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Foreword

In the United States, the use of geosynthetics, which began in the early 1970s, has grown phenomenally. Their success comes not without some concern on the part of owners of facilities in which geosynthetics are used. One of the major concerns is about the durability of geosynthetics; this matter has necessitated investigations, both nationally and internationally, by local, state, and federal governments, academia, manufacturers, and industry. The nine papers in this volume, which were presented at TRB's 73rd Annual Meeting in January 1994, collectively provide valuable information on the durability of geosynthetics used in various applications, methods used to test the geosynthetics, and various sources that could affect the properties of these novel materials.

Durability Study of Typar 3401 Twenty Years After Installation: The Smyrna Road Project

STEPHEN J. DEBERARDINO AND WILLIAM M. HAWKINS

In 1972 a test section was initiated as part of an evaluation of the newly emerging geotextile industry. The purpose was to determine which of the family of spunbonded products best fit the end use, performing the required functions. Several materials were installed and evaluated in different locations. One location still in operation is Smyrna, Delaware. In June 1992 this original site was located and samples were exhumed. Installation conditions, field performance, and current status are reviewed for this test section, one of the oldest accessible geotextile separation applications in this country. Through evaluation of physical properties, the geotextile's survivability and durability are evaluated and compared with what is experienced today. Electron microscope results are compared, and the evolution of ultraviolet stabilizers is highlighted. Results show property retention after this extended period.

In 1972, the nonwoven fabrics "Cambrelle" from ICI and "Bidim" from Rhone-Ponlenc were being used in Europe in road support applications on soft soils and construction sites. The results were better roads. Recognizing this, DuPont, already a nonwoven fabric producer, established a program to develop a product for use in these applications. Today these permeable separation materials are commonly known as geotextiles.

As part of that program several fabrics were evaluated under roads at four different locations:

- A plant road in Delaware,
- A local school road in Delaware,
- A farm road in Wisconsin, and
- A private road in Smyrna, Delaware.

As a result, a 4-oz/yd^2 thermally spunbonded polypropylene fabric was selected as the preferred geotextile: Typar 3401.

As part of the DuPont Company's 20th anniversary in the geotextile industry and as a result of increasing interest in durability, the authors located one of the original sites, the road in Smyrna, Delaware. Pictures of the site were taken, samples of geotextile and core samples were obtained for later testing, and a general evaluation was made of the field performance of the test section.

The source of most of the historical information is the original test evaluation report and discussions with its author, Dick Hutchins (*1*). Mr. Hutchins was present when the geotextile was installed in the Smyrna road over 20 years ago, and was also present in June 1992 when the site was revisited and samples exhumed.

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1972 INSTALLATION DESIGN

In the early 1970s the DuPont Company ran various tests on its thermally spunbonded fabrics to determine their ability to enhance road performance. The Smyrna test section used a Delaware farm road over a sandy clay soil with a load bearing capacity of 1 CBR (California bearing ratio) when wet (6 CBR when dry). This road has been used actively over the past 20 years. The original road design using a geotextile as a design enhancement is one of the first of its kind in the United States.

Unlike the other test sections run by DuPont, the Smyrna road was completely controlled by the designers. During the test the Smyrna road was not subject to repair. The DuPont test program focused on providing useful information for predicting performance of potential geotextile materials placed under the base course. The Smyrna test used a 3.7- × 310-m section of road that was intentionally underdesigned. Using 40-kN wheel loads above the low-load-bearing soil calls for a 38-cm gravel base; however, only 15 cm of gravel base (40 percent of design) was used (*1*). The idea was to encourage or accelerate failure so that the test geotextiles could be evaluated. The tests were run in two stages: a dry run, in which the loaded vehicle traverses while the road is dry, then the samples are excavated; and a wet test, in which loaded vehicles are run after a heavy rain, then samples are excavated.

Normal road construction techniques were used, and a control section was installed where no fabric was placed under the 15-cm gravel base (*1*).

The dry run (142 passes of loaded vehicles) produced no noticeable differences between the section where fabric was used and the control section. After a 6.5-cm rain deluge, the wet test was carried out. Results were as follows:

- Soft spots only occured after 120 passes in the Typar 3401 section, and
- Complete failure occurred after 20 passes in the control section.

After the wet test, Typar 3401 was excavated. The fabric maintained sheet integrity, and it was concluded that Typar 3401 provided the best permeable separation and stabilization of all products used at the Smyrna road project. The relative strength loss after excavation was 50 percent for trapezoid tear and 35 percent for puncture. It was also concluded that, for heavy-duty construction stresses, fabric weights less than 135 g/m^2 should not be used due to lack of survivability. The ability of a fabric to resist tearing was noted as highly desirable (*1*).

EXHUMATIONS

In June 1992 a team returned to the Smyrna road project site to determine the performance status of the road and the condition of the geotextile and to retrieve a representative sample of the 20-year-old geotextile for evaluation.

Upon arrival at the site the various test plots were located, specifically the Typar 3401 plot. Several photographs were taken to characterize the general area conditions as well as the specific plot.

Sample exhumation followed. Pick and shovel were required to break up the unpaved road surface, which was well compacted considering that the exhumation was done in the most critical area; the tire tracks. After locating the geotextile elevation about 15 cm down, where it was initially placed, careful removal of the fill by hand and brush proceeded over an area of 2 m^2. More photos were taken and the sample was removed and stored in a plastic bag and paper tube.

Later the site was revisited and the team used Shelby tubes to take core samples. Fill was removed down to 4 or 5 in., leaving approximately 1 in. of fill. Core samples of two controls (with no geotextile) and six samples with geotextile were taken with 3 to 5 cm of fill above and 3 to 5 cm of subgrade below. It was not possible to get totally undisturbed core samples because of the rocky nature of the fill and the inability of the Shelby tube to cut cleanly through the geotextile.

The core samples were encased in plastic for observation and possible analysis. In general it was possible to reconstruct the cross-section to demonstrate the separation.

TECHNICAL EVALUATION

Site Evaluation

Photographs confirm that even though the geotextile was installed 20 years ago and the project was grossly underdesigned, the geotextile has survived and endured to effectively perform its primary function as a permeable separator. At the site it was obvious where the geotextile was used: there is no significant rutting at those locations. It was equally obvious where no geotextile was used, as indicated by the rutted areas.

Shelby Tube Evaluation

Eight Shelby tube samples of subgrade, geotextile, and fill were taken from the site. The tubes are separated into two groups: two without geotextile and six with geotextile. Although analysis continues, preliminary results indicate the following:

- Most of the geotextile samples were disturbed because the tube was inserted by impact. When the tube reached the geotextile, it did not cut cleanly. The fill and soil next to the geotextile were disturbed, making it difficult to observe the exact soil structure next to the geotextile. It was clear in all cases that there was effective separation of subgrade.
- The sample with no geotextile showed significant intermixing resulting in ruts and potholes (poor road performance).

Physical Characteristics

The geotextile samples were brought to the lab to compare their current physical characteristics with those of 1972. Grab tensile was not tested in 1972; however, curent results show a 50 percent strength retention and a 35 percent elongation retention compared with historical production data on first-quality Typar 3401 produced in 1972. Current trapezoid tear stength retention was approximately 40 percent, which compares favorably with the 50 percent strength retention observed after exhumation 20 years ago. The permeability values of the recovered soil-impregnated geotextile samples are above the range of values of the soils present (sandy clay, .00005 cm/sec). Overall the physical characteristics of the geotextile remained stable over a 20-year period in this underdesigned roadway.

Photomicrographs of Exhumed Geotextile

Exhumed geotextile samples were returned to the lab in plastic bags for testing and evaluation. Analysis of the magnified polypropylene filaments showed little to no degradation over time. For photomicrograph analysis of the geotextile polymer, it was necessary to remove as much dirt and other interference as possible. Repeated attempts to clean the dirt from the samples by washing failed. A DuPont technical assistant suggested and demonstrated that ultrasonic cleaning was effective, and using that procedure many photos of the exhumed geotextile fiber structure were taken and compared with unused, recently produced geotextile. There was no indication of polymeric deterioration observed even when magnified 3,000 times. Only mechanical damage to the outer layer of the geotextile surface fibers was apparent. In addition that damage appeared more obvious on the fill side of the geotextile as a result of the fill texture being harsher than the subgrade. The damage is also more severe than normal because of the underdesign fill thickness (15 cm versus 38 cm needed for proper design) and overloaded conditions permitting more stress to reach the geotextile.

Overall it appears that photomicrograph analysis may be more sensitive to polymer deterioration than other methods under consideration. That combined with stabilizer analysis may give more conclusive evidence of projected life expectancy.

Polymer Evaluation

One of the goals of this study was to determine whether the same amounts of antioxidants and ultraviolet stabilizers are present today as were added to the material in production 20 years ago. In pursuing this goal, it became clear that current techniques (gel-permeation chromotography and oxidation induction) cannot provide any relevant information for comparison. We are, however, able to review the heat flow (melting) curve, review the thermo oxidative breakdown of polypropylene, and compare the 1972 stabilizer package with today's stabilizer package.

Differential scanning calirometry (DSC) was performed on the 20-year-old samples. The melting point for polypropylene is not a sharp peak, and the peak melting point varies from pellet to pellet. The melting point observed (about 160°C) and the resulting heat flow curve for the 20-year old sample is typical for polypropylene.

Polypropylene is a long chain polymer in which structural damage can be initiated by heat or light with time by the development of free radicals. The presence of oxygen is a fundamental requirement for breakdown to occur (John Daniel, unpublished data).

Oxygen content in soils is quite low and continues to diminish exponentially the deeper one goes below the surface. This helps explain the relatively undisturbed condition of the filaments examined in the photomicrographs.

The potential breakdown is a surface phenomena caused by the presence of all the necessary ingredients (heat, light, oxygen) at the surface of a filament. Once the free radical development begins, the next reaction will occur with the nearest neighbor molecule (John Daniel, unpublished data). This explains the circumferential crack development of a well-oriented polypropylene fiber where heat or light degradation, or both, has occurred.

Manufacturers add stabilizers to polymers to retard and quench the free radical reaction. These stabilizers either hinder the development or react with free radicals to make them nonreactive.

The stabilizer package of 1972 is radically different and much less effective than today's stabilizer packages. The 1972 Typar 3401 stabilizer package consisted of the following:

- UV 531—Absorbs ultraviolet light and releases it as energy. Major drawback was that it could easily be washed away.
- Dilaurylthiodiproprionate (DLTDP) and Topanol CA—Thermal stabilizers interact with free radicals, quenching the thermal degradation process.

Today Typar uses the latest in hindered amine light stabilizer packages (HALS). HALS act as free radical scavengers no matter what type of free radical is developed; they quench the degradation process and in the process regenerate themselves. Typar's specific HALS package is proprietary.

Even though the 1972 Typar sample had a stabilizer package inferior to what is available today, the photomicrographs clearly show that polymer degradation is minimal to nonexistent. From the standpoint of long-term durability, this is primarily the result of the lack of oxygen in the soil, with secondary support from the stabilizer package.

CONCLUSIONS

Site inspection and Shelby tube samples indicate that the thermally spunbonded nonwoven geotextile is still performing the function originally intended when it was installed 20 years ago, even though grossly underdesigned.

By the indicators available and used, including DSC, the scanning electron microscope, and physical testing, the polymer had no measurable deterioration, and the only damage to the geotextile was mechanical—primarily to the upper filament surface layer—even though the geotextile was only 15 cm below the surface. Further examination of the geotextile is continuing. For future studies ultrasonic cleaning of field samples is effective in removing soil embedded in the geotextile matrix to permit more accurate analysis and more revealing photomicrographs. Cross-sections of fibers make it possible to see any sign of polymeric breakdown. Even though the installation was intended to be temporary and was grossly underdesigned, overall it has continued to function effectively.

For over 20 years, a geotextile has ably performed the job of permeable separation in Smyrna, Delaware. The positive results of the 1972 test section are strongly supported by the current analysis. Current visual inspection of the site clearly shows that the use of a geotextile dramatically increases the performance of an unpaved road over an extended period of time. Moreover analysis of the polypropylene geotextile shows little to no degradation over its 20-year life. It is concluded that the relative lack of oxygen in the soil surrounding the geotextile over the past 20 years is the main contributor to the insignificant degradation. Similar physical index test results from 1972 and 1993 coupled with more revealing photomicrograph analysis of the polymer confirm this insignificant degradation. Advances in stabilizer packages such as the HALS package currently used in Typar products will only increase a geotextile's effective design life.

Polypropylene geotextiles such as Typar offer strong evidence that a geotextile can be used effectively over a long period of time.

ACKNOWLEDGMENTS

Special thanks go to Richard Hutchins, a DuPoint retiree who installed the fabric 20 years ago, for location and exhumation assistance; Shobha Bhatia at Syracuse University for mounting and evaluating Shelby tube samples; Charlie Cruise of DuPont's scanning electron microscope laboratory for his technical assistance; and John Daniel of Reemay, Inc., for his valuable polymer information.

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Construction Damage Assessment of a Nonwoven Geotextile

MARK H. WAYNE AND RICHARD J. BARROWS

Current state-of-the-practice geosynthetic design has evolved to the "design-by-function" concept. When geosynthetic materials are incorporated as reinforcing elements in highway widening projects the designer will often use a default value for the partial factor of safety associated with construction durability (i.e., installation damage). Since Task Force 27 has recommended a default value of 3.0, it is beneficial for the manufacturer to determine the influence of construction-induced stresses on their materials. A test pad was constructed and a geosynthetic test program conducted to determine the actual partial factor of safety associated with installation damage. This information will aid future designers, specifiers, and manufacturers in developing such a test program and will enhance the data base for future investigators.

When geosynthetics are used as reinforcing elements within earth structures the designer must consider the influence of construction damage, aging, temperature, creep, and confining stresses on the allowable design strength, T_a. In the absence of sufficient test data, T_a can be calculated by using the following simplified expression (*1,2*):

$$T_a = \frac{T_{\text{ult}}\ (\text{CRF})}{\text{FD} \times \text{FC} \times \text{FS}} \leq T_s$$

where

T_{ult} = ultimate (or yield tensile strength) from wide-width strip tensile tests (ASTM D-4595);
T_s = long-term tension capacity of geosynthetic at a selected design strain (usually 5 percent or less);
FD = durability factor of safety (dependent on susceptibility of geosynthetic to attack by microorganisms and chemicals, thermal oxidation, and environmental stress cracking and can range from 1.1 to 2.0. In the absence of product-specific durability information, use 2.0);
FC = construction damage factor of safety (Task Force 27 recommends a minimum value of 1.25 when specific backfill source is unknown but construction installation damage test data are available. In the absence of product-specific construction damage information, use 3.0);
FS = overall factor of safety to account for uncertainties in geometry of structure, fill properties, reinforcement properties, and externally applied loads (for permanent, vertically faced structures, FS should be a minimum of 1.5); and
CRF = creep reduction factor (CRF = T_l/T_{ult}, where T_l is creep limit strength obtained from creep test results). If CRF value for specific reinforcement is not available, Task Force 27 recommends 0.2 for polypropylene, 0.4 for polyester, 0.35 for polyamide, and 0.2 for polyethylene.

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On the basis of this information it is in the best interest of the manufacturer to work with the designer to establish the appropriate partial factor of safety values.

Of all the values indicated, emphasis is placed on the determination of a partial factor of safety associated with construction damage, FC. The construction damage assessment program should take into consideration the supporting subgrade conditions, gradation and angularity of backfill, geotextile properties, method of backfill placement, lift thickness, and compaction. These conditions will have an effect on the post-construction mechanical properties of the geosynthetic and will be dependent on site conditions and construction requirements established within project specifications.

PROJECT DESCRIPTION

The Salmon–Lost Trail Pass Highway project is an experimental project initiated by the FHWA Western Federal Lands Highway Division to evaluate the use of nonwoven geotextiles for the construction of steep slopes. The project is located in Idaho's Salmon National Forest and involves the widening of Idaho Forest Highway 30. A portion of the highway widening involves the construction of a 45-degree permanent geotextile-reinforced slope 172 m (565 ft) long, 2 to 15 m (5 to 50 ft) high.

TEST PROGRAM

A construction damage assessment program should include an up-front site evaluation, construction of a test pad, geotextile testing, and development of a partial factor of safety after evaluation of test data. This process is broken down into the following eight steps:

1. Evaluate subgrade conditions,
2. Evaluate backfill soil,
3. Conduct geotextile placement,
4. Conduct backfill placement,
5. Perform compaction,
6. Determine lift thickness,
7. Conduct geotextile testing, and
8. Develop partial factor of safety.

Each of these steps is dependent on the intended end use of the geosynthetic. In the following paragraphs these steps are examined for the case of a nonwoven geotextile used as reinforcement in the construction of a steep slope on the Salmon–Lost Trail Pass Highway project.

Subgrade Conditions

Cut material that would be removed prior to embankment construction was deemed as an appropriate embankment construction material. As a result the most economical means of performing the construction damage assessment program involved construction of a test pit. By excavating a pit 0.5 m (1.5 ft) deep along the shoulder on Highway 30, the undisturbed soil simulated actual construction conditions. Soil removed from this test pit was stockpiled for use in the test program.

Backfill Soil

The cutbank soil evaluation included visual observation, compaction, Atterberg limit testing, mechanical sieving, and pH testing. On the basis of visual observation it is expected that gravel and cobbles will be present throughout the proposed fill material. To limit construction damage potential, all material larger than 102 mm (4 in.) will be removed during construction of the geotextile embankment. Compaction tests indicated that a maximum density range of 18 to 21 kN/m^3 (115 to 130.6 lbf/ft^3) can be achieved at a moisture content range of 13.5 to 9.5 percent, respectively. From Atterberg limit testing, the cutbank soil was found to exhibit a liquid limit of 28 and a plasticity index that ranged from 7 to 10. Based on results of Atterberg tests and mechanical analysis, the cutbank soil is described as a silty sand with gravel (SM) in accordance with the unified soil classification system. Results of a pH test indicate that cutbank soil exhibits a pH range of 5.8 to 7.1. Because of the short-term exposure to soil during this test program, the influence of soil chemistry on the mechanical properties was deemed negligible.

All soil evaluation tests were conducted by the materials section of FHWA's Western Federal Lands Highway Division. Testing protocol and results associated with this work are found elsewhere (*3*).

Geotextile Placement

A 15.25-m (50-ft) by 3.96-m (13-ft) sample of an enhanced-modulus nonwoven geotextile was submitted by the manufacturer to FHWA. This geotextile was designated as geotextile Type IX within the project specifications and is a 407-g/m^2 (12-oz/yd^2) polypropylene continuous-filament needle-punched nonwoven geotextile manufactured for reinforcement applications. The properties required for this project are given in Table 1. At the site, 7.62 m (25 ft) by 3.96 m (13 ft) of the sample was placed on the undisturbed soil within the test pit. The remaining material was set aside to be used as the control in the testing program.

Backfill Placement, Compaction, and Lift Thickness

The geotextile sample was divided into three zones each measuring approximately 2.44 m (8 ft). Within each zone stockpiled soil was placed with a rubber-tire front-end loader to a loose depth of 152 mm (6 in.), 305 mm (12 in.), and 457 mm (18 in.). All stones and cobbles larger than 102 mm (4 in.) were removed from the backfill in accordance with project requirements. A fully loaded 10-yd, 10-wheel dump truck was then used to simulate compaction. A total of 25 passes were made across the entire section, resulting in lift thicknesses of 102 mm (4 in.), 203 mm (8 in.), and 305 mm (12 in.), respectively.

The compacted soil was loosened with a pick and removed within the trafficked areas with shovels. A geotextile sample was then removed from each section and labeled accordingly.

Geotextile Testing

The purpose of geotextile testing was to determine the influence of construction activities on the ultimate strength, T_{ult}, of the geotextile in the direction of load application, namely the machine direction. To aid in the interpretation of this information, additional testing was deemed appropriate.

In accordance with survivability requirements established by FHWA (*4*) and AASHTO (*5*), the Mullen burst, puncture resistance, trapezoidal tear, and water permeability values were evaluated as part of this investigation. Results of testing are presented in Table 2. Each test series was conducted in accordance with the ASTM method designated in Table 2. A compilation of geosynthetic testing procedures is presented in *ASTM Standards on Geosynthetics* (*6*).

Partial Factor of Safety

As indicated in Table 2, for the soil and placement conditions considered in this study, there is no reduction in machine-direction

TABLE 1 Construction Geotextile Property Requirements

Property	Test Procedure	Value[a]	Units
T_{ult} [b]	ASTM D 4595	20,000 (115)	N/m (lbf/in)
Mullen Burst	ASTM D 3786	2756 (400)	kPa (psi)
Puncture Resistance	ASTM D 4833	601 (135)	N (lbf)
Water Permeability	ASTM D 4491	0.30	cm/sec

[a] Minimum Average Roll Values: the sample average test results for any roll tested within a lot designated as first quality, tested in accordance with ASTM D 4759-88, must meet or exceed the values listed.

[b] Machine direction strength

TABLE 2 Geotextile Construction Damage Test Results

Property	ASTM Test Procedure	Control[a]	102 mm (4 in) lift	203 mm (8 in) lift	305 mm (12 in) lift	Units
T_{ult} [b]	D 4595	22565	22022	22232	23984	N/m
		128.8	125.7	126.9	136.9	lbf/in
Mullen Burst	D 3786	4747	3864	3278	3934	kPa
		689	560.8	475.8	570.9	psi
Puncture Resistance	D 4833	838	761	861	769	N
		188.3	171	193.4	172.9	lbf
Water Permeability	D 4491	0.329	.443	.438	.468	cm/sec
Trapezoidal Tear [c]	D 4533	453	635	567	594	N
		101.9	142.7	127.5	133.5	lbf
Trapezoidal Tear [d]	D 4533	555	695	638	582	N
		124.8	156.2	143.3	130.7	lbf

[a] Sample average test results for the field sample which was tested in accordance with ASTM D 4759-88.

[b] Machine direction strength

[c] Machine direction strength

[d] Cross direction strength

wide-width strip tensile strength for the 305-mm (12-in.) lift thickness. For the more severe conditions involving smaller lift thicknesses of 102 mm (4 in.) and 203 mm (8 in.), test data indicate reductions of 2.4 and 1.5 percent, respectively. Comparison of T_{ult} for the 102-mm (4-in.) and 203-mm (8-in.) lift thickness values against that of the 305-mm (12-in.) lift thickness indicates 8.2 and 7.3 percent losses, respectively. On the basis of this information, the construction damage partial factor of safety, FC, was set at 1.1, the lower limit established by Christopher et al. (*1*). Results of additional tests indicated a slight increase in permeability and trapezoidal tear stength. In contrast, puncture and Mullen-burst testing led to reductions in strength for the 305-mm (12-in.) lift thickness of 8 and 17 percent, respectively, when compared with the control sample. Because all values are well above those required by AASHTO for a high-survivability geotextile, it appears that the current AASHTO criteria are useful as a starting point.

CONCLUSIONS

A construction damage assessment program should replicate conditions that may exist during construction. The key steps in this process are the evaluation of subgrade and backfill soil, geotextile placement, backfill placement, compaction, determination of lift thickness, geotextile testing, and determination of the partial factor of safety. Because it is not possible to precisely model the field conditions it is important for designers to include a minimum construction installation damage factor, FC. As recommended by Christopher et al., this factor should not be less than 1.1.

A construction damage assessment program was examined for a high-strength nonwoven geotextile to be used in the construction of a reinforced soil structure. For nonwoven geotextiles used in these applications, evaluation should include the ultimate strength evaluation (i.e., wide-width strip tensile strength) in the direction of stress transfer, along with key index properties. For this particular needle-punched nonwoven geotextile, influential index properties were found to include the Mullen burst, puncture resistance, and permeability.

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Evaluation of Nonwoven Geotextile Versus Lime-Treated Subgrade in Atoka County, Oklahoma

DAVE GURAM, MARK MARIENFELD, AND CURTIS HAYES

The use of chemically stabilized subgrades in routine maintenance and new construction of roadways represents an expensive way to address the problem of stabilization and separation from base materials. In 1984 a nonwoven Supac 8NP was installed on secondary state highway SH-131 in Atoka County, Oklahoma, to investigate a more cost-effective means of separation and stabilization. Roadway performance of the geotextile test sections was compared with the traditionally used 610-mm (24-in.) lime-stabilized subgrade control sections, as both the geotextile and the stabilized subgrade were covered by the same pavement structural section. Also evaluated was geotextile survivability and performance after the rigors of construction, the stress of traffic, and aging. Roadway history, fabric and soil sampling and testing, road conditions, and estimation of fabric durability are examined. Geotextile durability is determined by removing the fabric from the roadway and testing the exhumed samples. Data are compared with the original, unaged samples.

According to the 1974 National Highway Needs Report, federal-aid highways are deteriorating at a rate of 50 percent faster than they are being rebuilt. Today this percentage could be much higher. It is therefore imperative that more efficient and effective highway construction and reconstruction technologies be developed. Although Oklahoma has an excellent system of highways, better, more durable roadway systems are being investigated. One promising development is the use of geotextiles for separation and stabilization.

Incorporation of a geotextile in the pavement design can improve performance and service life. Geotextiles are cost-effective alternatives to stabilization methods such as demucking, placement of thick structural fill, lime stabilization, or other expensive manipulation operations. All roadway systems, whether temporary or permanent, derive their strength and support from the subgrade. The misconception in layered roadway designs, such as AASHTO pavement design, is that respective layers of various pavement components will remain "as placed or constructed" over the existing subgrade throughout the service life of the pavement. Because of changes in load and environmental factors, however, pavement system failures do occur at the aggregate base subgrade contact point. This is a result of the intrusion of low-strength subgrade material into the aggregate base and base material into the subgrade. The intermixing of two dissimilar materials causes a net reduction in the effective thickness of the base and initiates a progressive failure mechanism, resulting in the need for continual road maintenance. A study conducted by Hicks et al. (*1*) clearly shows that a base contamination of as little as 10 percent subgrade soil fines can destroy the structural strength of the base layer.

Traditional solutions to this contamination problem include using a well-graded base, which helps choke off the migration of fines but is less strong and not free draining; stabilizing the subgrade to limit its migration; or stabilizing the base stone to make it less affected by fines migration. A better solution is the placement of geotextiles as a separation and stabilization layer between the subgrade and overlying base, preventing base contamination due to subgrade intrusion into the subbase or base. Use of a geotextile for separation and stabilization has been proven technically effective and is a widely used alternative. Work done by Barenburg et al. (*2*) clearly showed that incorporation of a geotextile could significantly improve the stability of the roadway system or would allow the system to be constructed with a thinner structural section and still achieve the same performance level. One reason thinner sections can be used is that the AASHTO methodology, which evolved over time based on performance, actually compensates for base thickness loss due to contamination, and a geotextile eliminates this contamination (*3*). The other reason that geotextiles allow the use of a thinner structural section is the stabilizing effect the geotextile has on the subgrade.

This paper discusses the work done to evaluate and compare a nonwoven needle-punched polypropylene geotextile, Supac 8NP, with 610 mm (24 in.) of in-place lime-stabilized subgrade. The durability of the geotextile and the performance of both sections were monitored over a period of 9 years. Durability is defined as the geotextile's resistance to damage due to initial installation and construction and other mechanical and chemical factors during the service life. Polyester fabric was intentionally not installed because of its chemical incompatibility with lime on the construction site.

The durability of the lime-treated subgrade material was not evaluated.

PURPOSE

The purpose of the study was to compare the relative performance of Supac 8NP, a nonwoven needle-punched polypropylene geotextile, nominal weight 271 g/m^2 (8 oz/yd^2), with lime-treated subgrade soil. Long-term performance questions were also addressed by examining the long-term durability of the geotextile section. Specifically, would the less expensive geotextile system placed over a highly plastic clayey subgrade be equivalent to 24 in. of

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lime-treated subgrade? The same road base and surface treatment were applied over both the lime-stabilized and the geotextile-stabilized section to allow a fair comparison.

PROJECT LOCATION

The project is located on secondary highway SH-131, 0.8 km (0.5 mi) east of the town of Wardville, extending east to US-69 in northern Atoka County, Oklahoma. Two traditionally used lime-treated sections, one on each end of the Supac 8NP section, were selected as the control. The geotextile section was 183 m (600 ft) long by the full width of the roadway. Both lime sections used a layer of lime-stabilized subgrade 610 mm (24 in.) thick. The project was built by Honegger Construction Company of Oklahoma City, Oklahoma, as a part of the Oklahoma Department of Transportation (ODOT) project SAP-3(168).

SITE CONDITIONS

The subgrade soils were of poor quality with low bearing capacity. The site has a perched water table at a depth of approximately 0.6 m (2 ft) to 0.9 m (3 ft) during winter and spring. Soil classification and mechanical analysis data for the site are given in Table 1. According to ODOT guidelines, these soils require special treatment to increase the subgrade strength and prevent local and general damage to the pavement system. Originally the project was set up to require lime treatment for subgrade support to the pavement because of poor soil conditions. The project was modified to include Supac 8NP as part of an experiment.

CONSTRUCTION

Lime Treatment Section

Approximately 2 mi of the 8.5-km (5.27 mi) project were treated with quicklime. The modification was based on subgrade soil properties and the Oklahoma subgrade index (OSI). The OSI is used to determine whether a subgrade requires any modification, and is calculated from the liquid limit, plasticity index, and percentage passing the No. 200 sieve. As a rule of thumb any subgrade with an OSI of 15 or more requires lime treatment in order to carry the design load. More than 40 percent of the soils exceeded 15 OSI on the project.

The lime-treated areas were constructed according to ODOT Standard Specifications 706.02 and 307. The treated subgrade was placed and compacted in three lifts of 8 in. each. Lime quantities were estimated at 58 kg/m^2 (108 lb/yd^2) for the 610-mm (24-in.) layer. The concentration of the lime incorporated into the treated subgrade was approximately 5 percent. Figure 1 shows a cross-section of the lime-treated area.

Fabric Treatments

Supac 8NP, a nonwoven geotextile, with a width of 3.81 m (12.5 ft), was placed over the existing subgrade. An initial 3.81-m

TABLE 1 Soil Classification, Physical and Mechanical Analysis

AASHTO Soil Group	Station	Subgrade Soil Classification	Depth (in)	Liquid Limit(%)	Plasticity Index	Physical & Mech. Analysis: Percent Passing No.10	No.40	No.200	OSI[b]
ORIGINAL SUBGRADE SOIL - 1983									
A-6(13)	176+00	Silty Clay	0-6	34	19	98[a]	95	81.4	14
A-6(16)	Underlay	" "	6-12	36	20	100	97	84.8	15
A-6(17)	"	" "	12-18	37	22	100	96	82.4	16
A-6(16)	"	" "	18-24	37	20	100	97	85.2	15
A-2-6(0)	178+00	Sandy Clay and Gravel	0-6	26	12	57[a]	38	24.6	4
A-7-6(20)	Underlay	Silty Clay	6-12	43	28	94[a]	92	78.6	20
A-7-6(23)	"	" "	12-18	47	31	95[a]	90	78.5	22
A-7-6(27)	"	" "	18-24	49	32	100	97	84.7	23
SUBGRADE SOIL UNDER VARIOUS SECTIONS - 1993									
A-4(1)[c]	177+50	Silty Clayey Sand	0-6	32	5	78[a]	58	38.2	2
A-4(0)[c]	-	" " "	6-24	NP	NP	84[a]	64	42.4	0
A-7-6(19)	Below Lime Treatment	" " "	24-36	49	27	95[a]	87	72.3	21
A-7-6(21)	195+16	Lean Clay w/Sand	0-6	46	28	96[a]	95	79.0	20
A-7-6(37)	-	Fat Clay	6-24	58	37	100	99	90.6	27
A-7-6(42)	-	" "	24-36	63	43	100	99	98.8	30
A-6(1)[c]	227+95	Clayey Sand	0-6	38	12	77[a]	73	62.7	5
A-7-5(2)[c]	-	" "	6-24	47	12	91[a]	47	21.8	13
A-7-6(34)	Below Lime Treatment	Fat Clay	24-36	59	32	99[a]	81	68.3	24

[a]Percent Passing 1-inch Sieve
[b]Oklahoma Subgrade Index
[c]Lime Treated Sections

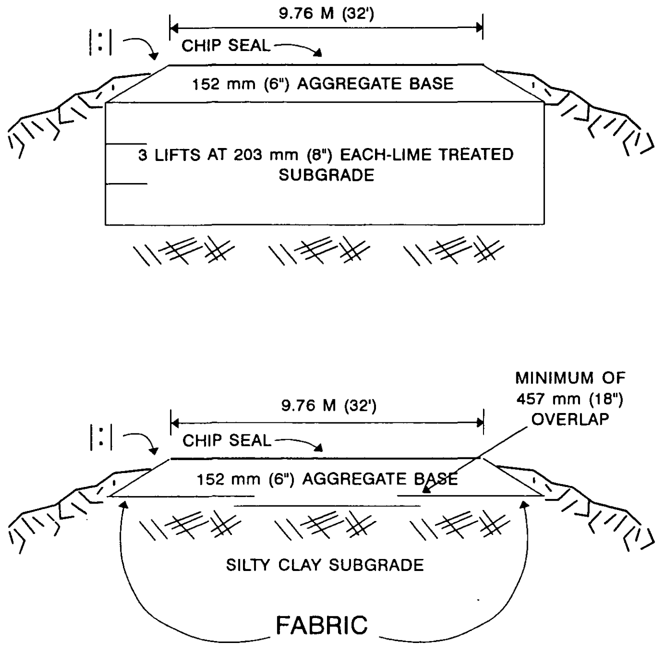

FIGURE 1 Typical cross-section of pavement.

(12.5-ft) roll was placed along the centerline of the roadway and two additional rolls were overlapped along either side. The overlap ranged from 457 mm (18 in.) to 610 mm (24 in.), providing a minimum effective width of 9.8 m (32 ft). The geotextile design cross-section is also shown in Figure 1.

The 183-m (600-ft) length of geotextile was installed by hand and took a six-man crew less than an hour. No special tools or equipment were necessary for the installation.

Pavement Structure

The same pavement structure was placed over the lime-treated and the geotextile-covered sections. Six inches of aggregate base, Type A (based on ODOT Standard Specification, 1976, Section 303) was placed by dump trucks. The aggregate was backdumped, placed in two layers, and compacted to 100 percent density per AASHTO T-180. Construction traffic did not tear or abrade the geotextile.

The surface of the aggreagate base wsa primed with 1.0 L/m^2 (0.22 gal/yd^2) of SS-1 emulsion. The primed surface then received a single bituminous surface treatment (chip seal). A CRS-2 emulsion tack coat was applied at a rate of 1.7 L/m^2 (0.38 gal/yd^2). It was followed by 16 kg/m^2 (30 lb/yd^2) of 16-mm (5/8-in.) cover aggregate and pneumatic rollers.

Exhuming of Geotextile

The following procedure was followed to exhume the geotextile. Since the roadway was kept open during removal of fabric, one lane of the test section was closed and traffic was diverted to the other lane by flag persons.

An approximate outline of the section to be removed was marked. With the help of a jackhammer the outer edges of the section were penetrated to the aggregate base. The backhoe operator cautiously removed the pavement and some aggregate base. The remaining aggregates were removed with pick and shovel to within 25 to 50 mm (1 to 2 in.) of the geotextile. The final layer of aggregate was removed by hand. Approximately 1 yd^2 of the geotextile was then cut and removed from the subgrade. The subgrade and base aggregate nearest to the fabric were visually inspected for contamination or slurry buildup underneath. No base contamination or slurry material was found. Samples of the subgrade soil were brought back to the laboratory from both sections, and test results are given in Table 1 for comparison with the original test data. The digout section was backfilled with a full-depth asphalt patch after another piece of geotextile was installed. The exhumed fabric sample showed no damage upon initial observation except two approximately 1-mm holes. The samples were brought back to the Phillips Fibers testing laboratory to determine mechanical loss in strength compared with original samples. The comparative strength properties of virgin samples and of samples removed in 1989 and 1993 are presented in Table 2.

Cost Analysis

Table 3 gives a summary of the materials and in-place 1984 costs for the project. The Supac fabric (geotextile) was furnished for free by Phillips Fibers Corporation. However, Supac 8NP fabric

TABLE 2 Supac 8NP Physical Properties

PROPERTY	TEST PROCEDURE	VIRGIN[a] SAMPLE	1988 (After 4 YRS)	1993 (After 9 YRS)
Tensile Strength, KN (lb)	ASTM D-4632	0.89 (200)	1.04 (234[b])	1.0 (199[b])
Elongation, %	ASTM D-4632	50	62	67
Puncture Strength, KN (lb)	ASTM D-4833	0.50 (125)	0.75 (170)	0.7 (158)
Mullen Burst, KPa, (psi)	ASTM D-3786	2618 (380)	3714 (539)	3259 (473)
Coeff. of Perm., cm/sec.	ASTM D-4491	0.4	–[c]	–[c]
Permittivity, sec^{-1}	ASTM D-4491	1.0	"	"
AOS	ASTM D-4751	7.0	"	"

[a]minimum average roll values, weakest principal direction
[b]the value is in the weakest direction
[c]hydraulic properties were not tested due to soil residue in the exhumed geotextile

TABLE 3 Pavement Layer Cost, 1984

Layer	Cost ($/m^2)
Aggregate Base, 152 mm (6") Thick	3.56
Prime Coat	0.27
Tack Coat	0.32
Cover Aggregate No. 3 (5/8")	0.39
Total	4.55

costs were $1.39/m^2 ($1.17/yd^2). Today the cost of Supac is less, making it an even more cost-effective option. The installation cost was estimated to be $0.12/m^2 ($0.10/yd^2), bringing the total installed geotextile cost to $1.51/m^2 ($1.27/yd^2).

The installed bid price for 610-mm-thick (24-in.-thick) lime-treated subgrade was $12.77/m^2 ($10.73/yd^2). The total cost for the pavement construction, including a 6-in.-thick base, prime, and surface treatment was $4.53/m^2 ($3.81/yd^2). Table 3 gives detailed pavement costs.

DISCUSSION OF RESULTS

To determine the durability and performance of geotextile, the following data were evaluated: soil sampling and testing results, fabric sampling and testing results, roadway conditions, ride quality, roadway maintenance history, and review of the pavement design that is based on Benkelman beam deflection measurements. Observations were made on the conditions of the exhumed geotextile samples, subgrade beneath the fabric, and base above the fabric. In combination this information provides adequate backup to characterize performance and durability.

Pavement performance and pavement deterioration were examined first. In 1986 the lime-treated sections experienced extensive pothole damage and underwent repairs. This was reportedly due to a wet spring in 1985. There was only one isolated pothole in the geotextile-stabilized section. In 1988 1½-in.-thick ODOT Type C hot-mix asphalt concrete was placed on the entire roadway even though the geotextile section had almost no pavement distress.

To evaluate the rideability, tests were conducted with the Mays ridometer in February 1993. The average pavement serviceability index (PSI) for the entire project was 3.7. The Supac section individually had a PSI of 3.6. According to ODOT, roadway repairs are usually required when the PSI is 2.5 or less. When the pavement was visually examined and evaluated, the lime-treated control section and the fabric section showed no visible differences.

The Benkelman beam deflection measurements were made in June 1993 and compared with 1984 test data, which were developed after the construction of the project. The 1993 deflection measurements were made after an early morning rainfall. The average deflections in 1993 were somewhat higher than in 1984 for both sections. Review of the deflection data shows that the geotextile-stabilized section is more flexible than the lime-stabilized subgrade section. Supac in this application is functioning as a separation and stabilization fabric by providing a barrier against intermixing of subgrade and base aggregates while letting the moisture seep through. The 610-mm-thick (24-in.-thick) lime-treated section is providing a beam effect due to higher stiffness; that is why the Benkelman beam deflection measurements are lower in these sections. However in 1985 the lime-treated section developed many potholes even though deflection data already indicated adequate roadway strength. Similarly 1984 data for the section show moderate deflection and yet no potholes.

The 6-in. pavement structure placed over the fabric is a limited section. Although it has performed well, this structural section could be increased significantly and the cost benefit of the geotextile section would still be evident. Increasing the structural section over the fabric would produce a road that would show less deflection, which, as can be seen, may have little relevance to actual road performance.

The Mays ridometer data and visual inspection of both the control and Supac sections clearly show that these sections have equivalent performance. Review of the 1984 and 1993 overlay design based on deflections indicates that the fabric section requires strengthening while the lime-treated sections require no strengthening. However in 1988 the 1½-in. overlay was placed due to potholes in lime-treated section. This clearly shows that input parameters need to be modified for the design of an overlay that is based on deflection data. Stiffness is not always an indicator of stronger properties or performance characteristics. As shown in this case, a slightly forgiving subgrade can provide better performance, although according to the design based on deflection data the nonwoven section would need strengthening to carry design loads.

Durability of Fabric

The nonwoven geotextile polypropylene did not show any loss of strength. Instead it showed some gain in mechanical properties. This finding agrees with the work done by Brorsson et al. (*4*) of the Swedish National Road Administration. That study also showed that after 10 years, nonwoven needle-punched polypropylene geotextiles kept their original strength fairly unchanged while the thermally bonded nonwoven geotextiles lost approximately 50 percent of their original strength. These data also agree with other work done by Phillips Fibers Corporation to evaluate the life expectancy of Supac (*5*). The virgin data shown in Table 2 represent the "minimum average roll values" in the weakest principal direction, and test data for exhumed samples are the actual test data for field samples. Another good indication of the excellent durability characteristics of Supac is the percent of elongation. Generally in the degradation process the fabric loses elongation properties and strength. Review of the elongation data shows no change in elongation properties compared with the unused samples. To further evaluate the chemical durability of the polypropylene resin, infrared analysis shows that no oxidation of the geotextile occurred. The infrared data show that there is no change in the molecular weights of three samples tested (i.e., virgin material) and the fabrics exhumed in 1988 and 1993.

Performance

Although subgrade soils were fat clays, which are known to hold moisture and are prone to pumping, none of these problems was observed below the nonwoven geotextile section. In comparing overall project conditions (including no potholes in the fabric section compared with the lime-treated control, surface conditions as good as control, and same current PSI) it is clear that the geotextile section has performed its separation and stabilization function

TABLE 4 Cost Analysis, 1984

Type of Treatment	Treatment Cost/M²	Pavement Layers Cost/M²	Total Cost/M²
Supac 8NP Fabric	$ 1.52	$4.55	$ 6.07
Lime - 610 mm (24") Thick	$12.82	$4.55	$17.38

Calculated per mile cost of subgrade treatment and pavement layers, 7.32 M (24 ft) wide.

Supac 8NP fabric..........................$ 71,526
Lime Treatment - 610 mm (24") thick........$204,723

for the past 9 years. From a pavement life-cycle analysis it is clear that the geotextile section required less maintenance and should continue to show a longer pavement life due to its long-term establishment of a separation and stabilization layer, compared with the lime-stabilized system, which will continue to break down over time.

Cost-Effectiveness

The cost of the lime-treated section was $11.26/m² ($9.46/yd²) higher than the geotextile section. Table 4 presents a cost analysis. After 6 months, the lime-treated section pavement surface failed due to severe spring weather conditions and required 38 mm (1½ in.) of additional overlay. The geotextile section did not need the additional overlay, but because it was in the middle of the lime-treated sections it received the same overlay treatment. Had the fabric section been left unpaved, which might not have been practical, the cost-benefit ratio would have been much higher. The approximate additional cost was $2.97/m² ($2.50/yd²) for an overlay plus the maintenance cost of repairing potholes before an overlay. These additional costs are not included in the table.

CONCLUSIONS

- The Supac 8NP nonwoven needle-punched polypropylene geotextile did not lose any strength properties during 9 years of service life.
- Based on infrared test data, there is no chemical degradation or change in molecular weight of the polypropylene resin in the geotextile.
- Supac 8NP performed its intended function of separation and stabilization.
- Actual pavement performance, based on PSI data and visual inspection of the control and Supac sections, is the same for all practical purposes; that is, the basic performance of the Supac 8NP section is equivalent to that of the 24-in. lime-treated subgrade.
- There is a strong need to develop new design methods based on Benkelman beam deflection measuring devices for designing geotextile-incorporated pavements.
- Significant savings in both construction and maintenance costs can be realized by incorporating a geotextile in road design. Geotextiles bring down the life-cycle cost of pavements.

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Durability of Geotextiles in Railway Rehabilitation

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The durability of geotextiles installed in railway rehabilitation applications is investigated by examining the track conditions and the change in properties of exhumed geotextiles at different time intervals. Properties examined include soil fouling content, change in permeability ratio, and change in geotextile strength. These properties are related, as appropriate, to characteristics such as filtration opening size. The results are discussed in terms of the geotextile's primary functions of filtration, drainage, separation, abrasion resistance, and elongation. Data collected before 1982 were used to develop a track rehabilitation geotextile specification for use without a capping sand. Data collected after 1982 were used to support and confirm the validity of the specification. This specification was adopted and has been in use in the Canadian National Railways Maritime Region since 1981. It has also been used for rehabilitation of other railway company tracks. Correctly installed geotextiles meeting the specification have given satisfactory performance and have been cost-effective.

Geotextiles were introduced into North American railway tracks in the 1970s to correct some of the problems related to track support. Similar problems were being addressed on European, Japanese, and other railways that are subject to lighter axle loads. Most of the problems were related to inadequate internal track drainage, whether because of the topography or because of created drainage problems. In North America these problems were, and still are, aggravated by the use of heavier freight cars and greater freight quantities. This results in more frequent repetitive loading by larger loads, which have also increased since the original designs.

Initially the technical recommendations for selecting track geotextiles were adapted from applications that were not railway-proven. Experience soon indicated that the North American track environment is much more abrasive and demanding on geotextiles than originally thought. Consequently, a project was funded in 1980 by Canadian National Rail, Canadian Pacific, and Transport Canada through the Canadian Institute for Guided Ground Transport with Queen's University. The objective was to develop guidelines for use of geotextiles in North American railway track rehabilitation applications.

A literature review was conducted and North American railways were assessed on their use of geotextiles for track rehabilitation. Visits were then made to a number of Canadian sites and excavation were made to exhume geotextiles (*1*,p.153). The author also visited two U.S. locations (2,p.35) as a guest of the Consolidated Rail Corporation (Conrail).

After development of the specification, excavations were made to confirm its validity. In addition excavations were made at other sites where geotextiles were installed. These results all added support to the findings of the prespecification study.

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PRELIMINARY ASSESSMENT ON CANADIAN TRACKS

The first task undertaken was to obtain details of geotextiles installed on Canadian railways. Preliminary assessments were then made to record the surficial conditions at some selected sites. Selection was based on type of geotextile, ease of accessibility, geographical location, and the like. Sixteen sites were visited in June and July of 1981. A geotextile had been installed at all sites within the previous (to 1981) 5 years. Visual examination of the track adjacent to that containing the geotextile demonstrated that there were poor drainage conditions at all sites. All were located in areas susceptible to pumping fines from either fouled ballast (fouled from any one of a number possibilities) left at the track undercut interface, subballast with excess fines, or the subgrade. The most obvious observation made was that the installations were at areas that were hard to maintain and drain, including grade crossings and track switches. The fact that a geotextile was used was indicative that a ballast support stability problem had been identified.

Fifty percent of the sites exhibited reasonably stable track structure conditions. These had all been rehabilitated with a nonwoven geotextile of mass/unit area of 500 g/m^2 or greater.

The balance of the sites all exhibited at least some surficial pumped fines at the track surface. As noted, all were in areas of poor drainage; however, this was also true of all the sites in reasonable condition. At some of the sites in poor condition, the ballast was close to being completely fouled, despite having been rehabilitated within the previous 5 years. Some of these poorly performing sites exhibited varying degrees of tie movement or wear, track out of gage, and some already required track upgrading. All these poorly performing sites had been rehabilitated with nonwoven geotextiles of mass/unit area of 400 g/m^2 or less.

From the information gathered on site installation location and from the 16 sites visited, conclusions were formulated relating to theoretical considerations for geotextile selection. These are given later.

ASSESSMENT OF CANADIAN 500 g/m^2 GEOTEXTILES

Six Canadian sites having had geotextiles installed before the start of this study were selected from those exhibiting reasonably stable track structure conditions for further site investigation. All had been rehabilitated earlier with the heaviest mass/unit area geotextile in use on Canadian National and Canadian Pacific railways before 1981. At these six sites excavations were made during the summer of 1981 to exhume a sample of the previously installed geotextile. The exhuming consisted of pulling one tie and excavating the distance of one tie width and two crib widths. Great care was taken not to damage the geotextile during excavation and removal. In all cases the excavations were continued to the subgrade, and in all cases the subgrade was protected by an intake filter subballast layer that was found below a fouled ballast layer. No evidence was found of fines migrating from the subgrade. In most cases the migrating (pumping) fines through the geotextile consisted of degradated ballast fines migrating from the fouled ballast surface left after undercutting.

The universal problem noted at all sites was a partial lack of drainage considerations during installation. In many instances the geotextiles were installed so that they were unable to drain moisture to the shoulders and then into the side ditches. Preventing bathtub and canal effects at all rehabilitation sites, whether with or without a geotextile, is now considered of paramount importance.

One factor contributing to a geotextile's performance was the depth at which the geotextile was installed below the base of the ties, because abrasive action increases as the geotextile is placed closer to the base of the tie. Even minor worn-through areas (i.e., holes), in the presence of water, will permit migration of fines that damage the ballast. Thus any damage reduces dramatically the function of the geotextile, as clearly demonstrated by the sites visited. The percentage of measured, completely worn-through areas (i.e., holes) of the worst 300 mm × 300 mm of each exhumed geotextile was plotted against exhumed depth below the tie base. The values range from 0.3 percent at a depth of 350 mm to 4.1 percent at a depth of 175 mm. The damage increased rapidly when the exhumed depth was less than 200 to 250 mm.

ASSESSMENT AT LOUDONVILLE ON CONRAIL TRACK

Five visits were made at 6-month intervals to a geotextile test site at Loudonville, Ohio, as a guest of Conrail. At this site eight different types of geotextiles were installed, all with a mass/unit area less than 500 g/m^2. During the first site visit the geotextiles were at a depth 200 mm below the base of the ties. These included woven and thin heat-bonded as well as needle-punched nonwoven geotextiles. Again the fouling fines were degraded ballast fines from the ballast adjoining and below the undercut surface.

The condition of the exhumed geotextiles after 2.5 years is shown in Figure 1. Clearly the condition of the geotextiles shown in Figure 1 confirms the findings made from the Canadian sites (i.e., if a nonwoven geotextile were to be used it would need to have a mass/unit area of at least 500 g/m^2).

In addition the results at Loudonville indicated that the woven and thin heat-bonded geotextiles plugged and acted as plastic sheets, leaving the underlying surface coated with a thin layer of wet plastic slime of moist fines. These observations were also noted at other sites, resulting in the conclusion that track rehabilitation geotextiles are best selected from needle-punched geotextiles.

ASSESSMENT AT SALEM ON CONRAIL TRACK

Three visits at 6-month intervals and one at 45 months were made to a geotextile test site at Salem, Ohio, as a guest of Conrail. At this site six types of nonwoven geotextiles, five with a mass/unit area greater than 500 g/m^2, were installed. During the first site visit the geotextiles were a depth of 150 mm below the base of the ties. The exhuming method was the same as at Loudonville. Again the fouling fines were degraded ballast fines from the ballast adjoining and below the undercut surface. The condition of these geotextiles after 1.5 years is shown in Figure 2.

Two points are worth noting from Figure 2. First is the damage on each side of the area (outlined by shading at the edge of the

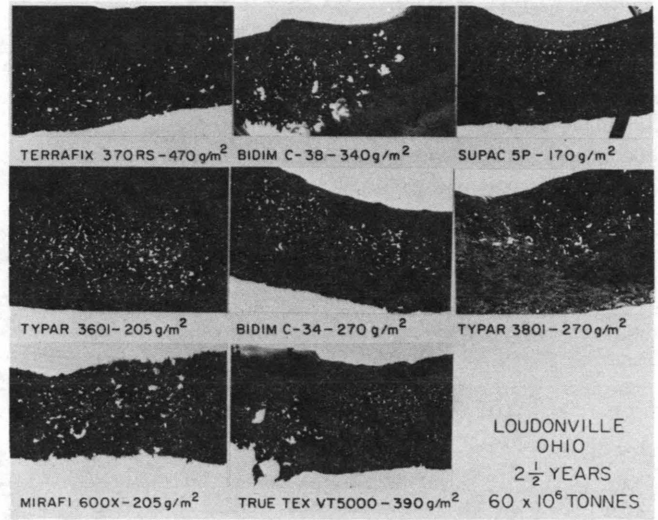

FIGURE 1 Condition of rail seat area of geotextiles exhumed after 30 months from Conrail's Loudonville site.

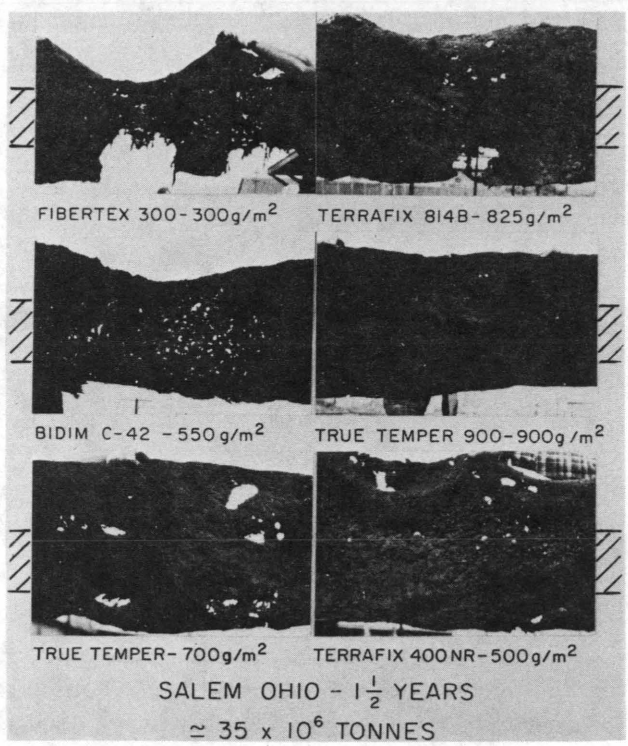

FIGURE 2 Condition of rail seat area of geotextiles exhumed after 18 months from Conrail's Salem site.

photographs) directly below the tie. This damage was done by the tamper tines, supporting the trend of the data that an installation depth of 150 mm below the base of the tie is insufficient. Second is the condition of the area directly below the tie. This area shows damage in the form of holes (not caused by the tampers) in the geotextiles of less than 900 g/m^2 mass/unit area. This suggests that a mass/unit area of at least 900 g/m^2 is needed for a rehabilitation geotextile to remain durable to function as a separation and filtration layer.

INSTALLATION CONSIDERATIONS FOR SELECTION OF TRACK GEOTEXTILES

One aspect of geotextile use that was not appreciated until some time into the project study was the method of track rehabilitation for fouled ballast. In general the track is not removed, but instead the ballast is undercut with either a chainsaw-type blade that extends under the track from one side only, or a chainlike belt feeding down under one side of the track and up the other side. A less used form is to plow or sled the old ballast flat without removing the rail. In this method new ballast is added to raise the rail above its prerehabilitation elevation. In all cases, where the track is not removed, the surface produced has ballast-size particles protruding from the surface. While some of these particles may be removed with a rake, numerous angular particles are left in place, as seen in Figure 3. Figure 3 shows the fouled ballast surface of an undercut turnout where a geotextile is to be installed. Incidently, about 100 mm below this surface is a clean filter sand used to prevent subgrade fouling of the ballast. No subgrade fouling existed at this site.

From observation of the fouled ballast surface onto which the geotextile is to be placed it is clear that a further requirement for track rehabilitation geotextiles (in addition to those already stated) is that the geotextile must be able to span and elongate around the freestanding or protruding sharp ballast particles. Because of the sharpness of the crushed particles, geotextiles that span and cannot elongate are cut from the impact of repetitive axle loading. Thus only geotextiles that can elongate around the particles were found to be satisfactory.

FIGURE 3 Condition of undercut ballast surface showing protruding particles.

A final consideration is the tamper tines used to compact ballast below the tie. These tines project about 130 mm below the base of the tie. As the tines move back and forth ballast particles are moved to some depth, typically about 200 mm below the tie base. Any geotextile used must be placed at a depth greater that 200 mm and be abrasive resistant (i.e., resistant to the abrasion of the relative movement between particles on both sides and in contact with the geotextile caused by tamping or by moving traffic).

A complete outline of problems and recommended techniques for installing geotextiles has been presented elsewhere (*3*) and is beyond the work reported here. Installation technique and careful handling are immensely important. An incorrectly installed geotextile can be a detriment to good performance rather than a help. A well installed, correctly specified geotextile had been observed to be highly cost-effective.

PERFORMANCE CONSIDERATIONS

From the data gathered in the preliminary assessment and from observing geotextiles being installed it was evident that in-track rehabilitation geotextiles are mainly employed as a means of preventing new, clean ballast or old undercut and cleaned ballast from being fouled with fines accumulated in the underlying dirty ballast or dirty subballast (i.e., separation and filtration that requires abrasion resistance). In rare situations fouling is from the subgrade, although all sites visited had considerable depths of ballast and subballast, suggesting that none of the fouling was subgrade fines. In the presence of water most internal contaminating fines below clean or cleaned ballast, subject to repetitive loads, will migrate upward through the track structure. Thus drainage improvement was established as an essential item. Such drainage provisions include the following, as appropriate:

- Internal drainage: Drainage of the track's internal stressed volume, whether ballast, subballast, or subgrade,
- Ditch drainage: Adequate side ditch drainage to deal with surface water, and
- Groundwater drainage: The lowering of the groundwater to increase the subgrade strength.

Clearly geotextiles should never be used in place of good drainage practice. From these considerations a set of functional requirements may be stated.

GEOTEXTILE FUNCTIONAL AND PERFORMANCE REQUIREMENTS

The basic functional and performance requirements of geotextiles in railway bed track rehabilitation can be summarized as follows:

- Drainage: The ability to drain water away from the track roadbed, on a long-term basis, both laterally and by gravity, along the plane of the geotextile without buildup of excessive hydrostatic pressures.
- Filtration: The ability to filter or hold back soil particles while allowing the passage of water.

- Abrasion resistance: The ability to withstand the abrasive forces of moving aggregate caused by the tamping/compacting process during cyclic maintenance, by tamping during initial compaction, and by the passage of trains on a frequent basis.
- Separation: The ability to separate two types of soils of different particle sizes and grading that would readily mix under the influence of repeated loading and water.
- Elongation: The ability to elongate around protruding large gravel-sized particles while resisting rupture or puncture.

REQUIRED PROPERTIES OF GEOTEXTILES

In order to perform the basic functions identified in the functional and performance requirements it has been established from exhumed geotextiles by Raymond and Bathurst (*4*) that the geotextile must have the properties discussed in the following paragraphs.

Permeability

At all sites examined, drainage was considered paramount. With time the ballast above the geotextile will foul. The ability of a geotextile to conduct water through its fiber matrix, whether normal or in its in-plane direction, is essential for good performance. In-plane coefficients of permeability of some new, unused geotextiles have been presented by Gerry and Raymond (*5*). It should be noted that for the woven geotextiles the weave will transmit water in the laboratory but not in the field, where soil penetrates the weave overlaps, closing the water passage existing between the flush boundaries of the permeameter. For unused nonwoven needle-punched geotextiles, the in-plane permeability is close to that of a clean sand. This is large in comparison with the permeability of fine sand, silt and clay produced by ballast degradation, transported sources, and subballast fines that, in the presence of water, migrate upward, fouling the clean ballast. Thus virtually any nonwoven needle-punched geotextile in a clean condition should pass any realistic in-plane permeability criterion.

Geotextiles installed in a track environment are subject to fouling. Tests on newly manufactured specimens should only be used to reject a geotextile that would not perform satisfactorily in the field. Of more value are tests performed on track-fouled geotextiles. Figure 4 shows results of tests conducted on thick nonwoven geotextiles exhumed at 6, 18, and 45 months from Conrail's Salem test site. All the geotextiles are nonwoven and were subject to the same tonnage. It is seen that the loss of permeability under load is related to the degree of fouling measured by the soil content of the exhumed geotextile.

The best predictor of fouling was found to be the geotextile's measured filtration opening size (FOS). The results were similar to those obtained by Gerry and Raymond (*6*) using a 5 percent passing equivalent opening size criteria.

It is concluded that a new geotextile's in-plane permeability was reduced less by products with low FOS. In general these products were constructed of fibers whose linear density was 0.7 tex or less, whose fibers had been tightly mechanically bonded by needle-punching, and were often resin bonded. Note that tex is the mass in milligram/meter length.

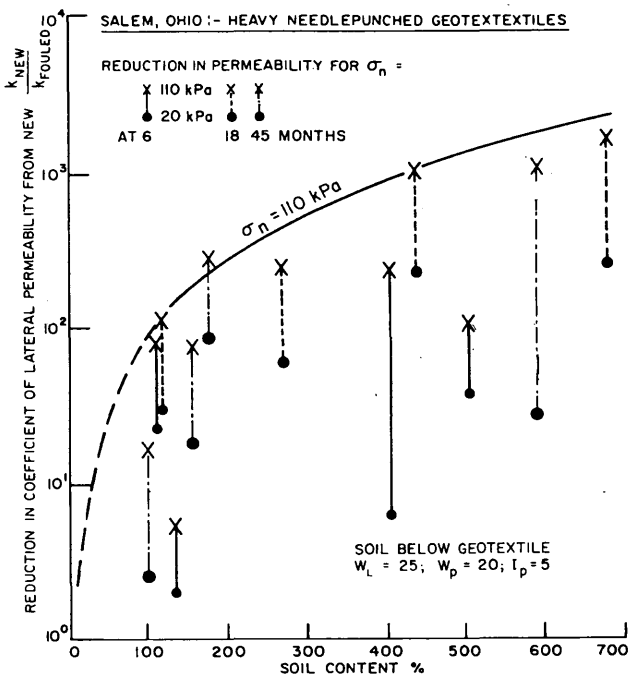

FIGURE 4 Permeability change ratio of exhumed geotextiles heavier than 500 g/m² when new.

Filtration Opening Size

The permeability, filtration, and separation performance of geotextiles is commonly related to one of the following similar quantities: FOS, apparent opening size (AOS), or effective opening size (EOS). Because of the wet environment associated with track geotextiles the FOS is considered the most appropriate test. As just illustrated, the reduction of in-plane permeability of in-track geotextiles due to fouling is dependent on the geotextile's FOS.

In practice soil particles push their way into a geotextile, thus increasing the geotextile's in-track FOS. These penetrating particles are abrasive. The effect of this abrasion is illustrated in Figure 5, which compares two 500-g/m² nonwoven geotextiles (with scrim) of the same trade name and manufacture, one taken from track (250 mm below the base of a tie rail seat) and ultrasonically cleaned and one that is new. They have both been photographed

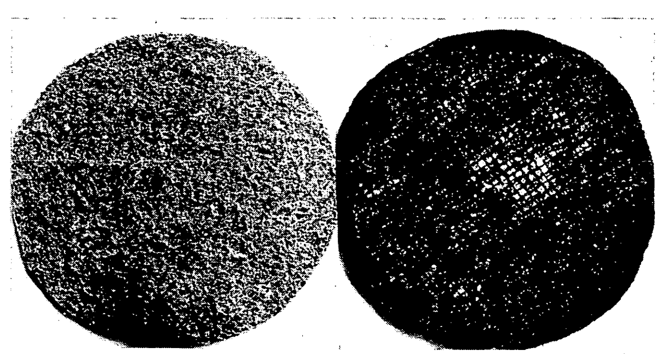

FIGURE 5 Comparison of new (*left*) and used (*right*) geotextile internal fiber wear.

in front of a bright light. The one taken from below the track shows light through the particle penetration holes, while the new one shows no light. A small FOS will reduce the size of particle that can penetrate the geotextile and hence will reduce the abrasiveness to the geotextile's fibers. Obviously any process that physically bonds fibers so as to increase the resistance to particle penetration will also be beneficial to geotextile durability. These processes are considered to be resin bonding of fibers, heat bonding of fibers, and mechanical bonding by needle-punching.

A further factor that decreases FOS is the fiber size. Hoffman (*7*) showed that both smaller fibers and decreased porosity reduced a geotextile's EOS. The trends should be valid for a geotextile's FOS. Figure 6 gives results of FOS tests performed on experimental geotextiles of similar mass/unit area and manufacture except for fiber size. The results confirm the general trend found by Hoffman regarding fiber size.

In conclusion durability will be increased by reducing the track geotextile's FOS. This will be enhanced by resin, heat, or mechanical bonding, smaller fiber size, and decreased geotextile porosity.

Abrasion Resistance

A geotextile placed in a track environment must resist abrasion from large stone particle movement on its surfaces and from small particles that penetrate its fiber matrix. If the geotextile is installed at too shallow a depth the tamper tines will cut and tear the geotextile, as seen in Figure 2. At depths just below the penetration of the tamper tines the ballast is agitated during tamping. At even greater depths particle movement, although less abrasive, still occurs. Geotextiles in a track environment must clearly be abrasive resistant. An assessment of a geotextile's ability to resist abrasion was initially reported by Van Dine et al. (*8*) and extended by Costa and Raymond (*9*). Costa and Raymond recommended testing in the laboratory using the Taber Abrasor (ASTM D-3884, Rotary Platform Double Head Method) fitted with two H-18 Calibrade stones, each carrying a 1000-g mass. Figure 7 shows the results of abrasion testing on a 1000-g/m^2 needle-punched polyester fiber geotextile using different percentages of resin treatment. Clearly even a little resin caused a major increase in abrasion resistance. The porosity of the untreated geotextile is about 85 percent. Because the resin has a specific gravity similar to polyester the resin treatment has little effect on the geotextile's porosity and in-plane permeability. Excess resin makes the geotextile too stiff to handle and install in the field, limiting the amount to 20 percent by weight.

The degree of needle-punching a nonwoven needle-punched geotextile receives during manufacture determines the amount of mechanical interlock between fibers, which influences the geotextile abrasion resistance. Results were obtained from abrasion tests performed on a number of geotextiles manufactured using a variable amount of needle-punching. While needle size, needle penetration, and other factors are manufacturing variables, the amount of needling was found to be important. From the test results a minimum of 80 penetrations per square centimeter (80 p/cm^2) should be specified. Further factors investigated included fiber strength and length. For the best mechanical fiber interlock a 100-mm minimum was established along with a minimum fiber strength of 40 μN/tex.

In conclusion both the amount of needling during manufacture and the resin treatment increase the abrasion resistance of geotextiles subject to particle penetration.

Impact Resistance

When ballast is placed it is dropped about 1 m onto the geotextile. No evidence was obtained that suggested that geotextiles that meet an abrasion criterion would not be suitable to resist impact.

Elongation

Undercut ballast has protruding aggregate particles, as illustrated in Figure 3. Track geotextiles must be able to elongate around

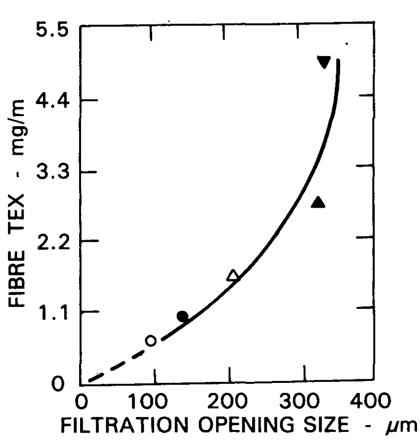

FIGURE 6 Effect of polyester fiber tex on measured filtration opening size of similarly manufactured 600 g/m^2 mass/unit area nonwoven needle-punched (80 p/cm^2) geotextiles.

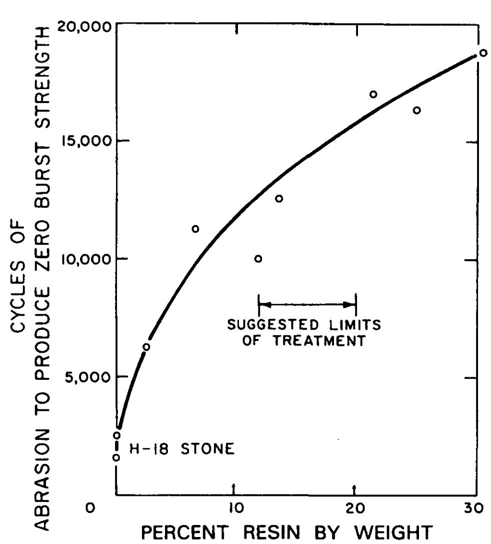

FIGURE 7 Cycles of abrasion required to cause zero burst strength for a 1000 g/m^2 polyester geotextile treated with different percentages by weight of resin.

these particles. If a nonwoven geotextile is reinforced with a woven scrim, protruding particles were found to rupture the scrim. The scrim, however, prevented adjoining portions of the geotextile from elongating and the particles penetrated through the geotextile, as seen in Figure 8. On the other hand exhumed nonwoven geotextiles that elongate 60 percent or more in a grab test (ASTM D-4632) were found to be unruptured.

In conclusion a track geotextile should elongate 60 percent or more in a grab test (ASTM D-4632).

Chemical Resistance and Polymer Type

Little is known about the long-term chemical effects on polymer types in a track environment, in particular the effects of long-term seepage of fuel oil and herbicides common to most track. Resin treatment is believed to protect fibers from the detrimental effects of track pollutants. The only known data relating to the time/traffic deterioration of geotextiles in a track environment were obtained from Conrail's test sections at Loudonville and Salem, Ohio. Mullen burst test (ASTM D751) results from samples extracted at 6-month intervals after installation are shown in Figure 9. Figure 9 shows the burst strength ratios (exhumed burst strength–initial burst strength) against time. The data were subject to regression analysis (including the initial 100 percent value), and the regression fits are shown along with their statistical significance in the figure. Examination of plotted data shows that only well-needled polyester fiber materials showed little to no loss of burst strength and were statistically significant. These were Truetex VT5000 and Terrafix 370 (*RS* at r^2 = 99 percent significance) from Loudonville, and True Temper 900 (at r^2 = 99 percent significance) and True Temper 700 (at r^2 = 90 percent significance) from Salem. The other polyester fiber materials were the three Bidim products that were manufactured as spun-bonded geotextiles with only a small amount of needle-punching. All the other geotextiles, made partly or wholly from other than polyester fiber, recorded between 25 and 70 percent loss in burst strength. The test data for these other geotextiles showed considerable scatter, having a statistical significance between 80 and 90 percent.

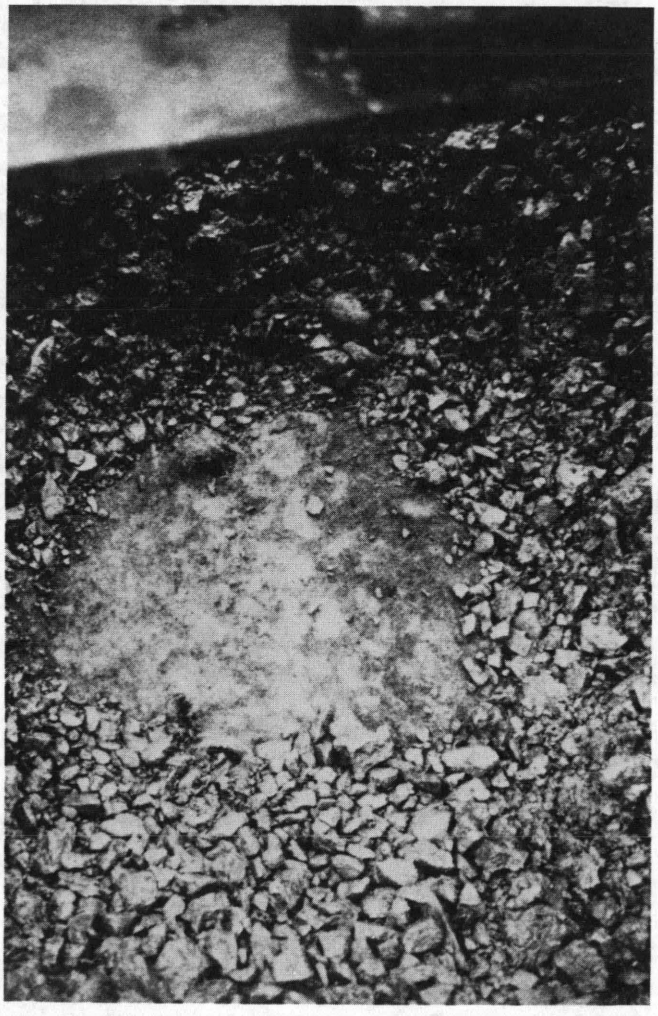

FIGURE 8 Ballast particle penetrating a 1000 g/m² composite (nonwoven with woven scrim) geotextile due to woven's inability to elongate.

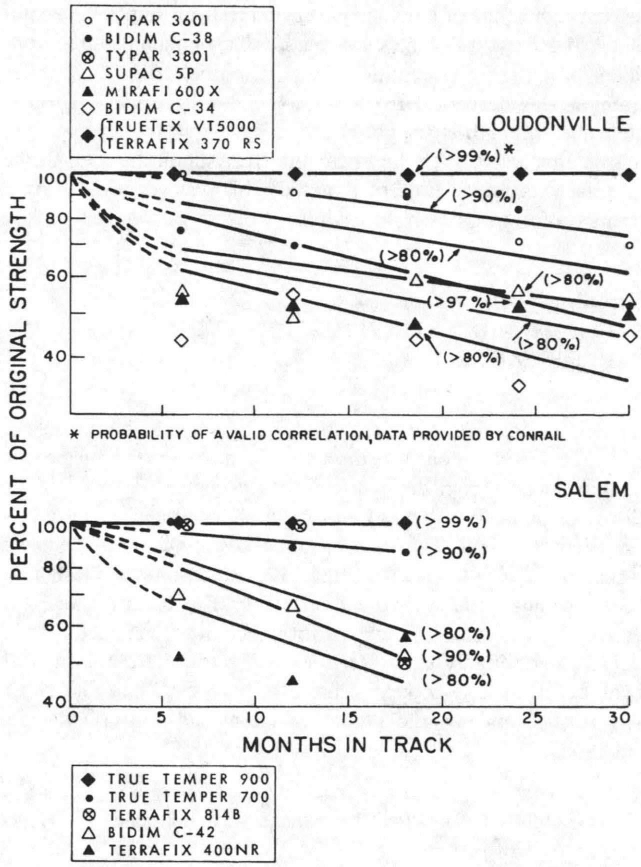

FIGURE 9 Relationship of time variation on Mullen burst strength of exhumed geotextiles from Conrail's two sites.

It is concluded from these initial data that the most environmentally stable geotextiles for these two Ohio sites were manufactured as well-needled geotextiles from polyester fibers.

Fiber Strength

The unit strength or failure stress of the individual fiber is perhaps the key to a major portion of a geotextile's strength and abrasion resistance. Results of laboratory abrasion tests on similarly needle-punched geotextiles of the same mass/unit area manufactured from fibers of different strengths showed a tenfold loss as the fiber strength was decreased from 40 μN/tex to 10 μN/tex. Clearly, high fiber strength is important.

It is concluded that a high-strength fiber should be used in the manufacture. At the time of the study this was represented by a strength of not less than 40 μN/tex.

RECOMMENDED SPECIFICATION AND PERFORMANCE

In 1981 a possible track rehabilitation geotextile based on the previous considerations was discussed with a number of geotextile manufacturers. From these discussions a geotextile specification (listed in point form below) was recommended and was used by Canadian Railways. This has been the basis of about 50 geotextile installations per year for more than 12 years. Some of these have been exhumed and all were intact (*10*). The ballast above the geotextile was clean, track performance was satisfactory and judged to be highly cost-effective, and geotextile durability has been excellent. The geotextile has also been used on other North American Company tracks with excellent and cost-effective results (*11*).

- Type: Needle-punched nonwoven with 80 penetrations per cm^2 (800 p/cm^2) or greater.
- Fiber size: 0.7 tex or less.
- Fiber strength: 40 μN/tex or greater.
- Fiber polymer: Polyester.
- Yarn length: 100 mm or greater.
- Filtration opening size: 75 μm or less.
- In-plane coefficient of permeability: 50 μm/sec or greater.
- Elongation: Sixty percent or more to ASTM D4632.
- Seams: No longitudinal seams permitted.
- Color: Must not cause "snow blindness" during installation.
- Packaging: Must be weatherproofed and clearly identified at both ends stating manufacturer, width, length, type of geotextile, date of manufacture.
- Wrapping: 8-mil black polyethylene or similar.
- Abrasion resistance: 1050 g/m^2 geotextile must withstand 200 kPa on 102-mm burst sample after 5,000 revolutions of H-18 stones each loaded with 1000 g of rotary platform double-head abrasor (ASTM D3884).
- Width and length without seaming: To be specified by client.
- Fiber bonding by resin treatment or similar: 5 to 20 percent by weight low-modulus acrylic resin or other suitable non-water-soluble resin that leaves the geotextile pliable.
- Mass: 1050 g/m^2 or greater for track rehabilitation without the use of capping sand.

CONCLUSIONS

From the 1981 surficial inspection of Canadian tracks it is clear that if a nonwoven geotextile were used for track rehabilitation, it would need to have a mass/unit area greater than 500 g/m^2. This assessment was confirmed by the excavations at other locations, such as those presented herein for Loudonville, Ohio.

Excavations made on Canadian tracks of geotextiles have a mass/unit area of at least 500 g/m^2 confirmed this conclusion, as even when 500 g/m^2 samples were exhumed at a depth of 350 mm there was damage. This assessment was confirmed by the excavations at other locations, such as those presented herein for Salem, Ohio.

From the excavations made on Canadian tracks in the summer and fall of 1981 it was concluded that geotextiles should be installed at a depth of 250 to 300 mm below the tie base and tamping should not be permitted until that depth of ballast was in place. The conclusion was confirmed by later excavations made at numerous other sites, in Canada, the United States, and other areas of the world.

The excavations at Salem, Ohio, indicated that a mass/unit area of at least 900 g/m^2 is needed for a rehabilitation geotextile to remain durable to function as a separation and filtration layer.

In addition, laboratory studies were carried out and observations made of geotextile installation techniques. All these factors resulted, in late 1981, in a geotextile specification for a track rehabilitation geotextile, the main requirements of which are listed in the previous section.

The specification has been in use since 1981 in Canadian National Rail's Maritime Region, where about 50 installations have been made annually. Excellent performance has been obtained, and the few excavations made to exhume geotextiles manufactured to this specification have all shown complete geotextile integrity. Geotextiles installed without a capping sand, meeting the specification, are showing excellent durability after 12 years of service in the physically harsh environment of North American track.

ACKNOWLEDGMENTS

The work reported forms part of general studies funded partially by Canadian Pacific Rail, Canadian National Rail, Via Rail, and Transport Canada Research and Development Center through the Canadian Institute of Guided Ground Transport and partially by the Natural Sciences and Engineering Research Council of Canada. Thanks is extended to Conrail and other rail companies for permission to visit their sites containing geotextiles.

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Durability of Geotextiles Used in Reinforcement of Walls and Road Subgrade

BILL POWELL AND JOHN MOHNEY

The USDA Forest Service began using geotextiles in the early 1970s, actively contributing to the early stages of technology development. As part of that development, two test projects were completed in the Olympic National Forest in the state of Washington. One project evaluated geotextiles used as reinforcment layers in a "fabric wall" and the other evaluated road subgrade reinforcement. The test wall was constructed in 1975 using two different nonwoven materials, one polypropylene and the other polyester. To help evaluate the long-term durability, samples were retrieved in 1993 and tested to determine changes in strength. Also the original wall design was compared to current design standards. The test road sections were constructed in 1976 using nonwoven fabric and were used for heavy timber hauling during 1977. In 1978 samples of fabric were extracted from the road and retested. Samples were extracted in 1993 to conduct tests for determining the existing fabric strength, which should provide an indication of the long-term durability of the materials. Testing was completed under the FHWA Pooled Fund Study for durability of geosynthetics for highway applications. Test results indicate a strength loss ranging from 20 to 50 percent. Most of the strength loss appeared to be a result of the angular fill material piercing the geotextiles. The FHWA design procedure for walls is more conservative than the procedure used for the test wall, primarily due to the creep reduction and lateral resistance factors of safety.

Geotextiles have been used for construction on USDA Forest Service projects since the early 1970s. The early Forest Service work resulted in development of comprehensive standards and guidelines published in 1977, *Guidelines for Use of Fabrics in Construction and Maintenance of Low-Volume Roads* (*1*). As part of the Forest service's technical development of the use of geotextiles, several construction projects were used to test the materials in actual applications. Before this time geotextiles had limited use, so there was little information on the long-term durability of the materials. The intent of this paper is to take two of these projects one step further and report on the durability of the materials since construction. Both projects are in the Olympic National Forest in Washington State. Project 1 is an internally reinforced "fabric wall" and Project 2 is a reinforced saturated subgrade for a logging road. Extensive testing and documentation was conducted on these projects at the time of construction because they were used as test structures for evaluating geotextile materials. Additional samples were obtained on both projects in May 1993 to determine the loss in strength over time.

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PROJECT INFORMATION

Fabric Wall

In conjunction with Oregon State University (OSU), the Forest Service completed the design and construction of a retaining structure used to correct an unstable fill on an existing Forest Service road in 1975. The wall had a maximum height of 5.6 m (18.5 ft) and the total length was 50 m (166 ft). Two types of geotextiles were selected for evaluation: a nonwoven polypropylene (Fibertex) and a nonwoven polyester (Bidim). Two different material thicknesses were selected for each geotextile type. Material properties are given in Table 1, and summaries of the 1993 tests are presented in Table 2.

Design Parameters

Design parameters are as follows:

- Live load: 50 000-kg (110,000 lb) loaded log truck.
- Backfill: crushed rock, angle of internal friction = 40 degrees, dry density = 2 g/cm^3 (125 lb/ft^3).
- Dead load: at rest pressure.
- Foundation: moist unit weight = 2.08 g/cm^3 (130 lb/ft^3), angle of internal friction = 35 degrees, cohesion = 0.

Wall design was completed in 1975 using the same standards outlined in *Guidelines for Use of Fabrics in Construction and Maintenance of Low-Volume Roads* (*1*). Detailed information for the design and monitoring of the wall is documented in the guidelines. The wall is 3.7 m (12 ft) in width, and the vertical spacing between geotextiles is 22.9 cm (.75 ft) and 30.5 cm (1 ft) in thickness. Figure 1 is a photograph taken when the wall was nearly complete, with emulsified asphalt being sprayed on the exposed front face to be used for ultraviolet protection. A cross-section of the wall is shown in Figure 2.

Using the same geometry and materials testing information, the wall was analyzed using today's design standards from *FHWA Reinforced Soil Structures, Volume 1, Design and Construction Guidelines* (*2*). The width of the wall would be the same (3.7 m) but it would require more layers or higher strength geotextiles. The layer thickness would be 30 percent less with the polyester (Bidim) and 65 percent less with the polypropylene (Fibertex). There would be 1.4 times as many layers with polyester and 2.8 times as many layers with polypropylene. To make this comparison, correlations were made among different testing methods used to determine strength. Extensive testing was completed for

TABLE 1 Material Properties of Fabric Wall

Trade Name	Thickness mils	Weight gm/m^2	1975 Grab Test ASTM D-1682 Mach Dir lbs.	1975 OSU Ring Test Mach Dir lbs./in	1993 Grab Test ASTM D-4632 Mach Dir lbs.	Strength Reduction 1975-93 %
Bidim C-28	95	200	198	62	99	50
Bidim C-38	114	420	--	106	--	--
Fibertex 420	190	420	550	67	327	40
Fibertex 600	250	600	--	95	--	

the original fabric wall design. The OSU ring test used in the wall design produced values that were about 33 percent of the grab test (ASTM D1682). The new design standards recommend using the wide width test (ASTM D4695-86), which is about 50 percent of the grab test using product information testing results or about 35 percent higher allowable strength than for the original design (OSU ring test result).

It is difficult to directly compare the design methods because different parameters are used in each method. The new design standards have higher requirements for reinforcement materials due primarily to the strength reduction factors for construction (creep and durability) referred to in Task Force 27 (*2*) (see Figure 3). Although the Olympic National Forest wall would be substantially underdesigned by today's design standards, it does not show any signs of distress after 18 years of use. The emulsified asphalt used to protect the front face has deteriorated and small holes in the fabric face are starting to appear. Although the holes are on the face, there are no deformities or loss of materials on the wall face. In time the face will be repaired by shotcrete sprayed on the surface.

Subgrade Reinforcement

The Quinault Test Road Project, located on the state of Washington's Olympic Peninsula, was constructed in October 1976 (*1*). As with the fabric wall, extensive testing was conducted as part of this project. During 1977 the road was used for log hauling. In

TABLE 2 1993 Fabric Wall Test Results, Site 5

Material	Weight gm *	Tensile Force, lbs	St. Dev.	lb.	Coeff. Var %	Strain %	Comments
	5.1	88	Orig			62	
	5.22	85	1975	10.6	5.4	74	
5A1	4.8	94				72	Layer 1
	5.02	96				78	
Bidim	6.79	135	1993	19.0	19.8	89	Ave. = 96
200	5.67	105				72	Range = 69 - 135
	5.47	99				69	
	4.72	69				65	
	5.7	87	Orig			48	
	5.2	83	1975	10.6	5.4	53	
5A2	4.75	122				64	Layer 2
	5.34	119				61	
Bidim	4.94	118	1993	24.7	24.2	56	Ave. = 102
200	5.7	55				55	Range = 55 - 122
	5.3	122				65	
	6.61	113				60	
	7.68	287	Orig			118	
	8.1	355	1975	15.1	2.8	169	
5B2	8.1	346				164	Layer 2
	8.6	333				125	
Fibertex	7.33	318	1993	27.9	8.5	158	Ave. = 327
420	8.2	327				134	Range = 279 - 358
	8.84	358				150	
	7.32	279				121	
	8.35	336				160	

* 4"x8" Sample Size

FIGURE 1 Geotextile wall, with exposed material being sprayed with emulsified asphalt after construction.

1978 the test sections were excavated to obtain geotextile samples for comparative strength testing. These test results have been reported (*3*), giving an indication of the construction damage sustained by the geotextiles. In May 1993 the test sections were again excavated to obtain geotextile samples for strength testing. These tests provide information on the deterioration of the geotextiles in the ground during the 15 years since the 1978 tests. Tables 3 and 4 present a summary of strength testing. Figure 4 shows rockfill material being placed on geotextile during initial construction.

Geotextiles used in the test sections were those available in the northwestern United States at the time of construction. These were

- Bidim needle-punched nonwoven polyester—150, 325, 420 g/m^2.
- Fibertex needle-punched nonwoven polypropylene—320 and 420 g/m^2.
- TYPAR heat-bonded nonwoven polypropylene and nylon—140 g/m^2.
- Mirafi heat-bonded polypropylene.
- SUPAC needle-punched polypropylene—140 g/m^2.

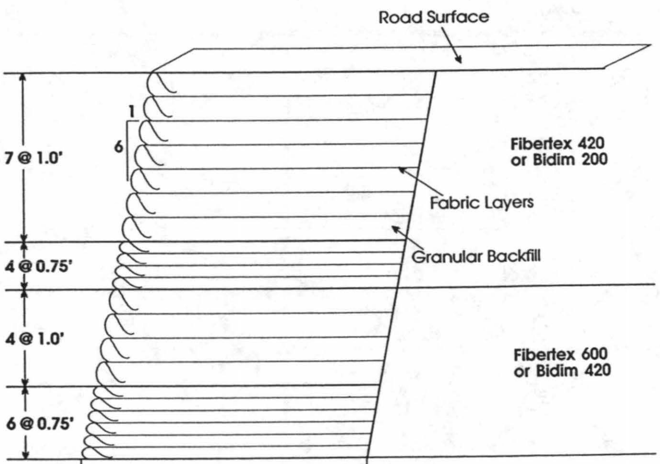

FIGURE 2 Cross-section of fabric wall.

$$X = \frac{S}{FS\,\sigma_h} = \frac{.33\ (\textit{Grab Strength})}{(1.75)\ (130)\ (.36)\ \textit{depth}} = \frac{.004\ (\textit{Grab Strength})}{\textit{depth}}$$

Where
X = Layer Spacing
S = Fabric Strength
OSU Ring Test = 33% of Grab Test
σ_h = Horizontal Stress (at rest K_o)
F.S. = 1.75 (Factor of Safety)

$$S = \frac{T_d}{\sigma_h} \qquad T_d = \frac{T_L\ CRF}{C_c\ C_r\ C_v}$$

$$= \frac{T_d}{(\textit{depth})\ (.22)\ (130)}$$

$$= \frac{.0014\ (\textit{Grab Strength})}{\textit{depth}}\ (\textit{polypropylene})$$

$$= \frac{.0028\ (\textit{Grab Strength})}{\textit{depth}}\ (\textit{polyester})$$

Where
T_d = Design Tensile Strength
T_L = Wide width Geotextile Strength
CRF = Creep Reduction Factor
= .2 for Polypropylene
= .4 for Polyester
Cc = Factor of Safety for construction
= 1.1 to 1.3
C_r = Factor of Safety for Durability
= 1.1 to 2.0
C_v = Uncertainty
= 1.5
σ_h = Horizontal Stress (active Pressure, K_a)

FIGURE 3 Layer spacing for fabric wall: *top*, Forest Service (*1*); *bottom*, FHWA (*2*).

Subgrade soils were highly organic clays and silts (Unified Soil Classification OH) with California bearing ratio (CBR) values less than 1. The geotextiles were placed directly on the subgrade. Then 0.5 to 1 m (2 to 3 ft of "shot rock" fill was dumped on the geotextile and spread at full depth using a Caterpillar D-6 dozer. The rock fill was surfaced with 0.2 to 0.3 m (8 to 12 in.) of glacial pit run gravels.

The original design called for 0.7 m (2 ft) of rock fill, and the rock contained pieces up to 1 m (3 ft) in size. Because of this the rock fill was generally thicker than the design and actual stresses under load were less than designed. Questions of construction damage and long-term durability can be answered, however. Traffic consisted of rock-hauling trucks during construction, and highway-legal (36 000-kg or 80,000-lb gross vehicle weight) log trucks during logging.

DURABILITY SAMPLING AND TESTING

In May 1993 samples of the buried materials were retrieved by excavating into the embankments of both the wall and the road. The excavation was started using a hydraulic excavator. To prevent damage to the geotextiles, final excavations were completed by hand digging. The samples were retrieved in cooperation with Earth Engineering and Sciences, the FHWA contractor for the Pooled Fund Study, then wrapped in black plastic. The 1993 samples retrieved were Bidim 150, Fibertex 420, Mirafi 140, Typar 140, and Supac 140. These samples were tested using the 102-mm (4-in.) grab test (ASTM D1682). The tests were performed at Polytechnic University of Brooklyn using the same test grips used for the original tests to limit the amount of testing error.

RESULTS

Fabric Wall

Test results for the wall are displayed in Tables 1 and 2. The test results are consistent, with minor variations. The loss in strength

TABLE 3 Results of Grab Tests, Quinault

Fabric	Wt. gm/m^2	4" Grab Test, lbs. 1976	1978	1993	% Lost 1976-78	% Lost 1978-93
Bidim	150	234	135	124	42	5
Fibretex	420	550	395	339	28	10
Mirafi	140	124	112	97	10	12
Typar	140	144	119	116	18	2
Supac	140	86	93	92	--	--

TABLE 4 Subgrade Reinforcement Test Results, Site 10

Material	Weight gm *	Tensile Force, lbs	St. Dev.	lb.	Coef. Var %	Strain %	Comments
	13.42	360	Orig			183	Ave. = 339
	13.18	345	1976	15.1	2.8	196	Range = 317- 360
A	14.01	352				185	
	12.92	317	1978	88.3	22.4	168	
Fibertex	13.25	329				179	
	11.72	240	1993	14.8	4.4	126	Slippage
	12.7	338				149	
	12.7	330				146	
	4.42	101	Orig			61	Ave. = 116
	3.6	125	1976	18.2	12.6	81	Range = 101 - 125
B	3.52	116				88	
	3.5	122	1978	18.6	15.7	90	
Typar	3.42	105				64	
	3.44	116	1993	8.6	7.4	68	
	3.31	86				40	Broke in jaw
	3.38	121				74	
	3.3	121				61	
	3.33	99	Orig			128	Ave. = 97
	3.54	102	1976	11.2	10.0	121	Range = 79 - 105
C	3.74	79				122	
	3.31	91	1978	18.0	16.1	129	
Mirafi	3.4	105				153	
	3.12	104	1993	9.1	9.4	102	
	3.14	76				77	Broke in jaw
	3.34	90				120	
	2.93	102				162	
	4.15	88	Orig			59	Ave. = 93
	3.98	100	1976	10.8	7.7	79	Range = 80 - 109
D	3.99	80				59	
	4.3	86	1978	8.9	9.6	65	
Supac	4.24	109				82	
	3.93	85	1993	9.6	10.3	69	
	4.05	93				73	
	3.35	68				85	Broke in jaw
	5.15	99				86	
	4.51	115	Orig			90	Ave. = 124
	4.51	113	1976	11.2	4.8	60	Range = 113 - 142
E	4.63	124				72	
	7.03	86	1978	10.0	7.4	57	Slippage
Bidim	7.18	88				39	Slippage
	5.1	142	1993	11.6	9.4	48	
	8.26	119				60	
	10.92	132				55	

* 4"x8" Sample Size

FIGURE 4 Dozer spreading rockfill on geotextile.

during the 18 years since the wall was built was 50 percent for the Bidim (nonwoven needle-punched polyester) and 40 percent for the Fibertex (nonwoven needle-punched polyester) and 40% for the Fibertex (nonwoven needle-punched polypropylene). Although there was a reduction in material strength, the wall has performed well and the original design is probably appropriate. If we consider that all losses of strength measured in the initial construction are due to construction damage, differences in the designs are a function of the creep reduction factors and the factor of safety for uncertainty. It would therefore be appropriate to further evaluate the creep reduction factors in a confined mode and conduct wall monitoring aimed at evaluating movement and stresses in the materials.

The original material testing done in 1975 had a low coefficient of variation: 5.4 percent for the Bidim 200 and 2.8 percent for the Fibertex 420. The coefficient of variation increased significantly for the 1993 tests: 19.8 percent and 24.2 percent for the Bidim and 8.5 percent for the Fibertex (see Table 2). Based on these results it could be concluded that the 1993 tests cannot be used to indicate chemical degradation of the geotextiles. The 1993 coefficients of variation for the Bidim and Fibertex from the wall are consistent with the data obtained in 1978 from the subgrade that was for construction damage. The material that was retrieved in 1993 had numerous holes caused by punctures from the sharp, angular rock material in the fill. Additional sampling may reduce the coefficient of variation, but not significantly enough to warrant a different conclusion.

Subgrade Reinforcement

The test results are summarized in Tables 3 and 4. Initial construction damage resulted in a loss of strength of 10 to 42 percent of the initial strength. It is interesting to note the two lightest weight geotextiles sustained less damage than the heavier geotextiles.

During the 15-year period from 1978 to 1993 an additional loss of strength ranged from 2 to 12 percent. Total strength loss from construction and long-term degradation was 20 to 47 percent for all geotextiles.

Generally the standard deviation of test results is consistent except for the Fibertex (see Table 4). The rock used in the road construction was larger than the material used in the fabric wall, but the individual particles were either rounded or angular and soft. Also there was a large amount of fines mixed with the rock, which had a cushioning effect, and there seemed to be substantially less damage to the fabric during construction.

Judging from the coefficient of variation, the 1978 and 1993 tests only varied slightly for the Supac and Bidim geotextiles, and it could be concluded that the 1978 to 1993 loss of strength of 8 percent for Bidim could be attributed to hydrolysis. No detectable loss of oxidation for the polypropylene could be inferred from the data by considering the coefficients of variation measured.

Although the geotextiles have a reduced strength they are still functioning as designed. Because geotextile strength is not an input in the design procedure, a large loss of strength is not critical to performance. The primary purpose of a strength requirement for subgrade reinforcement geotextiles is to ensure construction survivability. Stresses from traffic loads have not caused failure of the geotextiles.

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Laboratory Evaluation of Geosynthetic-Reinforced Pavement Sections

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Preliminary experimental and analytical investigations were conducted to evaluate the performance of pavements with and without geotextile or geogrid reinforcement materials. Four pavement sections were tested: one unreinforced section that served as a control and three sections reinforced with either one of two geotextiles or a geogrid. The pavement sections were constructed to model a typical secondary road in Virginia built over a weak granular (silty sand) subgrade material. Loading of the pavement sections was accomplished through the use of a computer-controlled pneumatic system that delivered approximately 55 kPa through a 30-cm rigid plate at a frequency of 0.5 Hz. The resulting displacement of the pavement surface was monitored by an array of linear variable displacement transformers. The performance of each pavement section was evaluated as a function of the applied number of cycles, the resulting surface deflection profile, and the layer deflection profile. It was concluded that geosynthetics can substantially improve the performance of a pavement section constructed on a subgrade soil with a low California bearing ratio. Also the reinforcing mechanisms of geogrids and geotextiles are different.

Geosynthetics have long been recognized as materials that can significantly improve the performance of paved and unpaved roads, especially those constructed on weak subgrades. Geogrids and geotextiles are the two types of geosynthetic most widely used in pavement systems. Geotextiles consist of synthetic fibers that are either woven into flexible, porous sheets or matted together in a random, nonwoven manner. Goegrids are usually manufactured from polypropylene, high-density polyethylene (HDPE), or high-tenacity polyester.

The first industrywide design standards for geotextiles were not established until Giroud and Noiray's landmark 1981 paper (*1*), almost 50 years after their first documented application in the United States. Until the mid-1980s the design of geosynthetic-reinforced pavements was poorly documented and based on empirical evidence. In 1985 the *Geotextile Engineering Manual* was published by FHWA (*2*). In 1990 Koerner published *Designing with Geosynthetics*, which explained the mechanical properties of geogrids and geotextiles on the basis of contemporary information (*3*).

The key functions of geotextiles in improving flexible pavement performance are separation, reinforcement, and filtration. Reinforcement is the most important function of geogrids. Through separation geotextiles inhibit two mechanisms that tend to occur simultaneously over time in pavements: soil fines attempt to migrate into the voids between the base course stones, thereby affecting the drainage capability of the pavement system and its structural capacity; and the stone attempts to penetrate into the soil, compromising the strength of the stone layer (*2*). In order to achieve proper separation geosynthetics should be designed for burst resistance, tensile strength requirements, puncture resistance, and impact resistance.

Through reinforcement geosynthetics distribute a concentrated load over a larger area of the subgrade and improve the strength of pavement systems built on weak soil or other disjointed and separated material (*3*). The dual mechanisms of reinforcement are a further spreading of the load to the subgrade, providing a more stable support condition; and development of an appreciable amount of tensile stress resistance in the fabric. If the geosynthetic has a sufficiently high tensile modulus, the tensile stress resistance may reduce the plastic deformation of the subgrade soil caused by vehicular loading (*2*).

Finally through the filtration mechanism geosynthetics may inhibit generation of excess pore pressures in the subgrade and may prevent migration of the subgrade fines migration into the base or subbase. The pore water pressure in the soil usually increases as a result of dynamic loading. At the point at which the pore pressure is greater than the total stress of the soil, a soil slurry is formed. When designed with the correct permittivity, geosynthetics filter the soil particles and pore pressure is allowed to dissipate. Christopher and Holtz (*2*) reported a case in which geotextiles were applied incorrectly and designed inadequately, which led to pore water pressure development and pumping in subgrades.

Reports from various studies indicate that pavement strength can be increased by placing geosynthetics at the hot-mix asphalt (HMA) base course interface, in the base course layer, at the base course–subgrade interface, and in HMA overlays to strengthen existing pavements. Although the resulting improvement in the pavement systems has not been well quantified, it has been reported that reinforced pavement strength increases as the position of the geosynthetic approaches the base course–subgrade interface (*4–6*). In general geosynthetics are said to increase initial stiffness, decrease creep, increase tensile strength, reduce cracking, improve cyclic fatigue behavior, hold cracked pieces together, and provide low life-cycle cost.

Many studies on the importance of geogrids have been reported recently (*1,5–12*). The studies conducted at the Royal Military College in the United Kingdom; the Ontario Ministry of Transportation and Communication in Canada; Gulf Canada, Ltd.; and the University of Waterloo (*4,7,8*) suggested that geogrids provide substantial savings in HMA thickness, double the number of load repetitions, prevent or minimize fatigue cracks in the HMA layer, and reduce permanent deformation in flexible pavement systems. Geogrid-reinforced pavement sections have been reported to carry three times the number of loads as conventional unreinforced pavements, and geogrid reinforcement allowed up to 50 percent reduction in the required thickness of the base course. Webster (*9*)

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evaluated the performance of geogrids in reinforcing flexible pavements, using a 134-kN single-tire load, to develop design criteria for reinforced flexible pavements used by light aircraft. He reported that using geogrid at the base course–subgrade interface would decrease the required base course thickness. This benefit was reported to decrease for stronger pavement sections.

Research on geotextiles has been less intensive. DeGiardiel and Javor (*13*) concluded from their study that the effectiveness of geotextiles increases with increasing deformation and suggested that a double layer of fabrics yielded the largest amount of subgrade strengthening. Resl and Werner (*14*) concluded from their study that the benefit of geotextiles is derived instead from their characteristics in separation, filtration, and drainage.

Case histories such as the Pan American Highway (*15*) have also shown the importance of the separation mechanism of geotextiles. Saxena (*16*) used pretensioned geotextiles to reduce potential rutting in a major roadway project in Florida. A field application combining both geotextiles and geogrids was reported to provide the benefit of both geotextile and geogrid mechanisms (*17*). Austin and Coleman (*10*) evaluated four types of geogrid, a geotextile, and a geogrid/geotextile for pavement reinforcement. The study showed that the geotextile performed better than the other systems, with the exception of one geogrid-reinforced section constructed on a subgrade with a higher California bearing ratio (CBR) value. The study emphasized the importance of geotextile as a separator.

In a comparison study of geotextile and geogrid performance, Barksdale et al. (*6*) reported that permanent deformation can be reduced substantially if geosynthetics are used on weak subgrades to reinforce thin pavement layers. The study suggested that under the testing conditions used the performance of geogrids is better than that of geotextiles and recommended using geotextiles in the middle of low-quality base course material. However the data in the report showed that if the geosynthetic did not mobilize its strength and separation was the mechanism that provided the performance enhancement (which is more likely), the geotextile would perform better. Prerutting was also found to improve the performance of geosynthetics. In all cases proper application is critical.

Field evidence suggests that both geogrids and geotextiles can improve the performance of pavement sections constructed on weak soil; however, it remains difficult to quantify the benefits that result from the application of these geosynthetics. In the absence of such quantification, a cost comparison is not possible. Also the mechanisms by which these materials enhance the performance of pavement sections is poorly understood.

The purpose of this ongoing research is to investigate pavement life-cycle improvement when geotextiles and geogrids are used to reinforce pavement cross-sections. Four pavement test sections were constructed to model typical secondary roads built over weak silty sand subgrades; one was a control section and the other three were reinforced with geosynthetics. Simulated traffic loads were applied and the performance of each test section was evaluated. This paper presents a detailed report of the experimental methods and a preliminary analysis of the results.

EXPERIMENTAL PROGRAM

Four different pavement sections were constructed in a reinforced concrete testing pit. One test section was unreinforced (the control section), two were reinforced with geotextiles, and one was geogrid-reinforced. The test sections were built to model typical secondary roads constructed on a weak granular subgrade material. Following the construction of each section the pavement surface was dynamically loaded via a rigid plate and the resulting displacement was continuously monitored and recorded. The following paragraphs describe the composition and construction of the test sections and the pavement loading system.

Test Facilities

The testing program was conducted at Virginia Polytechnic Institute and State University's (Virginia Tech's) Price's Fork Geotechnical Research Center. The Instrumented Test Facility at the research center was constructed for previous experimental programs (*18,19*). The facility's dimensions are 3.1 × 1.8 × 2.1 m deep, with the test pit floor located 1.2 m below grade. A schematic cross-section of the pit and pavement is shown in Figure 1. Access to the pit is gained by a ramp that facilitates soil placement and lift construction. The test pit walls are constructed of reinforced concrete. A load frame secured to the top of the east and west walls of the test pit provides a reaction force for the application of a vertical load of up to 62 kN. In this investigation, only 40 kN were required to load the pavement sections, representing dual-tire loading of an 80-kN axle.

Test Materials

The test sections consisted of a compacted silty sand subgrade, a well-graded gravel base course, and an HMA wearing surface. For the three reinforced sections, a geotextile or geogrid was placed at the subgrade–base course interface.

Subgrade Soil

The subgrade soil was Yatesville silty sand (YSS) obtained from alluvial deposits excavated during the construction of the Yates-

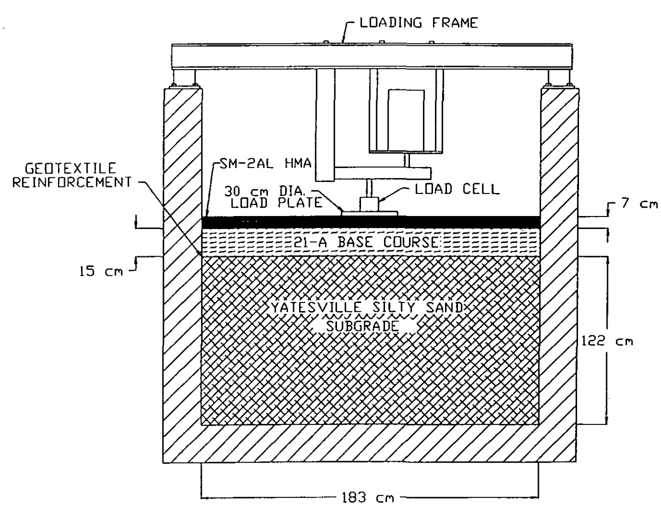

FIGURE 1 Schematic of test pit and pavement test section.

TABLE 1 Moisture-Density Relations for Yatesville Silty Sand (*18*)

Test Type	Compactive Effort (kN•m/m^3)	Maximum Dry Density (g/cm^3)	Optimum Water Content (%)
Mod. Proctor	2690	2.11	8.8
Std. Proctor	592	2.00	10.9
Low Energy	296	1.93	11.6
Very Low Energy	118	1.83	14.0

ville Lake Dam in Lawrence County, Kentucky. It is an A-4 soil, according to AASHTO classification, with a fines content of 40 to 47 percent. The fines are non-plastic, and the specific gravity (G_s) of its solids has been found to be 2.67. Moisture-density relations were established for a variety of compactive efforts, as summarized in Table 1.

The CBR (ASTM D1833-87) was used to characterize the subgrade soil index strength during the testing program. To place the lifts of YSS soil at a specific CBR instead of a standard dry density, it was necessary to evaluate CBR as a function of both compactive effort and the water content for that compactive effort (see Figure 2). To achieve this, two CBR (soaked) testing programs were devised and 43 tests conducted. Based on test results it was determined that it was possible to achieve any low CBR, modeling a weak subgrade material, given careful control of compactive effort and water content.

Base Course Aggregate

Granite aggregate was used to construct the base course in a trial test section. The base course aggregate met the Virginia Department of Transportation (VDOT) specifications for 21-A classification. The aggregate gradation is presented in Table 2. The material's moisture-density relationship was established by performing modified Proctor tests (ASTM D1557-91), and it was determined that the maximum dry density was 2.30 g/cm^3 at an optimum water content of 6.3 percent. The aggregate has a specific gravity of 2.81 and an absorption value of 0.4 percent (ASTM C97-90).

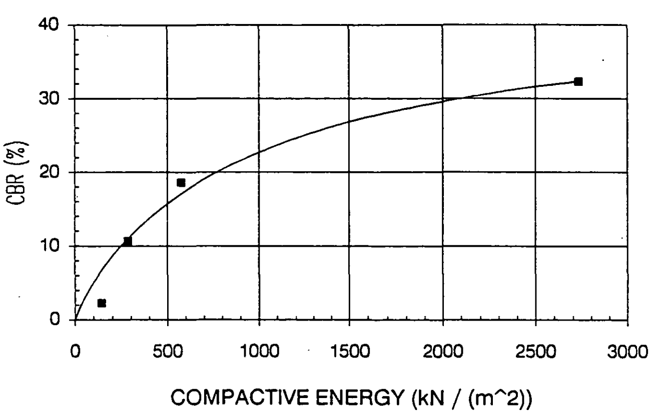

FIGURE 2 CBR as a function of compactive energy at optimum water content for Yatesville silty sand.

TABLE 2 21-A Base Course Aggregate Gradation

Sieve Size (mm)	Percent Passing (%)	Standard Deviation
50.8	100.0	0.0
19.1	90.0	0.8
12.7	74.6	2.9
9.5	65.4	2.7
4.75	48.9	2.2
2.36	39.6	1.9
0.60	28.1	1.4
0.30	19.8	1.4
0.15	13.9	0.8
0.075	7.4	0.6

HMA Wearing Surface

The HMA wearing surface used in the construction of the test sections met the VDOT material specification for SM-2AL, which is the same as that often used on secondary roads in Virginia. The aggregate used in this mix was a crusher-run dolomitic limestone. The asphalt content (AC-30) was 5.9 percent, HMA maximum specific gravity was 2.54, voids in the total mix were 4.5 to 5.1 percent, and voids in the mineral aggregates were 17.5 to 18.9 percent.

Geosynthetic Materials

Geotextile A and Geotextile B were used in two of the reinforced test sections. They have a woven structure and are manufactured from polypropylene. A biaxial geogrid was chosen to reinforce the third reinforced pavement section. This geogrid has a punched and sheet-drawn structure and is also manufactured from polypropylene. Table 3 summarizes the wide width strip tensile data for these geosynthetic materials, as provided by the manufacturers.

Pavement Section Design and Construction

The pavement test sections were designed to reflect the conditions typically encountered when constructing a secondary road over a weak subgrade material having a low CBR. To model these conditions, a subgrade section of YSS 1.22 m thick was compacted at a CBR of approximately 4 percent. The YSS subgrade was placed in uncompacted lifts 20 cm thick at a water content between 12.2 and 12.8 percent, which corresponded to the desired CBR value of 4 percent. Following placement the soil was tilled to break up soil clumps and then raked to a level surface. The

TABLE 3 Wide-Width Strip Tensile of Geosynthetics (ASTM D4595-86)

Type of Reinforcement	2% Strain (N/cm)	5% Strain (N/cm)	Ultimate (N/cm)
Geogrid	54	103	171
Geotextile A	39	89	256
Geotextile B	44	103	344

soil water content was again measured and if it was within acceptable limits the lift was compacted with a hand-operated Wacker model BPU 2440A compactor. A trial-and-error process yielded accurate estimates of the number of passes required to obtain the desired dry density, which was within the range of 1.85 to 1.89 g/cm^3. Each 15-cm-thick compacted lift was surveyed to determine total section thickness and evenness. To verify the water content and dry density each lift was evaluated by performing at least one sand cone test (ASTM D1556-90).

One test section, the control section, was unreinforced. In the other three sections, a geosynthetic was placed, without pretensioning, at the subgrade-base course interface. The three types of reinforcement were Geotextile A, Geotextile B, and the geogrid.

Following subgrade construction and geosynthetic placement, the base course aggregate stockpile was brought to a water content within 1 percent of the optimum value of 6.3 percent. Using procedures similar to those used for the subgrade material, the aggregate was placed and compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557-90. It was found that a loose lift 18 cm thick would compact to a compacted base course layer 15 cm thick. Again the water content and dry density were verified using sand cone tests. Layer thicknesses at different locations were verified by surveying.

Approximately 1.5 tons of HMA were used as a wearing surface on each test section. After delivery by a local contractor the HMA was placed using a front-end loader and hand tools. The HMA was compacted to a density of 2.16 g/cm^3 on the basis of the results of a Troxler Model 3440 nuclear density gauge and core verification. After compacting the HMA to a nominal thickness of approximately 7 cm, the pavement section was surveyed.

Loading and Instrumentation Systems

The pavement loading system was developed through the use of pneumatics and computer control. To simulate traffic loads a force of approximately 40 kN was applied to the pavement surface through a steel plate 30 cm in diameter (550 kPa pressure) at a cyclic rate of 0.5 Hz. The magnitude of the applied load was monitored and recorded through the use of a load cell. Deformation of the pavement surface was monitored and recorded through an array of linear variable displacement transformers (LVDTs).

The loading system used a Bellofram air cylinder to transfer a force, via a lever system, directly to a loading plate 30 cm in diameter placed at the center of the test section surface. The pressure applied to the Bellofram air cylinder was controlled by a Schrader Bellows PAR-15 digital pressure regulator operated by a personal computer with parallel printer interface. With this system virtually any loading pattern could be achieved, limited only by the speed at which air could enter and exit the air cylinder.

The instrumentation system consisted of eight LVDTs and one load cell, as illustrated in Figure 3. The LVDTs were mounted at 15-cm intervals along the longitudinal axis of the test pit and were secured by a frame that was isolated from the motions of the reaction frame and loading system, except for the two LVDTs on the two loading plates, which were mounted 2.5 cm from the edge of the plate. The LVDTs were used to measure pavement surface displacement and the load cell was used to monitor the loads applied by the air piston. The data acquisition system consisted of analog-to-digital converters and multiplexing cards that measured the resulting voltages, converted them to binary format, and stored them in a data file. Measurements were collected five times per second.

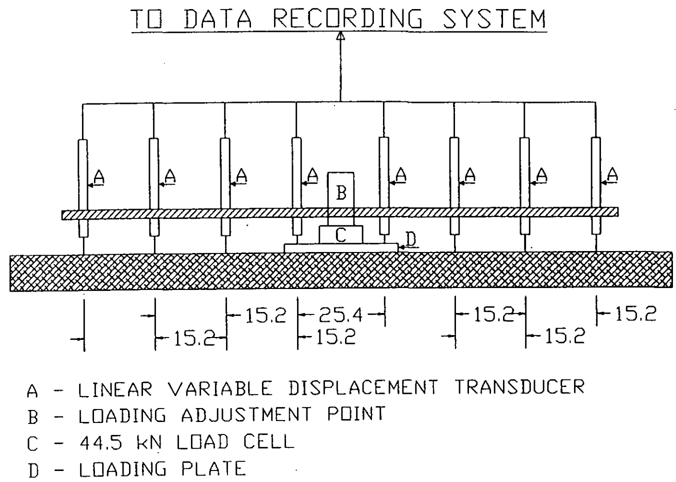

FIGURE 3 Schematic of instrumentation system.

PAVEMENT TESTING PROCESS

After paving a test section at least 24 hr were allowed to pass before loading commenced. During this period the loading system was installed. The cyclic loading was applied in 200-cycle increments, necessary because the loading pins connecting the Bellofram air cylinder and the load cell with the lever arm required adjustment after the pavement surface deflected. This loading process was continued until at least 25 mm of displacement had occurred at the pavement surface beneath the loading plate.

After the loading process the test pit was carefully excavated. The pavement was cut along the centerline and the materials were excavated from the front half of the pit, in layers approximately 15 cm thick, to a depth of 0.6 m. The condition of the final wearing surface, base course aggregate, and the subgrade were inspected, and the thicknesses and displacement profile were measured across the pavement section. Displacement of each layer at the center of the section was measured and is reported in Table 4. As each soil lift was excavated the water content of the lift was checked to verify that no downward seepage had occurred during the period between test section construction and completion of loading.

RESULTS AND ANALYSIS

Performance of the test sections was evaluated by studying the following relationships:

- Effect of loading cycles on displacement,
- Displacement profile at 800 cycles, and
- Displacement progress.

Visual observations and measurements of the excavated profiles were also useful in the evaluation.

TABLE 4 Thickness and Displacement of Each Layer of Evaluated Sections

Reinforcement Type	AVG Applied Press./cycle (KPa)	Subgrade CBR (%)	HMA Thickness (cm)	Base Thickness (cm)	Displace* in HMA (cm)	Displace* in Base Layer (cm)	Displace* in Subgrade (cm)
None	543	4.4	7.6	14.8	0.4	1.2	1.3
Geogrid	525	5.7	7.3	14.5	0.8	0.3	1.3
Geotextile A	525	4.5	7.2	13.4	0.3	0.4	1.9
Geotextile B	550	4.2	7.1	14.6	1.0	0.4	1.2

* Displacement taken at failure, see Table 6.

Effect of Loading Cycles on Displacement

A typical relationship between displacement and loading cycles is presented in Figure 4, showing that displacement increased with the increasing number of cycles, while the rate of displacement decreased. Initial large displacement after the end of the first 25 cycles can be attributed to load seating. A significant displacement was recorded in the first 200 cycles in each test section. At the end of each loading sequence, the pavement continued to rebound, and it was found that this may have continued for as long as 5 min. The rebound was recovered in the first 5 to 20 cycles of the next loading series.

Displacement Profile at 800 Cycles

The instrumentation scheme used for the test sections allowed profiles to be developed of permanent displacement for a given number of cycles, which is particularly valuable in comparing the reinforced sections with the unreinforced control section. Figure 5 shows the profiles of displacement at 800 cycles. Table 5 shows displacements and relative performance illustrated by this graph.

Displacement of the pavement section occurs up to a considerable distance away from the loading plate. This effect was verified by inspection of the cross-sections excavated after loading. Visual inspection indicated that the rutting that occurred in the different layers of the test sections was mostly consolidation rutting. The flow-type rutting observed by Webster (*9*) was minimal in this study. It is also apparent from Figure 5 that a small amount of tilt occurred in the loading plate by 800 cycles. This can be attributed to local variations in the density of the HMA, the base course, or the subgrade.

Displacement Progress

The performance of the test sections was compared by studying displacement beneath the center of the loading plate as a function of the applied number of cycles. This relationship is shown in Figures 6 and 7. Figure 6 plots the test data as measured; in Figure 7, the results are adjusted to account for load seating, which is considered to have occurred in the first 25 cycles.

The figures show that the geogrid and geotextiles all performed comparably, particularly as the number of cycles increased. It also appears that they substantially improved the pavement's resistance to displacement. It can be seen that the reinforcement had an almost immediate effect. For example the control section required only about 25 cycles for the first 12.5 mm of displacement, whereas the reinforced sections required approximately 200 cycles.

The improvement in performance of the reinforced sections can be quantified by comparing either the displacements measured at the loading plate for a given number of cycles or the number of cycles required to produce a given displacement. Table 6 gives the number of cycles required to obtain 25 mm of displacement and the performance of the reinforced sections relative to the control, with and without the effect of load seating.

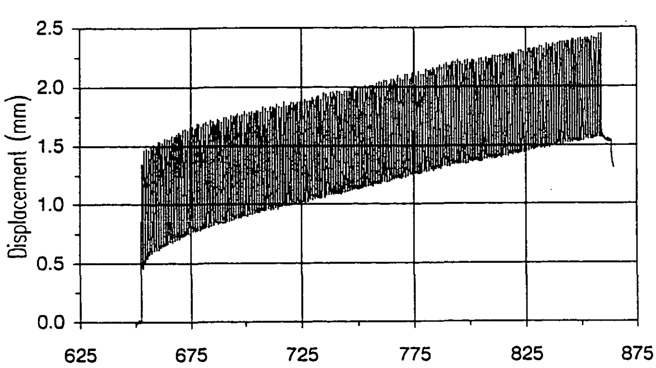

FIGURE 4 Typical relationship between displacement and loading cycles.

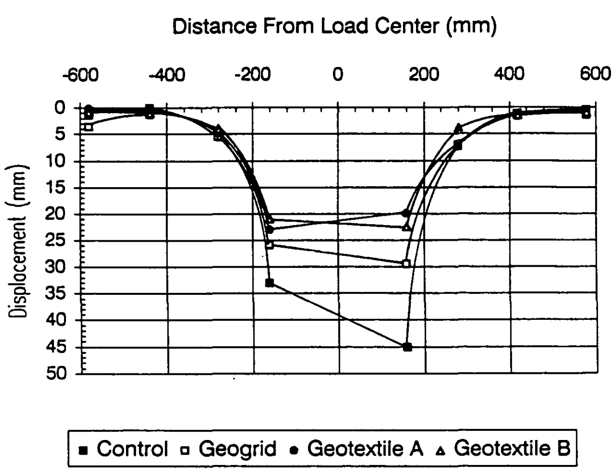

FIGURE 5 Permanent displacement profile at 800 cycles.

TABLE 5 Displacement Profile of Pavement Sections at 800 Cycles

Type of Reinforcement	Displacement (mm)		Improvement Over Control Section (%)	
	Before Load Seating	After Load Seating	Before Load Seating	After Load Seating
Control	38.9	25.7	-----	----
Geogrid	27.2	25.8	43.0	0.4
Geotextile A	21.5	19.9	80.9	29.1
Geotextile B	20.8	19.0	87.0	35.3

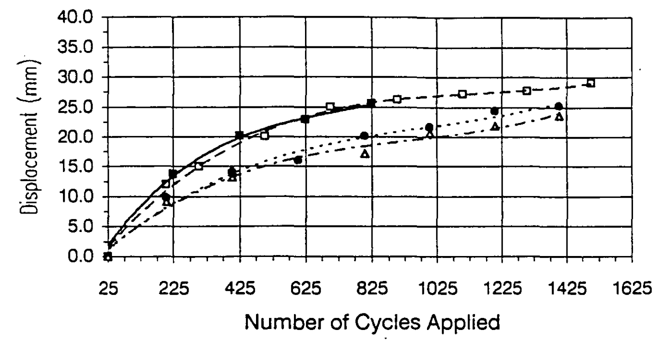

FIGURE 7 Progressive displacement for control and reinforced sections after load seating.

Observation of Excavated Sections

In an attempt to discern the amount of displacement that occurred in the wearing surface, the base course, and the subgrade, measurements were made of the final thicknesses of each of these components during the excavation process. The measurements indicate that most of the total displacement occurred in the subgrade. Table 4 shows the displacement in each layer of the evaluated sections.

An interesting observation was also made regarding the base course–subgrade interface in the excavated profiles. In the control and geogrid-reinforced sections the granite aggregate material had penetrated into the silty sand subgrade material and the silty sand had migrated in between the granite aggregate particles. It was obvious that the geotextiles were effective in preventing fines migration between the base course and subgrade layers. This observation was in agreement with the field study conducted by Austin and Coleman (*10*). It appears that the separation mechanism is more important in strengthening reinforced pavement sections than has been reported previously in the literature. This observation is supported by the results of the Falling Weight Deflectometer tests conducted by Barksdale et al. (*6*) and Webster (*9*), which did not show any significant difference between reinforced and unreinforced pavement sections.

CONCLUSIONS

The benefits provided by geosynthetic reinforcement of pavement sections constructed over weak subgrades must be understood and quantified if an adequate cost comparison is to be made. Until now the decision to use a given type of reinforcement has been largely based on field experience and empirical design methods, a nonmechanistic approach. This practice may result in unnecessary expenditures on geosynthetic materials that either are not required or are entirely inadequate.

The results of this ongoing research are preliminary in nature because they are effectively based on the results of four test sections; the data yielded by these sections will be further analyzed and other pavement sections will be investigated. In addition the results were collected using a small-scale pavement mode that was subjected to an accelerated loading process without having been subjected to some of the environmental factors that influence full-scale pavement section performance. Based on the testing and analysis performed thus far, the following conclusions can be stated:

1. Geotextiles and geogrids can offer substantial improvement to the performance of a pavement section constructed on a low-CBR subgrade.
2. It appears that the reinforcing mechanisms of geotextiles and geogrids are different. Geotextiles can provide substantial separation between the subgrade and the aggregate layers. This mechanism appears to be more important in improving pavement structural capacity than has been reported previously in the literature.

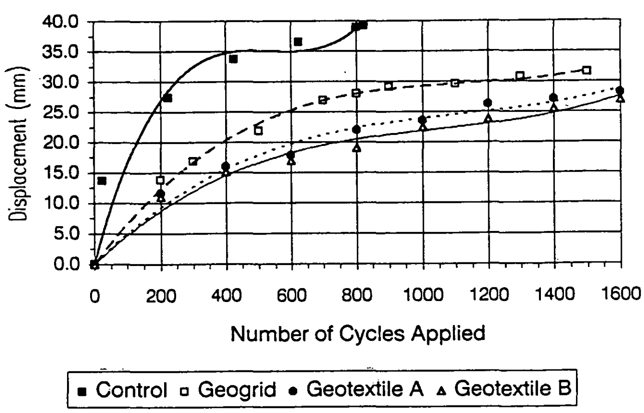

FIGURE 6 Progressive displacement for control and reinforced sections.

ACKNOWLEDGMENTS

This study was sponsored by the Virginia Center for Innovative Technology, Atlantic Construction Fabrics, Inc., and the Civil En-

TABLE 6 Number of Cycles Required To Obtain 25 mm of Displacement

Type of Reinforcement	Number of Cycles		Improvement Over Control Section (%)	
	Before Load Seating	After Load Seating	Before Load Seating	After Load Seating
Control	180	710	----	----
Geogrid	625	725	247	2
Geotextile A	1150	1420	539	100
Geotextile B	1285	1540	614	117

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Selecting Standard Test Methods for Experimental Evaluation of Geosynthetic Durability

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Geosynthetics have evolved from specialty materials considered state of the art in unique geotechnical designs to commonly used construction materials considered state of the practice in many transportation engineering applications. This relatively quick acceptance of geosynthetics can best be explained by their proven track record. Geosynthetics have generally performed as expected, though relatively few installations have yet reached their designed service lives. The term "durability" refers to the ability of geosynthetics to maintain satisfactory performance over time. Durability can be thought of as relating to changes over time of both the polymer microstructure and the geosynthetic macrostructure. The former involves molecular polymer changes and the latter assesses geosynthetic bulk property changes. Definitions of associated terminology, the identification of potential degradation processes, and the systematic selection of standard tests to evaluate both micro- and macrostructure components of durability as they relate to the use of geosynthetics in various transportation engineering applications are examined.

In 1987 the leadership of the ASTM Committee D35 on Geosynthetics recognized that confusion existed throughout the engineering community concerning the durability of geosynthetics. Yet the available standard test methods did not sufficiently address the question of geosynthetic durability. In fact the question itself appeared to require further definition before additional appropriate test methods could be developed. Therefore the Committee D35 leadership established a task group on durability with the following goals:

- Agree on terms and definitions,
- Perform a literature search,
- Identify potential degradation processes,
- Relate degradation processes to geosynthetic function,
- Identify existing test methods and parameters and propose new test methods, and
- Prepare a standard guide for selecting test methods for the experimental evaluation of geosynthetic durability.

All of these goals have been partially or completely satisfied, and a draft standard is in the balloting stage. It is through participation on this task group that the authors developed the basis for this paper.

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OVERVIEW OF GEOSYNTHETIC DURABILITY

History

Since the late 1960s planar materials constructed of synthetic polymers have been used in the construction of impoundments, roads, drainage systems, earth structures, and other civil engineering projects. These materials have become known as geosynthetics because they are synthetic materials used in conjunction with the ground. Geosynthetics are designed to perform a function or combination of functions within the soil/geosynthetic system. Such functions as filtration, separation, planar flow, reinforcement, or fluid barrier, as well as others, are expected to be performed over the life of the installation, which is often 50 to 100 years or more.

Geosynthetics are common materials and like all other materials they have limitations. Geosynthetics are made of polymers that can degrade in certain environments. For example, polyolefins such as polypropylene and polyethylene undergo oxidative degradation, whereas polyester (PET) can be hydrolyzed and polyamides degrade by both hydrolysis and oxidation. Also these degradative processes may be accelerated by exposure to, for example, transition metals in the case of oxidations and extreme pH conditions in the case of hydrolysis. It must be emphasized, however, that these reactions are usually slow and can be further retarded by the use of suitable additives (*1*).

Polymer Degradation

For geosynthetics, oxidation and hydrolysis are the most common forms of chemical degradation. Processes that involve solvents are also common. Generally chemical degradation is accelerated at elevated temperatures because the activation energy for these processes is commonly high. The moderate temperatures associated with most installation environments are therefore not expected to promote excessive degradation within the usual service lifetimes of most transportation engineering systems. Additionally most synthetic polymers are rather inert toward biological enzymatic attack. Yet prudent attention should always be given to unique environments to assess their potential for causing polymer degradation (*2*).

Because many users of geosynthetics are not familiar with polymer chemistry it is useful to assess geosynthetic performance on a functional basis and reserve polymer chemistry for interpreting unsatisfactory test results or performing forensic studies, if necessary.

Geosynthetic Performance

Geosynthetic performance is obvious to the geosynthetic user. Table 1 gives several geosynthetic failure mechanisms that result in unsatisfactory performance. In general long-term piping and clogging resistance and tensile and compression creep resistance are the most common properties related to durability in geotextiles, geogrids, geonets, and geocomposites. With geomembranes, development of openings that lead to leakage is a common concern.

Geosynthetic performance is dependent on the environment to which the geosynthetic is exposed. Therefore an understanding of the exposure environment is necessary before the user can select appropriate test methods to best simulate the aging of the geosynthetic.

Resistance to Aging

The exposure environment will generally be characterized by complex air, soil, and water chemistry as well as unique radiation, hydraulic, and stress-state conditions. The effect of this combination of exposures, over time, is called aging. Aging therefore includes both polymer degradation and reduced geosynthetic performance and is dependent on the specific exposure environment. Durability refers to a geosynthetic's resistance to aging—resistance to both polymer degradation and reduced geosynthetic performance.

Resistance to Polymer Degradation

A 1986 study by the U.S. Army Engineer Waterways Experiment Station found no reported cases of geotextile failure because of attack from chemicals present in a natural soil environment (*3*). However in cases of geosynthetic burial in soils having a very low or very high pH, consideration should be given to the composition of the geosynthetic selected. This should be a rare occurrence because most soils have a pH in the range of 3 to 10 (*4*). Geosynthetic composition should also be considered in cases of complex chemical exposures (e.g., leachate), burial in metal-rich soils, and extended exposure to sunlight. In order to evaluate these unique exposure conditions, tests are recommended that simulate actual exposure conditions on the geosynthetic selected. Accelerated tests should have a generally accepted relationship to real conditions.

Resistance to Reduced Geosynthetic Performance

Geosynthetics almost always encounter soil conditions that can cause reductions in geosynthetic performance. Such conditions may include gap-graded soil, which could lead to clogging of a

TABLE 1 Geosynthetic Failure Mechanisms

Function	Failure Mode	Possible Cause
Separation /Filtration	Piping of soils through the geotextile	Openings in geotextile are incompatible with retained soil. Openings may be enlarged as result of in-situ stress or mechanical damage.
Filtration	Clogging of the geotextile	Permittivity of the geotextile is reduced as a result of particle buildup on the surface of or within the geotextile. Openings may have been compressed as a result of long-term loading.
Reinforcement	Reduced tensile resisting force.	Excessive tensile stress/ relaxation of the geosynthetic.
Reinforcement	Unacceptable deformation of the soil/geosynthetic structure	Excessive tensile creep of the the geosynthetic.
Planar-Flow	Reduced in-plane flow capacity	Excessive compression creep of of the geosynthetic.
Protection	Reduced resis– tance to puncture	Excessive compression creep of the geosynthetic.
Fluid Barrier	Leakage through the Membrane	Openings are found in the geomembrane as a result of puncture or seam failure.

These failure mechanisms do not include polymer microstructure degradation mechanisms or installation damage and the resulting synergistic effects that may arise.

geotextile, or large embankment loads that must be resisted with little geosynthetic creep. Geosynthetic properties can be selected to protect against excessive reductions in performance, and prudent factors of safety can be used in designs incorporating geosynthetics.

Applications, Uses, and Primary Functions

In order to properly assess the effects of any given exposure environment on the performance life of the geosynthetic, a clear understanding of how the geosynthetic is to be used is required. For any given use there will be one or more primary functions that the geosynthetic will be expected to perform during its design life. Accurate identification of the application and the geosynthetic function is essential. Table 2 presents and defines primary geosynthetic functions, and Table 3 relates these functions to transportation engineering applications. It is the ability of the geosynthetic to perform satisfactorily the required primary functions during the design life that constitutes acceptable geosynthetic durability. The approach is consistent with "design by function" engineering, which is the preferred design approach for geosynthetics.

TABLE 2 Primary Geosynthetic Functions

Function	Definition
Erosion Control Device (ECD)	Restricts movement and prevents dispersion of soil particles subjected to erosion actions for an indefinite period of time.
Filter (F)	When placed in contact with a soil, allows liquid seeping from the soil to pass through while preventing most soil particles from being carried away by fluid flow.
Fluid barrier (FB)	Substantially prevents the migration of fluids through it.
Fluid transmission medium (FTM)	Collects a liquid or a gas and conveys it toward an outlet.
Permeable container (PC)	Encapsulates materials such as sand, rocks, and fresh concrete while allowing fluids to enter and escape. (During the placement of fresh concrete in a geotextile flexible form, the geosynthetic functions temporarily as a filter to allow excess water to escape.)
Protection layer (PL)	When placed between two materials, alleviates or distributes stresses and strains transmitted to the material to be protected.
Reinforcement (R)	Improves the mechanical stability of an earth structure through its tensile strength and physical interaction with soil.
Screen (Scr)	When placed across the path of a flowing fluid (groundwater, surface water, wind) carrying particles in suspension, retains some or all fine soil particles while allowing the fluid to pass through. After some period of time, particles accumulate against the screen, which requires that the screen be able to withstand pressures generated by the accumulated particles and the increasing fluid pressure.
Separator (S)	When placed between a fine and coarser material, prevents the fine soil and the coarser material from mixing.
Surface stabilization medium (SSM)	When placed on an incline, restricts movement and prevents dispersion of surface soil particles subjected to erosion actions (rain, wind), often while allowing or promoting vegetative growth.
Vegetative reinforcement medium (VRM)	Indefinitely extends the erosion control limits and performance of vegetation.

TERMS AND DEFINITIONS

Exposure Environment

The exposure environment in which a geosynthetic is placed can be characterized by the following environmental elements:

- *Air chemistry* includes the identification of the following characteristics of the gases expected to be present or created: oxygen content, gaseous pollution (e.g., NO_x, SO_2), ozone, organics (e.g., methane).
- *Fluid content* is a measure of the amount of liquid, vapor, or both in the environment immediately surrounding the geosynthetic.
- *Geometry of exposure* is described by the angle of exposure and the degree of exposure (i.e., surface versus complete).
- *Liquid chemistry* includes the identification of the following characteristics of the groundwater or leachate: pH, electrolytic conditions, dissolved and suspended minerals, chemicals, biochemical oxygen demand, chemical oxygen demand, and dissolved oxygen.
- *Macroorganisms* that are or could be present in the environment shall be identified. Macroorganisms such as insects, rodents, and other higher life forms shall be considered.
- *Microorganisms* that are or could be present in the environment shall be identified. Possible microorganisms include bacteria, fungi, algae, and yeast.
- *Radiation* shall be considered to include ultraviolet radiation, ionizing radiation, and infrared and visible radiation.
- *Soil chemistry* shall include the identification of the following characteristics of the soil or waste: transition metals, soluble minerals, polarizability, and clay mineralogy.
- *Stress* shall be focused on mechanical forces applied externally to the geosynthetic/soil system, resulting in tensile stresses, compressive stresses, shear stresses, or all three on the geosynthetic. Stresses on the geosynthetic shall be described by normal stresses, planar stresses, surface stresses, intensity of stresses, variance of stresses with time (static, dynamic, periodic), and distribution of stresses over the geosynthetic.
- *Temperature of exposure* shall be defined as the temperature of the geosynthetic, which is not necessarily that of the surrounding medium.
- *Time of exposure* shall be defined by the duration of exposure to any specific set of environmental elements.

Degradation Processes

The effects of the exposure environment are characterized by the following degradation processes:

- *Chemical degradation* is the reaction between a chemical or chemicals and a specific chemical structure within a polymer resulting in chain scission, and a reduction in molecular weight and physical properties.
- *Chemical dissolution* is the physical interaction between a solvent and polymer whereby the polymer absorbs the solvent, swells, and eventually dissolves.

TABLE 3 Applications, End Uses, and Primary Functions of Geosynthetics Used in Geotechnical and Transportation Engineering

GEOTECHNICAL/TRANSPORTATION ENGINEERING

Application	Use	Primary Function(s)
Embankments	Horizontal drain between saturated soil and embankment, filter during consolidation.	F,FTM
	Separation of soft soil and embankment materials.	S
	Reinforcement to improve embankment stability.	R
	Tensioned membrane to bridge soft soils.	R
Slope Stabilization and Protection	Filter between earth embankment and slope protection.	F
	Placed over slopes to prevent erosion.	VRM
	Reinforcement of slopes.	R
Soil Retaining Structures	Reinforced soil walls.	R
	Retained and protected slopes.	R
	Wall waterproofing systems	FB,PL
Roads on Expansive Soils, Soft Soils, or Peat	Reinforcement of soft subgrades, bridging of soft mat'ls.	R
	Separation of pavement material from soft soils.	S
	Horizontal filters, drainage of saturated subgrade.	F,FTM
	Control of expansive soils	FB,PL
	Prevention of frost heave	FB,FTM,PL
	Prevention of enlargement of karst sinkholes	FB,PL
	Protecting frost sensitive soils by encapsulation	FB
Pavement	Placed between pavement layers to act as moisture barrier	FB
	Placed between pavement layers to deter reflective crack'g	R
Railroad Tracks	To separate ballast from embankment.	S
	Moistureproofing railroad subgrades.	FB,PL
	To reinforce track systems and distribute loads.	R
	To prevent upward groundwater movement in a railroad cut	FB,PL
	To prevent contamination in railroad refueling areas	FB,PL
Tunnel Lining	To prevent puncturing of geomembrane lining	PL
	To provide drainage of seepage waters.	FTM
	To prevent migration of seepage through the tunnel lining	FB,PL
Drainage	Filter to wrap gravel drains and pipes.	F
	Drainage medium to collect and transport groundwater	FTM
	Pipeline trench base reinforcement	R

- *Clogging* is the collection of soil particles, microbiological growth, precipitates, or combination thereof on or within the geosynthetic altering its initial hydraulic properties.
- *Creep* is the time-dependent part of a strain resulting from an applied stress.
- *Environmental stress cracking* is the development of external or internal cracks in a geosynthetic that are caused by tensile stresses less than the short-time mechanical strength and are accelerated by the exposure environment.
- *Hydrolysis* is the degradative chemical reaction between a specific chemical group within a polymer and absorbed water causing chain scission and reduction in molecular weight.
- *Macrobiological degradation* is the attack and physical destruction of a geosynthetic by macroorganisms leading to a reduction in physical properties.
- *Microbiological degradation* is the chemical attack of a polymer by enzymes or other chemical excreted by microorganisms resulting in a reduction of molecular weight and changes in physical properties.
- *Mechanical damage* is the localized degradation of the in-service geosynthetic as a result of externally applied load; abrasion, impact loads, and vandalism are examples. (Installation damage is excluded, but it is an important consideration in geosynthetic selection.)
- *Oxidation* is the chemical reaction between oxygen and a specific chemical group within a polymer converting the group into a radical complex that ultimately leads to molecular chain scission or cross linking, thus changing the chemical structure, physical properties, and sometimes the appearance of the polymer.
- *Photo degradation* is the change in chemical structure resulting in deleterious changes to physical properties and sometimes to the appearance of the polymer as a result of the irradiation of the polymer by exposure to light (primarily ultraviolet).
- *Plasticization* is the physical process of increasing the molecular mobility of a polymer by absorption or incorporation of materials of lower molecular weight. The effects are usually reversible when the materials are removed.
- *Stress relaxation* is the decrease in stress, at constant strain, with time.
- *Temperature instability* is the change in appearance, weight, dimension, or other property of the geosynthetic as a result of low, high, or cyclic temperature exposure.

TABLE 4 **Degradation Concerns Relating to Geosynthetic Functions**

FUNCTION	POTENTIAL DEGRADATION PROCESS: Abbreviation	Biological Degradation	Chemical Degradation	Chemical Dissolution	Clogging/Piping	Creep	Environmental Stress Cracking	Hydrolysis	Mechanical Damage	Photo-Oxidation	Plasticization	Stress Relaxation	Temperature Instability	Thermal-Oxidation	EXPLANATIONS OF PRIMARY LONG-TERM CONCERNS
Erosion Control Device	ECD	P[1,2]	S[3]	S[3]	N	N	N	S[4]	S[5]	S[6]	N	N	N	S[7]	Resist Erosive Forces
Filter	F	P[1,2]	S[3]	S[3]	A[8]	S[9]	N	S[4]	S[5]	S[6]	N	S[9]	N	S[7]	Maintain design filtration & resist deformation & intrusion
Fluid Barrier	FB	P[1,2]	S[3]	S[3]	N	S[16]	A[11,20]	S[4]	S[5]	S[6]	N	S[16]	S[12]	S[7]	Maintain intended level of impermeability
Fluid Transmission Medium	FTM	P[1,2]	S[3]	S[3]	A[13]	A[14]	A[20]	S[4]	S[5]	S[6]	N	A[14]	N	S[7]	Maintain flow under compressive loads
Permeable Container	PC	P[1,2]	S[3]	S[3]	S[15]	S[16]	N	S[4]	S[5]	S[6]	N	S[16]	N	S[7]	Remain intact & maintain filtration performance
Protective Layer	PL	P[1,2]	S[3]	S[3]	N	S[17]	N	S[4]	N	S[6]	N	S[17]	N	S[7]	Maintain protective performance
Reinforcement	R	P[1,2]	S[3]	S[3] P[18]	N	A[19]	P[20]	S[4]	P[18]	S[6]	P[21]	S[19]	S[19]	S[7]	Provide necessary strength, stiffness & soil interaction
Screen	Scr	P[1,2]	S[3]	S[3]	S[22]	N	N	S[4]	S[5]	S[6]	N	N	N	S[7]	Maintain filtration performance & resist deformation
Separator	S	P[1,2]	S[3]	S[3]	N	N	N	S[4]	P[23]	S[6]	N	N	N	S[7]	Remain intact
Surface Stabilization Medium	SSM	P[1,2]	S[3]	S[3]	N	N	N	S[4]	A[10]	A[10]	N	N	N	S[7]	Remain intact to resist erosive forces until vegetation is established
Vegetative Reinforcement Medium	VRM	P[1,2]	S[3]	S[3]	N	N	N	S[4]	A[10]	A[10]	N	N	N	S[7]	Remain intact throughout vegetation

KEY: N = Not a generally recognized concern; S = Sometimes a concern; A = Almost always a concern; P = Potential concern being researched

[1]Microorganisms have been known to attack and digest additives (plasticizers, lubricants, emulsifiers) used to plasticize a base polymer. Study is needed to determine relevance to polymers incorporated into geosynthetic products. Embrittlement of geosynthetic surfaces may influence interaction properties.

[2]Microbial enzymes have been known to initiate and propagate reactions deteriorative to base polymers. Study is needed to determine relevance to polymers used in geosynthetic products.

[3]Chemical degradation and/or dissolution, including the leaching of plasticizers or additives from the polymer structure, may be a concern for geosynthetics exposed to liquids containing unusually high concentrations of metals, salts, or chemicals, especially at elevated temperatures.

[4]Hydrolysis may be a concern for PET and PA geosynthetics exposed to extreme pH conditions, especially at elevated temperatures.

[5]When subject to rocking (abrasion), puncture (floating or airborne debris), or cutting (equipment or vandalism).

[6]When permanently exposed or during extended construction (>2-4 weeks) and in wrap-around construction, photo degradation may be a concern for exposed geosynthetics.

[7]Geosynthetics in applications such as dam facings and floating covers that result in exposure to temperatures at or above ambient must be stabilized to resist thermal oxidation.

[8]Clogging resistance of geotextiles can be assessed only by testing with site-specific soil and (sometimes) liquid.

[9]If a filter geotextile is used with a geonet, it is important to assess short-term extrusion and long-term intrusion into the net.

[10]Always exposed therefore resistance to photo oxidation and mechanical damage must be determined.

[11]Residual stresses and surface damage may produce synergistic effects with other degradation processes.

[12]Excessive expansion and contraction resulting from temperature changes may be a concern for geosynthetics without fabric reinforcement.

[13]Composite drains must resist clogging due to soil retention problems and intrusion of filter medium.

[14]Geosynthetics relying on a three-dimensional structure to facilitate flow must demonstrate resistance to compression creep.

[15]If select fill is not available, then a clogging resistance test should be performed with the job-specific soil.

[16]Geosynthetics in containment structures that require long-term strength characteristics should be designed using appropriate creep and stress relaxation criteria.

[17]Sufficient thickness must be maintained by a protective layer over an extended period of time.

[18]Chemical dissolution of or mechanical damage to geosynthetic coatings may affect their interaction properties (i.e., lead to surface or joint slippage).

[19]Geosynthetics creep and stress relax at different rates depending mainly on manufacturing process, polymer type, load levels, temperature, and application.

[20]Polyethylene geosynthetics may experience slow crack growth under long-term loading conditions in certain environmental conditions.

[21]Plasticization may be a concern for PET geosynthetics exposed to humidity or polypropylene and polyethylene geosynthetics exposed to hydrocarbons while under stress.

[22]If the screen is expected to operate indefinitely, then clogging should be assessed often. Commonly, screens are considered temporary.

[23]Holes resulting from mechanical damage may alter the effectiveness of separators.

• *Thermal degradation* is the change in chemical structure resulting in changes in physical properties and sometimes in the appearance of a polymer caused by exposure to heat alone.

SELECTING TESTS TO EVALUATE DURABILITY

Scope

This selection guide defines those factors of the appropriate exposure environment that may affect the post-construction service life of the geosynthetic. Test methods are recommended to facilitate an experimental evaluation of the durability of geosynthetics in a specified environment so that durability can be considered in the design process. This does not address manufacturing, handling, transportation, or installation conditions.

Summary of Selection Process

The effects of a given exposure environment on the durability of a geosynthetic must be determined through appropriate testing. Selection of appropriate tests requires a systematic determination of the primary function(s) to be performed and the associated degradation processes that should be considered. This selection guide provides a suitable systematic approach.

Primary functions of geosynthetics are listed and defined in Table 2. With knowledge of the specific geosynthetic application area and end use, the corresponding primary function(s) is identified. (Table 3 lists transportation engineering applications) Table 4 lists degradation concerns as they relate to geosynthetic functions. Table 5 gives the environmental elements that relate to the various degradation processes and the currently available ASTM D35 standard test method for the experimental evaluation of specific types of geosynthetic degradation.

Designers and specifiers of geosynthetics should evaluate geosynthetic durability as an integral part of the geosynthetic specification and selection process. The following procedure is intended to guide a designer or specifier through a systematic determination of degradation concerns based on the intended geosynthetic function. The procedure then provides a guide to selecting available test methods for experimentally evaluating geosynthetic durability and to identifying areas where no suitable test exists.

This guide does not address the evaluation of degradation resulting from manufacturing, handling, transporting, or installing the geosynthetic.

Step-by-Step Procedure

To use a structured procedure for selecting appropriate test methods, the geosynthetic designer or specifier must have knowledge of

• The intended geosynthetic application,
• The end use of the geosynthetic via its primary function(s),
• The specific environment to which the geosynthetic will be exposed,
• The types of geosynthetics that may or will be used, and
• The duration or time of use.

With this knowledge, the designer or specifier follows the following procedure:

1. Identify the primary functions to be performed by the geosynthetic in the specific application and end use intended. Primary functions are defined in Table 2. Table 3 provides guidance in identifying primary functions for transportation applications.
2. Identify in Table 4 the potential degradation processes that are almost always (A) or sometimes (S) concerns when a geosynthetic performs the primary function(s) that were identified in Step 1. Consult the notes for Table 4 to see whether those identified degradation processes that are sometimes a concern apply to the specific application environment expected.
3. Using Table 5, select the standard test method(s) that applies to the potential degradation processes identified in Step 2 as a concern in the specific exposure environment expected.

Guidance is given in Table 5 to identify the most important elements or variables relating to each degradation process.

TEST METHOD SELECTION PROCEDURES: EXAMPLES

Example 1

Select the appropriate standard test methods to assess the durability characteristics of a geotextile to be used to separate an asphalt-surfaced aggregate road structure from underlying soft soils. The design life of the road is 20 years.

Selection Procedure

• Application (see Table 3): road on soft soil.
• End use: separation of road structure from soft subgrade.
• Primary function(s): separator.
• Potential degradation processes (see Table 4)
–Biological degradation (potential being researched; not a documented concern at this time);
–Chemical degradation and dissolution (only seepage water is expected; therefore, chemical degradation and dissolution are not concerns);
–Hydrolysis (extreme pH conditions are not expected; therefore, hydrolysis is not a concern);
–Mechanical damage (potential being researched; not a documented concern at this time);
–Photo oxidation (extended ultraviolet exposure is not expected; therefore, photo oxidation is not a concern); and
–Thermal oxidation (extended exposure is not expected; therefore thermal oxidation is not a concern).

Summary of Required Tests

No durability tests are required.

Example 2

Select the appropriate standard test methods to assess the durability characteristics of a geotextile to be used to wrap aggregate pavement edge drains. The design life is 20 years.

TABLE 5 Environmental Factors and Tests Relating to Geosynthetic Degradation

POTENTIAL DEGRADATION PROCESS	ENVIRONMENTAL ELEMENTS RELATING TO DEGRADATION											STANDARD TEST METHODS IN ASTM D35 COMMITTEE ON GEOSYNTHETICS	
	Air Chemistry	Fluid Content	Geometry of Exposure	Liquid Chemistry	Macro-Organisms	Micro-Organisms	Radiation	Soil Chemistry	Stress	Temperature of Exposure	Time of Exposure		
Biological Degradation	X	X			X	X		X		X	X	None	Microbio. Attack
Chemical Degradation				X				X		X	X	ASTM D5322	Chemical Immersion
												ASTM D5496	Field Immersion
Chemical Dissolution				X				X		X	X	None	Effect of Solvents
Clogging/Piping				X		X		X			X	ASTM Proposed	Hydr. Conduct. Ratio
												ASTM D5101	Gradient Ratio
												ASTM D1987	Biological Clogging
												None	Precipitate Clogging
Creep			X						X	X	X	ASTM Proposed	Compression
												ASTM D5262	Tension
												ASTM D4716	Transmissivity
Envir. Stress Cracking	X			X				X	X	X	X	ASTM D5397	Stress Cracking
Hydrolysis		X		X				X		X	X	None	Effect of Water
Mechanical Damage			X						X		X	ASTM D4886	Abrasion
												None	Fatigue
												ASTM D4833	Puncture
Photo-Oxidation	X						X				X	ASTM Proposed	Outdoor Exposure
												ASTM D4355	UV Resistance
Plasticization		X		X						X	X	None	Effect of Liquids
Stress Relaxation			X						X	X	X	ASTM Proposed	Compression
												ASTM Proposed	Tension
Temperature Instability										X	X	ASTM D4594	Temperature Stability
Thermal Oxidation	X						X			X	X	None	Effect of Heat

<u>Note</u>: This table provides the standard test methods current at the time of the writing of this article. ASTM Standards are in constant development, review, revision, and replacement. It is the responsibility of the geosynthetic specifier to identify the most current applicable standard test method.

Selection Procedure

- Application (see Table 3): drainage.
- End use: filter to wrap gravel drains.
- Primary function(s): filter.
- Potential degradation processes (see Table 4)

–Biological degradation (potential being researched; not a documented concern at this time.);

–Chemical degradation and dissolution (only seepage water is expected; therefore, chemical degradation and dissolution are not concerns);

–Clogging (always);

–Creep (because the geotextile will be used adjacent to aggregate, no bridging is required; therefore, creep and stress relaxation are not concerns);

–Hydrolysis (extreme pH conditions are not expected; therefore, hydrolysis is not a concern);

–Mechanical damage (no rocking, impact, or cutting is expected because of complete burial and negligible stress; therefore, mechanical damage is not a concern);

–Photo oxidation (extended ultraviolet exposure is not expected; therefore, photo oxidation is not a concern); and

–Thermal oxidation (extended exposure is not expected; therefore, thermal oxidation is not a concern).

Summary of Required Tests

- Potential degradation process: clogging, and
- Standard test methods: ASTM D5101 and D1987.

CONCLUSIONS AND RECOMMENDATIONS

Although geosynthetics have generally performed as expected, some uncertainty exists as to how long these materials can be expected to continue to perform; that is, how durable they are. This paper attempts to provide guidance for the experimental evaluation of geosynthetic durability by outlining a procedure for selecting appropriate standard test methods. The selection of appropriate tests has been related to the primary functions that the geosynthetic is expected to perform and to the specific degradation processes that can be expected for the anticipated exposure environment. Detailed associated terminology has been proposed to facilitate continued discussions of geosynthetic durability among design engineers and polymer scientists.

It is hoped that the proposed selection procedure and terminology will assist engineers in including durability in their "design by function" use of geosynthetics and also provide a basis for defining the scope of new tests to evaluate specific degradation processes.

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Large Strain Measurements in Geogrid Reinforcement

KHALID FARRAG, JOHN OGLESBY, AND PAUL GRIFFIN

Strain gauges are usually used in measuring deformations of geosynthetics in reinforced soil walls, where maximum strains around 2 percent are usually monitored. For the higher strain levels encountered in confined extension and pullout tests, extensometers and linear variable differential transformers (LVDTs) are usually used. The use of strain gauges in monitoring larger strains (5 percent and higher) requires a special procedure for the attachment of the gauges and a correlation between strain gauges and LVDT readings in confined soils for the proper interpretation of the measurements. A standardized procedure for attaching strain gauges to geogrid reinforcement was developed to monitor large strains (in excess of 8 percent) in confined conditions. The procedure was examined for different types of geogrids, adhesives, and protective coatings. It was first evaluated in unconfined extension tests. The correlation between strain gauge measurements and strains measured across the specimen length (cross-head strains) was investigated. Strain gauges monitored up to 16 percent cross-head strain with a linear relationship up to 10 percent strain. Strain measurements under confined conditions were evaluated in confined extension tests and pullout tests. The strains between the geogrid transversal elements (element-strains) were obtained from LVDT measurements. Strain gauge readings were correlated to element strains at different locations along the specimen. Strain gauge measurements were less than those calculated from LVDT measurements. The relationship between both was linear up to 8 percent strain. A correlation factor to correct strain gauge measurements to element strains is used and a numerical procedure is used to estimate the tension forces at various locations along the geogrid reinforcement from strain measurements.

Geosynthetics strains monitored under working stress conditions in reinforced soil walls are usually small because of the high factors of safety inherent in the design of such structures. Maximum tensile strains are usually about 2 percent in typical reinforced walls and may reach 3 to 4 percent in the base reinforcement of embankments above soft soils (*1,2*). Strain gauges are typically used in the field to monitor such strain levels in geogrid reinforcements (*3*), woven geotextiles (*4*), and nonwoven fabrics (*5*).

Higher strain levels up to 10 percent are usually reached in model walls tested under ultimate loading conditions (*6*) and in confined extension and pullout tests (*7*). For the higher strain levels encountered in such conditions, extensometers and linear variable differential transformers (LVDTs) are usually used. The use of a sacrificial array of strain gauges to measure large strains offers an economical solution for monitoring large deformations in the laboratory as well as in the field. However, the use of strain gauges requires a special procedure for the attachment of the gauges and a correlation between strain gauges and LVDT readings in confined soils for the proper interpretations of the measurements.

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A procedure was developed to attach strain gauges for measuring strains in excess of 8 percent. The procedure was examined in unconfined extension tests for two different types of geogrids and various adhesives and protective coatings. The performance of the strain gauges under confined conditions was evaluated in confined extension tests and pullout tests. The strain gauges were tested in compacted cohesive soil under various confining pressures. Strain gauge readings were correlated with the LVDT measurements at different locations along the specimen. The measurements were used in defining the confined stress-strain properties of the geogrid and in estimating the induced tension forces along the geogrid reinforcement in pullout tests.

STRAIN MEASUREMENTS IN UNCONFINED TESTS

The geogrid–strain gauge attachment procedure was investigated in two types of geogrids of different geometry, material properties, manufacturing processes, and surface texture: (high-density polyethylene (HDPE) geogrid Tensar UX1500 and woven fabric geogrid Conwed Stratagrid 6033. Two types of strain gauges from Micro Measurements (MM), namely, EP-08-250BG 120 ohm and EP-40-250BF 350 ohm, were used for both geogrids. Preliminary unconfined extension tests were conducted to investigate the effects of strain rate, surface preparation, gauge and adhesive type protective coatings, and water submergence. The tests were performed on specimens 18 in. long with three longitudinal ribs. Strains were monitored using the MM P3500 strain indicator. The gauges monitored up to 12 percent strain in most tests. The procedure for strain gauge attachment is discussed in detail elsewhere (*8*) and is summarized in the following sections.

Surface Preparation

A combination of 000 steel wool and 220 and 400 grit sandpaper was used for surface abrasion at the location of the strain gauges. Unconfined extension tests (*8*) showed no reduction in the tensile strength due to the surface abrasion of the specimens.

For the woven geogrid, the surface texture is rough and irregular, with a uniform cross-sectional area. A thin layer of MM A-12 adhesive was placed on the woven geogrid to create an adequate surface for gauge attachment. The adhesive was cured for at least 4 hr at 125°F and clamped between metal plates with sufficient pressure to allow the epoxy adhesive to impregnate the woven fibers. The surface was then abraded and cleaned.

Strain Gauge Attachment

MM A-12 adhesive, a two-part epoxy used for high strain conditions, was used to attach the gauges. After the gauges were glued, pressure was applied on the gauges by spring clamps while the adhesive cured. A neoprene sponge was placed on the gauges to protect them and to evenly distribute the pressure. The geogrid specimens were cured for at least 4 hr at 125°F. The preliminary tests showed that no apparent change in the ultimate geogrid strength was found when the geogrid was exposed to temperatures up to 165°F and for 24 hr. After curing the leads were soldered to the gauges.

Unconfined Extension Tests

After the preliminary tests to investigate the attachment procedure of strain gauges, additional unconfined extension tests were performed on the HDPE geogrid specimens instrumented with the MM EP-08-250BG 120-ohm strain gauges. The specimens were 0.15 m (6 in.) wide and 0.48 m (19 in.) long. In these tests strain gauges were attached at different locations along the geogrid longitudinal ribs. Figure 1 shows the geogrid specimen with the locations of the strain gauges. Typical test results on the HDPE geogrid are shown in Figure 2. The strain gauge readings are plotted with the cross-head strain measured along the overall length of the specimen. The strain gauges monitored up to 12 percent (corresponding to 16 percent cross-head strain) in most tests.

The results in Figure 2 show that strains are uniform within the gauges *A* to *C* and the location of the gauge along the longitudinal ribs has no effect on the measurements. However, strain gauge readings are not equal to the cross-head strain along the specimen length. This is mainly due to the varying geometry and stiffness modulus at the transversal ribs, which causes a nonuniform strain distribution along the specimen length. Within the range of the linear strain gauge reading (10 percent), a correlation factor of 0.8 relates the strain gauge reading to the cross-head strain for this specific type of geogrid. It should be noted that the correlation factor was approximately 1.0 for the Stratagrid woven grids of uniform cross-sectional area before the protective coating was applied.

STRAIN MEASUREMENTS IN CONFINED TESTS

Specimen Preparation

The procedure for strain gauge attachment to the geogrid is essentially the same for confined applications. To protect the gauges in the soil, a number of specialized protective coatings were tried. A layer of rubber cement covered by a layer of silicon rubber coating was found to be simple, inexpensive, and adequate. The coated gauges were tested on the HDPE geogrid specimens in a 2 percent saline solution. The gauge readings were monitored periodically for 2 weeks in the solution before testing. The coating was found to be adequate to waterproof the gauges.

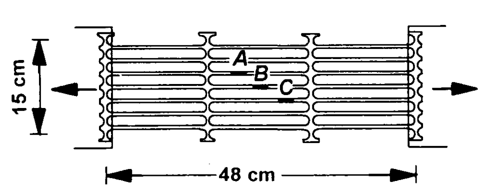

FIGURE 1 HDPE geogrid specimen and location of strain gauges.

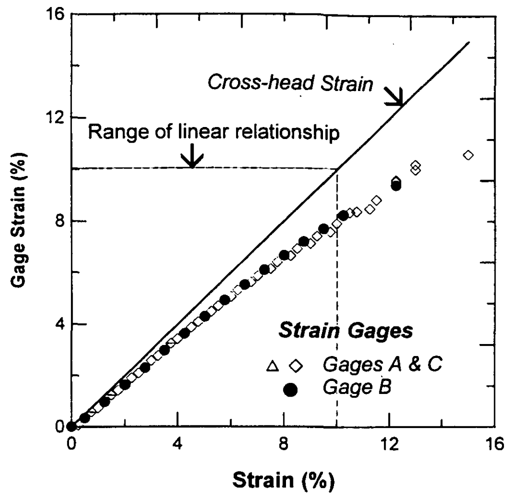

FIGURE 2 Strain measurements of HDPE geogrid in unconfined extension tests.

The coating system was also applied to the polyester yarn woven geogrids. Specimens were coated on both sides of the grid and soaked in the solution for various periods of time. High strain readings (approximately 5 percent) were monitored during the period of submergence, which suggested that the woven grid was absorbing water and swelling. When the geogrid was soaked in the solution it absorbed 16 percent water by weight after 40 hr.

In order to ensure that the saline solution was not getting to the strain gauges through the soaked woven grid, a strip of elastic foil, slightly larger than the gauge, was glued to the prepared surface before gauge installation. The gauge was glued directly to the strip and coated using the standard procedure. The strain readings increased to approximately 5 percent within 3 days of soaking, showing that the grid was swelling; however, the readings were stable, showing that the saline did not reach the gauges.

To protect the gauges during compaction, two pieces of plastic pipe 1 in. long and 0.5 in. in diameter were split and placed around the geogrid rib and the gauge. Compaction in the box was performed with a minimum soil thickness of 4 in. above the tubes.

Confined Extension Tests

In the confined extension tests, instrumented HDPE geogrid specimens of 0.3 m (1 ft) wide and 0.92 m (3 ft) long were tested in a box 1.22 m (48 in.) in length, 0.6 m (24 in.) in width, and 0.45 m (18 in.) in height. Figure 3 shows a schematic diagram of the confined extension box. The details of the box are presented elsewhere (7). The geogrid specimen was placed at mid-height of the box with one end clamped to the box. The soil was silty clay with a plasticity index of 24. The tests were conducted at the optimum water content of 22 percent, at 90 percent of the maximum dry

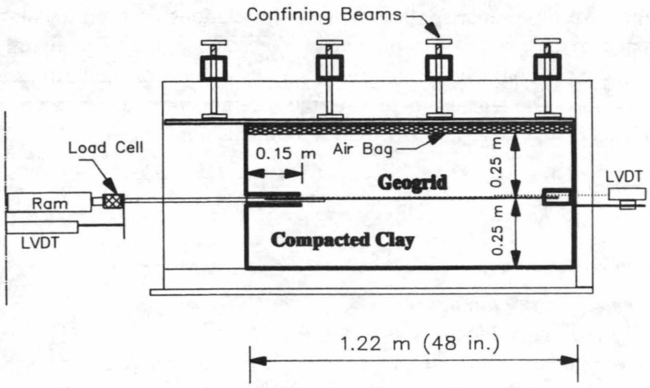

FIGURE 3 Schematic diagram of confined extension box.

FIGURE 5 Placement of instrumented HDPE geogrid in confined extension test.

density, and at different confining pressures. Four strain gauges were placed on the first two geogrid elements. Figure 4 shows a schematic diagram of the geogrid specimen and the location of the strain gauges and Figure 5 shows the placement of the geogrid in the large testing box.

The nodal displacements along the geogrid specimen were also monitored by the LVDTs. The strains ϵ_i between the grid nodes (element-strains) were calculated from the LVDT measurements from the relationship

$$\epsilon_i = [\delta_i - \delta_{i-1}] / \Delta x \quad (1)$$

where δ_i and δ_{i-1} are the displacements at two consecutive nodes and Δx is the element length.

The strain gauge measurements are plotted with the element-strains from the LVDT measurements in Figure 6. The strain gauge readings were stable up to 12 percent element-strain. The correlation factor between the element-strain and the strain gauge readings was 0.7, which is lower than that deduced from the unconfined tests. The reduction of the correlation factor is possibly attributed to the increase in the stiffness modulus of the geogrid due to the addition of the protection coatings around the geogrid ribs.

ESTIMATION OF TENSION FORCES FROM STRAIN MEASUREMENTS

The tension forces along the geogrid reinforcement during pullout can be estimated from strain measurements. In the pullout tests, testing parameters were identical to those in the confined extension tests. Geogrid specimens were tested in the box shown in Figure 3 with the back of the specimen not attached to the box.

A schematic diagram of the HDPE geogrid specimen and the locations of the strain gauges for pullout testing is shown in Figure 7. The strain gauge measurements during pullout are shown in Figure 8 while the LVDT measurements during pullout are shown in Figure 9. The displacement distribution along the geogrid length is plotted for different pullout loads in Figure 10. It can be seen from this figure that the displacement of each geogrid element between node i and $i - 1$ results from the elongation of the element ($\delta_{i-1} - \delta_i$) and the shear displacement δ_i. The strain ϵ_i between the grid nodes (element-strains) can be calculated from Equation 1.

Figure 11 shows the pullout load versus the strain of the first geogrid element, and the confined stiffness modulus of the geogrid (E) can be obtained from the initial slope of the relationship. For

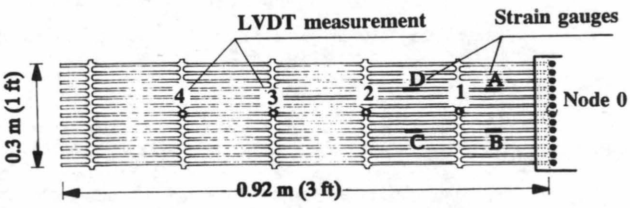

FIGURE 4 Locations of strain gauges and LVDT measurements.

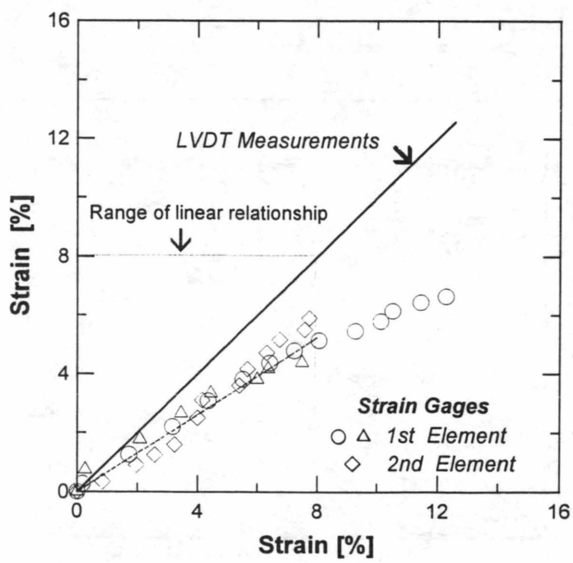

FIGURE 6 Strain measurements in confined extension tests on HDPE geogrid: confining pressure, 48 kN/m^2 (7 psi).

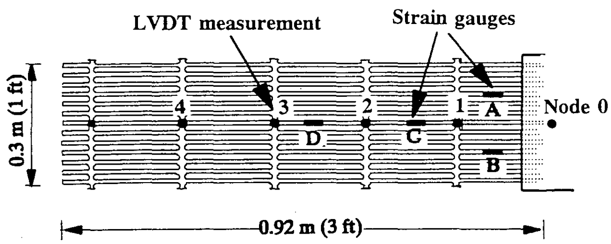

FIGURE 7 Locations of strain and LVDT measurements in pullout tests.

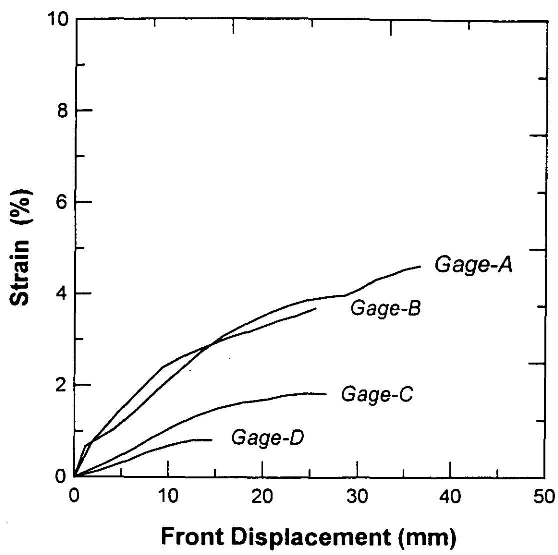

FIGURE 8 Strain gauge measurements in pullout tests on HDPE geogrid: confining pressure, 48 kN/m^2 (7 psi); geogrid specimen, 0.3 m wide × 1 m long.

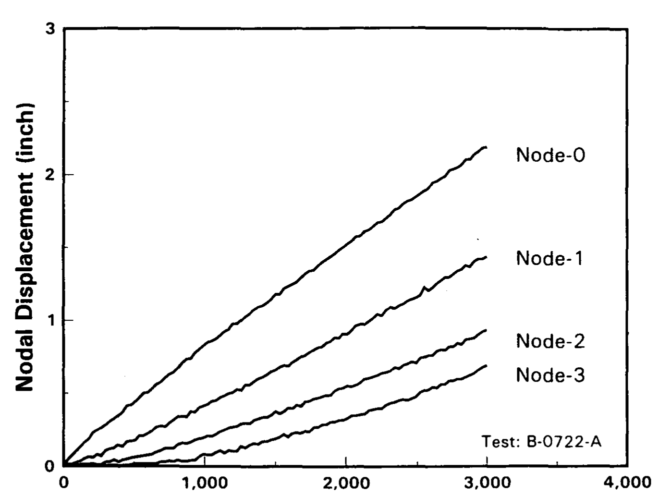

FIGURE 9 LVDT measurements during pullout test: HDPE geogrid, 3 ft long; cohesive soil, density 95 lb/ft^3, w/c 22 percent.

the sake of comparison the stess-strain relationship from unconfined extension tests is plotted in the same figure. The figure shows that confinement can result in an increase of the stiffness modulus. The tension force T_i at the locations of strain measurements can be determined from the relationship

$$T_i/b = E\ t\ (\epsilon_i) \tag{2}$$

where

b = geogrid width,
t = geogrid thickness, and
ϵ_i = geogrid strain at gauge i.

The calculated tension forces at the locations of the strain gauges are shown in Figure 12.

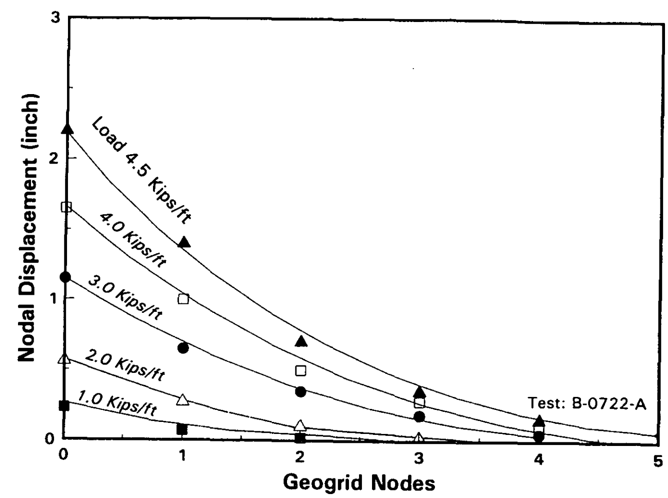

FIGURE 10 Displacement distribution along geogrid during pullout.

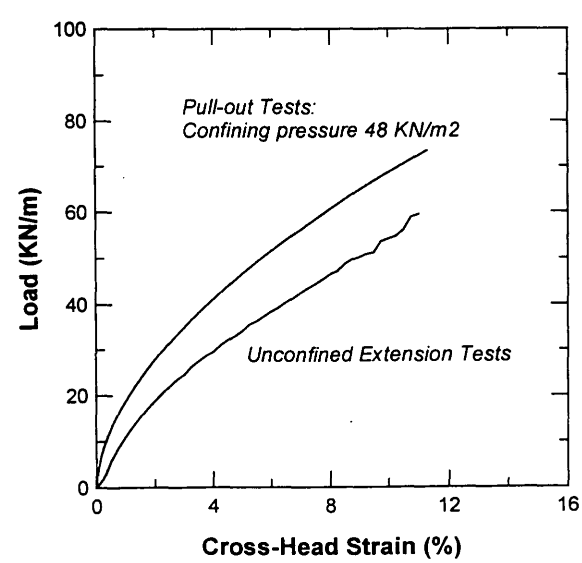

FIGURE 11 Confined and unconfined stiffness modulus of HDPE geogrid.

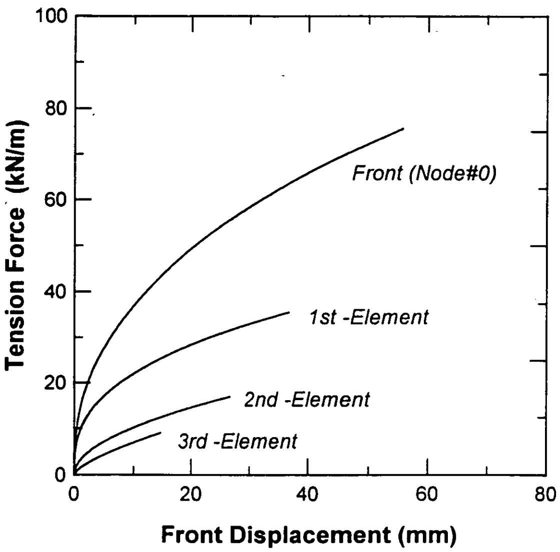

FIGURE 12 Estimated tension forces along geogrid elements during pullout test.

CONCLUSIONS

The geogrid reinforcement can be subjected to large strains during confined extension and pullout tests; moreover, large deformations may exist in reinforced-soil test walls under ultimate loading conditions. Deformations can be monitored by instrumenting selected sections of the geogrid reinforcement with a low-cost sacrificial strain gauge system. A standardized procedure for the attachment of strain gauges to monitor strains in excess of 8 percent has been developed.

The results of the unconfined extension tests showed that the strain gauges could monitor strains up to 16 percent cross-head strain. However strain gauge readings were linear up to 10 percent strain and they were not equal to the cross-head strains. The relationship between strain gauge reading and cross-head strain depends on the geogrid geometry and the change in the thickness and stiffness modulus along the geogrid length. The ratio between strain gauge reading and cross-head strain was close to 1.0 for the uniform Stratagrid woven grids and 0.8 for the HDPE geogrid.

The strain gauge installation procedure was also evaluated in confined applications. The results of the confined extension tests demonstrated the importance of protecting the gauge and the specimen from moisture and compaction damage. A lower correlation factor (0.7) between the strain gauge reading and element-strains was deduced from confined tests on the HDPE grids, possibly because of the addition of the protective coatings.

The deformation-induced tension forces in the reinforcement could be estimated from the strain gauge readings. The procedure demonstrated the importance of obtaining the strain measurements and the reinforcement stress-strain relationships from the appropriate tests in confined conditions.

ACKNOWLEDGMENTS

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Durability of Geosynthetic Soil Reinforcement Elements in Tanque Verde Retaining Wall Structures

DONALD G. BRIGHT, JAMES G. COLLINS, AND RYAN R. BERG

The findings of an investigation into the stability and durability of high-density polyethylene (HDPE) geogrid soil reinforcing elements used in grade separation structures on a project at the Tanque Verde–Wrightstown–Pantano Roads intersection in Tucson, Arizona, is documented. The project represents the first use of geogrid reinforcement in concrete-faced, mechanically stabilized earth retaining walls in a major transportation-related application in North America. The reinforced soil walls were constructed in 1984 and 1985, and the geogrids have been in service for 8 to 9 years in an elevated temperature environment that accelerates the mechanisms of degradation. The combined effects of age and temperature exposure made this project a candidate for evaluation of geosynthetic reinforcement stability and durability. A sample retrieved from the project was subjected to a series of laboratory procedures and tests. Results of the tests are presented as topological analysis, ultimate tensile strength, ultimate tensile strain, 1,000-hr creep response, melt rheology, melt temperature range, crystallinity, and oxidative induction times. Test results are compared to archived geogrid values. Soil samples also were retrieved and analyzed. It is concluded that after more than 8 years of exposure to an elevated temperature environment, the exhumed HDPE geogrid has not experienced any significant change in the physical and performance properties of the geogrid or the morphological properties of the HDPE.

The durability of a specific geogrid soil reinforcing element used in a specific project is examined. The geogrid elements have been in service for between 8 and 9 years in concrete-faced, mechanically stabilized earth (MSE) retaining wall structures. The project is located in the southwestern United States and is exposed to high summer temperatures.

A summary description of the project is presented and references providing detailed design and performance information are noted. The mechanical instrumentation program and monitored results are referenced. The procedure used for exhumation of a geogrid sample is described, and a description and illustration of the geogrid reinforcing element is presented. The test methods used in this durability evaluation and a rationale are summarized along with tabulated test results and a discussion of the findings. Conclusions are stated regarding the durability of the geogrid based on interpretation of test results from archived and exhumed samples.

PROJECT DESCRIPTION

Forty-six geogrid-reinforced retaining walls were constructed to created grade separations for the Tanque Verde–Wrightstown–Pantano Roads intersection in Tucson, Arizona (Figure 1). Approximately 1600 lineal meters (m) of wall were constructed between December 1984 and September 1985. The walls vary in height from 1 to 6 m.

The precast concrete wall face panels are 0.15 m thick, 3 m wide, and full height. Four cross-section wall geometries, as illustrated in Figure 2, were used for the 46 wall structures. Comprehensive details of the wall design are presented in a value engineering study, the design report (*1*), and in the project construction drawings (*2*). Two of the Type D wall sections were instrumented in September 1985 and have been monitored since then. The goal of the instrumentation program is to assess the performance of the wall structures. Stresses and strains in the soil and strain in the geogrid are monitored with pneumatic load cells, inductance coils, and resistance strain gauges. Internal and external wall temperatures are monitored with resistance thermometers. Panel displacements during and immediately after construction were monitored using optical survey techniques. Brief descriptions of the instrumentation program and instrumentation readings during and immediately after construction have been presented by Berg et al. (*3*). Reports of the post-construction monitoring program have been presented by Desert Earth Engineering (*4,5*).

A comprehensive report on the project, incorporating the Dames and Moore design report and the Desert Earth Engineering monitoring results and an interpretation, has been published by the FHWA (*6*) and by Fishman et al. (*7*). A summary of monitoring results through the first 4 years in service was presented at the 1991 TRB Annual Meeting (*8*). A summary of monitoring results through the first 8 years in service was presented at the Seiken Symposium in Japan by Collin and Berg (*9*).

Based on resistance thermometer readings from one of the two monitored Type D wall sections, transient temperatures up to approximately 49°C (120°F) on the concrete wall face panels were recorded, and up to approximately 38°C (100°F) behind the panels. Figure 3 shows the temperature readings at various elevations within a Type D wall section as recorded in March and June 1986.

Mechanical response over time is the focus of the still-active instrumentation program. Based on readings of the two monitored wall sections, the geogrid reinforcement has experienced low tensile loads, resulting in maximum strains of less than 1 percent. Typical strains of 0.5 percent and 1.0 percent have been recorded for the two instrumented sections.

Although instrument readings taken as recently as September 1992 indicate no significant change in performance over time, an investigation into the stability of the geogrid soil reinforcing elements was initiated in 1993. An analysis of the geogrids used in this project was deemed appropriate because it is the oldest struc-

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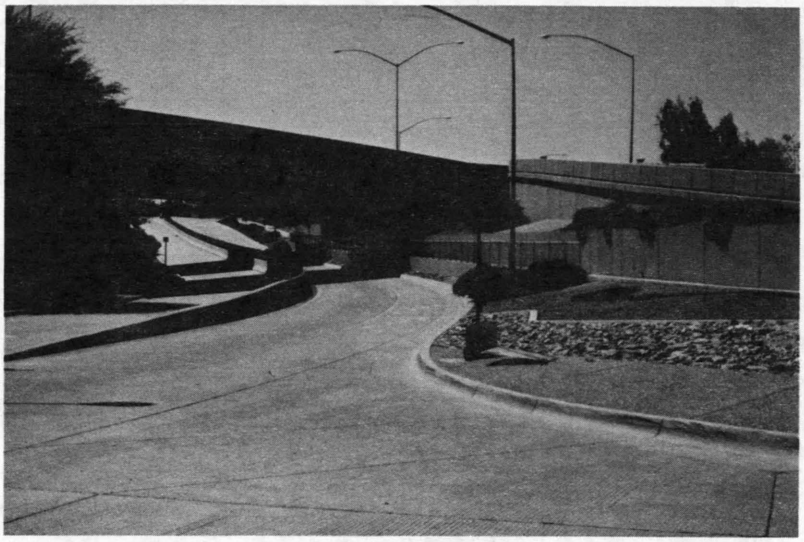

FIGURE 1 Tanque Verde - Wrightstown - Pantano Roads intersection.

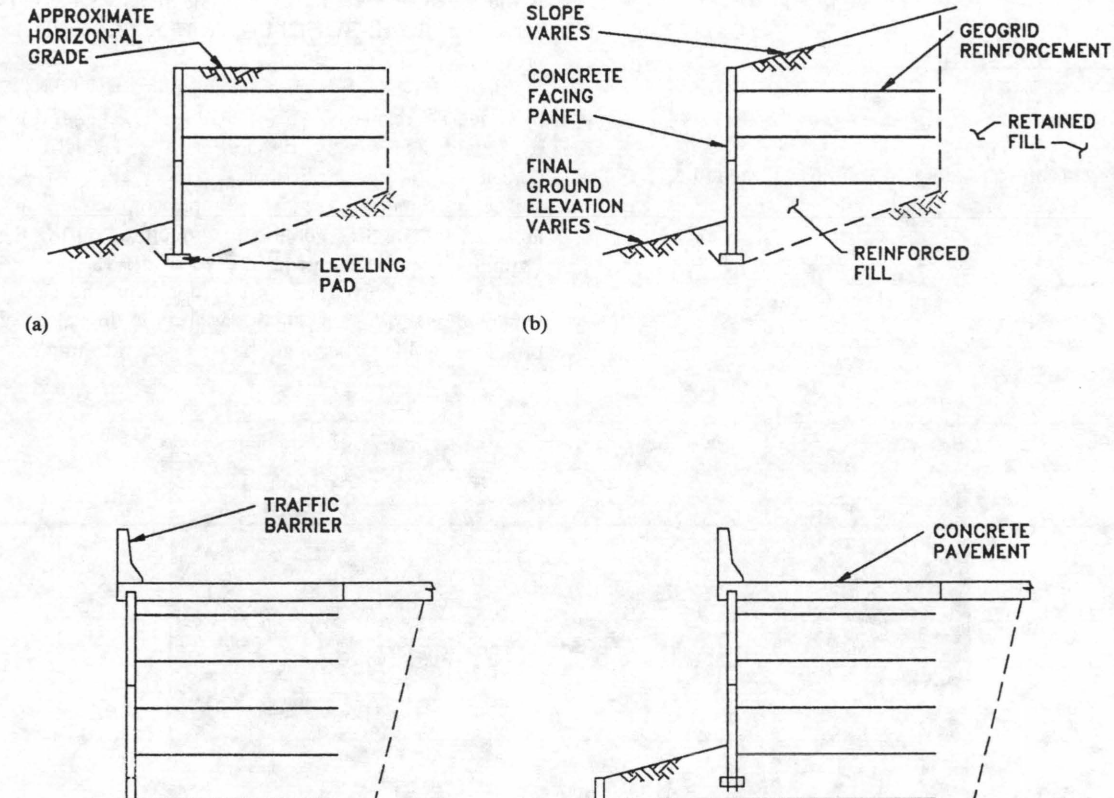

FIGURE 2 Four wall geometries used in Tanque Verde retaining wall structures. *a*, Type A wall; *b*, Type B wall; *c*, Type C wall; *d*, Type D wall.

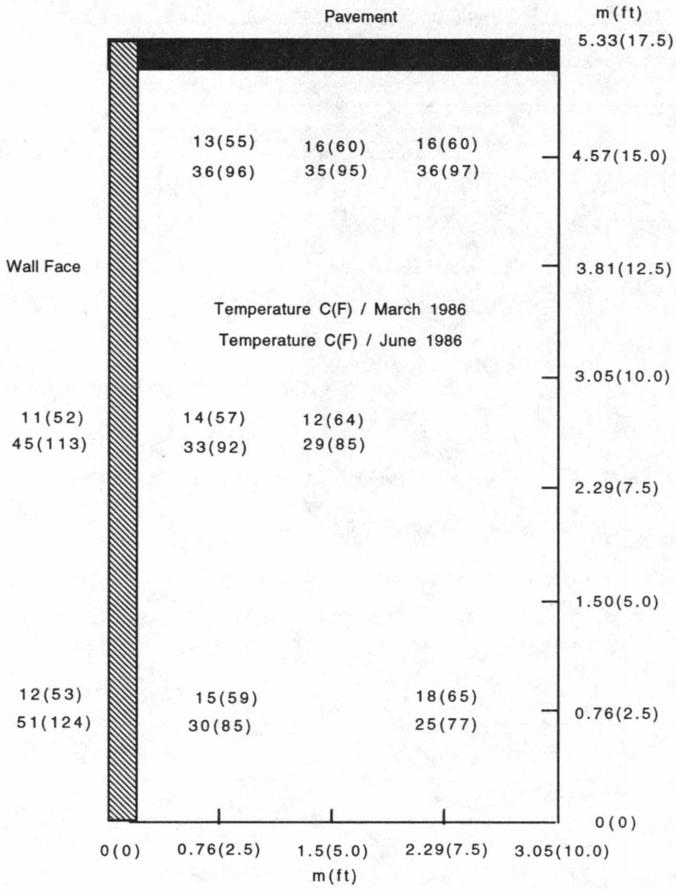

FIGURE 3 Temperature readings within a wall section.

ture of this type in service and because of the elevated temperatures associated with the project's location in the Sonoran Desert of Arizona.

EXHUMATION

A sample 1 m wide by 2.5 m long of the geogrid soil reinforcing elements from the Tanque Verde project was exhumed in August 1993. The sample was hand excavated from a Type A wall (Figure 2) adjacent to Tanque Verde Road (Figure 4). Figure 5 shows the ground surface before excavation; Figure 6 shows the texture and composition of the excavated reinforced fill. The sample was located 0.3 to 0.4 m beneath the finished grade behind the wall face, as shown in Figure 7. Once the bulk of reinforced fill was removed with shovels, the geogrid was brushed clean with a small whisk broom to avoid exhumation damage. Figure 8 shows the geogrid brushed clean before its removal. Figure 9 is a representative close-up of the geogrid before its removal. Also this geogrid sample was exhumed from above a very active ant nest.

GEOGRID MATERIAL

The soil reinforcing element used on this project is a Tensar SR2 geogrid. It is a uniaxial product fabricated by punching, reheating, and drawing an extruded sheet of high molecular weight, high-density polyethylene (HMW HDPE). The geometry of the geogrid is illustrated in Figure 10. Drawing increases the molecular orientation of the HMW HDPE, enhancing the tensile and modulus properties of the geogrid.

This HMW HDPE is classified as a Type III, Class A, Category 5, Grade E5 resin per ASTM D1248. The geogrid composition, by weight, and constituents are 97+ percent HMW HDPE, 2+ percent carbon black, plus antioxidants.

The specimens used for comparison testing are from an archived roll of SR2 geogrid manufactured during the same era (within 1 year) as the geogrid used in the Tanque Verde project. Because the archived geogrid is not from the same lot as the exhumed geogrid, some small variation in the values of measured properties is likely between the two geogrid samples.

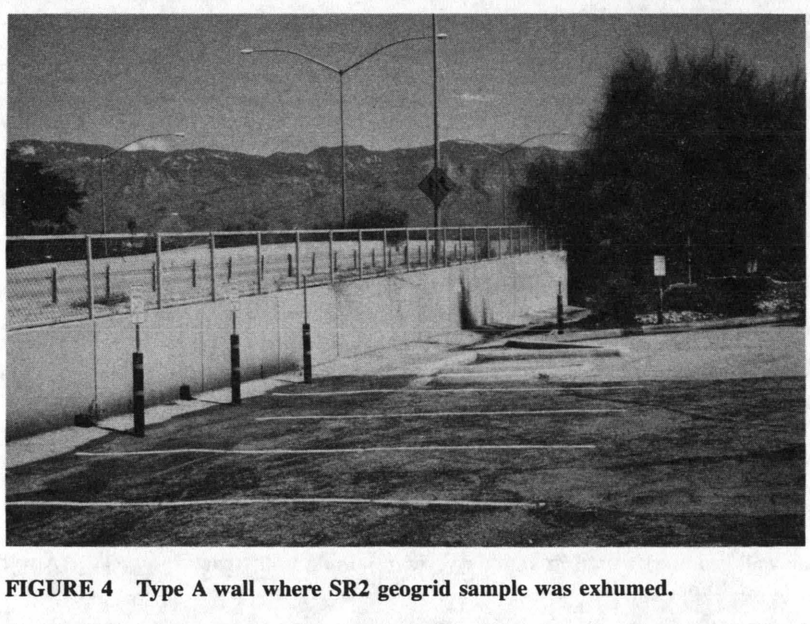

FIGURE 4 Type A wall where SR2 geogrid sample was exhumed.

FIGURE 5 Ground surface at excavation site of SR2 geogrid sample.

FIGURE 6 Representative texture and composition of excavated reinforced fill.

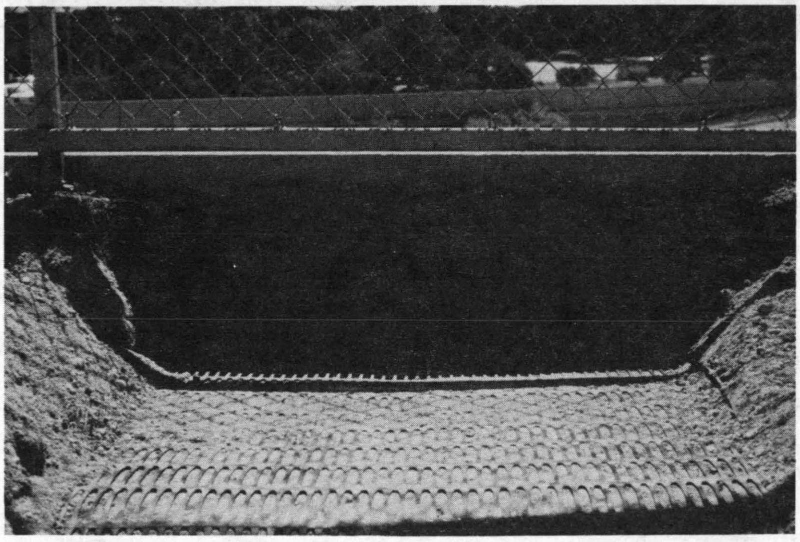

FIGURE 7 Uncovered SR2 geogrid toward Type A wall.

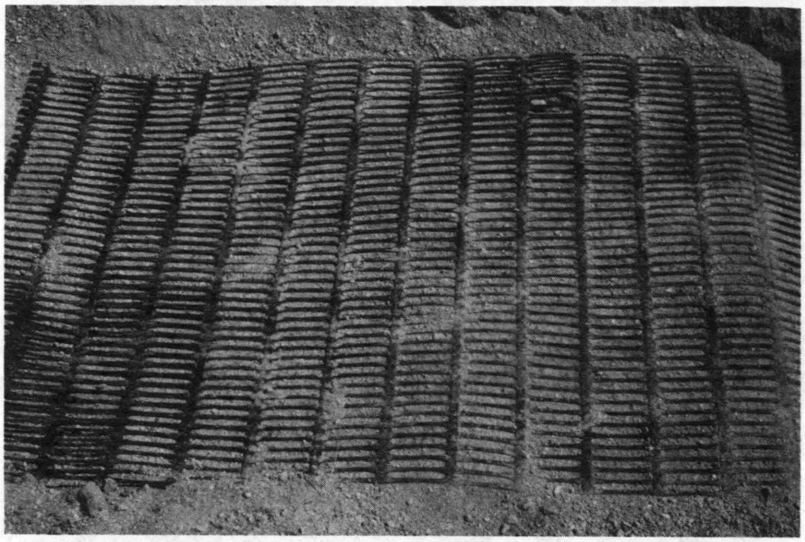

FIGURE 8 SR2 geogrid brushed clean, showing no evidence of physical damage.

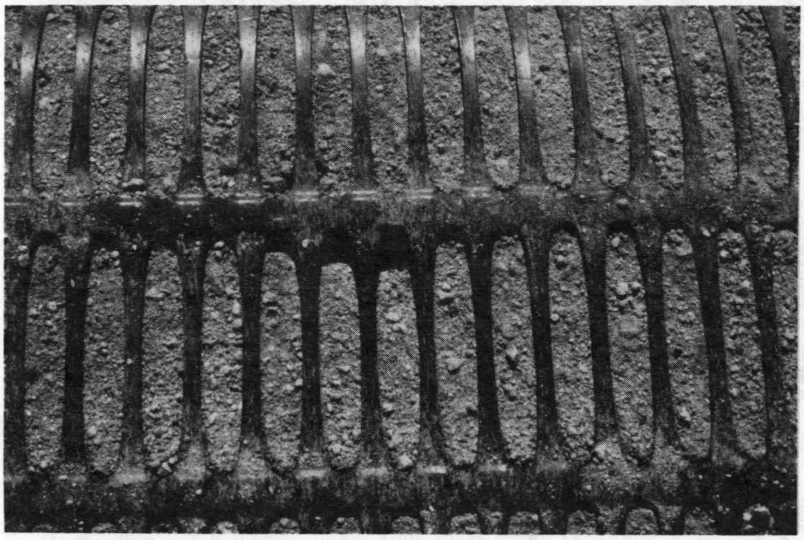

FIGURE 9 Close-up of SR2 geogrid, showing no evidence of physical damage and presence of a glossy surface.

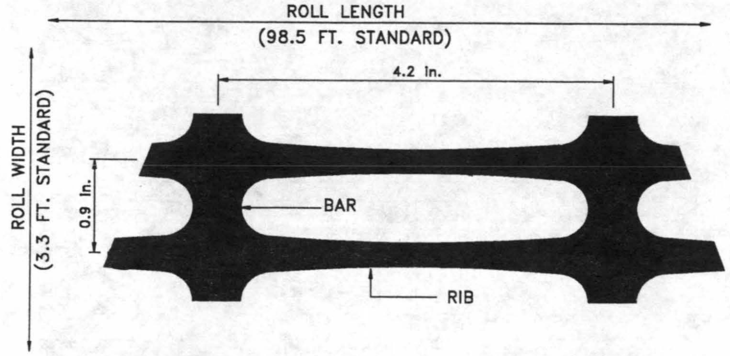

FIGURE 10 Geometry of SR2 geogrid

TEST PROTOCOL AND STANDARDS

Deterioration of geosynthetics may occur due to physical damage (i.e., installation damage), mechanical deformation (i.e., dimensional change, tensile and elongation behavior, creep response), thermal degradation (i.e., transition temperatures, crystallinity, oxidation), and biological degradation (i.e., attack by macro- and microorganisms). Thus visual, physical, mechanical, thermal, and compositional tests have been performed with specimens from the archived and exhumed geogrid samples.

Visual analysis, using a photographic record, assesses the presence and extent of installation, exhumation, and macroorganism damage across the surface topology. Scanning electron micrographs show the extent of such damage, that is, surface degradation due to attack by oxidation (surface dullness), soil chemistries, and microorganisms. Physical tests assess dimensional stability due to annealing, as well as subsequent densification via geogrid shrinkage due to prolonged exposure to an elevated temperature environment. Mechanical tests assess retention of tensile and elongation properties and the behavioral response to a constant sustained load. But these tests cannot necessarily differentiate between the mechanisms of mechanical deformation. Thermal tests, however, may assess any significant changes in the morphological status of the HMW HDPE that may relate to changes in mechanical properties. Composition tests indicate the residual amount of the principal additive, carbon black. Comparison with original formulations documents any concentration changes, thus indicating the duration of long-term stability.

The parameters evaluated, the related tests and applicable standards, and the number of specimens (i.e., archived and exhumed) per test are summarized in Table 1. The ASTM standards are taken from the *1993 Annual Book of ASTM Standards* published by ASTM, Philadelphia. The GRI standards are taken from *Test Methods and Standards* published by the Geosynthetic Research Institute at Drexel University in Philadelphia. The industrial standard for oxidative induction times (OITs) stipulates that a specimen be heated in a calorimeter at a rapid rate in an inert atmosphere (i.e., nitrogen) to a temperature usually above the melt range of the polymer (e.g., 200°C), allowed to thermally equilibrate, and then be switched to an oxygen atmosphere and timed and recorded to the onset of an exotherm. An exotherm indicates oxidative degradation in progress.

TEST RESULTS

Physical and mechanical test results and resin properties are summarized in Table 2. For tests using multiple specimens, results in Table 2 are reported as arithmetic averages with standard deviation. Figure 11 shows the unconfined tension creep evaluation of the exhumed SR2 geogrid in progress. Figure 12 shows the creep response of specimens taken from two rolls of SR2 geogrid manufactured in the 1984–1985 era, but within 1 year, and loaded to 31.7 kN/m, equivalent to 2,170 lb/ft. One roll was chosen for quality control (QC) testing following its manufacture; the other roll went into the Tanque Verde project and was exhumed more than 8 years later for evaluation. Figure 13 shows the creep response of specimens taken from two rolls of SR2 geogrid manufactured in the 1984–1985 era also, and within 1 year; both specimens are loaded to the design loading of 29 kN/m (2,000 lb/ft). One roll was archived; the other roll is the same as in Figure 12. Results of thermal analysis of geogrid components are summarized in Table 3. Melt transition and crystallinity sample sizes ranged from 9.4 to 10.2 mg. Only one thermogram was run per component because specimens were taken at random and results were typical of HMW HDPE.

A soil sample was taken from the exhumation site at Tanque Verde in Tucson, Arizona, for analysis; the soil fractions were 15.5/81.8/2.7, by weight percent, for gravel, sand, and fines, respectively. Soil pH was 8.0 and 8.7 in distilled water, and 7.6 and 7.8 in a 0.01 molal solution of $CaCl_2$, respectively.

DISCUSSION OF RESULTS

Visual Assessment

Photographs of the exhumation site at the Tanque Verde–Wrightstown–Pantano Roads intersection are presented as Figures 4 and 5. Figure 6 shows the texture and nature in the reinforced

TABLE 1 Properties and Related Tests and Specific Standards

Tests	Parameters	Standards	Number of Specimens Archived Sample	Number of Specimens Exhumed Sample
Physical	Mass / Area	ASTM D 3776	1	1
	Thickness, Rib	ASTM D 5199	10	10
	Thickness, Node	ASTM D 5199	10	10
Mechanical	Rib Strength	GRI GG1	10	10
	Junction Strength	GRI GG2	10	10
	Wide Width Strength	ASTM D 4595	10	10
	Tension Creep Behavior	ASTM D 5262	1	1
Thermal	Transition Temperatures	ASTM D 3418	3	3
	Heat of Crystallization	ASTM D 3417	3	3
	Oxidative Induction Time	Industry	3	3
Resin	Density/Specific Gravity	ASTM D 0792	2	2
	Melt Flow Index	ASTM D 1238	2	2
	Carbon Black Content	ASTM D 4218	1	1
Soil	pH	ASTM D 4972	n/a	2

TABLE 2 **Physical and Mechanical Test Results and Resin Properties**

Parameters	Number of Specimens		Number of Specimens	
	Average Value	Standard Deviation	Average Value	Standard Deviation
Mass / Area (g/m²)	912.	-	930.	-
Rib Thickness (in)	0.054	0.009	0.054	0.004
Junction Thickness (in)	0.178	0.002	0.177	0.001
Single Rib Strength (kN/m)				
Max. Load @ % Strain	85.0/15.2	1.79/1.15	85.0/14.0	0.45/0.64
Load @ 5 % Strain	46.8	0.85	48.6	0.35
Load @ 2 % Strain	26.2	0.65	26.5	0.29
Junction Strength (kN/m)				
Max. Load @ % Strain	84.6/16.4	0.63/0.59	83.7/16.0	0.42/0.55
Load @ 5 % Strain	46.1	0.40	47.0	0.57
Load @ 2 % Strain	26.1	0.42	25.9	0.41
Wide Width Strength (kN/m)				
Max. Load @ % Strain	78.0/15.3	3.0/1.28	78.0/14.0	2.2/1.21
Load @ 5 % Strain	44.6	0.40	43.3	0.30
Load @ 2 % Strain	25.6	0.45	23.6	0.37
Density (g/cc)	0.9530	0.0005	0.9595	0.0002
Melt Flow Index (g/10 min)	0.225	0.007	0.207	0.004
Carbon Black Concentration (%)	2.06	0.020	2.96	0.031

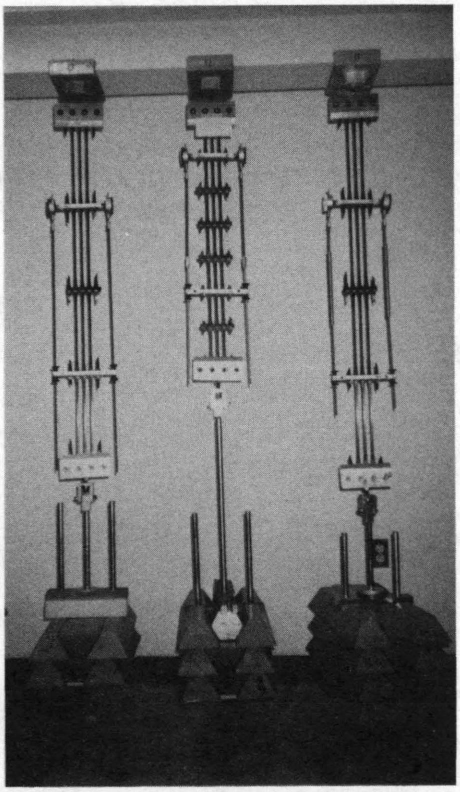

FIGURE 11 Unconfined tension creep evaluation of exhumed SR2 geogrid (center).

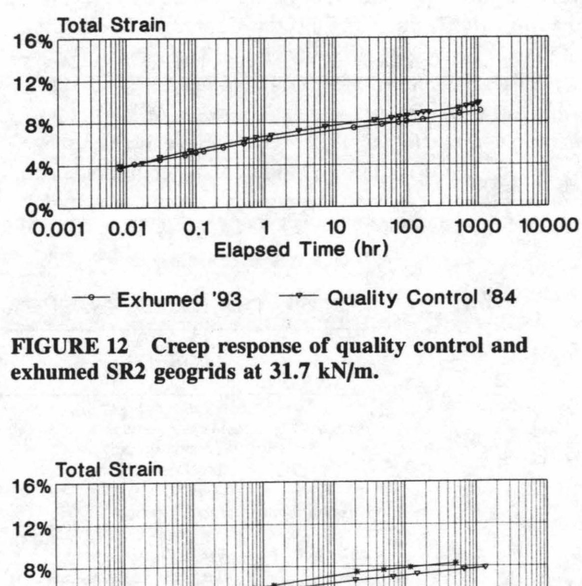

FIGURE 12 Creep response of quality control and exhumed SR2 geogrids at 31.7 kN/m.

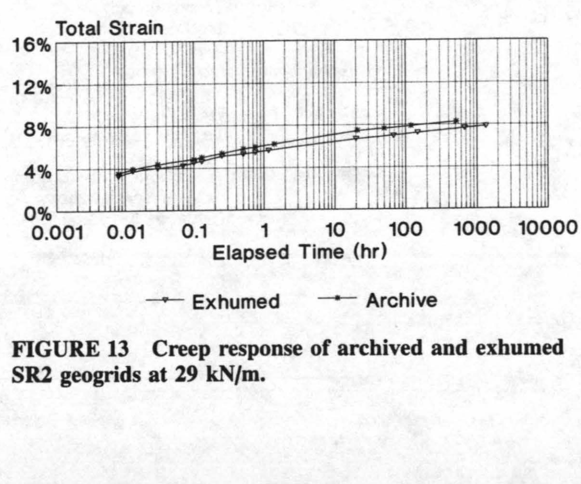

FIGURE 13 Creep response of archived and exhumed SR2 geogrids at 29 kN/m.

TABLE 3 Thermal Analysis Results

Parameter	Number of Specimens			Number of Specimens		
	Rib @ Mid Length	Node	In-Between Nodes	Rib @ Mid Length	Node	In-Between Nodes
Melt Range (°C)	90-139	93-134	91-136	90-134	93-133	92-133
Melt Peak (°C)	130	130	130	130	130	130
Crystallinity (%)	58	54	57	58	55	56
OIT (minutes)	6.7	8.0	7.4	4.7	4.3	3.9

fill, which was well-compacted and uniform. Figure 8 shows no broken or cut ribs over the surface of the exhumed geogrid sample; thus this sample experienced no significant installation or exhumation damage. There was no evidence of surface degradation due to attack by the resident ant colony; apparently the ants simply were not interested in a geogrid of HMW HDPE. Typically oxidation starts on the surface and progresses inward. An oxidized surface of a polyolefin, like polyethylene, will appear dull or tarnished. But surface quality is glossy, as evident in Figure 9, indicating no oxidative degradation in progress. Thus topological analysis by scanning electron microscopy was not warranted.

Physical Tests

Physical test results show no significant change in dimensional properties throughout 8-plus years. Rib and junction thicknesses exhibit zero change. The change recorded for mass per unit area is within the variance of specification for SR2 geogrid.

Mechanical Test Results

Test results show no significant change in tensile strength measured at 2 percent and 5 percent strain levels and at maximum loading and corresponding strain between the two samples of geogrid. The ultimate tensile strength values for a single rib are above the 79.0-kN/m product specification. The average tensile strength values plus one standard deviation for the exhumed sample fall within, or significantly overlap, the corresponding average values plus one standard deviation for the archived sample. Thus there is no significant change in single-rib, junction, or wide-width tensile strengths between archived and exhumed geogrid manufactured during the same era (within 1 year). There is also no indication of loss in ductility, or embrittlement, of the exhumed geogrid with time.

Creep Response

Figure 12 shows the creep response of two SR2 geogrid specimens manufactured in the 1984–1985 era; one taken from a roll selected for QC assessment and the other exhumed from the Tanque Verde project, both at the same loading of 31.7 kN/m. After 1,000 hr, both specimens exhibited a parallel response to the same constant, sustained loading, indicating that the mechanism by which creep occurs is the same. This behavior indicates that more than 8 years of exposure to an elevated temperature environment has not changed the creep response of aged versus production SR2 geogrid. Figure 13 shows the creep response of exhumed and archived SR2 geogrid specimens at the design loading, 29 kN/m for a total strain response of less than 10 percent. The archived geogrid has been in storage since its manufacture in 1984. At about 1000 hours, the two specimens are becoming asymptotic to less than 8.5 percent total strain. The two response curves are essentially parallel, indicating that the mechanism by which creep occurs is the same within the two geogrid specimens. Although these two geogrids are from different lot numbers, as discussed earlier, their response with time to a constant, sustained load is essentially the same; in addition, the mechanism by which the geogrid specimens respond is identical and thus has not changed over more than 8 years' duration and exposure to different aging temperatures, ambient and elevated, as documented in Figure 3.

Resin Properties

Resin density was determined from the extrudate from a melt flow index tester; no significant changes in morphology occurred over the duration. Melt flow index values indicate no change in the molecular weight of the resin over the duration. Any significant change in molecular weight would be reflected in corresponding changes in mechanical strength, for which there were no changes. However a 0.022-g/10-min difference does indicate that the samples came from different production lots, also indicated by the values on carbon black (CB) concentration.

CB specification for SR2 was 1 to 3 percent by weight in 1984, and 2+ percent is known and accepted to be sufficient to retard long-term degradation of HDPE due to exposure to ultraviolet light, which is of no concern here. The difference in CB concentration has not affected the mechanical properties of the exhumed geogrid relative to the archived geogrid; a higher CB concentration, if anything, would slightly lower mechanical properties initially. Any significant change in ductility or embrittlement would increase strength values with a corresponding decrease in strain values. As discussed earlier, this has not occurred.

Thermal Test Results

Melt range, peak temperature, and crystallinity data indicate no significant changes in the morphological state of the HMW HDPE geogrid, at various locations within the geogrid configuration, over the duration of the project in an elevated temperature environment. Crystallinity data verify that there was no significant change in the resin density reported in Table 2, because for semicrystalline thermoplastics, changes in morphological density are usually reflected in corresponding crystallinity. Any significant changes in either density or crystallinity should be reflected in mechanical property results, which is not the case. Such changes would not necessarily reflect a change in molecular weight.

OITs for the archived and exhumed samples are given in Table 3. A single test was run for each specimen identified by location within the basic geogrid configuration. OIT values of the archived SR2 sample are 7 to 8 min, typical of values reported by the resin supplier for the antioxidant package used in 1984 and evaluated at 200°C. OIT values for the exhumed sample are 4 to 5 min. For an average time duration of 6 ± 2 min, the OIT values of the archived and exhumed specimens are, within experimental reproducibility and significance for this test, essentially equal. The mass/area, melt flow index, and CB values clearly indicate that the test samples are from different production lots, as do the OIT values. Thus some difference in OIT values is expected between the archived and exhumed samples.

CONCLUSIONS

The Tanque Verde and archived geogrid samples were produced about 1984 but were from different production lots. The HMW HDPE geogrid experienced no significant installation or exhumation damage and exhibited no evidence of biological attack or surface oxidation. No significant change in physical, mechanical, thermal, or resin properties occurred throughout 8 or more years in service. The creep behavior of the archived and exhumed Tanque Verge geogrids is essentially identical and indicates that the mechanism by which creep occurs has not changed throughout the years in service. Thus assessment has shown that the first project using a geogrid of HMW HDPE as reinforcement in a concrete-faced, MSE retaining wall in a major transportation-related application in North America has not experienced any significant change in the performance and physical properties of the geogrid or in the morphological properties of its resin in more than 8 years of exposure to an elevated temperature environment.

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