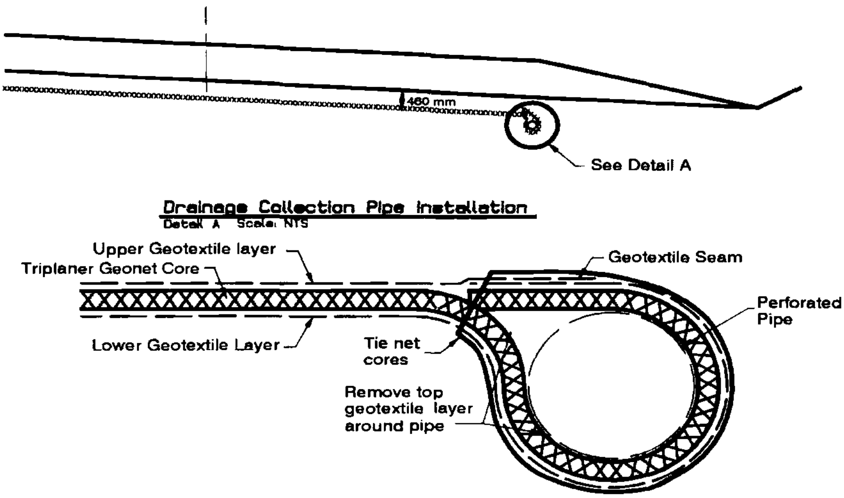


Figure 4. *Drainage Collection System Schematic*

Table 1. *Geonet Drainage Composite Properties*

GEONET			
Tensile Strength (MD)	ASTM D4595	kN/m	14
Compression Behavior % Retained Thickness @ 2400 kPa (short term) @1200 kPa (10,000 hours)	ASTM D1621	% %	50 60
Resin Density	ASTM D1505	g/cm^3	0.940
Resin Melt Index	ASTM D1238	g/10 min.	1.0
Carbon Black Content	ASTM D4218	%	2.0
Thickness	ASTM D5199	mm	7.0
GEOTEXTILE			
AOS	ASTM D4751	mm	0.125
Permitivity	ASTM D4491	sec^{-1}	1.26
Permeability	ASTM D4491	cm/sec	0.3
Grab Tensile Strength	ASTM D4632	Kn	1000
GEOCOMPOSITE			
Thickness	ASTM D5199	mm	9
Ply Adhesion	ASTM D413	g/cm	178
In-Soil Transmissivity @gradient 0.1and normal load 725kPa	ASTM D4716	m^3/sec-m	$2.2*10^{-3}$

Drainage Instrumentation

Water is allowed to drain from the drainage collection system at eight separate locations labeled outlet A through H. Monitoring stations were constructed at six outlet locations A through F (Table 2). At each of the monitored locations the outlet pipe is connected to a tilt bucket (Figure 5) housed in a protective wooden structure. A micro switch is positioned on the tilt buckets and is actuated every other time the tilt bucket dumps. The micro switch is, in turn, connected to a traffic counter which records the number of dump cycles of each tilt bucket. Data is collected continually 24 hours a day. The traffic counter software then provides daily or monthly reports, which presents the total number of dump cycles per hour, per day. This information is then downloaded over phone lines from the traffic counter on project to the Maine Department of Transportation offices.

Stand pipe type well points were installed outside the roadway at three locations along the drainage test section to provide ground water level information. These were read manually with a tape. Although the well points could not provide water level values directly under the roadway, they were useful in identifying the general relationship between ground water and roadway drainage.

Table 2. *Monitored Outlet Locations and Details*

Monitored Outlet Locations	Test Section	Drainage Pipe Location	Geocomposite Location
Outlet A 261+40 right	D-1	255+00 - 260+00 right 152 m drained section	Geocomposite is located (460 mm) below subgrade and is placed along the low side of a super elevated turn.
Outlet B 261+40 left	D-1	260+00 - 261+50 left 46 m drained section	Geocomposite located (460 mm) below subgrade in a standard section.
Outlet C 261+40 right	D-1	260+00 - 261+50 right 46 m drained section	Geocomposite located (460 mm) below subgrade in a standard section.
Outlet D 268+00 left	D-2	261+50 - 268+00 left 200 m drained section	Geocomposite is located at subgrade along the low side of a super elevated turn in areas.
Outlet E 268+00 left	D-3	268+00 - 269+00 30 m drained section	Geocomposite is located at subgrade in a standard section.
Outlet F 268+00 left	D-3	268+00 - 269+00 30 m drained section	Geocomposite is located directly beneath the pavement in a standard section.

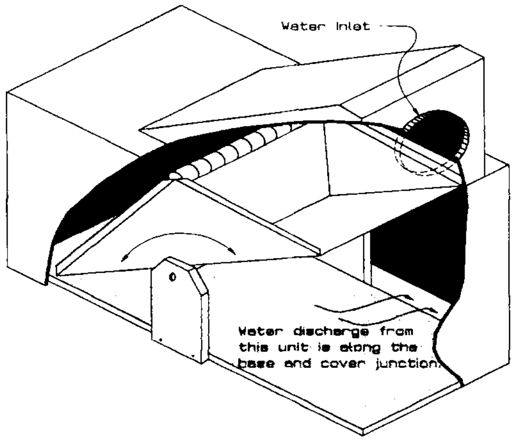


Figure 5. Tilt Bucket Schematic [13, revised from Wisconsin DOT]

Rainfall at the site and temperature were obtained from a weather station in the vicinity of the project. Thermocouples were also installed within the drainage sections to monitor frost penetration.

In order to evaluate the potential influence of the geocomposite on dissipation of pore pressure in the base course aggregate and the subgrade, vibrating wire piezometers (Roctest model PWS) were placed in the subgrade approximately 150 mm below the geocomposite and at two levels above the geocomposite in each drainage subsection and at three corresponding locations in the control section and in a test section where a geotextile was used as a separation layer between the base and subgrade. For more details see reference Hayden et al. [*13*].

The instrumentation was also complemented with a periodic survey of the pavement surface and falling weight deflectometer (FWD) tests. The pavement surface surveys were conducted between the months of December and April to measure frost heave. FWD tests were performed prior to reconstruction and in April 1998 soon after the end of the spring thaw. The pavement thickness varied in the test section from 127 to 254 mm prior to construction. The pavement was 146 mm thick after construction.

Drainage Geocomposite Test Section Results

Drainage Discharge Results

Data collection began in March 1998 for outlets B through F. Data collection at outlet A did not begin until late June. Discharge volumes in liter per meter of section length (l/m) from monitored outlets per length drained section are listed in Table 3 and plotted in Figure 6 along with monthly rainfall.

Table 3. *Discharge Volumes from Monitored Outlets Per Length of Drained Section*

Monitoring Period	Discharge Volumes Per Length of Drained Section (L/m)						
	Outlet A D-1	Outlet B D-1	Outlet C D-1	Outlet D D-2	Outlet E D-3	Outlet F D-3	Monthly Totals
March 98	-	77	1094	118	0	0	1289
April 98	-	0	0	91	0	0	91
May 98	-	0	0	94	0	0	94
June 98	0	4	63	73	6	0	146
July 98	2	0	339	56	0	0	397
August 98	0	0	7	0	0	0	7
September 98	0	0	80	0	0	0	80
October 98	0	0	22	79	17	0	118
November 98	0	0	51	102	0	0	133
December 98	0	0	18	21	0	0	39
January 99	0	0	0	0	0	0	0
February 99	0	0	1	0	0	0	1
March 99	0	16	843	464	41	0	1364
Totals	2	97	2518	1098	64	0	3779

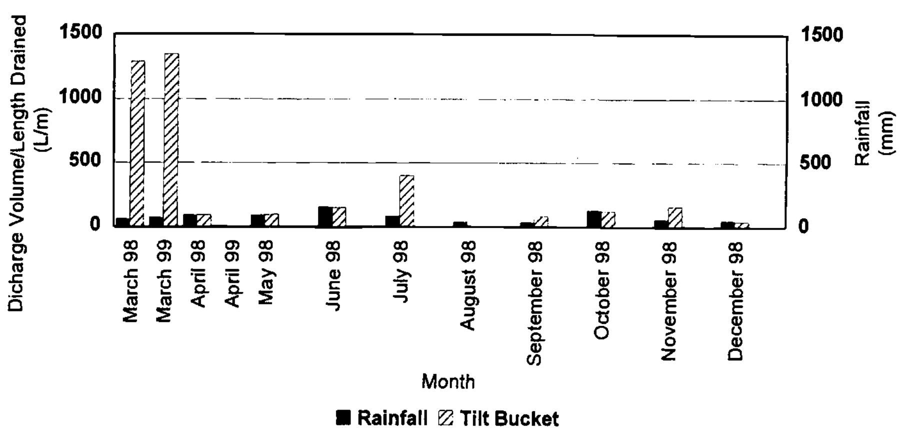


Figure 5. *Monthly Discharge Volumes (per Length of Drained Section) and Rainfall*

The volume (l/m) totals for each outlet indicates that outlet C recorded the highest volume per length drained section and outlet F recorded the least flow. The greatest monthly flow recorded at each of the monitored locations for 1998 was encountered during the month of March. Discharge volumes per length drained were considerably less for the other months. The volume (l/m) of discharge in the month of March accounted for 53% of the total one year discharge volume. Discharge during the months of April through August accounted for less than 10% of the total for the 1998 monitoring season except for the month of July which accounted for 20% of the total. This trend appears to be continuing in 1999 with March again showing a significant magnitude of flow.

Heavy flow activity during the month of March corresponds strongly with the thawing of ice lenses as verified by thermocouple readings and frost heave elevation surveys. The flow observations tend to confirm the results of drainage studies in cold regions performed by Hagen and Cochran [*16*] (1996). They also found that the highest flows out of pavement, higher than any rain event throughout the year, occurred during the spring thaw.

Outlet C recorded the greatest discharge of the six monitored outlets. Water discharged (l/m) from Outlet C is collected from a 45 m run of drainage geocomposite located 460 mm below subgrade in the thawing region. Outlet D recorded the second greatest discharge (l/m) during March. This outlet drains a 200 m length of road with the drainage geocomposite placed directly at subgrade.

Discharge during the first year of monitoring (March 1998 – March 1999) corresponds strongly with precipitation events and water table levels. Over 800 mm of rain fell on the project area during the monitoring year period. Tilt bucket activity would begin shortly after each rain event and ended the same day or the next day after the rain ceased. As the water table lowered through summer, the time between rainfall and tilt bucket activity increased. Based on the AASHTO [*17*] definitions for pavement drainage capacity, the quality of drainage in the geocomposite test section would be good (i.e., water removed within one day) to excellent (water removed within 1 day).

Outlet C and Outlet D recorded the greatest discharge (l/m) of the monitoring period with 67% and 30% respectively for the 1998 monitoring season respectively. Again, water discharged from Outlet C is collected from a drainage geocomposite located below subgrade and water discharged from Outlet D is collected from a drainage geocomposite located directly at subgrade. Outlets E and F recorded very little water discharge. Outlet E only recorded a total discharge volume of 64 liters per meter of section drained. Outlet F recorded even less with only a trace of water. Outlets E and F drain a 30 m length section constructed on a fill area. Outlet E collects water from the geocomposite placed at subgrade whereas Outlet F collects water from the geocomposite placed directly beneath the asphalt pavement.

Based upon tilt bucket limitations, the volumes recorded at each of the six monitored outlet locations could be considerably lower than the actual volumes. The tilt buckets were not able to dump when subjected to flow rates greater than approximately 7 l/min. Flow rates as high as 57 l/min were measured manually during site visits. At that rate, the geonet drainage composite is near its flow capacity. In addition, tilt buckets experienced difficulties in accurately measuring flow rates less than 0.25 l/min due to surface tension. Site visits revealed that water being discharged at these low flow rates failed to drop vertically into the collection bin but rather clung to the sides and traveled down the inner walls and exited the tilt bucket without ever entering the collection bin.

Falling Weight Deflectometer Results

The results of the back calculated structural number (SN) for all test sections in the project are shown in Figure 6. It is seen that the highest SN in April 1998, was obtained in the control sections (labeled F-1 and F-2) with values of 7.2 to 7.1, respectively. These results are not surprising considering each control section was unfortunately affected during construction. Control section F-2 was in a fill section and was constructed with a mixture of fill material containing the native clay soils and gravel particles from the old base. Control section F-2 failed during construction and was subsequently constructed with an additional 600 mm of subbase aggregate due to poor subgrade conditions. The next highest SN (6.9) was obtained in section D-1 and was essentially the same as measured in the F-1 control section. In this section D-1, the subgrade was undercut by 460 mm to allow placement of the drainage geocomposite. The subgrade was brought back to grade with the soil removed from that section. This compacted fill thus consisted of a mixture of clay, sand, and gravel. This mixture had a lower water content than the native subgrade soils. It is likely that this drier mixture was stiffer than the in-place subgrade soils. Even so, the modulus of this mixture should be significantly less than the backfill placed in the control section. It is also possible that consolidation induced by drainage had a stiffening effect on the soil. The SN for both the other two drainage sections was 6.0 which is in line with the SN values obtained for other geosynthetic reinforcement and separation test sections. The SN improvement from the original readings is significant, on the order of 75% for section D-1 and 50% for the other two test sections.

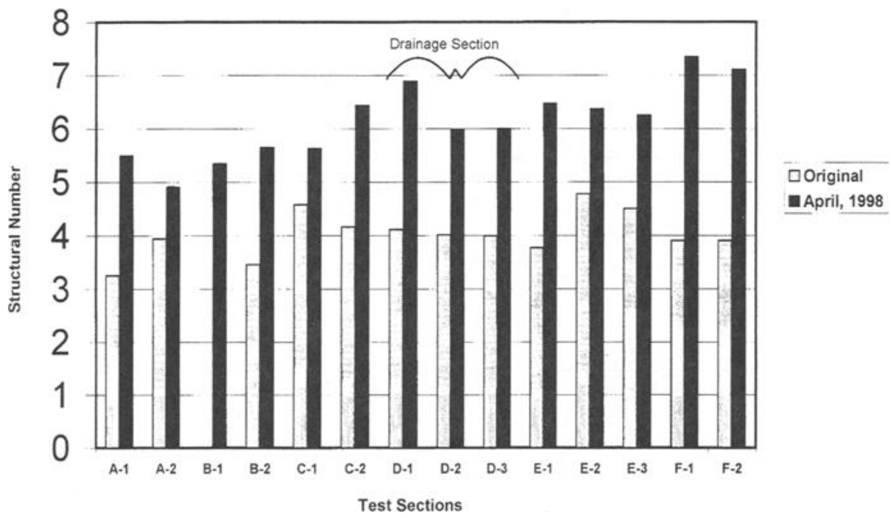


Figure 6. *FWD Results*

It should also be noted that some sections of the original road had up to 619 mm of asphalt. Additional FWD readings are being conducted within the drainage test sections during the spring thaw to investigate and compare the reflection curves to determine if there is a noticeable difference between these curves represented by the soils ability to regain it's strength as water is removed from the soil. Data are not available at this time. It should also be noted that some sections of the original road had up to 619 mm of asphalt. Additional FWD readings are being conducted within the drainage test sections

Piezometer Results

Unfortunately difficulties were encountered in reading the piezometers during the critical spring thaw due to both equipment and winter environment problems. Average readings during the fall of 1997 and the summer of 1998 can, however, be compared as shown in Table 4 [*18*]. These readings show that during the fall of 1997, the drainage sections D-2 and D-3 had lower pressure readings than the control section in both the base and the subgrade suggesting the geocomposite placed at the subgrade interface removed water from the pavement system. Section D-1 had a lower pressure in the base aggregate and the subgrade below the geocomposite but was higher in the clayey backfill over the geocomposite when compared to the control section. This is not surprising considering the subgrade in the control section was placed as compacted fill. Although the summer of 1998 readings indicate a significantly lower pore pressure in the geocomposite drainage section than the control section, the negative pore pressures which would normally indicate partially saturated conditions may also be the result of damaged transducers caused by freezing in the winter.

Table 4. *Average Values of Piezometers [18]*

Average piezometric values for dates	10/22/97 to 11/25/97			
	Piezometric Head (mm)			
	Section	Top	Middle	Bottom
Separation Geotextile at 820 mm	C-2	138	95	210
Geocomposite 460 mm below subgrade	D-1	29	359	3
Geocomposite at 820 mm	D-2	10	52	-15
Geocomposite at 180 and 820 mm	D-3	118	23	74
Control - No Geosynthetic	Control	414	348	149
Average piezometric values for dates	6/27/98 to 8/19/98			
Separation Geotextile at 820 mm	C-2	-138	-154	-224
Geocomposite 460 mm below subgrade	D-1	-82	-88	58
Geocomposite at 820 mm	D-2	-501	-110	-84
Geocomposite at 180 and 820 mm	D-3	-158	-154	-151
Control - No Geosynthetic	Control	237	247	378

Frost Heave

Minimal frost heave has been observed thus far in any of the test sections and it may take several additional seasons to provide discernible results. Freezing tests on the subgrade soil with and without the geocomposite as a capillary barrier were also conducted [*19*]. Analyses of frost heave will be reviewed and presented in a separate paper as soon as significant results have been obtained.

Conclusions

Data from drainage monitoring outlets indicate that a high flow capacity geonet drainage composite placed at subgrade or below subgrade is successful in rapidly removing water from beneath the roadway. The placement of such a drainage layer proved especially useful for removing the most damaging waters present during the spring thaw. More water was removed from the roadway section during the spring thaw than during the highest rainfall period throughout the year. A measured improvement in stiffness for the section in which the drain was placed in the subgrade may have been the result of this drainage as well as greater compaction of the soil placed over the geocomposite. Drainage during summer months was best realized where the geonet drainage composite was placed within cut sections in areas where the water table was relatively shallow. The drainage quality provided by the geonet drainage composite would be classified as good to excellent based on AASHTO [*17*] requirements.
In addition to providing improved drainage, the geonet drainage composite facilitated construction in areas where the subgrade was weak without requiring additional undercuts. In the control area where geosynthetics were not used an additional 600 mm of stabilization aggregate was required.

The magnitude of water discharging from the drainage section is significant and should significantly improve the long-term performance of the road in comparison to undrained sections. Drainage and associated rapid improvement in pavement support in the spring may also allow for a reduction or earlier removal of load restrictions, resulting in significant transport savings. Unfortunately we may not see this influence for a number of years. It is hoped that designers will take note and not wait that long to implement this technology.

Acknowledgments

This project involved the expertise from many other people besides the authors. In great appreciation we would like to thank and recognize the following individuals for their technical support: Dr. Dana Humphrey and Ms. Christine Fetten from the University of Maine; Mr. Phil Dunn, Mr. Steve Hall, Mr. Steve Colson, Mr. Terry White and Mr.

Victor Smith of MDOT; and Ms. Karen Henry and Ms. Rosa Affleck of CRREL. We would also like to thank the Maine DOT and the Tenax Corporation for their vision and financial support that enabled us to pursue this work.

References

[1] Cedergren, H.R., 1987, *Drainage of Highway and Airfield Pavements*, Robert E. Lrieger Publishing Co. Inc., Malabar, FL.

[2] Forsyth, R.A., G.K. Wells, and J.H. Woodstrom, 1987, "The Economic Impact of Pavement Subsurface Drainage," *Transportation Research Record No. 1121*, Transportation Research Board, Washington, D.C.

[3] Henry, K.S., (1991), "Effect of Geotextiles on Water Migration in Freezing Soils and The Influence of Freezing on Performance," *Proceedings of Geosynthetics '91*, Atlanta, Georgia, pp. 469-483.

[4] Christopher, B.R. and McGuffey, V.C., 1997, *Pavement Subsurface Drainage Systems*, National Cooperative Highway Research Program, Synthesis of Highway Practice 239, Transportation Research Board, National Academy Press, Washington, D.C., 44 p.

[5] Rankilor, P.R., *Membranes in Ground Engineering*, John Wiley & Sons, Inc., London, England, 1981.

[6] Henry, K.S. and Christopher, B.R., "Pavement Test Section to Determine the Effect of Geotextile on Frost Heave," *Proceedings of Geosynthetics '93*, Vol. 1, Vancouver, BC, Canada, 1993, pp. 95-109.

[7] Henry, K.S, "The Use of Geosynthetics to Mitigate Frost Heave in Soils," Ph. D. Dissertation, Civil Engineering Department, University of Washington, Seattle, WA., 1998, 333p.

[8] Gardoni, M. G. A. and Palmeira, E. M., "Transmissivity of Geosynthetics Under High Normal Stresses," *Proceedings of Geosynthetics '99*, Vol. 2, Boston, Massachusetts, 1999, pp. 769-782.

[9] Koerner, R. M., *Designing with Geosynthetics*, Fourth Edition, Prentice Hall, Englewood Cliffs, New Jersey, 1997..

[10] Zao, A. and Montanelli, F., "Effect of Soil Presence on Flow Capacity of Drainage Geocomposites Under High Normal Loads," *Proceedings of Geosynthetics '99*, Vol. 2, Boston, Massachusetts, 1999, pp. 799-812.

[11] Dempsey, "Fatigue Performance of a Fully Supported Concrete Beam with a Geocomposite Layer," unpublished report prepared for Barry R. Christopher, 1996, 6 p.

[12] Hayden, S.A., "A Subsurface Investigation for a 1.9 Mile Portion of Route 1A in the Towns of Frankfort and Winterport," unpublished report, State of Maine Department of Transportation, Report No. 96-11, Bangor, ME., 1996, 40 p.

[13] Hayden, S.A., Christopher, B.R., Humphrey, D.N., Fetten, C.P., and Dunn, P.A., Jr., "Instrumentation of Reinforcement, Separation and Drainage Geosynthetic Test Sections used in the reconstruction of a Highway in Maine," *Proc., 9th International Conference on Cold Regions Engineering*, Duluth, MN., 1998, pp. 420-433.

[14] Hayden, S.A., Humphrey, D.N., Christopher, B.R., Henry, K.S., and Fetten, C.P., "Effectiveness of Geosynthetics for Roadway Construction in Cold Regions: Results of a Multi-Use Test Section," *Proceedings of Geosynthetics '99*, Vol. 2, Boston, Massachusetts, 1999, pp.847-862.

[15] Holtz, R.D., Christopher, B.R., and Berg, R.R., *Geosynthetic Design and Construction Guidelines (Participant Notebook)*, NHI Course No. 13213, FHWA Publication No. HI-95-038, Federal Highway Administration, Washington, D.C., 1998, 396 p.

[16] Hagen, M.G., and G.R. Cochran, "Comparison of Pavement Drainage Systems," Transportation Research Paper #960203, Transportation Research Board, Washington, D.C., 1996.

[17] AASHTO, *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

[18] Fetten and Humphrey, "Instrumentation and Performance of Geosynthetics Beneath Flexible Pavements in Winterport and Frankfort, Maine," Report prepared for the Maine Department of Transportation, Bureau of Planning, Research, & Community Services, Transportation Research Division, Technical Report 97-14, 1998, 137 p.

[19] Henry, K.S., and Afflect, R., "Freezing Tests on Lean Clay with Tenax Tri-Planar Geocomposite as Capillary Barrier," Contract Report to Tenax, Inc., from U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, NH., 1998, 13 p.

Gerald P. Raymond[1] and Richard J. Bathurst[2]

Facilitating Cold Climate Pavement Drainage Using Geosynthetics

---

Reference: Raymond, G. P. and Bathurst, R. J., **"Facilitating Cold Climate Pavement Drainage Using Geosynthetics,"** *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390*, L. D. Suits, J. B. Goddard, and J. S. Baldwin, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: Good highway drainage has been recognized for many centuries. The theoretical concepts are simple and the technology applicable to highways built today (1999) is widely available in the technical literature. It is widely understood that efficient drainage is essential to good highway performance independent of aggregate compacted density or aggregate stability. While the theoretical concepts are simple they are often not effective in cold climates. Indeed, for cold climates, these simple concepts are shown by field excavations described herein to be lacking in a number of aspects. Based on field excavations and performance of some selected Ontario highway locations, involving both clay and sand subgrades, recommendations are presented for the design detailing, selection and installation of geosynthetic edge drains. Installation at the investigated sites was by various techniques that included: ploughed-in-place, trench excavation, and mechanical trencher and boot. All excavated edge drains were installed as retrofits either at the time of the original pavement construction or several years later. The retrofits used the existing excavated/displaced shoulder granular material as backfill. Frost action, despite what was considered good drainage practice at the time of installation, is shown to have had a major effect on field performance.

Keywords: highway, edge drain, retrofitted, geotextile filter, geocomposite, drainage

[1] Professor Emeritus, Department of Civil Engineering, Queen's University, Kingston, ON, K7L 3N6, Canada.

[2] Professor, Department of Civil Engineering, Royal Military College of Canada, Kingston, ON, K7K 7B4, Canada.

Introduction

The importance of good highway drainage has been recognized for many centuries. The theoretical concepts are simple and the technology applicable to highways today (1999) has been described by Cedergren et al. *[1]*, Moynahan and Sternberg *[2]*, and Ridgeway *[3]*. Correctly, these authors stress efficient drainage as essential to good highway performance independent of aggregate compacted density or aggregate stability. They also state that the theoretical concepts are simple to apply. In cold climates these simple good drainage concepts are, unfortunately, often not effective.

Good highway drainage provisions include surface drainage, ground water lowering and internal drainage. Internal drainage is the most difficult application, and is the focus of the current paper in the context of cold climate environments. The problems related to cold climate environments will be illustrated by the authors' experience related to the performance evaluation of some Ontario highways.

Internal drainage is the collection and discharge of water that may enter the pavement structure through the wearing course (e.g., surface cracks), granular shoulders or from the subgrade, and is facilitated by subdrains, french drains, geocomposite drains and/or open-graded drainage layers (OGDL).

The Ontario Environment

The minimum freezing index experienced by Ontario highways is 500 °C days and most experience a higher number of °C days (The Freezing Index is the difference between the maximum and minimum values on a plot of cumulative degree-days of below-freezing temperature for one freezing season. The value of a degree-day is the difference between the average daily air temperature and freezing temperature, which in SI units is 0 °C). Frost penetration during a typical winter will exceed 1 m below a snow-cleared highway. Since it is uneconomical to have granular covers equal to the frost penetration depth, most subgrades during freeze/thaw will dramatically affect highway performance, including internal drainage performance. Figure 1 illustrates the affect of frost on an Ontario highway in the form of measured Benkelman beam highway rebound values that resulted during severe weather conditions in 1963.

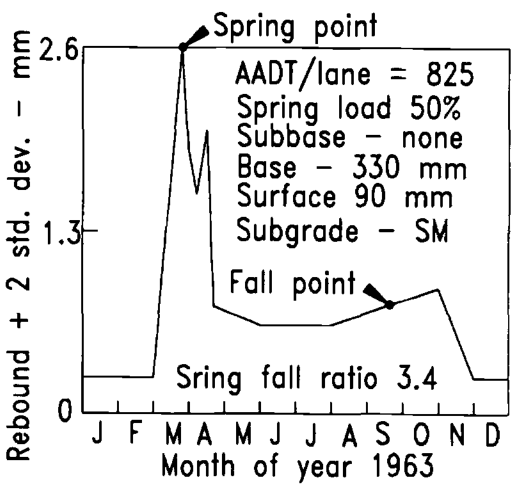


Figure 1 - *Highway seasonal movements.*

Ontario Freeway Standard Highway

Figure 2 illustrates typical Ontario freeway pavement design standards used to balance highway survival with acceptable capital costs. Ditch inverts are 500 mm or more below the lowest subgrade elevation. The subgrade is finished with a minimum 3% slope and is covered with a minimum thickness of 100 mm of densely graded granular aggregate (Granular 'A' similar to ASTM Specification for graded aggregate material for bases or subbases for highways or airports, D-2940). An open-graded drainage layer (OGDL) is provided directly below the pavement course on freeway standard highways.

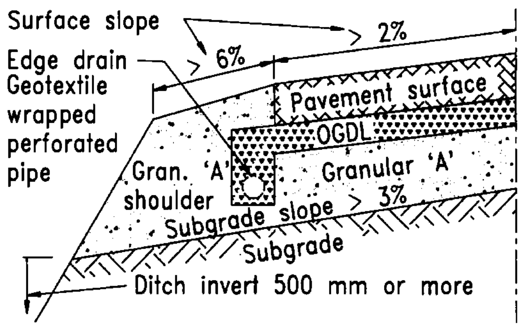


Figure 2 - *Typical highway dimensions.*

The OGDL consists of (generally bound) crushed aggregate graded from 14 mm down to 6 mm (i.e., 14/6 mm). When OGDL layers were first adopted for use under the pavement of Ontario freeways they were extended under the highway shoulder surface so as to daylight on the ditch slope surface. Due to the unsatisfactory performance (Hajek et al., *[4]*), these were discontinued for new pavements and replaced by edge drain systems. These edge drain systems incorporated geosynthetics. The pavement wearing surface is sloped, or centre-line crowned, ending with a 2% or more falling slope at the pavement edge. Initially highways were provided with 3 m wide shoulders that were unpaved (Granular 'A') and surface sloped from the pavement edge at 6%. Today (1999) the 3 m wide shoulders include a minimum 0.6 m wide partially paved portion made integral with and of the same thickness/slope design as the pavement. The unpaved portion has a 6% slope from the paved edge and is constructed of Granular 'A' to the subgrade elevation. Many of the initial pavements, constructed without partially paved shoulders have been retrofitted with asphalt concrete partially paved portions.

As stated above, earlier OGDL layers that were daylighted in the ditch slope surface proved unsatisfactory. Figure 3 (Hajek et al., *[4]*) illustrates a typical evaluation of one of these ditch daylighted installations. Water was introduced through holes made in the pavement to the

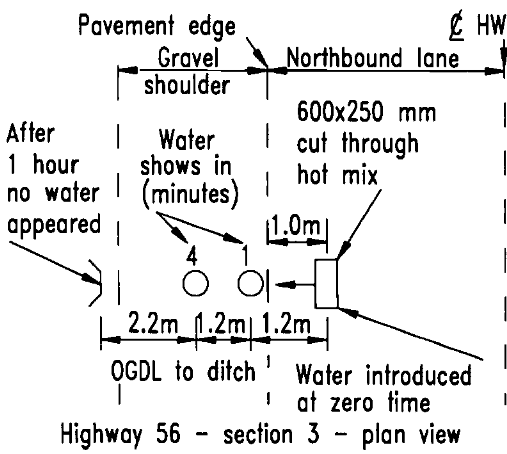


Figure 3 - *Daylighted OGDL drainage test details.*

depth of the OGDL layer and the velocity and direction of flow recorded. As recorded on the figure the water added to holes made in the pavement failed to appear at the shoulder daylight exit after 60 minutes. Shoulder daylighting of an OGDL was concluded to be unsatisfactory and this construction technique was superseded and replaced by retrofitted edge drain systems. These, unfortunately have not given uniformly good performance. This work presents investigations from numerous excavations dealing with edge drains and then presents the resulting present (1999) installation practice. The work updates the preliminary work presented earlier (Raymond et al., *[5, 6, 7, 8]*).

Properties of Geotextile Used in Edge Drain Systems

Three main types of geotextiles have been used for the construction of the edge drain systems used on Ontario highways. (1) A sock geotextile is used to wrap a 100 mm internal diameter perforated pipe drain. It is made from a 4.5 tex polyester yarn knitted to give a Filtration Opening Size (FOS, CGSB *[9]*) of 600 μm or less, grab strength (ASTM Test Method for Grab Breaking Load and Elongation of Geotextiles, D-4632) of 225 N or more at a break elongation of 75% or more, Mullen Burst (ASTM Test for Hydraulic Bursting Strength of Knitted Goods and Nonwoven Fabrics: Diaphragm Busting Strength Testing Method, D-3786) of 690 kPa or more. (2) The aggregate geotextile-wrap used for french drain type drainage are nonwoven needlepunched geotextiles with a FOS of 100 μm or less; mass/unit area of 230 g/m^2 or more; grab strength of 550 N or more with elongation at failure of 75% or more. A geotextile satisfying this specification has been used in one installation to wrap an unbound aggregate designed to function as an OGDL. (3) The geocomposite edge drain system is made of a plastic core 300 mm wide (placed vertically) with 25 mm high cusps all wrapped in a nonwoven geotextile. The geotextile is required to have an FOS of 200 μm or less, mass/unit area of 135 g/m^2 or greater, and a grab strength of 400 N or greater with elongation at failure of at least 50%.

Ploughed-in-Place Method of Installation of Pipe Edge Drains

At all sites having clay or silt subgrades where the retrofit pipe edge drains were installed by the ploughed-in-place method, migrated clay fines were found within the geotextile-sock wrapped pipe edge drains at all excavations. Considerable mixing of subgrade and backfill resulted from this method of installation resulting in a reduced permeability for the backfill. Common to most sites investigated was the observation that the pipe had risen as a result of frost heave or was installed above the subgrade elevation.

At one installation with a flat ground surface the ploughing of a pipe to a constant depth below the pavement surface installation resulted in a zero longitudinal hydraulic gradient. All the recovered geotextile-sock samples were found to have holes 5 mm to 10 mm in size. During heavy rain storms all working outlets were observed to be discharging dirty water.

Geocomposite Edge Drain and Clay Subgrade

Raymond et al. *[10]* have reported on a site typical of the performance of geocomposite edge drains (GED) installed in an area where the subgrade was clay. Where a GED was installed at a site where the subgrade was clay or silt the site showed evidence of pumping. At some excavations the GED was completely plugged with clay fines.

Two main problems resulted from frost heave that occurred during cold weather. First, the lower cusps of some of the cores were displaced and bent upwards. Figure 4 is a photograph showing a typical crushed condition at the bottom of a geocomposite edge drain. This often resulted in an invert above subgrade elevation. This occurred even when the geocomposite edge drains were installed in an open trench excavated to below subgrade elevation. All the geotextile wraps were cut by the geocomposite cores at their base. Second, where the geocomposite edge drain was installed next to a concrete pavement, frost heave caused holes to be worn through the geotextile wrap at the areas between the cusp tops and the pavement edge.

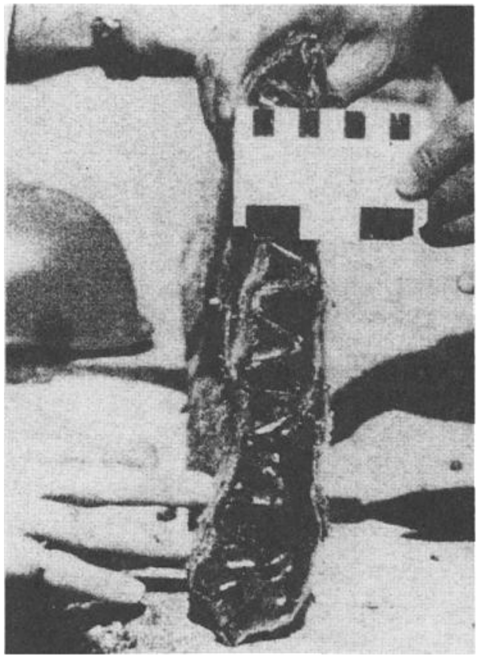


Figure 4 - *Photo of typical crushed condition at the bottom of a geocomposite edge drain.*

Geotextile-Wrapped Aggregate Edge Drain

Raymond et al. *[11]* have reported on a site typical of the performance of a geotextile-wrapped open-graded aggregate edge drain with pipe (unwrapped) installed in an area where the subgrade was a gravelly clay (GC). Geotextile-wrapped open-graded aggregate drains were installed in a trench whose base was below subgrade elevation. The drains all failed to arrest subgrade pumping where subgrade pumping was or could occur due to a deficient subbase/base/separator graded filter. Typically the sides and top of the geotextile were observed to be relatively clean. The main fouling occurred on the portion of the geotextile above the trench base and within the aggregate below the pipe, where the main function of the aggregate bedding is to trap fines. Figure 5 is a photograph of the typical

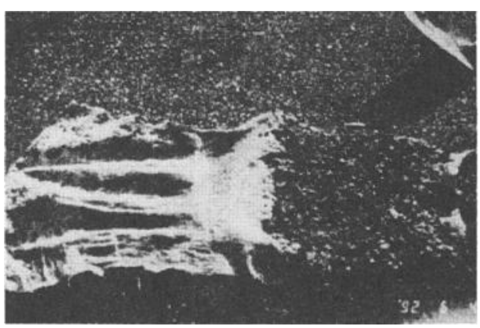


Figure 5 - *Photo of typical conditions of trench geotextile-wrap.*

condition of the base of the trench geotextile-wrap showing the trapped fines below the pipe elevation. At the outlets, the perforated drainage pipes were correctly turned down to the base elevation of the geotextile and then connected through T-junctions to a non-perforated outlet pipe. Figure 6 shows a longitudinal section of an outlet and Figure 7 shows a transverse cross section. The T-junction was wrapped in geotextile that was bedded on about 150 mm of Granular 'A' (similar to ASTM Specification for graded aggregate material for bases or subbases for highways or airports, D-2940) with a very low permeability relative to the open-graded aggregate material of the edge drain.

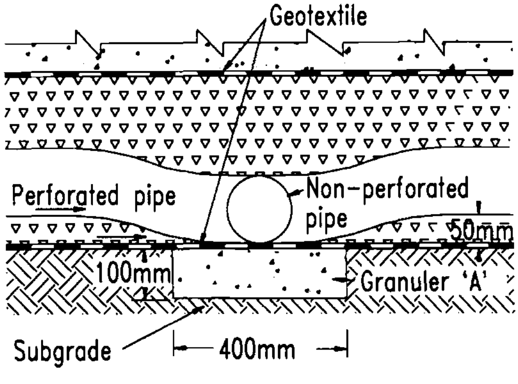


Figure 6 - *Longitudinal edge pipe outlet connection.*

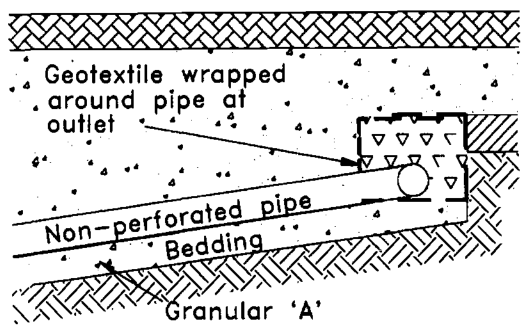


Figure 7 - *Transverse edge pipe outlet connection.*

The excavations uncovered evidence of: a) pumping, b) differential heave/settlement at the shoulder/pavement edge boundary, and c) deterioration of the quality of the cement-treated base aggregate. The edge drains did not stop subgrade pumping. No damage to the geotextile was evident (FOS = 140 μm; mass/unit area = 200 g/m^2; grab strength = 500 N with elongation at break of 70%) or of the pipe.

Geocomposite Edge Drain with Sand Subgrade

Raymond et al. *[6]* have reported on a site typical of the performance of geocomposite edge drains (GED) and sand subgrades. Excavations were made at or near the lowest elevation of the portion of highway being investigated. An in-situ water test showed that the geotextile-wrap around the plastic core was badly fouled and almost impermeable. After the water test, removal of sections of the geocomposite showed that the void area of the plastic core was clean. It was observed that the backfill had been correctly installed only on the shoulder side of the geocomposite trench and that the backfill had a low permeability. The cut face of the installation trench was covered with fine soil, which after cleaning, exposed the OGDL (composed of particles up to 14 mm in size) and Granular 'A' (composed of particles up to 27 mm in size), initial trench faces. These were seen to be very rough and the OGDL at the cut face was plugged with fines.

Figure 8 is a photograph showing a typical fouled condition of the pavement/edge drain trench wall. It is believed that as water entered the cracked pavement surface, it drained downslope within and across the OGDL and then discharged from the very permeable OGDL to the edge drain trench. Here the water flow was resisted by the less permeable geotextile (particularly at the lowest elevation). This resulted in the water becoming turbulent in the voids between the rough cut surface of the OGDL and the geotextile causing fines from the subgrade to be deposited between the OGDL and the geotextile. With a build up of fines, drainage was impeded and the geotextile fouled.

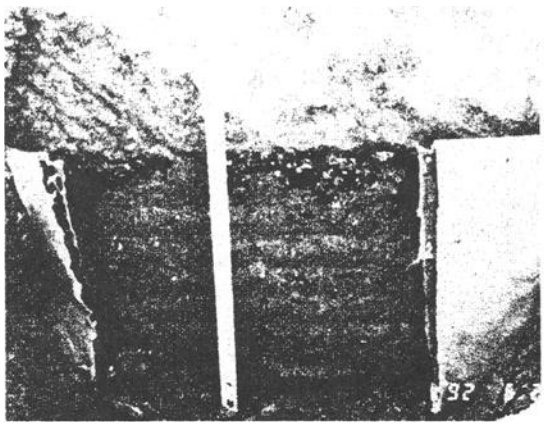


Figure 8 - *Photo of typical fouled condition of pavement/edge drain trench wall.*

New Concept

The difference in field frost heave between the loaded pavement and unloaded shoulder is analogous to the difference in heave between two laboratory frost heave test soil specimens; one loaded and one unloaded. Figure 9 illustrates this effect for an unloaded shoulder and loaded pavement. The upper left diagram shows the initial

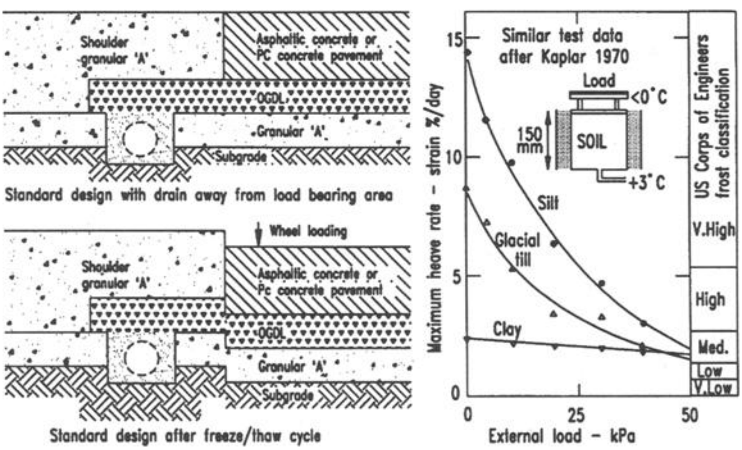


Figure 9 - *Concept of observed frost heave behavior.*

construction, while the lower left diagram shows the effect of winter heave and the diagram on the right shows laboratory experimental observations by Kaplar *[12]* of the reduced frost heave resulting from surface loading. It is clear from this figure that retrofit drain trenches must be excavated as close to the pavement edge as possible. New drains should be placed below the pavement, or partially paved shoulder where the paved shoulder portion is made integral with the pavement. It is also evident that the GED performance concept is contrary to generally accepted geotextile filter concepts that require the permeability of the soil to be at least an order of magnitude less than the permeability of the geotextile, (e.g., Giroud *[13]*).

A new concept is needed if a GED is to be used. Consider allowing the OGDL to discharge into a trench filled with a high permeability backfill with the GED on the shoulder side of the trench. The surface area of the GED geotextile receiving water would then be considerably larger than its area contacting the OGDL. Under proper construction, voids would not develop between the OGDL and the backfill. With a surrounding sand, rather than gravel, and care during installation, a low mass per unit area geotextile (200 to 230 g/m^2), as presently used by the Ministry of Transportation, Ontario (MTO), should be sufficient.

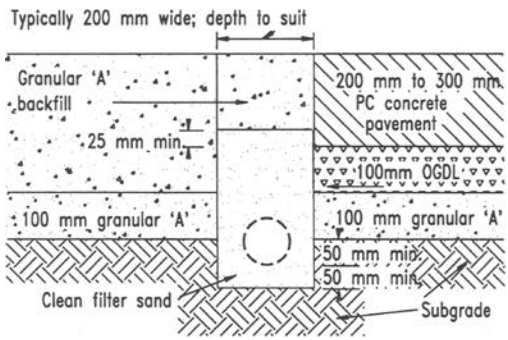


Figure 10 - *Pipe edge drain installation technique for retrofit highway construction undertaken by Ministry of Transportation of Ontario.*

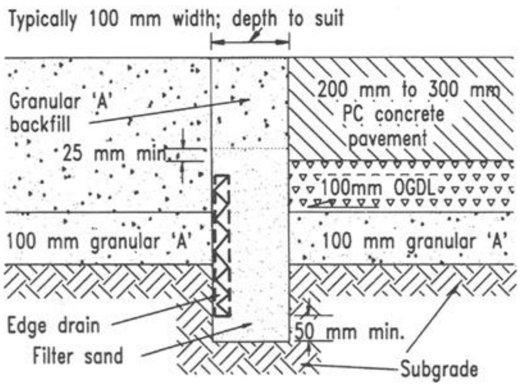


Figure 11 - *Geocomposite edge drain installation technique for retrofit highway construction undertaken by Ministry of Transportation of Ontario.*

Recommendations

It is recommended that all retrofit edge drainage conduits such as pipe edge drains or geocomposite edge drains be installed in an open trench, or with a mechanical excavator and boot with placement of the geocomposite, if used, on the shoulder side of the excavation. These recommendations for the installation of retrofit drains are illustrated in Figures 10 and 11. To prevent debris from falling back into the trench, excavated soil should be removed from the

Table 1 - *Grading curve for filter sand to prevent pumping of silt or clay.*

Sieve Size Micrometer (μm)	Sieve Size ASTM* E-11	Percent Passing by Weight
9500	3/8"	100
4750	No. 4	95-100
2360	No. 8	80-100
1180	No. 16	50-85
600	No. 30	25-60
300	No. 50	10-30
150	No. 100	2-10
75	No. 200	0-2

* ASTM Specification for wire-cloth sieves for testing purposes

trench opening surface. Trenches should be filled no later than the same day as excavated. The backfill should consist of an extra clean sand, preferable manufactured and graded to that of concrete sand (ASTM Specification for concrete aggregates, C-33) with the additional requirement of 0-2% at the 75 μm sieve size. Table 1 gives a recommended grading for this clean sand. The trench base should have a bedding layer where practical to protect the conduit from infiltration of subgrade fines. In addition, the outlet inverts must be dropped to the invert elevation of the trench in much the same way as illustrated in Figures 6 and 7. Where practical, the drainage conduit should have a minimum longitudinal grade of 1.0% (100H:1V).

Based on observations at excavated drain sites and recommendations summarized above, MTO has revised their edge drain design for new highway construction. Based on the performance of the partially paved portion of the shoulder at the sites investigated, all new partially paved shoulders are now made integral with the pavement and with the same structural design. These details are given in Figure 12 showing that the edge drain is positioned below the integral partially paved shoulder pavement slab. The method prevents detrimental frost heave at the pavement slab edge.

Similar recommendations (for different reasons) have been arrived at for geocomposite drain installations by Koerner et al. *[14]* and Koerner and Koerner *[15]*. Koerner et al. *[14]* describe one method of installation in an open trench that has been in use by the Kentucky Department of Transportation since 1992 (Allen and Fleckenstein, *[16]*). The sand backfill was compacted by hydraulic sluicing.

In addition to the recommendations related to edge drains, it was apparent that both the bases and edges of OGDL must be protected from the intrusion of fine-grained soils and fine sands by an appropriate non-cemented graded granular filter layer.

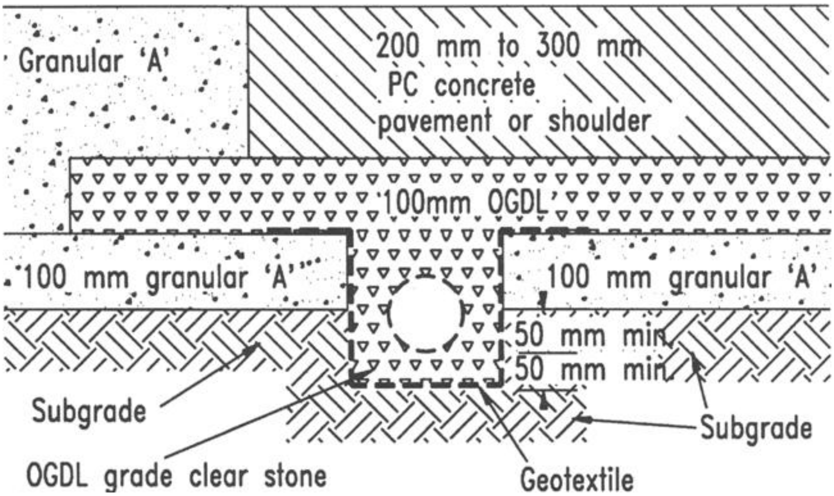


Figure 12 - *Edge drain details for new highway construction undertaken by Ministry of Transportation of Ontario.*

Typically, below an OGDL the filter layer material should conform to ASTM Specification for graded aggregate material for bases or subbases for highways or airports D-2940 and be a well-graded non-cement-treated gravel (GW, ASTM Classification of soils for engineering purposes, D-2487) or, where frost effects are not a concern, GW-GM. Below and around edge drainage conduits this material would be concrete sand conforming to ASTM Specification for concrete aggregates C-33 with the additional requirement of 0 to 2% at the 75 μm sieve size. Lean concrete layers and/or geotextiles are unacceptable as filters to prevent pumping of minus 75 μm sized subgrade fines. Considerable differential settlement due to frost action was recorded at all asphaltic concrete shoulder/cement concrete pavement boundaries. It is recommended that for all future concrete pavements built with paved or partially paved shoulders the shoulder paving should be a cement concrete shoulder constructed integral with the cement concrete pavement.

Acknowledgment

Gratefully acknowledged is the considerable cooperation and help received by the Ministry of Transportation, Ontario personnel who supervised and provided the excavation equipment and manpower. Particular thanks are extended to Dr. J. Hajek, Ms. P. Marks and Mr. S. Senior of the MTO for their significant contributions to this research study.

References

[1] Cedergren, H. R., Arman, J. A. and O'Brian K. H. *Development of Guidelines for*

the Design of Subsurface Drainage Systems for Highway Pavement Structural Sections, Final Report No. FHWA-RD-73-14., FHWA, Washington, DC, 1973, 193 p.

[2] Moynahan Jr., T. J. and Sternberg, Y. M. "Effects on Highway Subdrainage of Gradation and Direction of Flow Within a Densely Graded Base Course Material," *Transportation Research Record No. 497*, Transportation Research Board, Washington, DC, 1974, pp. 50-59.

[3] Ridgeway, H. H. *Pavement Edge Drainage Systems*, Synthesis of Highway Practice Report No. 96, National Cooperative Highway Research Program, Transportation Research Board, Washington, DC, 1982, 38 p.

[4] Hajek, J., Kazmierowski, T. J., Strum, H., Bathurst, R. J. and Raymond, G. P. "Field Performance of Open Graded Drainage Layers," *Highway Research Record 1354*, Transportation Research Board, Washington, DC, 1992, pp. 55-64.

[5] Raymond, G. P., Bathurst, R. J. and Hajek, J. "Evaluation of Pre-1994 Ontario Highway Edge Drain Practice," *Recent Developments in Geotextile Filters and Prefabricated Drainage Geocomposites, ASTM STP 1281*, Bhatia and Suits, Eds., American Society for Testing and Materials, West Conshohochen, PA, 1995, pp. 222-233.

[6] Raymond, G. P., Bathurst, R. J. and Hajek, J. "Evaluation of Highway Edge Drains Incorporating Geosynthetics," *Proceedings International Symposium on Unbound Aggregates in Roads UNBAR4*, Nottingham (UK), 1995, pp. 75-85.

[7] Raymond, G. P., Bathurst, R. J. and Hajek, J. "Evaluation of Geocomposite Edge Drain on Sand Subgrade," *Geofilters'96, Proceedings 2nd International Conference on Filtration and Drainage in Geotechnical Engineering*, Montreal, Quebec, 1996, pp. 369-378.

[8] Raymond, G. P., Bathurst, R. J. and Hajek, J. "Geosynthetics Use in Cold Climate Pavement Drainage," *Proceedings 49th Canadian Geotechnical Conference*, St. Johns, Newfoundland, 1996, pp. 369-378.

[9] CGSB 148.1. *Method of Testing Geosynthetics: Geotextiles Filtration Opening Size*, Canadian General Standards Board, Ottawa, Canada, 1993.

[10] Raymond, G. P., Bathurst, R. J. and Hajek, J. "Geocomposite Edge Drain Evaluation on Ontario Highway 402 at Strathroy," *Proceedings 5th International Conference on Geosynthetics*, Singapore, Vol. 2, 1994, pp. 743-746.

[11] Raymond, G. P., Bathurst, R. J. and Hajek, J. "A Geotextile-Wrapped Aggregate Highway Drain Evaluation," *Proceedings 6th International Conference on Geosynthetics*, Atlanta, GA, Vol. 2, 1998, pp. 929-938.

[12] Kaplar, C. W. "Phenomenon and Mechanism of Frost Heaving," *Highway Research Record 304*, Transportation Research Board, Washington, DC, 1970, pp. 1-13.

[13] Giroud, J-P., "Filter Criteria for Geotextiles," *Proceedings 2nd International Conference on Geotextiles*, Las Vegas, Nevada, Vol. 1, 1982, pp. 103-108.

[14] Koerner, R. M., Koerner, G. R., Fahim, A. K., and Wilson-Fahmy, R. F. *Long Term Performance of Geosynthetics in Drainage Applications*, National Cooperative Highway Research Program Report 367, Transportation Research Board, Washington, DC, 1994, 54 p.

[15] Koerner, R. M. and Koerner, G. R. "Lessons Learned from Geocomposite Edge Drain Clogging due to Improper Installation," *Proceedings Geosynthetics: Lessons Learned from Failures*, Industrial Fabrics Association International, Nashville, Tennessee, 1995, pp. 135-141.

[16] Allen, D. L. and Fleckenstein, L. J. "Field and Laboratory Comparison of Pavement Edge Drains in Kentucky," *Transportation Research Record No. 1425*, Transportation Research Board, Washington, DC, 1993, pp. 1-11.

L. John Fleckenstein[1] and David L. Allen[2]

Development of a Performance-Based Specification (QC/QA) for Highway Edge Drains in Kentucky

---

Reference: Fleckenstein, L. J. and Allen, D. L., **"Development of a Performance-Based Specification (QC/QA) for Highway Edge Drains in Kentucky,"** *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390,* L. D. Suits, J. B. Goddard, and J. S. Baldwin, American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: The Kentucky Department of Highways (DOH) has been trying to remove water from its pavements since the early 1970s. Aggregate-filled trenches were used to remove water in the early 70s and perforated pipe edge drains were introduced in the mid-1970s. These systems remained relatively unchanged until the late 1980s. In 1987, after a number of localized pavement failures throughout Kentucky, the Kentucky Transportation Center (KTC) and the Kentucky DOH started evaluating the effectiveness and performance of these edge drain systems. It was found that most of the panel edge drains were significantly damaged during installation and that approximately 50% of the outlet pipes were crushed and/or the headwalls were clogged. As a result of these findings, several design changes were made in 1989. Research conducted between 1989 and 1991 indicated that the performance of the system had improved but failures still existed. In 1997, after the completion of an in-depth research study on the performance and construction of these systems the Kentucky DOH required that all edge drain outlets be inspected with a pipeline inspection camera. The contractor was made responsible for inspecting and repairing his own work. This was the early stages of a quality control (QC) program for edge drains in Kentucky. The camera inspections decreased the number of edge drain outlet failures from approximately 20% to approximately 3% to 5%. Of the 3% to 5% found damaged, the contractor was responsible for complete repairs. Current failure rates for mainlines are approximately 2%. At this time only 1/3 of the mainline is being inspected (headwall to approximately 150 ft into the mainline). In May 1998, the Kentucky DOH established a mission to incorporate quality into their construction projects by transitioning, where possible, from method specifications to QC/QA, performance related, warranty, and other innovative specifications for the year 2000. In 1998, a team was formed to evaluate the current edge drain specification and make recommendations and revisions towards a full fledged QC/QA specification. This paper discusses the historical performance of edge drains in Kentucky and the recent changes in the specification.

Keywords: pavement drainage, edge drains, performance, construction, QC/QA, specification

---

[1]Senior Research Investigator, University of Kentucky, Kentucky Transportation Center, 279 Raymond Building, Lexington, KY 40506-0281.

[2]Chief Research Engineer, University of Kentucky, Kentucky Transportation Center, 279 Raymond Building, Lexington, KY 40506-0281.

Background

Since the early 1970s the Kentucky Department of Highways (DOH) has been trying to remove water from its pavements. Aggregate-filled trenches were used to remove water in the early 70s. In the mid 70s the first perforated pipe edge drain systems were installed. Approximately 10 years later Kentucky installed their first panel drains. Prior to 1989, the Kentucky DOH installed panel drains on the inside of the trench and backfilled with existing trench material. Round perforated pipe edge drains were backfilled with sand or No. 57 stone. The outlets for both of these systems consisted of single-wall, non-perforated pipe backfilled with existing trench material connected to a precast concrete headwall. The headwall spacings averaged approximately 229 m (750 ft.) or more, and at times exceeded 452 m (1500 ft.) A majority of these installations were retrofit edge drains installed along existing interstates and parkways. These systems remained unchanged until 1989.

In 1987, after a number of localized pavement failures throughout Kentucky, the Kentucky Transportation Center (KTC) and the Transportation Cabinet started to evaluate the overall effectiveness and performance of the edge drain systems. It was found that most of the panel edge drains were significantly damaged during installation and that the outlet pipes were crushed and/or the headwalls were clogged (Figures 1 and 2). The net result was that approximately 43 % of the edge drain outlets were not functional.

In 1989, several design changes were made to the edge drain systems. These included moving the panel drains to the backside of the trench, backfilling (flushing) with sand and using a stiffer double-walled polyethylene outlet pipe. In addition, the headwall spacings were reduced to a maximum of 152 m (500 ft.) and the outlets were backfilled with a granular backfill (processed stone).

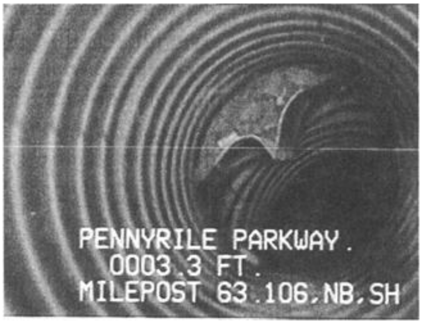


Figure 1 - Crushed and ripped outlet pipe.

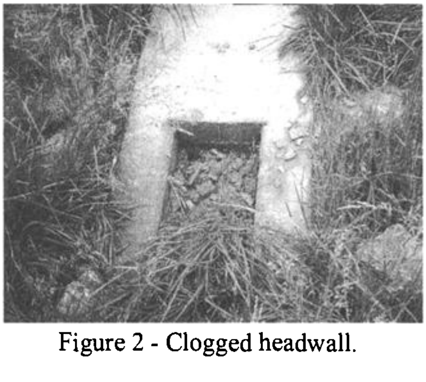


Figure 2 - Clogged headwall.

Research conducted between 1989 and 1991 indicated that the performance of the system had improved but failures still existed. Visual inspection of headwalls and pipeline camera inspections of the outlets indicated that approximately 20% of the outlets were not functional (outlet pipe <40% open and/or headwall plugged) and 36% were partially functional (outlet pipe 40-60% open and/or headwall covered). On several projects it was observed that the weaker single-wall pipe was still being utilized in the precast headwalls. A large percentage of the failures were occurring on the backside of the headwall where the single wall flexible pipe "pig tail" (approximately one foot long) exited the back of the headwall. In addition to the headwalls being clogged with debris, it was observed that several of the headwalls were located too low in the ditchline allowing water to pond over the headwalls. Backward settlement was also observed in 20% of the headwalls, causing a negative slope in the invert of the headwall.

Extensive research was also conducted on the performance of the panel drains. Moving the panels to the backside of the trench and backfilling with sand vastly improved the performance of the panels but distress was still apparent with some of the less rigid materials. Borescope inspections and laboratory testing indicated that even with the sand backfill the less rigid post and cuspated panel had core (open area designed for water transport) reductions of five to 15%. The more rigid solid core products performed substantially better.

In 1992, No. 2 stone was placed two feet around the edge of the headwalls to reduce erosion and vegetation (Figure 3). In addition, some maintenance was initiated on existing headwalls.

Figure 3 - No. 2 placed around the headwall.

Research and Development

In 1992, a four-year research project was established to determine the lateral effectiveness of the edge drains, to verify that pavement edge drains improve performance, to determine the cost effectiveness of pavement edge drains, to evaluate the major in-service problems of edges drains, and to develop a generic specification for pavement edge drains.

Results of the study indicated that pavement edge drains reduced the subgrade moisture by 30%, and this removal of water appeared to have significantly increased the subgrade strength. Evaluation of ride index (RI) data for a pavement with and without edge drains, and before and after edge drains indicated that the rate of deterioration was less for pavements with edge drains. The data indicated pavements with edge drains would have an average extended life of approximately 7 years [*1*]. Life-cycle cost analysis indicated a cost savings of $148,733 per km ($239,892 per mile) for a pavement with edge drains [*2*].

The study also determined that the specification and construction changes since 1989 did have a positive impact on the performance of these systems but further changes were needed. These changes were addressed in the recommended generic specification. The specification recommended a redesigned headwall. The redesigns include shortening the headwalls to allow for easier placement into the ditchlines and redistributing the mass (centroid) to help assure against backward settlement. A slanted invert of 42 mm per meter (0.5 inch per ft.) was recommended over tilting the headwalls to achieve positive flow. A loop-type edge drain system was also recommended. As shown in Figure 4, the up-grade end of the system would be not dead ended, but would tie back into the up-grade headwall. This would allow for the entire system to be inspected and would permit better maintenance in the future. In addition to redesigning the headwalls, it was recommended that a stiffer grade outlet pipe and/or flowable fill be used.

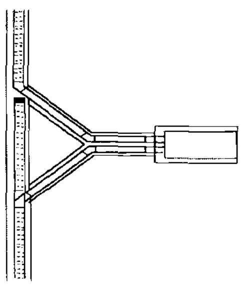


Figure 4 - Loop System

The specification also recommended design changes for edge drains discharging into shallow ditchlines. A collector pipe system was recommended over headwalls for these locations due to maintenance and construction problems. The specification also recommended using cameras to inspect the edge drains after installation. The specification included a test procedure for testing panel edge drains in vertical compression. This procedure was adopted by ASTM and is designated as ASTM Standard D6244-98, "Test Method for Vertical Compression of Geocomposite Pavement Panel Drains".

Specification Amendment, 1997

In 1997, the Kentucky DOH required that all edge drain outlets be inspected with a pipeline inspection camera. The mainline was required to be inspected 45m (150 feet) from the outlet into the mainline. The DOH made the contractor responsible for inspecting and repairing his own work (Figures 5 and 6). This was the early stages of a quality control (QC) program for edge drains. The camera inspections decreased the number of failures. Currently approximately three to 5 % of the outlet pipes that are inspected by the contractor during their initial inspection are not fully functional (Figure 7). Of the three to five percent found damaged, the contractor was responsible for complete repairs. Construction traffic is a large contributor to these failures. The number of failures has been reduced to approximately 1% in one of the highway districts where the outlet pipes are backfilled with flowable fill. Approximately 2% of the mainline pipes that are inspected state wide are damaged. This number is likely higher since (at this time) only about 1/3 of the entire mainline system is being inspected.

Figure 5 - Contractor personnel pushing camera into edge drain outlet for QC inspection.

Figure 6 - Contractor personnel recording and logging edge drain (QC) inspection from inside of inspection vehicle.

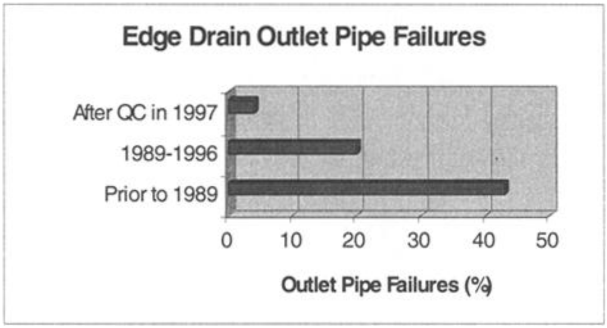


Figure 7 - Edge Drain Outlet Pipe Failures.

Development of a Performance-Based Specification

In May of 1998, the Kentucky DOH established a mission to incorporate quality into KyTC construction projects by transitioning from method specifications to QC/QA, performance related, warranty, and other innovative specifications that allow contractors the freedom to utilize their expertise and that promote efficient use of department personnel. The DOH appointed 10 teams to evaluate existing specifications and to transition a reasonable number of these specifications, in order of priority, into appropriate innovative specifications in time to be incorporated into the new specifications book for the year 2000.

The drainage team was assigned the duty to evaluate the possibility of developing the edge drain special note written in 1997 into a full fledged QC/QA specification. The team met over the course of a year reviewing other agencies specifications, the generic specification written by KTC, and evaluating problems and different construction techniques being utilized throughout the state. This information was compiled into a draft QC/QA Specification for Highway Edge Drains. The specification was written not only to address new construction but also rehabilitation work where it may be necessary to inspect and repair a system, or inspect and replace an existing system. The QC portion of the specification remained under the contractor, and the State became responsible for conducting the QA prior to the project being finalized. The proposed QA will be conducted by the state using the contractor's equipment and personnel. It is proposed that the QA will encompass approximately 20 percent of the installation. Disincentives and incentives are currently being discussed.

With the draft QC/QA specification in the final stages a pilot project was initiated through a change order for a current rehabilitation project on Interstate 64 in Franklin County in March 1999. Approximately 1/4 of the project contained the new recommended design changes (approximately 8 km (5 miles of interstate)). The design changes included: new headwall (Figure 8), loop type system, flowable fill for outlets, ditchline collector system, and inspection ports for the ditchline collector pipe and the edge drain loop. Channel lining was also utilized in the ditches along the cuts to decrease erosion and maintenance.

The recommended new headwall designs were submitted to the local manufacturer for review. The manufacturer was able to incorporate several of the design changes without significant modifications to the existing headwall forms.

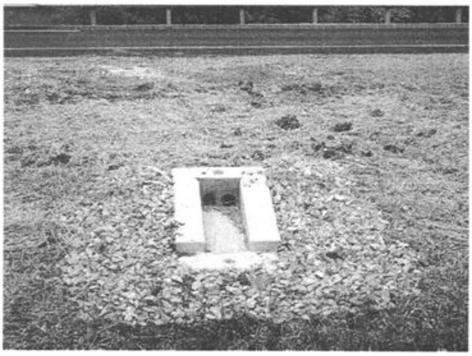


Figure 8 - Loop System Headwall

The edge drain collector pipe inspection port was designed and constructed by the contractor (Figure 9). The inspection port was constructed so that the edge drain loop system and the 203 mm (8 in.) collector line could be inspected from the same location. Figure 10 shows a pipeline camera (mounted on a motorized tractor) inspecting the 203 mm (8 in.) collector. Figure 11 shows significant compression found in the 203 mm (8 in.) collector during the QC inspection. Figure 12 shows the location of the edge drain inspection port and the manhole installed. Figure 13 shows the headwall and outlet for the edge drain collector system.

Figure 9 - Construction of ditchline collector system, and vertical inspection port.

Figure 10 - Pipeline camera inspecting 8-inch collector pipe.

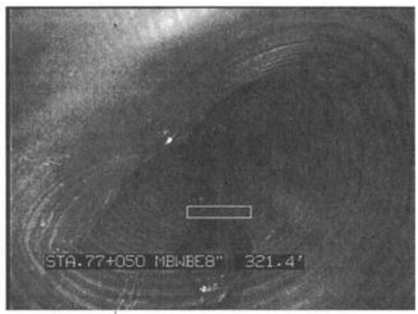


Figure 11 - Failure observed in collector pipe.

Figure 12 - View of temporary cover for top of edge drain collector and inspection port.

Figure 13 - Headwall and outlet for edge drain collector system.

Conclusions

The construction and design changes to pavement edge drains over the last 12 years have decreased the number of edge drain failures throughout the state. The use of pipeline cameras to inspect edge drains has provided vital information in the construction, performance, and design of these systems. The establishment of the QC program in 1997 has had the greatest impact on reducing these failures. The establishment of a QC/QA program for pavement edge drains and the proposed design changes should further decrease the percentage of failures and increase the performance of these systems.

Several of the design changes should reduce the maintenance requirement of these systems, but not entirely. Maintenance programs should be established for these systems. It is recommended that the headwalls and rodent screens be inspected and cleaned two to three times a year. The outlets and mainlines should be flushed every three to four years. Further research is needed to evaluate required maintenance intervals.

Reference

[1] Fleckenstein, L. J., and Allen, D. L., "Evaluation of Pavement Edge Drains and Effect on Pavement Performance," Research Report KTC-94-20, Kentucky Transportation Center, University of Kentucky, Lexington, KY, 1994.

[2] Fleckenstein, L. J., and Allen, D. L., "Performance and Cost Effectiveness of Pavement Edge Drains," Research Report KTC-97-5, Kentucky Transportation Center, University of Kentucky, Lexington, KY, 1997.

Mohamed K. Elfino,[1] David G. Riley,[2] and Thomas R. Baas[3]

Key Installation Issues Impacting the Performance of Geocomposite Pavement Edgedrain Systems

---

Reference: Elfino, M. K., Riley, D. G., and Baas, T. R., "**Key Installation Issues Impacting the Performance of Geocomposite Pavement Edgedrain Systems,**" *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390*, L. D. Suits, J. B. Goddard, and J. S. Baldwin, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: This paper addresses the key installation issues impacting the performance of geocomposite pavement edgedrain systems. This includes maintaining the verticality of the drain panel in the trench, proper positioning of the drain panel within the trench, backfilling with open-graded coarse aggregate, timely installation of outlet fittings and pipe, and the use of outlet pipes with adequate pipe stiffness. Three highway rehabilitation projects involving the installation of approximately 120 km (400,000 L.F.) of geocomposite edgedrains in Virginia and Ohio are investigated, and lessons learned documented in this paper. Actual cost savings, Up to 50%, were realized due to the use of geocomposite edgedrain compared to conventional edgedrains. Conclusions and recommendations are presented in support of successful installations, and lasting performance to meet pavement design life requirements.

Keywords: Edgedrain, Pavement Drainage, Installation of geocomposite.

Introduction

The performance of any construction product and/or system is largely dependent on the installation technique and the properties of the associated materials used and specified. This is particularly true for pavement edgedrain systems, whether we are speaking of

---

[1] PE, Ph.D., Assistant State Materials Engineer, Virginia Department of Transportation, Richmond, Virginia 23219.

[2] PE, Roadway Hydraulic Engineer, Ohio Department of Transportation, Columbus, Ohio 43223.

[3] Regional Sales Manager, Contech Construction Products, Inc. Oak Brook, Illinois 60523.

vitrified clay pipe and aggregate; fabric sock wrapped pipe, plastic pipe and aggregate or a geocomposite pavement edgedrain system.

Geocomposite pavement edgedrain systems were first introduced in the mid-1980s. This somewhat radical approach of installing a complete pavement edgedrain system in one operation, as shown in Figure 1, offered the pavement engineer numerous advantages.

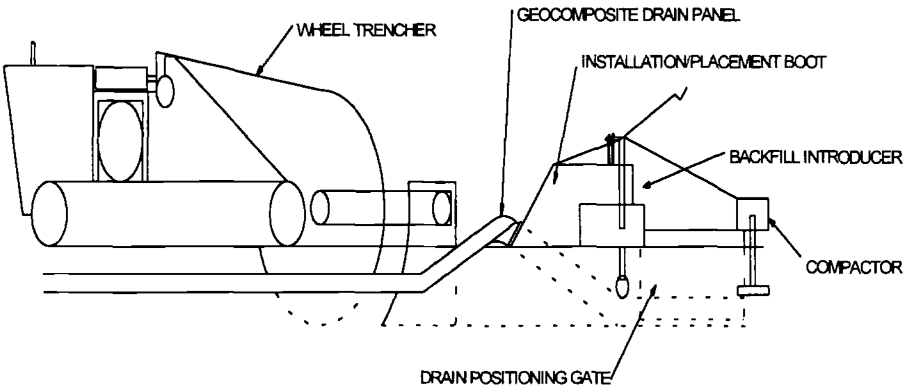


Figure 1- *Schematic for Installation of Geocomposite Drain Panel in One Operation.*

Key performance advantages such as faster response time, positive filtration and efficient collection and disposal of water within the pavement section met the needs and infrastructure retrofit performance criteria of the pavement engineer. The economics provided by geocomposite pavement edgedrains (up to 50% less, installed cost, than other conventional edgedrain systems), as well, fueled the interest, rapid acceptance, and use of geocomposite pavement edgedrain systems.

The initially developed installation technique for geocomposite pavement edgedrains was sound and would allow for the installation of a performance oriented geocomposite pavement edgedrain system. However, the rapid growth, use, and increased number of installing contractors using geocomposite edgedrains led to some unforeseen installation problems. These problems were common to all the available geocomposite edgedrain products in some form or fashion. Most problems were caused from poor and/or incorrect installation techniques, use of backfill materials containing high amount of fines, as well as the improper timing and installation of the outlet fittings and pipe and not meeting the specifications of the designer of the geocomposite pavement edgedrain system. As a result, the ability to maintain verticality of the geocomposite drain panel within the trench was often compromised yielding diminished flow capacity.

As expected, the rapid use and acceptance of geocomposite edgedrain systems being installed, without attention to the problems briefly outlined above, were questioned and led to a growing concern about the effectiveness and prudent use of geocomposite pavement edgedrains. Proper installation techniques and appropriate inspection of the installation is a must if the pavement engineer expects a geocomposite edgedrain system or any pavement edgedrain system to perform to his expectations and design criteria.

A properly installed and performance oriented geocomposite pavement edgedrain system results when the following key installation issues are addressed:

- Maintaining the verticality of the drain panel in the trench.
- Proper position of the drain panel within the trench.
- Backfilling the trench with open-graded coarse aggregate.
- Timely installation and proper stiffness of outlet fittings and pipe.

Maintaining the Verticality of the Drain Panel in the Trench

Geocomposite pavement edgedrains by their typical shapes and geometry are designed to be installed vertically, Figures 2 and 3, without excessive deformation in either the vertical or horizontal plane of the drain. The installation technique used must insure that during the initial placement and backfilling of the drain, the drain panel verticality in the trench is maintained. The use of installation equipment which use a drain panel positioning plate or wheel to push the drain panel up against the trench wall during backfill placement is effective in obtaining drain panel verticality within the trench, as shown in Figures 4 and 5. This positioning plate or wheel is particularly important during the backfilling operation due to the dragdown forces exerted on the drain panel during the backfilling operation. Figure 6a shows a drain panel without deformation, while Figures 6 b, and figure 6 c show vertical deformations, of the drain panel, in the form of "C ing" and/or "J ing" respectively.

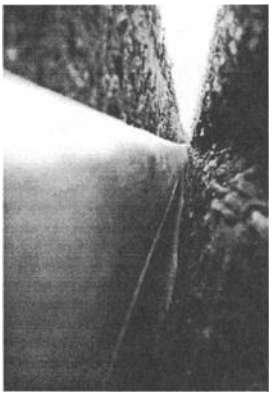


Figure 2- *Internal View of Trench Showing the Vertical Installation of Drain Panel.*

Figure 3- *Overview of Trench and Drain Panel.*

This deformation can occur during the backfilling operation, and result in a loss of flow capacity. This potential vertical deformation can be significantly reduced and/or eliminated by using installation equipment with vertical positioning wheels and plates. The potential for vertical deformation can also be reduced by placing the backfill in multiple lifts, which results in a reduction of the dragdown forces present during the backfill operation.

Figure 4- *Drain Panel Positioning Plate.*

Figure 5- *Drain Panel Positioning Wheel.*

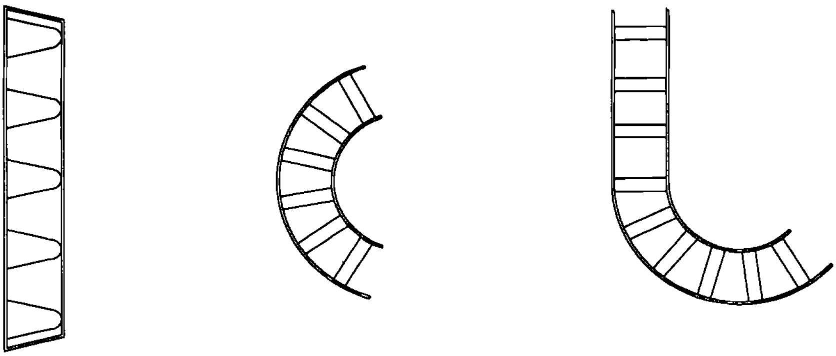


Figure 6- *Drain Panel (a) without deformation, (b) C-ing Deformation, (c) J-ing Deformation*

Proper Position of the Drain Panel within the Trench

The geocomposite pavement edgedrain can be placed against either the pavement side (Virginia practice) or the shoulder side of the trench walls (Ohio Practice), as shown in Figures 7 and 8 respectively. In both cases, drains with symmetrical support column design should be placed against the trench wall with the flat back of the drain exposed to the backfill placement operations. Exposing the flat back of the drain to the. backfilling operations reduces drain panel deformation in both the vertical and horizontal plane of the drain.

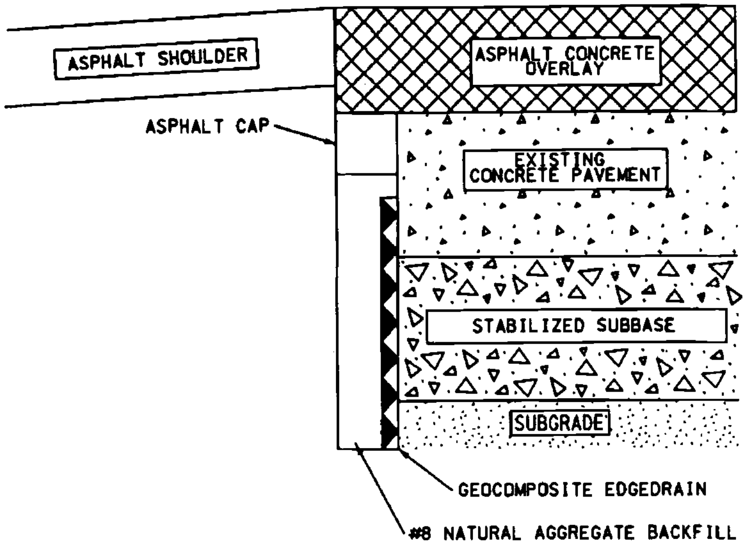


Figure 7- *Drain Panel Facing Pavement side, Virginia Practice.*

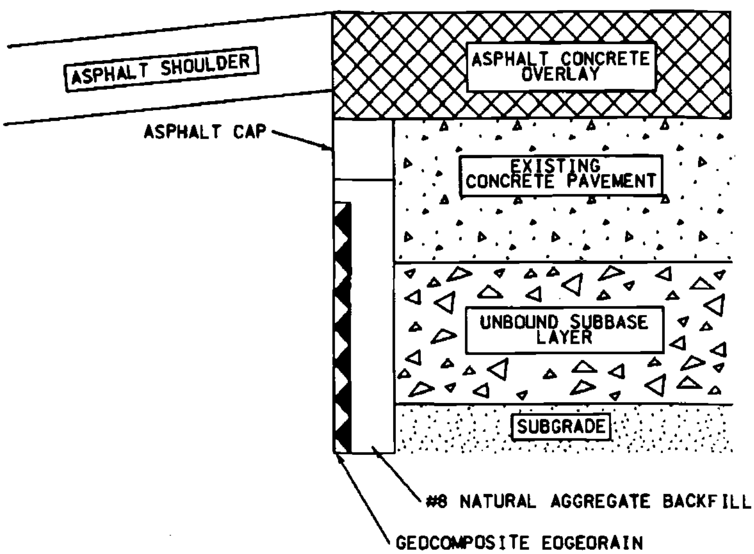


Figure 8- *Drain Panel Facing Shoulder side, Ohio Practice.*

Numerous field investigations [1 and 2] have found that the unbound aggregate base courses of some of the pavements being retrofitted with geocomposite edgedrains may show signs of sloughing during the trenching operation before the placement of the drain takes place. This sloughing of an unbound aggregate base course may create a void under the edge of the existing pavement. This void creates an opportunity for vertical and horizontal deformation of the drain during the backfill operations, Figure 9. When this condition is present the drain panel should be placed against the shoulder sidewall of the trench and backfilled with open-graded coarse aggregate, as shown in Figure10. This sloughing of the base course does not generally take place if a stabilized base course is present within the existing pavement section; therefore, the drain panel should be placed against the pavement side under this circumstance.

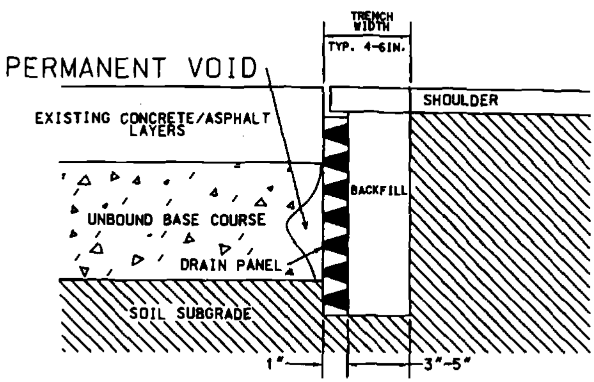


Figure 9- *Permanent Void Due to Sloughing of Unbound Aggregate Course.*

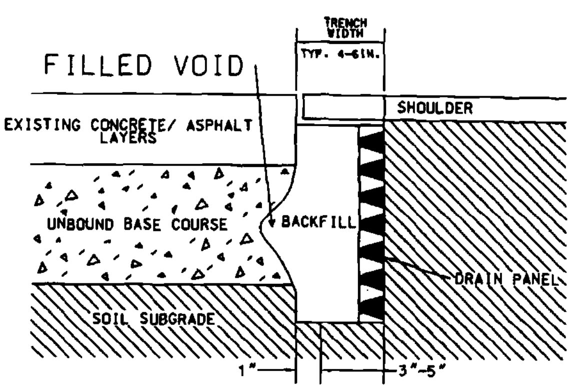


Figure 10- *Filled Void Due to Backfill Compaction.*

Clean trenches, stable trench walls, and proper positioning of the drain panel within the trench limits contribute greatly to the performance of a geocomposite pavement edgedrain.

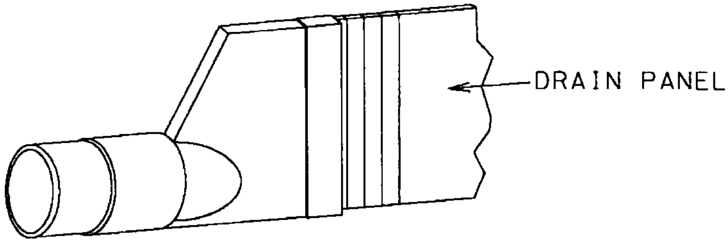
Backfilling the Trench with Open-Graded Coarse Aggregate

When first introduced, installation techniques for geocomposite edgedrains used the excavated in-situ material as backfill. Several concerns may arise from the use of excavated material as backfill. The "drainage" quality and gradations of the in-situ material is generally inadequate and inconsistent (2). Second, the gradation variation may allow for larger sizes of backfill and pavement materials to be placed against the drain panel as backfill.

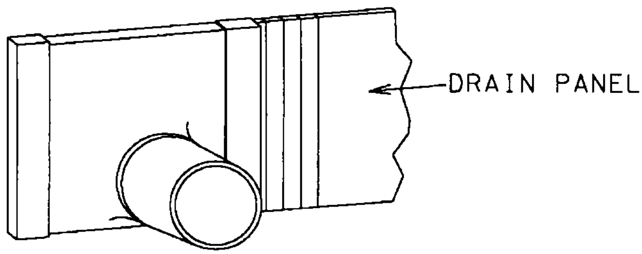
The use of open-graded coarse aggregate as backfill (AASHTO #8 or #78 type material) has a gradation that promotes additional drainage and water flow within the pavement/drain section. The use of open-graded coarse aggregate backfill material also reduces the potential for clogging and blinding of the geotextile overwrap of the drain panel that can occur from the use of the excavated material as backfill. The 9.5 mm (.375 inch) minus gradation allows for consistent placement, distribution and compaction of a graded backfill next to the drain panel and within the trench limits. Potential vertical deformation of the drain panel is also significantly reduced due to the ability to obtain more consistent and higher compaction densities of the backfill when open-graded coarse aggregate material is used as backfill.

Timely Installation and Proper Stiffness of Outlet Fittings and Pipe

The outlet fittings for a geocomposite pavement edgedrain system, Figure 11, must be installed within 24 hours of all mainline drain installed each day. Construction traffic and possible detour traffic may be operating on the pavement and the pavement drainage system should be in operating status under these conditions. The outlet fitting should be placed at the end of each segmented run of a length of 100 - 150 meters (300 - 500 ft). Two styles of outlet fittings are available to provide for either 90 degree (used at the vertical curves i.e. sag points) or 45 degree (used at tangents) relative to the drain mainline. The outlet pipe used should be a smooth wall pipe with a minimum pipe stiffness rating of 485 KPa (70 psi) that allows for positive alignment, positive grade maintenance, and deflection resistance during installation. The outlet pipe should be installed in a method similar to all other flexible drainage pipe structures with aggregate backfill for support around the outlet pipe. Precast concrete headwalls should be installed at the discharge point of the outlet pipe to provide deflection and damage protection from construction and maintenance traffic and mowing operations. A minimum free board of 150 mm (6 inches) above the 20-year storm in the ditch, is a must, so there is no water backing up into the system.



END OUTLET FITTING FOR 45 DEGREE TANGENT



SIDE OUTLET FITTING FOR SAG POINTS

Figure 11- *Outlet Fittings for Tangent and Sag Points.*

Two Case Histories

Virginia Department of Transportation Experience
Virginia Department of Transportation (VDOT) installed prefabricated geocomposite pavement edgedrains in two rehabilitation projects in 1996.

First Project - Location:
I-295 Hanover County, VA
(4 lanes in each direction)
Typical pavement section
150 mm (6 inches) cement treated aggregate base
200 mm (8 inches) continuously reinforced concrete pavement
Shoulder:75 mm (3 inches) asphalt concrete course
150 mm (6 inches) variable depth of aggregate base
History:
Built - 1979 (no edgedrain)
Type of edgedrain- prefabricated geocomposite pavement edgedrain 300 mm (12 inches)
Length of project - 9.65 Km (6 + miles)

Sequence of Construction: The trench is located at the edge of the slab and extends 425 mm (17 inches) below the surface. The panel is placed against the pavement side of the trench making sure it intercepts each pavement layer interface and extends 75 mm (3 inches) below the subgrade. The trench was cut through the shoulder using a wheel trencher to a depth of 425 mm (17 inches) at a width of 100 mm (4 inches). The drain was fed through a placement boot and laid vertically in the trench prior to backfilling. The panel was held against the pavement side of the trench using equipment with a drain panel-positioning wheel to maintain drain panel verticality in the trench. Backfilling the trench with open-graded aggregate (Virginia #78) with typical gradation is shown in Figure 12. The aggregate backfill is placed 50 mm (2 inches) above the top of the drain panel, then the asphalt concrete cap is placed to match the asphalt concrete shoulder elevation. At the end of each run of 150 m (500 ft) a 45° trench is dug through the shoulder and the panel is bent and extended approximately 0.6 m (2 ft) then capped with an end outlet fitting, as shown in Figure 13. A 100 mm (4 inches) diameter rigid, smooth bore polyethylene pipe is attached to the outlet fitting. The outlet pipe is then connected to the concrete headwall.

The above sequence of construction provides a performance-oriented drainage system. However, it was found that during the trenching operation, spoils of excavated material which were generated at each side of the trench were pushed across the open edgedrain trench to the unpaved shoulder area as fill material for build-up of the unpaved shoulder. In this process the excavated material got dumped into the open trench (i.e., before applying the asphalt cap), causing contamination of the top 50 to 75 mm (2 to 3 inches) of # 78 backfill material. Samples then were taken from the backfill material to check for the amount of contamination. The backfill failed to meet the gradation. The aforementioned practice was stopped immediately and three options were examined to eliminate backfill contamination.

1. Delay the sweeping across the trench, until the trench is covered with asphalt cap.
2. Overfill the trench with aggregate then push the spoil and the excess aggregate across the trench, and then sweep the top of the trench, to get rid of the contamination.
3. Use a conveyor belt to pick up all excavated material generated during trenching.

The second option was used by the contractor and yielded acceptable results as verified by our field inspection, resampling, and sieve analysis of the corrected backfill as shown in Figure 12. Both visual and mini camera inspections showed that the installation of the geocomposite edgedrain was vertical with little or no deformation and in intimate contact with the pavement layers and backfill.

Second Project – Location:
Loops and Ramps of VA Route 288 and 76
Chesterfield County, VA
Pavement Typical Section
150 mm (6 inches) cement treated aggregate base
200 mm (8 inches) continuously reinforced concrete pavement
Shoulder: 75 mm (3 inches) asphalt concrete course
150 mm (6 inches) variable depth of aggregate base

History
Built - 1989 (no edgedrain)
Type of edgedrain- prefabricated geocomposite edgedrain 300 mm (12 inches)
Length - 29 Km (18 miles)
Sequence of Construction : Trench and drain panel locations and depth were the same as the first project. However, the width of the trench was different in the second project. The contractor on this project excavated a trench width of 175 mm versus 100 mm (7 inches versus 4 inches) width, which meant more aggregate (Virginia #78) and asphalt cap. Because this project was not scheduled for overlay and there was no need to build up the existing unpaved shoulder, the contractor used a belt conveyor to haul the excavated material away

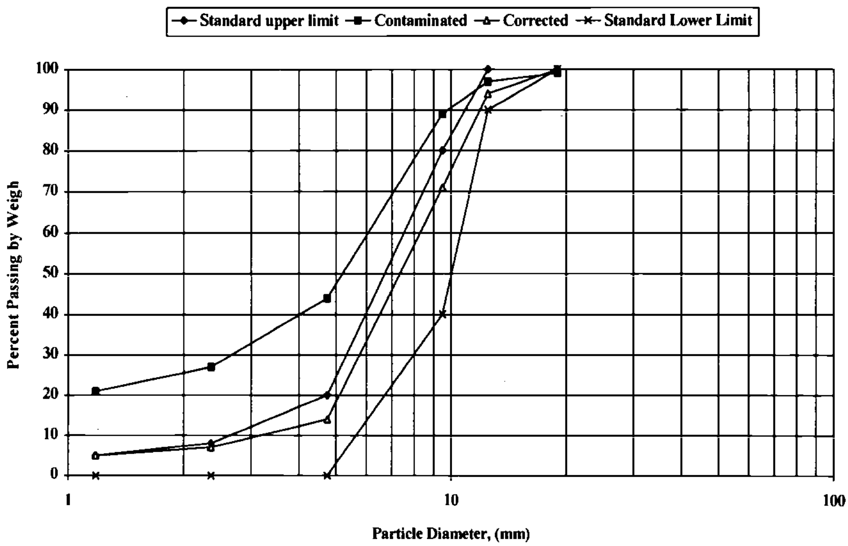


Figure 12-*Grain Size Distribution, Virginia #78 Coarse Aggregate.*

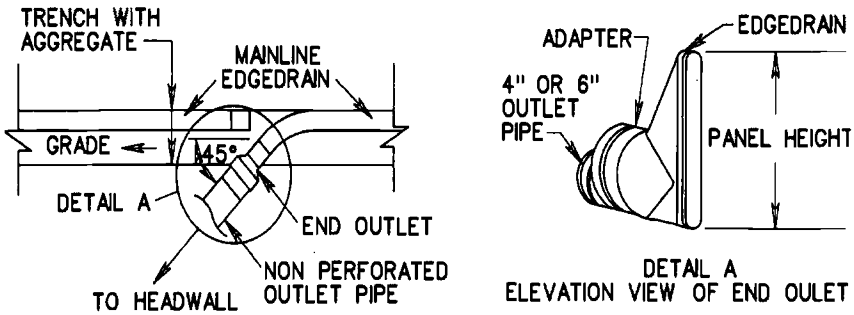


Figure 13- *Plan View of Edgedrain Connection to Outlet Pipe.*

On this project the Pavement Design and Evaluation Section, at VDOT, was involved from the point of contract preparation to attending the preconstruction conference. Emphasis was placed on contamination free backfill, straight cut of the trench walls, drain verticality and placement of outlet fittings and pipes at the end of each working day. Some fine contamination to the aggregate backfill due to sweeping the residual excavated material across the open trench was noticed. This practice was brought to the attention of the contractor and he delayed the sweeping until the asphalt cap was placed.

In Virginia, the policy is to place the drain panel against the pavement side of the trench due to the presence of stabilized subbase layers below the concrete slab. It also provides intimate contact with the water exiting from the pavement layer interfaces.

From both projects, it was concluded that the placement of outlet fittings and outlet pipe is very critical. Most contractors do not pay attention to the quality of the connection between the drain, outlet pipe and headwalls. Several locations were inspected and found that outlet pipes were crushed or disconnected. Attention is needed to maintain properly installed outlets.

Headwall installations were inspected and revealed instances of inadequate freeboard to allow for positive drainage. Maintaining the elevation and grade of the outlet pipe is essential to performance and maintenance of the edgedrain system.

Upon inspection of the asphalt cap, it was apparent settlement due to inadequate compaction of the aggregate backfill had occurred. A point to be made here is that there is no practical method to measure the density of open-graded coarse aggregate in a narrow trench (100 mm wide including the drain panel). Most contractors have a vibrator plate with a foot welded to it, which has a width equal to the backfill width behind the panel. The option of placing the backfill in more than one lift may solve this problem but slows the contractors production. This issue needs to be addressed in the field to avoid excessive settlement at the top of the trench, especially if no asphalt overlay is scheduled.

Both visual and mini camera inspection showed that the installation of the geocomposite edgedrain was vertical with little or no deformation and in intimate contact with the pavement layers and backfill.

The installed cost of the geocomposite pavement edgedrain system was $6.00/linear meter ($1.80/L.F.), while the installed cost was $15.00/linear meter ($4.50/L.F.) for conventional pipe and aggregate drain system, with considerable total savings of $347,850 for both projects.

Ohio Department of Transportation Experience

A large pavement rehabilitation project using prefabricated geocomposite pavement edgedrains was undertaken by the Ohio Department of Transportation (ODOT) in August of 1994.

Location:

Hancock County, Ohio

Interstate 75, Project I-75 -(3.31)

Project 842 (94)

Four lanes of pavement rehabilitation and overlay

Typical pavement section

150 mm (6 inches) aggregate base

225 mm (9 inches) reinforced concrete pavement

150 mm (6 inches) asphalt concrete, overlay

Shoulder: 300 mm (12 inches) aggregate base

225 mm (9 inches) asphalt concrete

History:

Built in 1965

Original construction used vitrified clay pipe segments with aggregate backfill for deep underdrain.

Length of Project:

Approximately 40,000 linear meters (130,000 L.F.) of various types of 450 mm (18 inches) geocomposite pavement edgedrains were used.

The underdrain trench is located at the edge of the pavement slab with the geocomposite drain panel placed against the shoulder side of the trench . The drain extends down into the subgrade approximately 75 mm (3 inches) below the base course and up approximately 25 mm (1 inch) above the bottom elevation of the existing pavement.

Sequence of Construction: The trench was cut using a wheel trencher to the design depth and to a 100 mm (4 inches) width. The drain panel was fed through a placement boot and into the trench prior to the placement of the ODOT #8 aggregate backfill , shown in Figure 14.

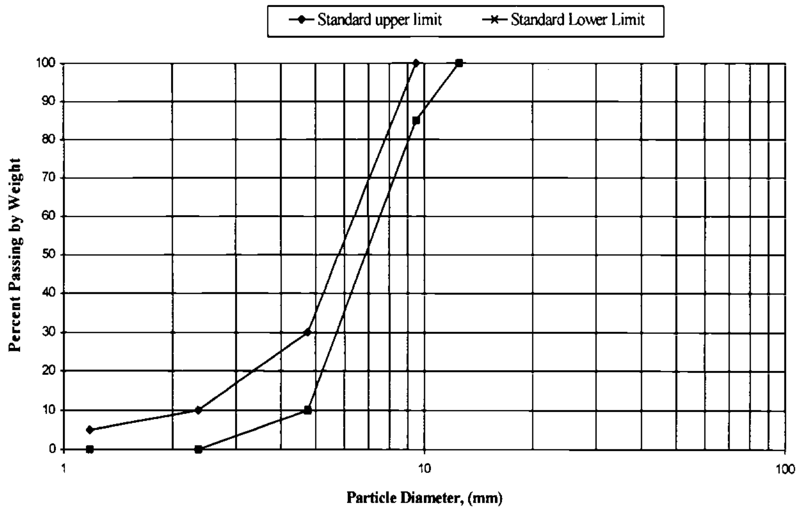


Figure 14- *Grain Size Distribution, Ohio # 8 Coarse Aggregate.*

The #8 aggregate backfill was placed using a road-widener unit equipped with a drain panel-positioning bar to aid in maintaining the verticality of the drain in the trench during backfill. This procedure and equipment help to hold the drain panel against the shoulder side trench wall during placement of the backfill. A production rate of approximately 3900 l.m. (13,000 L.F.) Per day was normally achieved throughout the project using this type of installation procedure and equipment. The backfill was brought to a final elevation of 50 mm (2 inches) above the top of the drain prior to the placement of the asphalt cap. Side outlet fittings with rigid plastic outlet pipes were installed. Concrete headwalls were installed at the outlet pipe. The outlets were placed on a 150 m (500 ft) maximum spacing or as determined by the ODOT field engineer. The outlet spacing was reduced to 91 m (300 ft) or less in the sag areas as required. The installed cost of the geocomposite pavement edgedrain system was $8.20 per linear meter ($2.50 per L.F.).

Conclusions and Recommendations

1- Prefabricated geocomposite pavement edgedrain systems are viable alternative to conventional pavement edgedrains.

2- Proper construction techniques are very important when using geocomposite pavement edgedrains.

3- Maintaining the verticality of the drain panel in the trench during the backfill operations by utilizing a drain panel- positioning wheel or plate is recommended.

4- Positioning the drain panel against the shoulder side or pavement side of the trench based on the prevailing trench wall conditions (stabilized versus unbound subbase) must be addressed before installation.

5- The support columns of symmetrical designed geocomposite edgedrains should be placed towards the trench wall with the flat back of the drain towards the backfill material.

6- Using an open-graded coarse aggregate as backfill for geocomposite pavement edgedrains enhances performance and reduces potential vertical and horizontal deformation of the drain panel during the backfill operations.

7- Proper spacing and installation of the drain outlets and outlet pipes enhances performance and reduce maintenance of geocomposite pavement edgedrains.

8- Having clear, concise specifications, and a pre-construction meeting, to outline the installation technique, are essential to the contractor, engineer, inspector, and the manufacturer to produce a well performing drainage system.

9- Periodic inspection of edgedrains using mini cameras and/or borescope should be made by the engineer for evaluation, and providing feedback to contractors .

10- Geocomposite pavement edgedrains are an economical alternative to conventional pavement edgedrain systems as well as economical insurance on pavement rehabilitation projects.

Acknowledgment

The authors would like to express their appreciation to David Simmons, Mike Jennings, and Cleo Hill with the Virginia Department of Transportation, and Steve Spaganle, and Margie Wells with Contech Construction Products, Inc. for their technical and editorial assistance.

References

[1] Koerner, R., Koerner, G., Fahim, A., and Wilson-Fahmy, R. "Long Term Performance of Geosynthetics in Drainage Applications," NCHRP Study 15-13, 1993.

[2] Koerner, R., "Designing with Geosynthetics," Prentice Hall Inc., 1990.

Testing

J. Jay Swihart[1]

Full-Scale Laboratory Testing of a Toe Drain with a Geotextile Sock

Reference: Swihart J. J., "**Full-Scale Laboratory Testing of a Toe Drain with a Geotextile Sock,**" *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390,* L. D. Suits, J. B. Goddard, and J. S. Baldwin, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: This paper describes the full-scale laboratory testing (pipe box testing) of a 380-mm-diameter, corrugated, polyethylene toe drain with a knitted geotextile sock, backfilled with a sand envelope material. The test results are compared with previous small-scale and full-scale pipe box tests using pipe with 3-mm and 6-mm perforations, but no geotextile. Use of the geotextile optimized toe drain performance both with respect to flow and with respect to loss of the sand envelope. The long-term flow rate was 260 liters per minute per linear meter of pipe, which was significantly higher (by a factor of 3 to 12) than the earlier tests without geotextile. The total loss of sand envelope was only 165 grams per linear meter of pipe, which was significantly lower (by a factor of 4 to 17) than the earlier tests without geotextile. The test with geotextile was run for 31 days at a constant head of 0.75 m above the pipe invert with no indication of clogging. Based on these results, use of geotextile sock in conjunction with a sand envelope is recommended for future toe drain installations in areas with fine-grained native soils.

Keywords: geotextile, sock, knitted, drainpipe, toe drain, perforations, testing, clogging, laboratory testing, flow, soil retention, envelope

Introduction

For drains in the downstream toe of dams (toe drains), Reclamation (U.S. Bureau of Reclamation) traditionally uses a gravel or sand envelope around corrugated polyethylene pipe with slotted or circular perforations. The perforations in the drainpipe must meet Reclamation's perforation criteria: perforation size $\leq \frac{1}{2}D_{85}$ [*1*]. This perforation criteria generally agrees with other published criteria [*2*]. However, previous tests [*3,4*] have shown that this perforation criteria (when used with a sand envelope) only addresses retention of envelope material, but not flow rate or clogging of the pipe perforations. Therefore, use of a geotextile sock was investigated with hopes of both increasing flow rates and decreasing loss of envelope material.

[1]Materials Engineer Technical Specialist PE, Materials Engineering Research Laboratory, US Bureau of Reclamation, D-8180, PO Box 25007, Denver, CO 80225.

Note that traditional 2-stage filters (consisting of a gravel filter surrounded by a sand filter) are sometimes used in areas with fine-grained native soils. However, installation of a traditional 2-stage filter is difficult and expensive because of the required bench-cut construction. Use of a geotextile as one stage of the 2-stage filter solves the constructability problem by allowing installation with traditional trenching equipment. Another option is to place the geotextile in the trench perimeter, but the geotextile is difficult to install in deep trenches and more likely to clog with native soil. By using a geotextile sock around the pipe, the geotextile and envelope material can both be custom designed for maximum compatibility (soil retention) and performance (flow rate).

Previous Testing

In 1997, a total of 650 linear meters of 460- and 600-mm toe drain were installed at Lake Alice Dam near Scottsbluff, Nebraska. Because of the silty native soils, a sand envelope (rather than gravel) was specified for soil retention. The pipe was specially perforated with 3-mm circular perforations to match the D_{85} of the sand envelope (perforation size = $\frac{1}{2}D_{85}$). However, within a few weeks, most of the 3-mm perforations had clogged with sand particles either plugging or blinding the pipe perforations. Small-scale laboratory tests [*5*] indicated that both 3-mm circular perforations and 3-mm slotted perforations would readily clog with the sand envelope.

To further explore this perforation clogging, a full-scale pipe box test apparatus was developed, and two full-scale pipe box tests were performed using the sand envelope with 3-mm and 6-mm perforations [*3,4*]. The perforation open area for both tests was 8500 mm^2 per linear meter. The first test with the 6-mm circular perforations was not stable and demonstrated a decreasing flow rate with time (indicating clogging). After 24 days, the flow rate had decreased by 24% to 80 liters per minute (Lpm) per linear meter, which was only about half the desired flow rate of 150 Lpm per meter. The 6-mm perforations also showed extensive loss of envelope material, losing 2800 grams of sand envelope per linear meter of pipe. The envelope loss was continuing to increase with time. The test results for the 6-mm circular perforations are shown in Figure 1.

The second test with 3-mm slotted perforations also demonstrated decreasing flow with time (again indicating clogging). After only 8 days, the flow rate had decreased by 43% to 22 Lpm per linear meter, which was far below the desired flow rate of 150 Lpm per meter. The 3-mm slotted perforations did demonstrate adequate retention of the sand envelope, stabilizing at 660 grams of lost envelope material per linear meter. The test results for the 3-mm slotted perforations are shown in Figure 2.

This paper describes a third full-scale pipe box test incorporating a knitted geotextile sock around the perforated drainpipe.

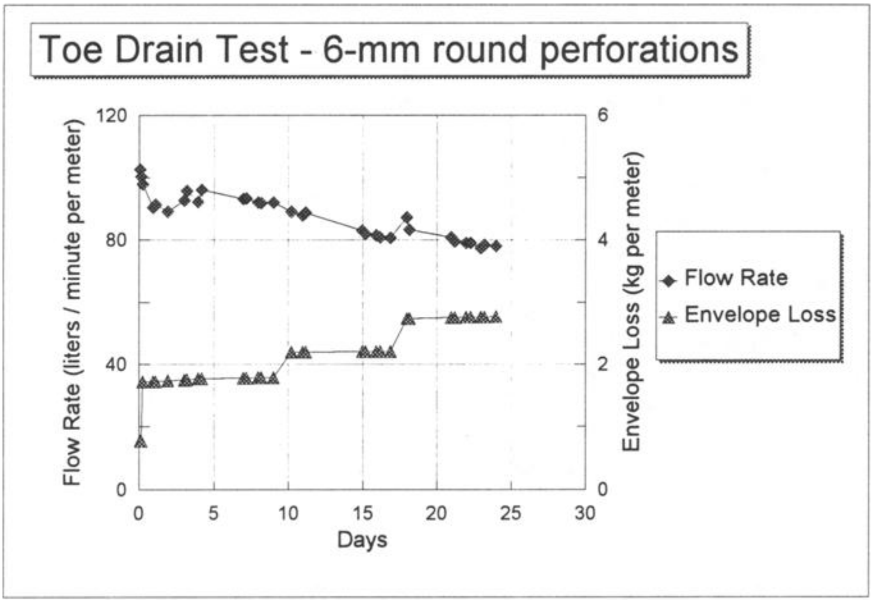


Figure 1 - *Previous Toe Drain Test with 6-mm Circular Perforations - No Geotextile.*

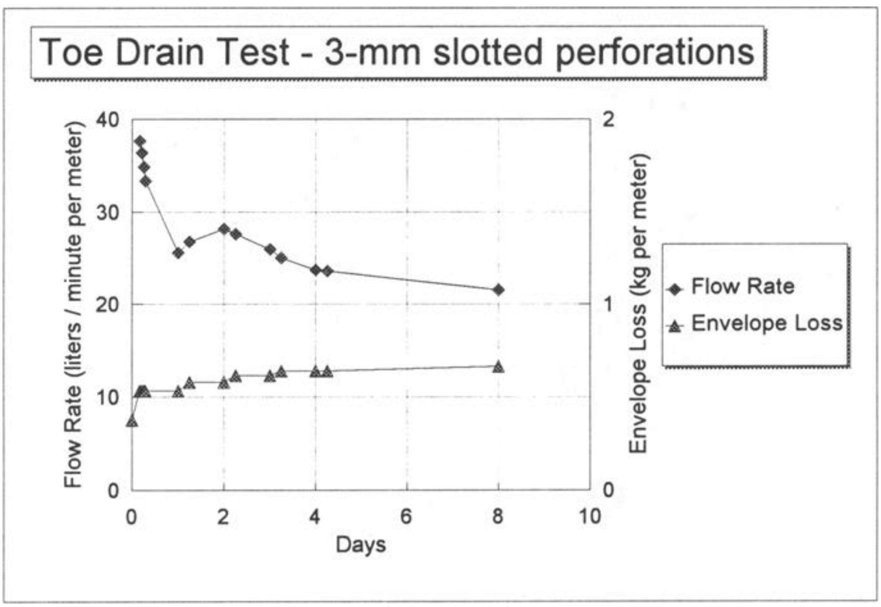


Figure 2 - *Previous Toe Drain Test with 3-mm Slotted Perforations - No Geotextile.*

Test Apparatus

The test apparatus (pipe box) used for all the full-scale toe drain tests is shown in Figures 3 and 4. The nominal box dimensions are 1.2- by 1.2- by 0.9-meters (height by width by length). The drainpipe measures 1.5-m long, and the length of pipe covered with geotextile inside the test box is 0.70 meters. As shown in the figures, water is pumped out of the sump in the floor drain, and into the 50-mm diameter PVC standpipes. The standpipes are connected to a PVC well screen network located around the bottom perimeter inside the box. The water flows out of the PVC well screen, upward through the sand envelope, through the geotextile, through the pipe perforations, and into the drainpipe. The lower half of one end of the drainpipe is blocked-off, forcing all the water to flow out the opposite end, where it is sieved for particle size analysis, and metered through an exit flowmeter. The outflow water is then returned to the floor drain for recirculation. The water level in the box is maintained at a constant head (0.75 m above the pipe invert) by overflows located on each corner of the box. Overflow water is again returned to the floor drain for recirculation.

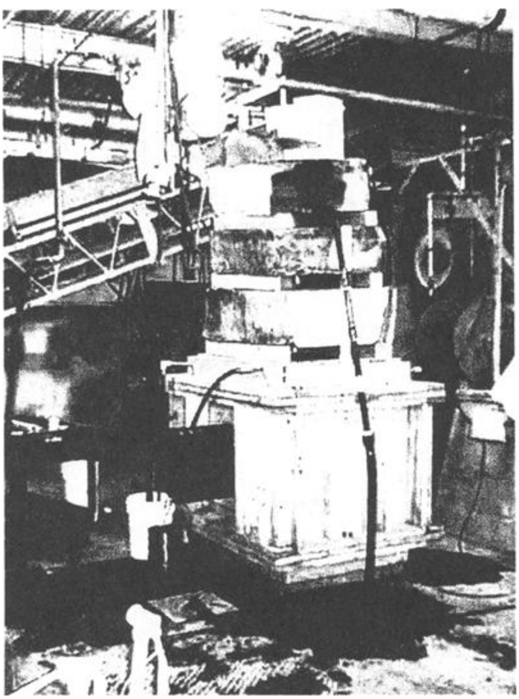


Figure 3 - *Pipe Box Test Apparatus.*

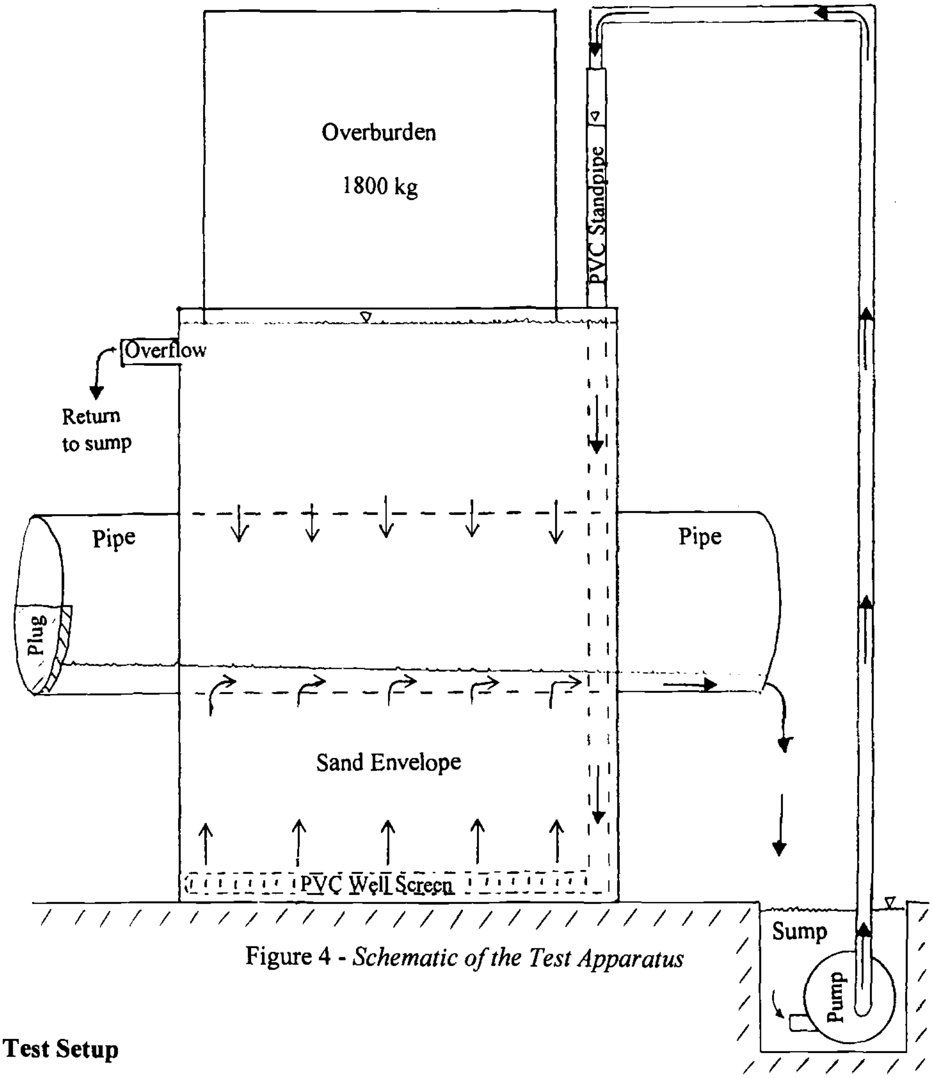


Figure 4 - *Schematic of the Test Apparatus*

Test Setup

A 380-mm-diameter dual-wall pipe with 8-mm perforations and 4500 mm^2 open area per linear meter (66 perforations in 0.70 linear meters) was selected for testing. The sock is a knitted polyester geotextile (100g/m^2) with AOS of 0.6 mm (#30 sieve), and is the only geotextile pre-installed on the pipe at the factory.

The required AOS of the geotextile to adequately retain the sand envelope was calculated by several methods as shown in table 1. All the methods indicate that the knitted geotextile sock with AOS of #30 sieve (O_{95} = 0.6 mm) should function adequately. The Federal Highway Administration (FHWA) method based on dynamic flow may be the most appropriate since the sand envelope is compacted by wet sluicing.

Table 1 - *Calculation of Minimum AOS (O_{95}) and Factor of Safety (FS).*

Method	Formula	Calculated O_{95} (mm)	FS for geotextile with 0.6 mm O_{95}
Giroud, 1982 [*6*]	$O_{95} \leq 13.5\ d_{50}/C_U$	3.6	6.0
Luettich et al, 1992 [*7*]	$O_{95} \leq 13.5\ d'_{50}/C'_U$	3.7	6.2
FHWA steady flow, 1985 [*8*]	$O_{95} \leq 8\ d_{85}/C_U$	7.7	12.8
FHWA dynamic flow, 1985 [*8*]	$O_{95} \leq 0.5\ d_{85}$	2.9	4.8

The pipe box was backfilled with the same sand envelope material used in the previous tests. The envelope gradation and the specification limits from Lake Alice Dam are shown in Figure 5. The sand envelope was placed in 150-mm lifts, and each lift was compacted by flooding with water at about 100 Lpm. Sand envelope that washed into the pipe through the geotextile was collected, weighed, and sieved. The box was backfilled flush to the top, and then loaded with 1800 kg of ballast to simulate a total earth cover of about 1.4 m over the pipe. As in the previous two tests, the important test criteria were adequate flow, plugging of geotextile (clogging), and excessive loss of envelope material.

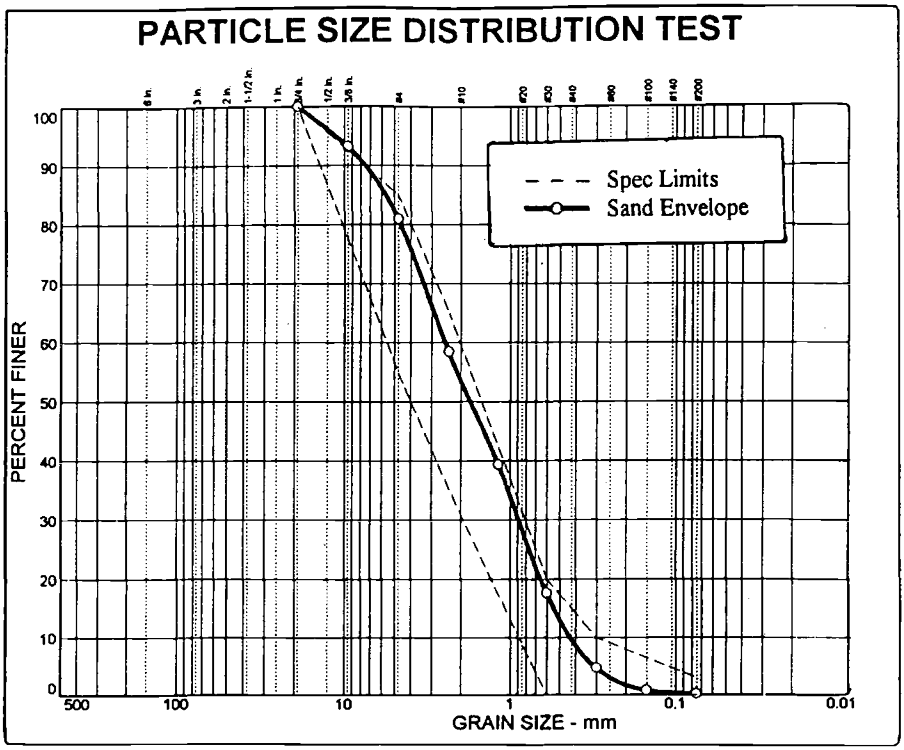


Figure 5 - *Gradation and Specification Limits of Sand Envelope.*

Outflow Data

All the testing (this test and the previous two tests) were performed under constant head of 0.75 m above the pipe invert. Two pumps were used to achieve a total inflow rate of about 300 Lpm per linear meter of pipe. The inflow was slowly ramped-up over a 3-day period, and then the inflow rate and head were maintained throughout the 31-day test duration. Outflow measurements were typically taken twice a day using a 50-mm flowmeter. In addition, the outflow was continuously sieved to collect any envelope material washing through the geotextile and into the pipe. Typical water flow through the geotextile and 8-mm circular perforations is shown in Figure 6. After peaking at about 290 Lpm per meter, the outflow stabilized at about 270 Lpm per linear meter with no indication of clogging. This flow rate is almost twice the desired flow rate of 150 Lpm per linear meter. The test results are shown in Figure 7.

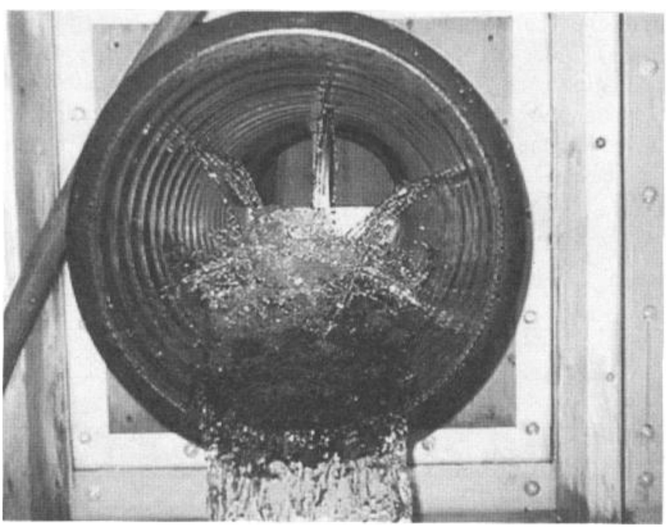


Figure 6 - *Protected by the geotextile, all the 8-mm diameter perforations are abundantly flowing water. Flow rate is approximately 270 Lpm per linear meter of drainpipe.*

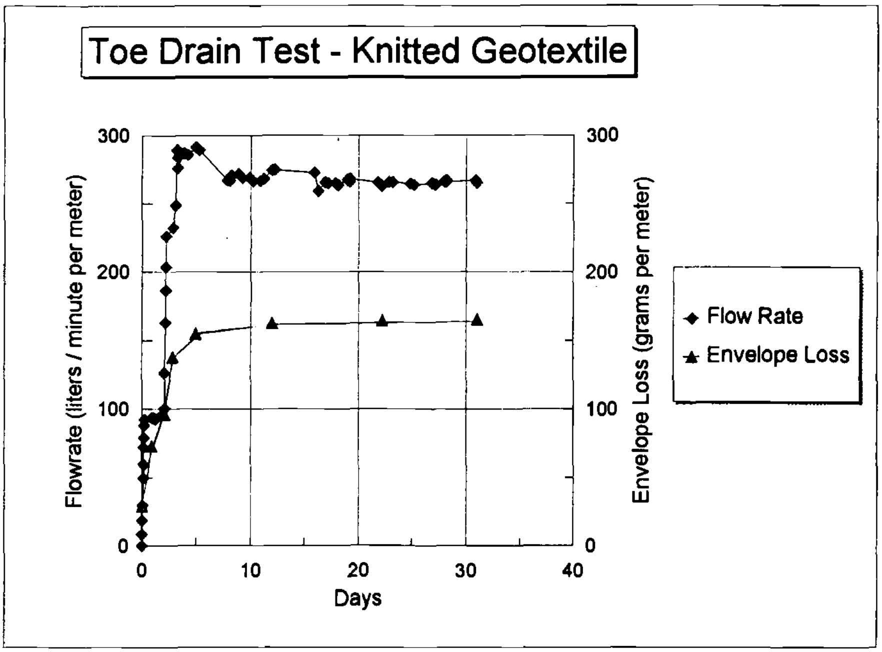


Figure 7 - *Toe Drain Test Using Knitted Geotextile Sock*

Discussion

Table 2 summarizes the results from all three pipe box tests. The first two tests demonstrate the classic trade-off between flow rate and envelope retention. To maximize flow (minimize clogging), large pipe perforations are required. Conversely to maximize envelope retention (minimize envelope loss), small perforations are required. The third test shows that by adding the geotextile sock, we are able to optimize performance by simultaneously increasing flow rate and decreasing loss of the sand envelope.

Table 2 - *Summary of Results from All Three Tests.*

Test Configuration	Flow Rate (Lpm per meter)	Sand Envelope Loss (grams / meter)	Test Duration and Comments
3-mm slots	22 Not Stable	660 Stable	8 Days - Not Stable
6-mm holes	80 Not Stable	2800 Not Stable	24 Days - Not Stable
Geotextile Sock	270 Stable	165 Stable	31 Days - Stable
Improvements with Geotextile (factor)	3 to 12	4 to 17	Stable

Several factors contribute to this optimized performance:

1. Improved retention of the sand envelope is achieved by using geotextile with small AOS (0.6 mm) compared to pipe perforations measuring 3 to 6 mm.

2. Improved flow rate is achieved by creation of a clean, unobstructed flow path through the pipe perforations.

3. Flow rate is further improved by a 100-fold increase in the effective open area from 4500 mm^2 per linear meter (perforation area) to about 600 000 mm^2 per linear meter (geotextile open area).

4. Perforation clogging is avoided because the geotextile removes the sand particles from the vicinity of the pipe perforations, preventing plugging or blinding of the perforations.

5. The increased open area also decreases the flow velocities at the geotextile/sand interface, which reduces the potential of particle movement and clogging of the geotextile.

Conclusions

1. Use of the geotextile sock in conjunction with the sand envelope optimized performance of the toe drain. Compared to previous tests without the geotextile, the flow rate increased by a factor of 3 to 12, while loss of the sand envelope decreased by a factor of 4 to 17. The flow rate was stable over the 31-day test, showing no indication of clogging of the geotextile when used with a sand envelope.

2. Use of a geotextile sock in conjunction with a sand envelope is recommended for future toe drain installations in areas with fine-grained native soils. Use of the geotextile sock will improve toe drain performance by increasing flow rates and decreasing loss of envelope material. Experience gained with the use of geotextiles in toe drains will also increase our ability to use geotextiles in other applications.

3. A traditional 2-stage filter (consisting of a gravel filter surrounded by a sand filter) is sometimes used in areas with fine-grained native soils. However, traditional 2-stage filters require expensive open-cut excavation to construct. Use of a geotextile sock as one stage of the 2-stage filter allows for toe drain construction with trenching equipment at significant cost savings.

4. The knitted geotextile sock used in this study is the only geotextile pre-installed on the pipe at the factory. The knitted sock is only available with AOS of 0.6 mm (#30 sieve) which limits design options. Other geotextile products (such as monofilament woven) would have to be attached to the pipe in the field, but are available in a wide range of AOS, which increases design flexibility.

Future Studies

1. The poor performance of the sand envelope at Lake Alice Dam was not anticipated. The dramatic improvement in performance with the addition of the geotextile sock was equally surprising. These results raise questions about whether similar improvements would be seen by using a geotextile sock with a traditional gravel envelope. Additional testing is required to evaluate the performance of a gravel envelope both with and without a geotextile sock. Installation stresses and damage will also be considered.

2. This testing program began with a small scale test apparatus based on the US Bureau of Reclamation "Soil Filter Test" (USBR 5630-89). However, the small-scale test did not model all the parameters, and the client requested full-scale tests to more accurately represent field conditions. Future studies will use the results from these full-scale tests to develop and calibrate a small-scale, less-expensive test. The small-scale test will then be used to evaluate other geotextiles and additional toe drain design parameters.

References

[1] U. S. Bureau of Reclamation, *Design Standards - Embankment Dams No. 13,* Chapter 5 Protective Filters, Denver CO, 1987, p 5.12.

[2] Vlotman, W. F., Willardson, L. S., and Dierickx, W., "Draft Report - Envelope Design for Subsurface Drains", International Institute for Land Reclamation and Improvement (ILRI), The Netherlands, March 1998, pp 127-132.

[3] Swihart, J. J. "Laboratory Pipe Box Tests on Toe Drains at Lake Alice Dam and Enders Dam," U.S. Bureau of Reclamation, Denver Technical Service Center (TSC), Materials Engineering Research Laboratory (MERL), Report No. MERL-97-04, Denver CO, December 1997.

[4] Swihart, J. J. "Laboratory Pipe Box Tests on Toe Drains at Lake Alice Dam and Enders Dam," *Filtration and Drainage in Geotechnical/Geoenvironmental Engineering, Geotechnical Special Publication No. 78*, ASCE, Boston MA, October 1998, pp 137-150.

[5] Szygielski, M. B. "Filter Testing on Lake Alice Dam Drain Material and Geomembrane, Lake Alice Dam, North Platte Project, Nebraska," U.S. Bureau of Reclamation, Denver Technical Service Center (TSC), Earth Sciences and Research Laboratory, Report No. 8340-97-34, Denver CO, November 20, 1997.

[6] Giroud, J.P., "Filter Criteria for Geotextiles", *Proceedings of 2nd International conference on Geotextiles*, Las Vegas, Nevada, Volume 1, pp. 103-109, August 1982.

[7] Luettich, S.M., Giroud, J.P., and Bachus, R.C., "Geotextile Filter Design Guide" *Journal of Geotextiles and Geomembranes*, Volume 11, No. 4-6, pp. 19-34, 1992.

[8] Federal Highway Administration, Christopher, B.R. and Holtz R.D., *Geotextile Engineering Manual,* Report No. FHWA-TS-86/203, Federal Highway Administration, Washington, DC, March 1985.

Soon-Hoe Chew,[1] Swee-Fong Wong,[2] Tann-Loo Teoh,[3] G.-P. Karunaratne,[4] and Siew-Ann Tan[4]

Influence of Test Apparatus on the Measurement of Transmissivity of Geosynthetic Drains

Reference: Chew, S.-H., Wong, S.-F., Teoh, T.-L., Karunaratne, G.-P., and Tan, S.-A., **"Influence of Test Apparatus on the Measurement of Transmissivity of Geosynthetic Drains,"** *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390*, L. D. Suits, J. B. Goddard, and J. S. Baldwin, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: The transmissivity of geosynthetic drains differs significantly under different test conditions. Test conditions can vary in terms of confining medium, the length of the specimen, and the direction of flow and properties of the fluid. This paper presents the findings on the determination of transmissivity of two prefabricated vertical drains and a geonet obtained using two different test apparatus. The first apparatus, in accordance with ASTM, comprises two stiff platens which press directly against a 300-mm-long specimen placed horizontally between two foam rubber layers. The second apparatus involves the use of compressed air which acts as the confining medium on a specimen enclosed in an elastic membrane. Flow of de-aired water for this setup is vertically upward. This paper seeks to compare objectively the differences in the performance of these two drainage testing apparatus for planar geosynthetic drainage products. A comparative study of the transmissivity of the prefabricated vertical drains and geonet under different test conditions is also presented.

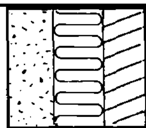
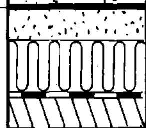
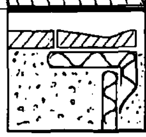
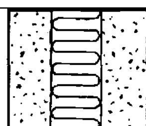
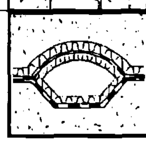
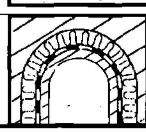
Keywords: Prefabricated vertical drains, geonet, transmissivity

[1]Assistant Professor, [2]Research Scholar, [3]Student, [4]Associate Professor
Department of Civil Engineering, National University of Singapore, 10, Kent Ridge Crescent, Singapore 119260.

Introduction

A multitude of geosynthetic products such as prefabricated vertical drains (PVD) and geonets is available to suit various subsurface drainage applications. Table 1 summarizes the main areas of application of drainage materials [*1*]. Attendant to this progress, it is imperative that appropriate means of determining the transmissivity or related performance criteria of the products be established. With the derivation of meaningful design parameters, engineers can then most effectively address any design problem.

Table 1 – *Main applications of drainage materials* [a]

Application	Adjacent constructions	Construction	Pressure (kPa)	Hydraulic gradient	Material orientation
Basements	Hard ↕ Flexible		< 100	1.0	Vertical
Roof gardens	Flexible ↕ Hard		< 20	0.03	Horizontal
Roads (Findrains)	Hard or Flexible ↕ Flexible		< 200	0.03	Horizontal
	Flexible ↕ Flexible		< 100	1.0	Vertical
Soil consolidation	Flexible ↕ Flexible		< 350	1.0	Vertical
Waste disposal	Hard or Flexible ↕ Hard or Flexible		Capping < 20 Bottom < 800	0 – 0.4 0 – 0.7	Horizontal to slope
Tunneling	Hard ↕ Hard or Flexible		< 200	0 1	Horizontal to vertical

[a] After Berkhout [*1*]

Much work had been done to investigate the factors influencing the testing of drainage materials to give realistic simulation of actual field conditions. Hansbo recommended that testing of vertical drains be carried out by placing the specimens in impervious soils [*2*]. The effect of specimen length, the confining pressure and the test duration were studied in detail by many institutions [*1-7*].

In addition, several attempts were made to develop an apparatus that can effectively reflect the in-situ conditions of the drainage material. Koerner summarized the findings on transmissivity values for geotextiles determined from different apparatus and test conditions [*3*]. The filtration characteristics of the geotextiles used in different PVDs and the discharge capacity were also investigated by using a modified triaxial cell [*4*]. The setup tested vertical flow and incorporated 10% kinking in the 200 mm long specimen. A modified oedometer test frame was also used to evaluate the discharge capacity of drains [*5*]. Ali and Karunaratne also evaluated the performance of several PVDs using two different horizontal setups which tested the drains under different forms of confining pressure [*6, 7*].

In general, the above works involved testing commercially available PVDs under unique test conditions. To date, no attempt has been made to systematically compare the performance of a set of drainage materials using two distinctly different test setups. This paper thus seeks to compare the differences in the transmissivity of four configurations of PVDs and a geonet determined from using two different apparatus – the ASTM constant head transmissivity test apparatus and the National University of Singapore (NUS) test apparatus. The research focuses on two of the applications of drainage materials, namely soil consolidation, where the direction of drainage is predominantly vertical, and the use of drainage materials in landfills and road pavements, where the drainage flow is predominantly horizontal. It is hoped that the findings presented can contribute towards establishing a single test apparatus that can be used to test different drainage materials under a range of test conditions appropriately.

Test Apparatus

ASTM Test Apparatus

The apparatus in study is in accordance with ASTM Test Method for Determining the (In-plane) Flow Rate per Unit Width and Hydraulic Transmissivity of a Geosynthetic Using a Constant Head" (D4716-95). Figure 1 shows a schematic sketch of the constant head transmissivity test apparatus fabricated in NUS. The constant head reservoir is maintained at a hydraulic gradient of 1.0. Pressure on the specimen (length 300 mm) is applied using a pneumatic piston. The piston presses on an aluminium block of surface area equivalent to the base holding the specimen.

For a typical setting, the specimen is placed between two layers of highly compressible closed-cell foam rubber. Each layer of foam rubber, which extends the entire width and length of the base, is 10 mm thick. To test for the transmissivity of the specimens in the presence of clay, two layers of clay at least 5 mm thick were placed immediately above and below the specimen. A series of tests was also conducted by removing the foam rubber, thus putting the specimen in direct contact with the smooth rigid base and the aluminium block. This arrangement investigates the transmissivity of specimens loaded between stiff platens.

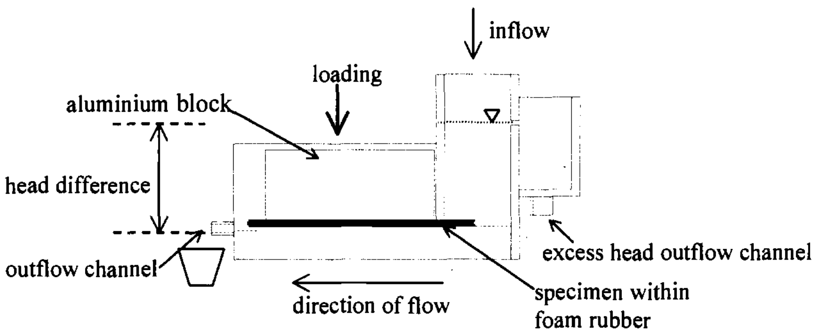


Figure 1 - *ASTM testing device*

NUS Test Apparatus

This new apparatus is a modified version of the first drain testing apparatus designed in NUS [*8,9*]. Figure 2 is a schematic diagram of the apparatus. The principal modification lies in making the specimen vertical instead of lying horizontally, and introducing upward flow through the specimen. Bearing in mind that the apparatus would be used to study the transmissivity of vertical drains, whereby the effect of gravity on the drain opposes the direction of flow, this modification thus gives a better simulation of the actual field condition.

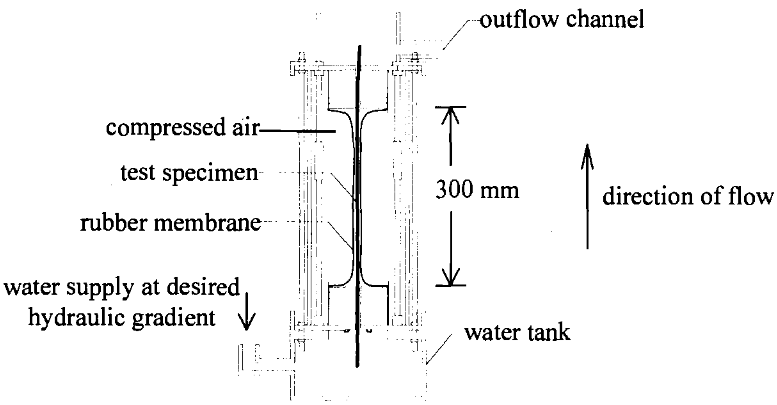


Figure 2 - *Schematic diagram of new transmissivity apparatus*

The new model comprises three sections: the constant head inlet water tank, outlet water tank and the transparent cylindrical compressed air chamber. It essentially retained all the advantages of the previous model but introduced the flexibility of testing specimens of different lengths in the vertical direction. To achieve an objective comparison between the two apparatus, the length of the specimen exposed to the compressed air in this setup is kept at 300 mm.

In a typical setting, the specimen is enveloped within a flexible rubber membrane to isolate the specimen from the compressed air within the cylindrical chamber. The rubber membrane has an approximate thickness of 0.8 mm, thus ensuring good compressibility under pressure. When testing in the presence of clay, a uniform layer of remoulded clay of at least 6 mm is placed in direct contact with the specimen within the rubber tube.

Test Materials

One geonet and four configurations of PVDs were tested using both apparatus. The geonet comprises a HDPE diamond-shaped net sandwiched between two non-woven, polypropylene, needle punched filters (Figure 3). The filter, comprising staple fibers, has an apparent opening size (AOS) equivalent to US sieve 80 and thickness of 100 mil. Two different sets of cores and filters were combined to form the four configurations of PVDs. Table 2 describes the components tested. Table 3 summarizes the core-filter combinations.

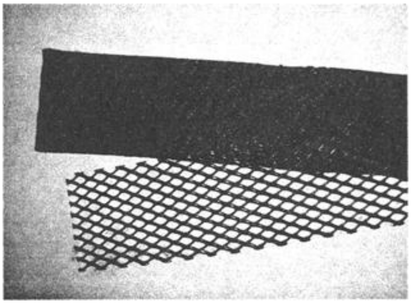


Figure 3 - *HDPE geonet with non-woven filter*

Table 2 – *Description of drain components*

Core type	**C1**: High density polyethylene (HDPE)	**C2**: High molecular polypropylene
Core structure	 Double cuspated continuous channel	 Corrugated continuous channel
Sleeve filter	**F1**: Polypropylene non-woven needle-punched (loose)	**F2**: Polypropylene non-woven heat bonded (tight)
Dimensions (width mm × thickness mm)	100 × 4	100 × 3.5

Table 3 – *Summary of core-filter combination*

	PVD 1	PVD 2	PVD 3	PVD 4
Core	C1	C1	C2	C2
Filter	F1	F2	F1	F2

Test Results

Comparison between Different Cores and Filters

A study was undertaken to investigate the performance of the four configurations of drain cores and filters using both the ASTM and NUS setups. Figure 4 summarizes the results.

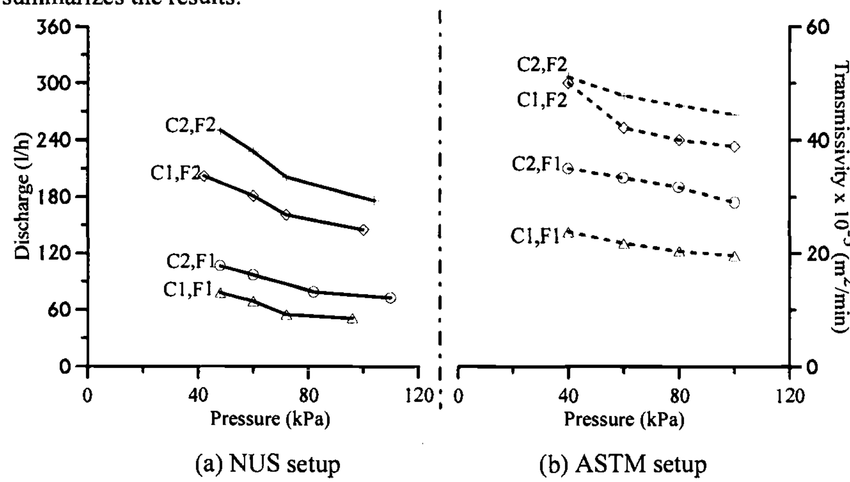


Figure 4 - *Comparison of different core and filter types for ASTM and NUS setups*

Clearly, filter F2 gives higher discharge than filter F1 regardless of core type or apparatus used. This can be attributed to the relative stiffness of F2. Under high pressure, the stiffer F2 is inhibited from 'pressing' into the core channels. On the other hand, given its flexibility, F1 can freely 'fold' into the grooves of the core channels. This in turn leads to a larger reduction in the effective cross-sectional area that permits the flow of water across the core channels (Figure 5).

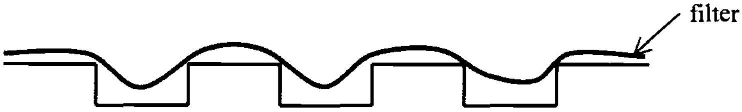


Figure 5 – *Core intrusion due to flexible filter*

For both apparatus, core C2 gives the higher discharge than C1 irrespective of the filter used. This suggests that C2 with rectangular continuous channels is more effective in transmitting water than C1, which comprises double cuspated continuous channels. A plausible explanation for this is that C2 has a higher effective cross-sectional drainage area than C1.

Comparison between NUS and ASTM Setups

Figure 6 and 7 summarizes the results obtained from testing PVD 4 and the geonet under various settings for both the NUS and ASTM setups respectively. Both figures illustrate the effect of varying degrees of core intrusions on the discharge of specimens. For specimens loaded directly under the stiff platen, the transmissivity measured are higher because the degree of core intrusion is minimal as compared to other loading conditions.

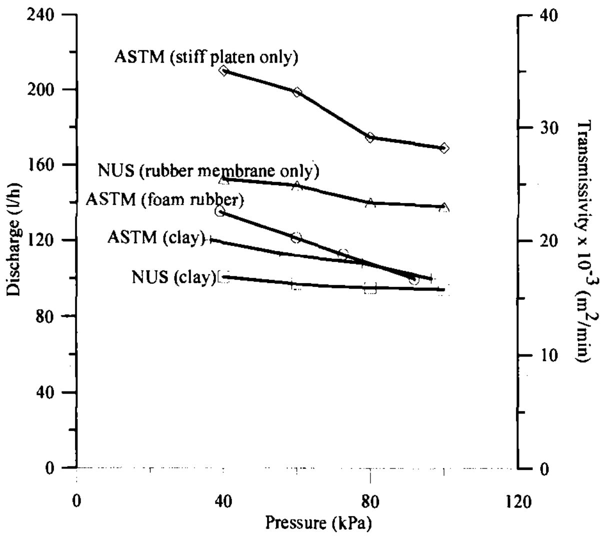


Figure 6 - *Transmissivity of PVD 4 for both NUS and ASTM setups*

Comparing between the effect of wrapping the specimen in a rubber membrane in the NUS setup against placing the specimen between foam rubber layers, the former gives a higher transmissivity. This suggests that the extent of core intrusion due to the rubber membrane is not as significant as the foam rubber because it is unable to conform to the core shape under pressure as easily as the foam rubber.

On the other hand, it was observed that the extent of core intrusion in a specimen is most significant when the specimen is wrapped in clay (Figure 8). This phenomenon is more pronounced in the NUS setup because unlike the ASTM setup, the specimen and clay are not enclosed in a restricted compartment, hence the clay is able to mould itself freely around the specimen under pressure. This freedom in movement enables the clay to achieve what is possibly the maximum degree of core intrusion.

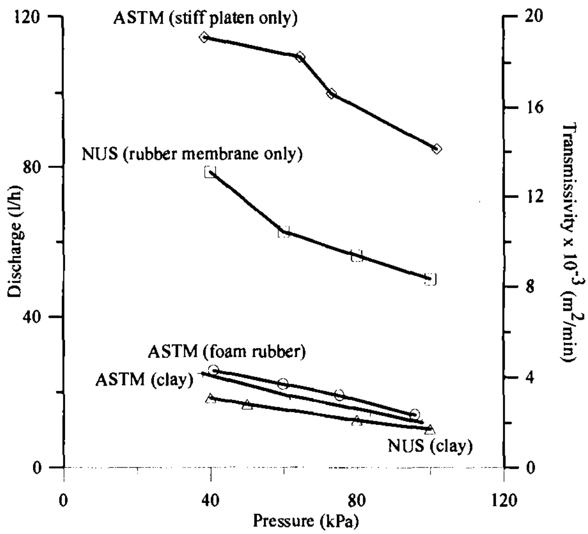


Figure 7 - *Transmissivity of geonet for both NUS and ASTM setups*

The results also revealed that under high pressure, the discharge measured for a specimen wrapped within foam rubber in the ASTM setup compares well with that of a specimen wrapped within clay in both the ASTM and NUS setups. This implies that under high pressure, the foam rubber chosen can effectively simulate the clay behaviour, and that the effect of gravity opposing flow is less significant.

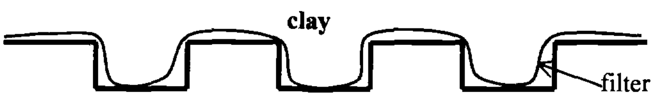


Figure 8 - *Severe core intrusion due to presence of clay*

Discussions

The results suggest that for different design applications, it is important to address the actual field conditions in which the prefabricated vertical drains or geonets would be used before deciding on an appropriate method of determining the transmissivity of the product.

The ASTM setup can realistically simulate the field conditions for geonets used to improve the drainage in applications such as slope failure problems and pavement under-drains. In these applications, the direction of flow of fluid is predominantly horizontal. On the other hand, in the case of vertical drains and behind retaining walls, the NUS setup would give a better simulation because the fluid flows vertically upwards or downwards in the field. In addition, it is necessary to introduce clay on either one or both sides of the specimen as in the application of geonets behind retaining walls.

In general, designing with the transmissivity obtained from the ASTM setup with the specimen loaded directly under a stiff platen would constitute a less than conservative estimate because the tests with stiff platen always produced higher discharge capacities. On the other hand, it is not economically viable to always adopt the transmissivity determined by packing clay around the specimen using either the ASTM or NUS setups. Hence, a more practical general approach would be to test the specimen in a rubber membrane in the NUS setup.

In view of the fact that the in-situ lateral pressure acting on vertical drains is not constant with increasing depth, it is also recommended that the design criteria take into consideration the transmissivity of the drains under different pressure at different depths operating in a given application.

Conclusions

Evaluation of Different Core-Filter Combinations

It had been shown that the flexibility of the filter and the core structure can significantly affect the discharge attainable in any combination of prefabricated vertical drains. In essence, the transmissivity is largely influenced by the effective cross-sectional drainage area of the drain. This in turn is influenced by the extent of core intrusion under lateral pressure. The results indicated that the degree of core intrusion is less significant when a stiff filter is used.

Evaluation of Transmissivity of PVDs and Geonet using NUS and ASTM Setups

. Comparing the five different test conditions, the transmissivity values of the PVDs and the geonet were the lowest when tested with clay using the NUS setup. In contrast, the transmissivity values obtained from tests using the ASTM setup with stiff platen only were the least conservative. Hence, it is essential to address the actual site conditions and select the appropriate test conditions in order to determine a reasonable design value.

References

[*1*] Berkhout, H. C., 1994, "In-Plane Permeability Measurement of Geosynthetic Drains Used in Civil Engineering and the Construction Industry", *Geosynthetics World*, Vol. 4, No. 5, pp. 4-6.

[*2*] Hansbo, S., 1983, "How to Evaluate the Properties of Prefabricated Drains", *Proceedings VIIIth ECSMFE*, Helsinki, Vol. 2, pp. 621-626.

[3] Koerner, R. M., and Bove, J. A., 1983, "In-plane Hydraulic Properties of Geotextiles", *Geotechnical Testing Journal*, GTJODJ, Vol. 6, No. 4, pp. 190-195.

[4] Chang, D. T. T., Liao, J. C., and Lai, S. P., 1994, "Laboratory Study of Vertical Drains for a Ground Improvement Project in Taipei", *Proceedings of Fifth International Conference on Geotextile, Geomembrane, and Related Products*, Singapore, Vol. 2, pp. 807-812.

[5] Broms, B. B., Chu, J., and Choa, V., 1994, "Measuring the Discharge Capacity of Band Drains by a New Drain Tester", *Proceedings of Fifth International Conference on Geotextile, Geomembrane, and Related Products*, Singapore, Vol. 2, pp. 803-806.

[6] Ali, F. H., and Huat, B. K., 1992, "The Influence of Filter Wrapping on the Flow Capacity of a Prefabricated Drain", *Proceedings of International Symposium on Applications of Geosynthetics Technology*, Jakarta, pp. 213-218.

[7] Karunaratne, G. P., Tan, S. A., Chew, S. H., and Loh, S. L., 1998, "Critical Evaluation of Laboratory Drain Tests Simulating Field Conditions", *Proceedings of Sixth International Conference on Geosynthetics*, Atlanta, Vol. 2, pp. 855-858.

[8] Loh, S. L., 1996, *Vertical Drain Capacity Test*, B.Eng Thesis, National University of Singapore.

[9] Teoh, T. L., 1999, *Analysis and Testing of Vertical Drains*, B.Eng Thesis, National University of Singapore.

Dave Ta-Teh Chang,[1] Chiwan Hsieh,[2] Shih-Yueh Chen,[3] and You-Quan Chen[3]

Review Clogging Behavior by the Modified Gradient Ratio Test Device with Implanted Piezometers

Reference: Chang, D. T.-T., Hsieh, C., Chen, S.-Y., and Chen, Y.-Q., "**Review Clogging Behavior by the Modified Gradient Ratio Test Device with Implanted Piezometers,**" *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390*, L. D. Suits, J. B. Goddard, and J. S. Baldwin, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: A modified implanted Gradient Ratio (GR) test system was proposed in the study. In comparison with the conventional GR test device, all the piezometers are implanted into the soil specimen, and an additional piezometer is installed at soil-geotextile interface, in order to precisely measure the pore pressure variation of the soil-geotextile system. By using the modified GR test apparatus, 120 tests were conducted to study the clogging behavior of soil-geotextile system. Four types of needle-punched nonwoven geotextiles and five gap-graded soil mixtures were used in the program. These soils are the mixture of the Ottawa sand and various percentages of weathered mudstone. Three hydraulic gradients were used in the GR tests. The results of the study indicated that the modified implanted GR test system is able to provide pore pressure head measurement within the test specimen. In general, GR values obtained from the implanted GR test system are greater than those obtained from the conventional GR tests.

Keywords: piezometer, geotextile, mudstone, clogging, gradient ratio

Introduction

To obtain the best drainage and filtration performance, geotextiles should include the following characteristics [*1, 2, 3*]: (1) satisfy soil retention, (2) have sufficient permeability, (3) clogging resistance, and (4) durability. However, the effectiveness of the geotextile is mainly dependant upon its clogging potential . At present, the Gradient Ratio (GR) test [*4*] and Hydraulic Conductivity Ratio (HCR) test [*5, 6*] are

[1]Professor, Department of Civil Engineering, Chung Yuan University, Chung Li, Taiwan, R.O.C.
[2]Associate Professor, National Pingtung University of Science and Technology, Pingtung, Taiwan, R.O.C.
[3]Graduated Students, Dept. of Civil Engineering, Chung Yuan University, Chung Li, Taiwan, R.O.C.

the most common methods to evaluate the long-term flow compatibility and clogging potential of geotextiles. Because the GR test is simple to use and less time consuming than the HCR test, it is the most common method for evaluating the clogging potential of geotextiles. However, there are several disadvantages in the current GR test method [*7, 8, 9*].

The objectives of the study are to refer to the modified principle presented in Chang and Neih [*9*] study, to propose a revised test device, and perform a series of GR tests using that device. Based upon the test results, a comparison analysis was the difference between the conventional and modified GR test systems. Furthermore, the criterion for determining the clogging resistance capability of geotextile was also revised.

Background Review

Filtration and Drainage

Rollin, et al. [*10*] reported that when placing a geotextile into a soil as a filter soil, water would carry particles through the geotextile and may cause soil blinding as well as geotextile clogging and blocking. These phenomena would block the flow path and affect the drainage capacity of the soil-geotextile system. In order to maintain the geotextile long-term drainage/filtration capacity, the geotextile opening size and particle distribution of surrounding soil should be compatible with each other. If a geotextile incompatible with the soil is used, it will cause the loss of soil particles or clogging within the geotextile and result in the failure and close up of the soil-geotextile system. In general, bridge network formation and vault network formation are the most common mechanisms to create soil/geotextile system as a stable filtration system [*10*].

Clogging Resistance Criterion

In 1982, Haliburton and Wood [*11*] used a mixture of a silt and Ottawa sand to conduct a series of GR tests to simulate a worse-case of soil-geotextile filtration condition. The results of the study indicated that the GR value increases rapidly as silt content increases, which indicates that the clogging resistance of the soil-geotextile system also decreases rapidly. Therefore, the U.S. Army Corps of Engineers defined GR values as less than and equal to 3 as the criterion for acceptance of the clogging resistance capability of the geotextile [*11*]. Williams and Luettich [*6*] reported that the geotextile filtration and drainage design generally follows a principle similar to the conventional granular filter design. They also mentioned that the soil, geotextile properties, and soil-geotextile interface behavior were not considered in the design. Therefore, a larger factor of safety was commonly used in the design.

Gradient Ratio Test

The original GR test device was developed by Calhoun [*4*]. The GR is defined as the ratio of the hydraulic gradient between the soil and the soil-geotextile system above the geotextile. Method of Measuring the Soil-Geotextile System Clogging Potential (By the Gradient Ratio) (ASTM D5101 - 90) is the most common test method for measuring the soil-geotextile system permeability and clogging potential. The test method requires setting up a cylindrical clear plastic permeameter with a geotextile and soil, and passing water through this system by applying various differential heads. Measurements of differential heads and flow rates are taken at different time intervals to determine hydraulic gradients. The schematic view of the geotextile permeameter is shown in Figure 1(a). The GR of the soil-geotextile system can be calculated by the following formula. Figure 1(b) shows the definition of each terms for GR calculation formula. The expression of this formula is given below.

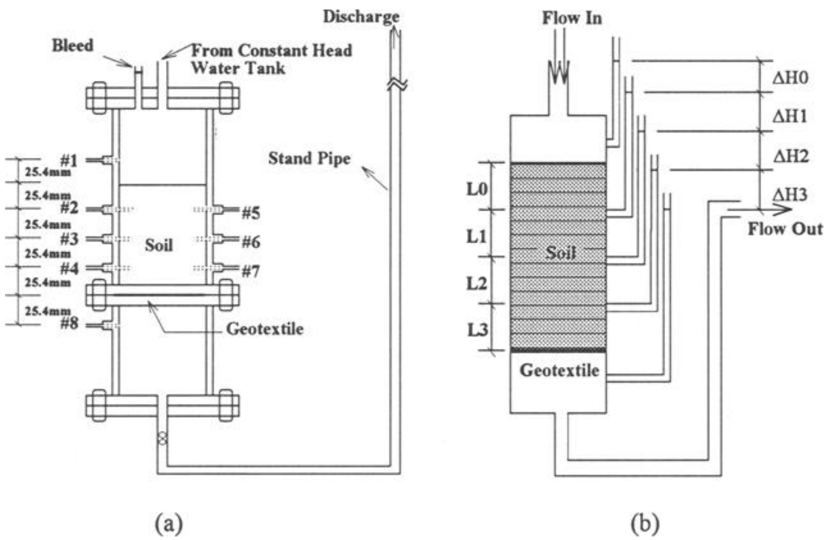


Figure 1 - *(a)Layout of GR Device, (b)Definitions of all terms for GR Calculation formula*

$$GR = \frac{\dfrac{\Delta H3}{L3}}{\dfrac{(\Delta H1 + \Delta H2)}{(L1 + L2)}} \tag{1}$$

Modified Device with Implanted Piezometer

Chang and Neih [*9*] indicated that the current GR device is unable to clearly identify geotextile clogging conditions. In addition, improper preparation of the soil specimen would cause unexpected soil particles movement, geotextile clogging, and improper pore water measurement during testing. Any air bulbs trapped within the manometer plastic tubing could not be removed easily and would affect the water pressure head measurement. Therefore, a modified GR test system was developed herein.

The most important revisions of the system included seven implant piezometers. From Figure 2, eight piezometers (#1~#8) are located at the same position as the original setup, and an additional piezometer (#9) is placed at the location right on the top of the geotextile specimen. The piezometer tips are inserted into the soil specimen 10.0 mm from the edge of the permeameter in order to eliminate any disturbance of filtration flow within the soil specimen. The installation of the piezometer would assist in the variation of the pressure head for the soil-geotextile system.

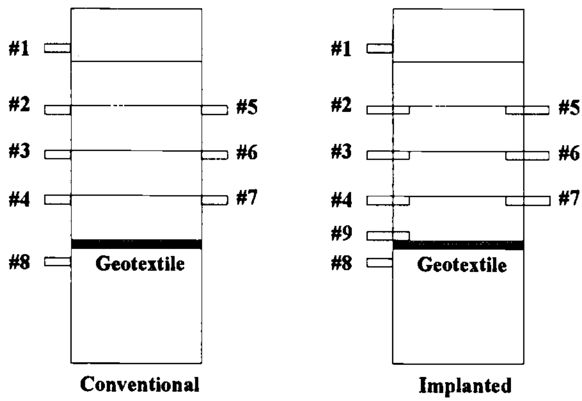


Figure 2 - *Comparison of Conventional and Implanted GR Test Set-up*

Performance Testing Program

A series of GR tests were performed to verify the performance of the implanted GR test system. Four different types of geotextiles, five gap-graded soils, and three hydraulic gradients (1, 5, and 10) were used in the performance tests. Totally, 120 different test conditions were tested in the program. The GR test condition is classified, based upon two variables, which include the geotextile type and the percentage of weathered mudstone contained in the mixture. For example, the GR test denoted as the A-10 test is associated with the test using geotextile A and 10% of weathered mudstone in the mixture. For implanted GR tests are marked with (I).

Test Materials

Since the fine particles of a gap-graded mixture are relatively easily to carry with filtration flow, five lab-made gap-graded soil mixtures were used in the test program to simulate the worse-case filtration/drainage condition within a soil-geotextile system. The test soils are the mixture of the Ottawa sand (C-190) with various percentages of the weathered mudstone. The mudstone is obtained from the southwest region of Taiwan, it contains more than 95% of fine grain soil (passing #200 sieve) and is classified as CL/ML. The physical properties of the weathered mudstone are listed in Table 1. The percentages of mudstone used in the mixture are 10%, 20%, 30%, 40%, and 100%. The gradation curves of the test soils are shown in Figure 3.

In order to identify the test geotextiles, four test needle-punched polyester nonwoven geotextiles were denoted as A (250g/m^2), B (350g/m^2), C (450g/m^2), and D (500g/m^2). The physical properties are summarized in Table 2. As shown in the table, the strength and mass per unit area of the geotextiles covers quite a wide range; however, the permeabilities of the test geotextiles are almost the same. For these needle-punched nonwoven geotextile, too thick to measure the Apparent Opening Sizes (AOS) and were not pressented in the table.

It is known that the viscosity of the test water is a function of several variables and would affect GR test results. These variables include atmosphere pressure, temperature, specific gravity, and dissolved oxygen content. Therefore, the GR performance tests were controlled at 1.0 atmospheric pressure and the test results would also be corrected to 20^{0}C by the correction factor (R_t) of water. The tested water was deaired through a double-activated carbon filtration system. The dissolved oxygen content of the tested water is in the range of 6 to 8 ppm.

Table 1 - *Physical Properties of the Weathered Mudstone*

Physical Properties	Test Results
Specific Gravity (Gs)	2.71
Liquid Limit (LL)	36.4
Plastic Limit (PL)	22.8
Plastic Index (PI)	13.6
Unified Classification (USCS)	CL
Permeability (cm/sec)	1.58×10^{-6}

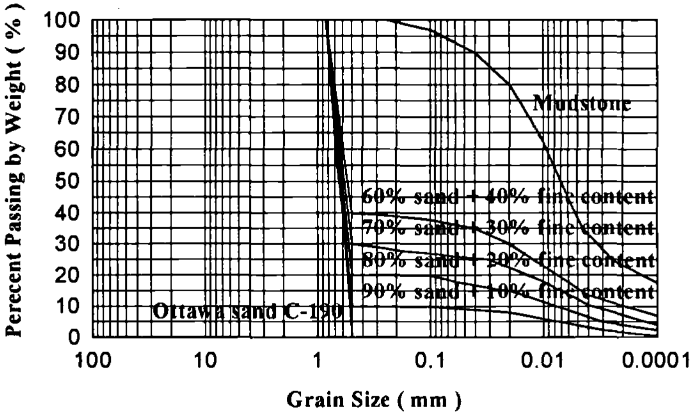


Figure 3 - *Grain Size Distribution for the Soil Mixtures*

Table 2 - *Basic Properties of Test Nonwoven Geotextiles*

Nonwoven Type	A-250	B-350	C-450	D-500
Thickness (cm)	0.278	0.352	0.473	0.553
Unit Weight (g/cm^2)	330.1	362.6	496.5	537.3
Permeability (cm/sec)	3.0×10^{-1}	4.0×10^{-1}	3.2×10^{-1}	4.7×10^{-1}
Tear Strength (kgf)	44.8	44.2	67.1	75.6
Grab Strength (kgf)	82.0	90.2	126.7	137.4
Elongation (%)	93.2	88.3	85.8	84.1
AOS	Not Available	Not Available	Not Available	Not Available

Specimen Preparation

The test soils were either fine-grained or medium Ottawa C-190 sand (#20-#40). When the ASTM D5101 procedure (dry pulverization sample preparation) to prepare the test sample was followed, the separation of fine and coarse particles was clearly observed in the specimen preparation. Therefore, a revised specimen preparation procedure was used in the test program. First, the piezometers were placed in the specified locations, ensuring that the pressure tips were 10.0 mm inside the permeameter. Then the well-mixed soil was placed one spoon at a time (approximate 30 cm^3) into the permeameter. The placement is divided into three layers, and the specimen height is about 11.0 cm. The rest of the preparation procedure is the same as that specified in the ASTM D5101 method. It was found that the use of this procedure significantly eliminated soil particle separation.

Results and Analysis

One hundred twenty GR performance tests were performed. The variation of pressure head within the soil specimen, the location of clogging, and the relation between clogging behavior and flow rate were analyzed and discussed herein.

Variation of Pressure Head and Clogging Location

An implanted additonal piezometer (#9) is placed at the soil-geotextile interface in order to investigate the variation of hydraulic properties of the test specimen at the location near the soil-geotextile interface. The typical GR values, the average hydraulic gradient for soil specimen (i_s) and soil-geotextile system (i_{sg}) are summarized in Table 3. In which i_s is defined as the hydraulic gradients for 25.4 mm to 101.6 mm above the geotextile, and i_{sg} is determined within 25.4 mm above geotextile and geotextile itself. More details about these definitions are given in Figure 4. Symbols in Figure 4 are defined as below.

M1~M9 : #1~#9 piezometers water head reading (mm).
L0~L2 : Thickness of soil zone "0" (S0) to zone "2" (S2), (76.2mm).
L3 : 25.4mm.
Tg : Thickness of geotextile.
ΔH0 = M1-M2, head loss through S0 zone (mm).
ΔH1 = M2-M3, head loss through S1 zone (mm).
ΔH2 = M3-M2, head loss through S2 zone (mm).
ΔH3 = M4-M9, head loss through S3 zone (mm).
ΔHst = ΔH0+ΔH1+ΔH2+ΔH3, head loss through S0, S1, S2, and S3 zones (mm).
ΔHg = M9-M8, head loss through geotextile (Sg) (mm).
ΔH = Total head loss = (ΔH0+ΔH1+ΔH2+ΔH3+ΔHg) =ΔHst+ΔHg (mm)
i_{S0} = ΔH0/L0, gradient within S0.
i_{S1} = ΔH1/L1, gradient within S1.
i_{S2} = ΔH2/L2, gradient within S2.
i_{sg} = (ΔH3+ΔHg) /(L3+ Tg), gradient within S3 and Sg.
i_g = ΔHg/Tg, gadient within Sg.
i_s = (ΔH0+ΔH1+ΔH2)/ (L0+L1+L2), gradient within S0, S1, and S2 zones.
i_{st} = ΔHst/(L0+L1+L2+L3), gradient within S0, S1, S2, and S3 zones.
i_{sys} = ΔH/(L0+L1+L2+L3+ Tg) ; system gradient.

According to the ASTM D5101, the GR value obtained from the GR test can be used to identify the clogging condition of a soil-geotextile system. If the GR value is equal to 1.0, it implies that the geotextile has similar filtration capability as the soil. If the GR value is less than 1.0, it indicates that the upstream fine grain soil particles are carried away through the opening of the geotextile. Moreover, if the GR value is greater than 1.0, it implies that the soil-geotextile system is clogged or blinded. If the geotextile is treated as an equivalent soil layer, the soil-geotextile system of the GR test can be divided into 5 layers, S0, S1, S2, S3, and Sg (Figure 4). Based upon the ASTM definition of clogging, if the hydraulic gradient of a soil layer is greater than that of the subsequent bottom soil layer, it implies that the upper soil layer is clogged. Since the similar findings are involved from 120 GR tests, only typical representatives, 20% and 40% mixtures are summarized in the following section.

1. Clogging occurs within S0 layer: Figure 5 shows the GR test results of geotextile A with 20% of mudstone mixture. The results of the GR test associated with a hydraulic gradient of 5 shows that the gradient slope of S0 soil layer (0 to 25.4 mm) is significantly greater than that of S1 soil layer (25.4 to 50.8 mm).
2. Clogging occurs within the S1 layer: As shown in Figure 5 the results of the GR test with a hydraulic gradient of 10, the gradient slope of S1 soil layer (25.4 to 50.4 mm) is greater than that of S2 soil layer (50.8 to 76.2 mm).
3. Clogging occurs within S2 layer: Figure 6 shows the GR test results of geotextile B with a mixture of 40% of mudstone. As shown in the figure, the curve associated with the hydraulic gradient of 5 shows the gradient slope of S2 soil layer (50.4 to 76.2 mm) is significantly greater than that of S3 soil layer (76.2 to 101.6 mm).

4. Clogging occurs in S3 soil layer: Figure 7 shows the GR test results of geotextile C with a mixture of 40% of mudstone. The gradient curve associated with the hydraulic gradient of 10 shows that the gradient slope of S3 soil layer is significantly greater than that for Sg (geotextile specimen).
5. Clogging occurs at Sg geotextile layer: As shown in Figure 6 for the GR curve associated with the hydraulic gradient of 10, the gradient slope for the Sg layer is significantly greater than that of the S3 layer.

Based upon the results of these 120 test cases, it was found that clogging occurring within S3 layer is more than that occurring within the other soil layers. Moreover, the Sg layer is the second highest location for clogging.

Table 3 – *Conventional and Implanted Results of GR* 、 *i_s and i_{sg}*

Test	Implanted Type			Conventional Type		
Condition	GR	i_{sg}	i_s	GR	i_{sg}	i_s
A-10	1.19	7.52	6.32	0.78	7.07	8.81
A-20	1.11	8.71	7.64	2.06	10.64	4.96
A-30	1.13	12.57	10.84	8.02	19.32	2.34
A-40	2.32	16.30	6.84	1.45	10.86	7.30
A-100	4.21	21.64	5.01	3.15	17.88	5.52
B-10	0.96	6.13	7.36	0.76	6.78	8.96
B-20	0.83	9.27	7.01	0.70	6.37	8.98
B-30	2.44	8.41	8.43	1.87	14.67	7.82
B-40	2.67	24.25	3.95	1.79	14.30	8.02
B-100	7.56	24.28	4.52	5.29	29.34	1.92
C-10	0.83	6.13	7.36	0.58	5.16	8.88
C-20	1.31	9.27	7.01	1.04	9.31	9.01
C-30	0.99	8.41	8.43	2.13	5.17	2.42
C-40	6.14	24.25	3.95	4.67	19.39	4.15
C-100	5.36	24.28	4.52	2.85	20.45	7.18
D-10	1.12	6.39	5.74	0.75	4.82	6.45
D-20	1.04	6.63	6.37	0.83	5.20	6.25
D-30	1.08	9.02	8.31	0.39	3.79	9.83
D-40	2.15	11.81	5.48	1.57	13.46	8.57
D-100	5.47	23.02	4.21	3.75	21.72	5.78

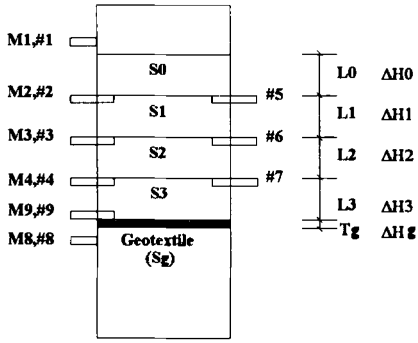
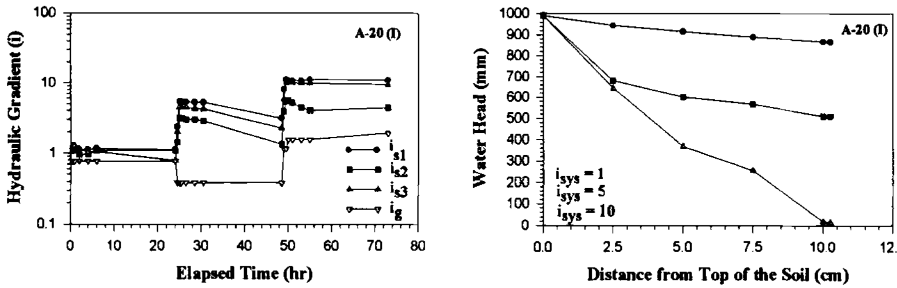
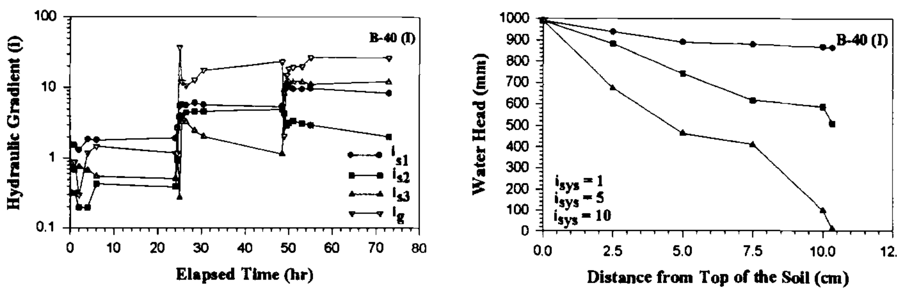


Figure 4 -Detailed *Definitions for Conventional and Implanted System*



(a) (b)

Figure 5 - *Geotextile A with a Mixture of 20% of Mudstone. (a) Hydraulic Gradient Variation within Soil Specimen, (b) Pressure Head Distribution within Soil Specimen at 24 hrs*



(a) (b)

Figure 6 - *Geotextile B with a Mixture of 40% of Mudstone. (a) Hydraulic Gradient Variation within Soil Specimen, (b) Pressure Head Distribution within Soil Specimen at 24 hrs*

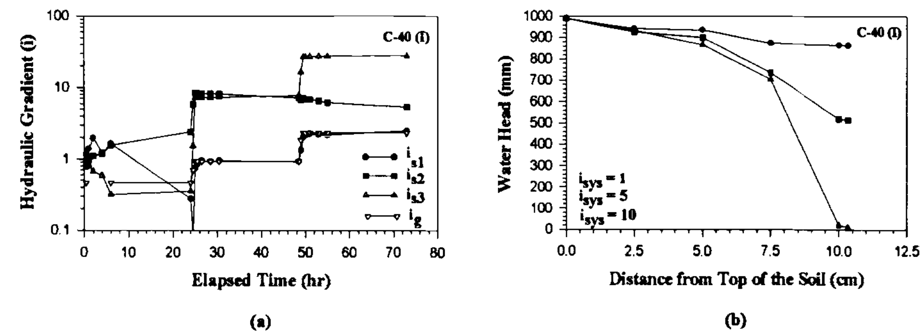


Figure 7 – *Geotextile C with a Mixture of 40% of Mudstone. (a) Hydraulic Gradient Variation within Soil Specimen, (b) Pressure Head Distribution within Soil Specimen at 24 hrs*

System Flow Rate and Clogging Behavior

The system flow rate of a GR test represents the discharge flow rate within a unit time interval. Typically, cc/sec is the most common unit used for system flow rate. Figures 8 and 9 show the typical system flow rates for the GR tests using a conventional GR device and the implanted GR test system. As shown, the system flow rate will increase as the hydraulic gradient ratio increases for the GR tests using both systems. In addition, if the hydraulic gradient changes rapidly at any location, the system flow rate at the location would also increase. However, the system flow rates for the GR tests using the conventional and implanted GR test systems are almost the same, but for GR values, the implanted system provided higher levels. These findings are the conclusion from most of the test results.

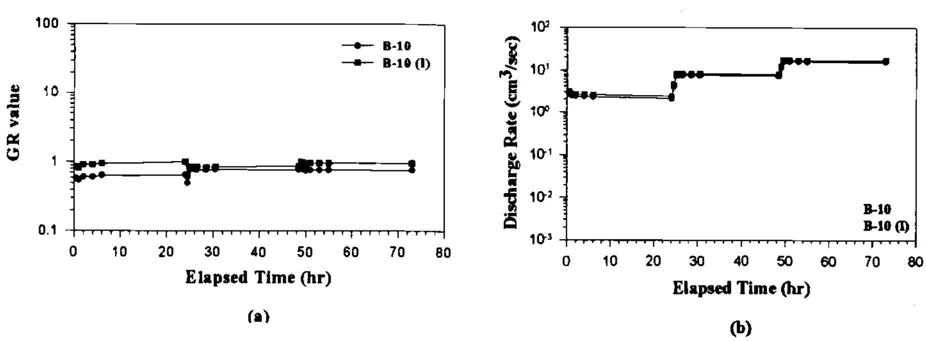


Figure 8 - *Comparision with (a) GR Value, (b) Discharge Rate, for Conventional and Implanted Systems. (Geotextile B with 10% of Mudstone)*

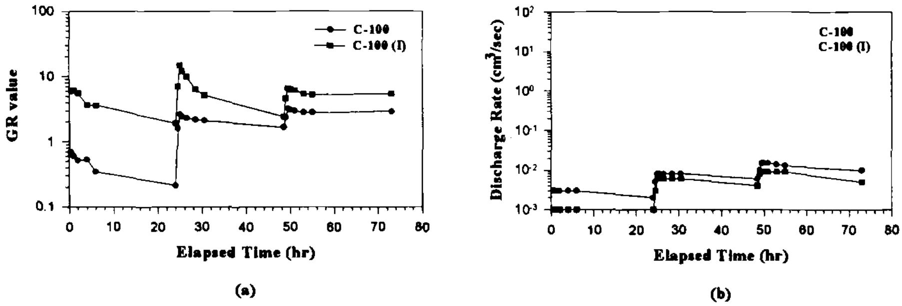


Figure 9 - *Compare with (a) GR Value, (b) Discharge Rate, for Conventional and Implanted Systems. (Geotextile C with 100% of Mudstone)*

Gradient Ratio Evaluation Program

Through this study, the criterion for evaluating the clogging conditions can be discussed. Based upon the principle of GR, a modified "GR_g" value for geotextile is proposed.

$$GR_g = i_g / i_{st} \tag{2}$$

$i_g = \Delta Hg/Tg$, the gradient within the geotextile (Sg).

$i_{st} = \Delta Hst/(L0+L1+L2+L3)$, the gradient for entire soil specimen (S0, S1, S2, and S3 zones).

According to the definition of GR_g, for GR_g=1, no clogging of geotextile occurs; and vis-a-vis for $GR_g > 1$. Based upon the results of the test program, soil blinding is an inevitable phenomenon in a soil-geotextile filtration system. Unfortunately, research studies related to soil blinding (sediment formation) are very limited. Up to now, the reasons for the development of soil blinding caused by soil itself or geotextile is still unclear to us. Therefore, a conservative assumption is made. It is assumed that soil blinding within a soil-geotextile system is mainly cause by the presence of a geotextile. As mentioned earlier, soil sediment mainly occurs within the S3 layer. The modified GR_s value follows a similar definition as that of the conventional GR value. The proposed GR_s value is defined in the following equation and can be used to evaluate the condition of geotextile clogging or soil blinding within a soil-geotextile system.

$$GR_s = i_{sg} / i_s \tag{3}$$

$i_{sg} = (\Delta H3+\Delta Hg) /(L3+ Tg)$, hydraulic gradient of the geotextile and the soil layer 25.4 mm above the geotextile.

$i_s = (\Delta H0+\Delta H1+\Delta H2) /(L0+ L1+ L2)$, the hydraulic gradient for the soil layer from 25.4 mm to 101.6 cm above the geotextile.

Based on equations (2) and (3), two steps are involved in the filter selection procedure. The first step is to perform the implanted GR test for determining the GR_g value. The second step is to calculate the GR_s in order to evaluate the clogging condition of the soil-geotextile system. If the geotextile is able to pass both evaluation procedures, it implies that the use of this candidate geotextile as a filter material would not cause geotextile clogging and soil blinding (or sediment formation) within the soil-geotextile system.

Summary and Conclusion

A modified implanted GR test system was proposed in the study. In comparison with the conventional GR test device, the following conclusions are made:

1. In general, the GR values obtained from the implanted GR test system are greater than those obtained from the conventional GR tests.
2. The developed implanted GR test system is able to identify what occurrs within the soil-geotextile system. It was also found that soil blinding normally occurs in the soil layer very near (within 25.4 mm) the geotextile.
3. An additional pizeometer (#9) placed on the geotextile is suggested to use, which can provides more valuable data to identify the clogging occurrence from the geotextile itself.
4. In the process of selecting a geotextile as filter material, both the clogging potential for the geotextile itself (GR_g) and the soil-geotextile (GR_s) system should be evaluated.

Reference

[1] Li, B., Curiskis, J. I. and Griffith, R. E., "A Study of Some Key Factor on Geotextile Hydraulic Property Measurement," Fifth International Conference on Geotextile, Geomembranes and Related Products, Singapore, 5-9 September 1994.

[2] Bhatia, S. K., Qureshi, S., and Kogler, R. M., "Long-Term Clogging Behavior of Non-Woven Geotextile with Silty and Gap-graded Sands," Geosynthetic Testing for Waste Containment Application, ASTM STP 1081, Robert M. Koerner , Ed. , American Society for Testing and Materials, Philadelphia, PA,1990.

[3] Faure, Y. and Gourc, J. P., "Soil-Geotextile Interaction in Filtration Systems," 3rd International Conference on Geotextile, Vienna, Austria, 1986.

[4] Calhoun, Jr. C. C., "Development of Design Criteria and Acceptance Specifications for Plastic Filter Cloths," Technical Report S-72-7, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, June 1972.

[5] Williams, N. D., and Abouzakhm, M. A., "Evaluation of Geotextile/Soil Filtration Characteristics Using the Hydraulic Conductivity Ratio Analysis," Transportation Research Board, 1989, pp. 1-26.

[6] Williams N. D. and Luettich S. M., "Labortory Measurement of Geotextile Filtration Characteristics," 4th International Conference on Geotextiles Geomembranes and Related Products, the Hague, Netherlands, 1990, pp. 273-278.

[7] Fannin, R. J., Vaid, Y. P., and Shi, Y., "A Critical Evaluation of the Gradient Ratio Test," Geotechnical Testing Journal, GTJODJ, Vol.17. No 1, March 1994, pp.

35-42.
[8] Fannin, R. J., Vaid, Y. P., Palmeria, E. M., and Shi, Y. C., "A Modified Gradient Ratio Test Device," Recent Developments in Geotextile Filter and Prefabricated Drainage Geocomposites, ASTM STP 1281, Shobha K. Bhatia and L. David Suits, Eds., American Society for Testing and Materials, 1996.
[9] Chang, D. T. T., and Nieh, Y. C., "Significance of Gradient Ratio Test for Determining Clogging Potential of Geotextile," Recent Developments in Geotextile Filters and Prefabricated Drainage Geocomposites, ASTM STP 1281, Shobha K. Bhatia and L. David Suits, Eds., American Society for Testing and Materials, Philadelphia, PA, 1996.
[10] Rollin, A., Andre, L. and Lombard, G., "Mechanisms Affecting Long-Term Filtration Behavior Geotextile," Journal of Geotextile and Geomembranes, Vol.7, 1988, pp. 119-145.
[11] Haliburton, T. A and Wood, P. D., "Evaluation of the U.S. Army Corps of Engineers Gradient Ratio Test for Geotextile Performance," Proceedings of the 2nd International Conference on Geotextiles, Las Vegas, Vol 1, 1982, pp. 97-101.

ISBN 0-8031-2860-6