

UNIVERSITY OF MINNESOTA

**INTERIM REPORT:
FABRIC FOR
REINFORCEMENT AND SEPARATION
IN UNPAVED ROADS**

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**Department of
Civil Engineering
Institute of Technology**

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FABRIC FOR
REINFORCEMENT AND SEPARATION
IN UNPAVED ROADS**

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**The Minnesota Department
of Transportation**

by

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EXECUTIVE SUMMARY

The use of fabric in unpaved roads has proven to be cost-effective by reducing the amount of aggregate needed structurally—the reinforcement mechanism—and by eliminating the need for additional aggregate because of mixing at the soil-gravel interface—the separation mechanism. Continued cost savings also have been recognized through the loss of less aggregate over the road's lifetime and through the stabilizing effect of the fabric. Many agencies have successfully used geotextiles for reinforcement and separation purposes in unpaved road design. Some agencies have also experienced failures. Through both the successes and failures, it is clear that many questions remain as to how reinforced unpaved roads are designed. For example: What are the mechanisms that control road design? Is there some system property that can be efficiently evaluated and used as a deciding parameter in design? This report provides an overview of the mechanics of the soil-fabric-aggregate system, and a reference for use in design.

A vast amount of information has been published in journals, periodicals, and theses pertaining to the use of fabrics for reinforcement and separation in unpaved roads. Key findings of the literature review include the following:

- Geotextile reinforcement may have a substantial benefit (possibly 75 mm of granular equivalence) when large rutting is allowed and the subgrade soil has an undrained shear strength $c_u < 30kPa$ or $\sigma/c_u < 8$, where σ is the vertical stress on the subgrade.
- Generally, lightweight ($<3.5 \text{ oz/yd}^2$) nonwoven fabric should not be used for separation because of damage during installation and placement of the aggregate. The separation function is often times the major benefit in using geotextiles.
- Besides aggregate and soil characteristics, the parameters of an unpaved reinforced road controlling design are fabric modulus (stiffness) and interface friction (shear resistance).
- Existing vendor information pertaining to geotextiles is vast, but a summary is provided in the Appendix for application to reinforcement and separation in unpaved roads.

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1. INTRODUCTION

This report summarizes a review of literature pertaining to the use of geotextiles for reinforcement and separation in unpaved roads, and is organized by the following four topics:

- Reinforcement in Unpaved Roads
- Factors Affecting Reinforced Unpaved Roads
- Separation in Unpaved Roads
- Case Studies

The concept of a geotextile providing support to unpaved roads started with work performed at the University of Illinois by Barenberg et al. (1975) and Kinney (1978), and led to the development of a reinforced unpaved low-volume road design approach by Giroud and Noiray (1981). After the publication of this paper, the research and interest in the topic of reinforcing unpaved roads with geotextiles quickly escalated. Based on observations of field performance, the U.S. Forest Service (1977) also suggested design procedures for unpaved roads using geotextiles for reinforcement. However, all of these design procedures assume large rutting is permitted so as to develop membrane support. More recent work (Espinoza, 1994; Espinoza and Bray, 1995) has recognized that even for low-deformation systems, where large rutting is not permissible and thus membrane support is not present, a reinforcement mechanism may exist due to an increased shear resistance at the aggregate-geotextile interface.

While the use of geotextiles for reinforcing unpaved roads continued to be investigated, others started to look at the use of geotextiles purely for separation in unpaved roads. Douglas et al. (1985) found that the reinforcement function of geotextiles in unpaved roads was insignificant and that the primary reason for improved performance

was separation. It is well known that many engineering properties of granular materials, such as shear strength, permeability, resilient modulus and frost action potential, are adversely affected by an increase in fines content. For example, about 20% by weight of subgrade soil mixed into the aggregate base can reduce the bearing capacity of the aggregate base to that of the subgrade soil (Yoder, 1959). Thus, the mixing of aggregate and subgrade compromise the structural integrity of the road.

Brorsson and Eriksson (1986) reported on the long-term properties of geotextiles and their function as a separator. Geotextiles were placed on soft and saturated frost susceptible clayey silt and silty clay subgrades to act as a separator between the aggregate base and subgrade. After five and ten years, geotextile samples were excavated and the subgrade and base materials were examined. All of the geotextiles sampled were found to have performed the separation function satisfactorily. It was observed that the subgrade material was firm, dry and well consolidated. These findings point to the idea that separation is a useful function while the subgrade is consolidating. Haliburton and Barron (1983) stated that, although the use of fabric for separation does not in itself strengthen the road system, it does allow for the dissipation of excess pore pressures and subgrade consolidation, which will lead to long-term subgrade strength improvement. This allows the road to improve, rather than degrade, with time and number of load repetitions.

2. REINFORCEMENT IN UNPAVED ROADS

This section deals specifically with the research on geotextile reinforced unpaved roads that was conducted by the following four groups:

- Barenberg (1975,1980) and Kinney (1979) – University of Illinois, Urbana-Champaign
- Giroud and Noiray (1981) – Woodward-Clyde
- Burd and Brocklehurst (1990) and Houlsby and Jewell (1990) – Oxford
- Espinoza (1994) and Espinoza and Bray (1995) – University of California, Berkeley

A vast amount of research on geotextiles for reinforcement in unpaved roads has occurred over the last 20 years. A major advancement in this area occurred with the development of an analytical treatment of unpaved road design with geotextile reinforcement performed by Giroud and Noiray (1981), based on the concept of membrane-type reinforcement. The membrane effect refers to the reinforcement provided by a geosynthetic when rutting (large deformation) is permitted, stretching and allowing the geotextile to provide some vertical (membrane) support.

Giroud and Noiray's design procedure for geotextile reinforced unpaved roads came shortly after Kinney (1979) completed a study on large deformation soil-fabric-aggregate systems (commonly referred to as SFA systems), where a so-called fabric tension model was introduced. This model used the notion that when a fabric is stretched over a curved surface, there is a higher normal stress on the aggregate side of the geotextile than on the subgrade side. Kinney (1979) defined this difference in normal stresses as the fabric induced normal stress. Giroud and Noiray (1981) defined it as membrane support. Prior to Kinney's work with SFA systems where large rutting was permitted, Barenberg (1975) developed a design procedure for the use of Mirafi[®] 140 in SFA systems. Membrane type reinforcement is now commonly recognized and applied to

the design of unpaved haul or access roads where large rutting is permitted. Since it is necessary to have rutting to generate membrane support, researchers have recently started investigating other possible reinforcing mechanisms that may be present without rutting. Burd and Brocklehurst (1990) and Espinoza (1994; 1995) looked at a possible shear reinforcing mechanism that may be present for small rut depths and unaffected by anchorage.

2.1 Membrane Reinforcement

2.1.1 Barenberg's Procedure

Barenberg et al. (1975) and Bender and Barenberg (1978) developed design criteria for the use of Mirafi® 140 and 280 in soil-fabric-aggregate (SFA) systems, from which design charts were generated. The design criteria were based on the findings of small scale laboratory tests on SFA systems (Fig. 1).

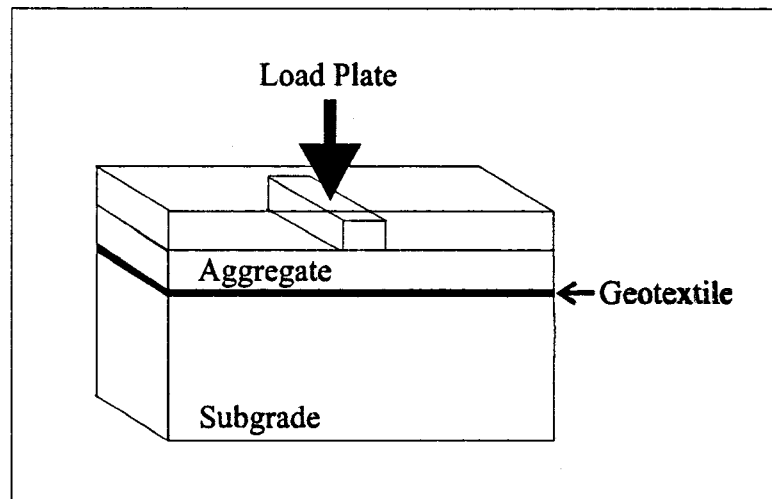


Figure 1. Testing Apparatus

All experiments consisted of a soft Goose Lake Clay subgrade soil and a layer of crushed limestone separated by a layer or layers of Mirafi® fabric. Different mathematical models were used:

1. Boussinesq – Yoder (1959)
2. Shear Layer Theory – Barenberg (1965)
3. Transformed Section Concept – Ferguson (1972)

The Boussinesq solution for determining stresses involved a uniform load applied over a circular area of an *elastic* half space. However, this theory is intended for single-layer elastic systems. The system Barenberg looked at was neither single-layered nor elastic.

Shear layer theory was developed by Barenberg (1965) and is based on the assumption that a layer of material can transmit stresses only by shear deformations when supported by a dense liquid subgrade. High horizontal stresses develop at the aggregate-subgrade interface, causing failure. This theory, however, does not take into account plastic deformations, and therefore may not be useful for predicting rut depth or the rate of rut depth development.

The transformed section concept used was the same as that used for reinforced concrete design. By transforming the SFA system to an equivalent cross-sectional area of a selected material, the system behavior can be analyzed. Barenberg (1975) found this concept useful in the elastic range of deformations. However, most systems are controlled by plastic deformations, and so this concept was not good for predicting rut depths.

For the 2D tests performed in the apparatus shown in Fig. 1, no **significant** difference in the performance of the systems with Mirafi[®] 140 and 280 was observed. The systems with fabric tended to reach some level of permanent deformation after which the system stabilized and further load applications of the same magnitude caused little or no additional permanent deformation. The critical stress level to cause excessive permanent deformation was found to be about $3.2c_u$ for systems without fabric and $6c_u$ for systems with fabric, where c_u is the undrained shear strength. It was recommended that if the number of load cycles expected over the roadway is in excess of 10,000, the depth of aggregate required to stabilize the roadway in terms of minimal rut depth should be increased by 10% above the value of aggregate depth calculated using the critical stress

levels given above. This correction factor should be applied for each increment of 10,000 loads above the initial level of excessive loading (that is, the first 10,000 loads). This correction is purely empirical and based on work performed by the Army Corps of Engineers (Ahlvin, 1962).

Barenberg (1975) also found that fabrics were effective in preventing the intrusion of subgrade soil into the aggregate layer and stabilizing the aggregate. By including fabric in the soil-aggregate system, an increase in the effective supporting capacity of the subgrade was realized. Under certain conditions, the use of fabric was found to result in significant aggregate savings, with approximately a *one-third* reduction in aggregate thickness.

2.1.2 Kinney's Fabric Tension Model

Kinney (1979) developed the fabric tension model, which is a mechanistic description of the behavior of the geotextile in an SFA system. It included methods to determine the geotextile induced shear and normal stresses and the strain energy stored in the geotextile. This model was based on the concept that outward movement of the aggregate and the clay causes outward shear stresses on the fabric, which creates tension in the fabric. When a fabric is stretched over a curved surface, there is a higher normal stress on the aggregate side of the fabric than on the subgrade side. The fabric resists stretching, causing inward-directed shear stresses on the gravel and the clay. Hence, the fabric indirectly causes changes in the behavior of the system.

The geotextile induced shear and normal stresses can be calculated if the tension in the geotextile and the deformed shape of the geotextile are both known. The sum of the shear stresses on the two sides of the fabric is equal to the rate of change in the tension per unit width in the fabric along the length of the fabric. The distribution of shear stresses between the top and the bottom of the fabric is not easily determined unless the fabric is slipping with respect to one or both of the materials. If slip does occur, the shear stress on

that side is equal to the maximum available, which is governed by the friction and adhesion between the fabric and the adjoining material and by the normal stress on that interface. The geotextile induced normal stress is equal to the tension in the geotextile per unit width divided by the radius of curvature.

The strain energy in the geotextile can be calculated if the tension-strain history of the geotextile is known. The general expression for the strain energy is

$$E_f = \int_S \left[\int_E T d\varepsilon \right] ds \quad (1)$$

where E_f = strain energy stored in geotextile
 T = tensile force per unit width
 ε = strain
 S = geotextile surface area

If it is assumed that the dynamic tension-strain relationship for the geotextile is linear and the problem is considered two dimensional, the following relationship results:

$$E_f = \frac{W}{2E_s} \int_L (T_p^2 - T_i^2) dL \quad (2)$$

where W = geotextile width being stressed
 T_p = tensile force per unit width under peak load.
 T_i = residual tensile force per unit width between loading
 E_s = modulus
 L = geotextile length

Using the technique outlined above the normal stress on the geotextile, shear stress developed by the geotextile on the aggregate and subgrade, and strain energy stored in the geotextile can be determined.

2.1.3 Barenberg's Revised Procedure

Barenberg's (1975) earlier design procedure for SFA systems was based on the concept of limiting, to acceptable values, the ratio of vertical stress (σ_z) transmitted to the subgrade to strength of the subgrade (c_u). A limitation of Barenberg's procedure was that it was based on tests performed on one fabric type, Mirafi® 140. At the time of Barenberg's (1975) work, the effect of fabric properties on the behavior and performance of an SFA system was unknown. Kinney (1979) found that the fabric's modulus significantly affected the performance of an SFA system. Fabrics with a higher modulus were found to perform better than fabrics with a lower modulus. The relationship between a fabric's modulus and road performance was further examined by Kinney (1979) and resulted in the development of the fabric tension model described in section 2.1.2.

Barenberg's (1980) revised design procedure incorporated the fabric tension model developed by Kinney (1979) with the design criteria and design procedures for Mirafi® 140 in SFA systems, to generate a more comprehensive design procedure for Mirafi® 500X fabric. Barenberg (1980) showed that when a load is applied to an SFA system, the normal stress transmitted to the subgrade may exceed the allowable stress. As the rut deepens, the fabric deforms and takes on a permanent tension, and a portion of the load is then carried by the fabric. Rutting continues to deepen with an increase in the fabric's tension until the normal stress transmitted to the subgrade is equal to the permissible subgrade stress. At this point, the system becomes stable and no further rutting occurs. Barenberg (1980) recommended that a permissible stress on the subgrade, σ , be adjusted to compensate for the effect of the fabric modulus on the failure criterion, that is $\sigma = A * \pi * c_u$, where A , a coefficient related to the confining effects of the fabric on the soil, is one when no fabric is used and varies up to two, and c_u is the undrained shear strength of the soil.

In this model, the geometry of the deflected shape of the fabric was assumed as a circular arc and the estimated rut depth was calculated based on this assumption. The differential normal stress is the normal stress difference between the top and bottom side of the fabric, the difference being the stress carried by the fabric. Design curves have been developed for different wheel loads using this model and assuming an average contact pressure per wheel of 550 kPa (80 psi).

Revised Design Procedure

1. Determine the wheel load and contact pressure anticipated on the surface of the SFA system.

For dual tires as a single load, a contact pressure of 0.7-0.8 times the air pressure can be assumed. All loaded areas are assumed to have uniform pressure over a circular area.

2. Determine the maximum allowable stress, σ_{all} , for the subgrade:

$$\sigma_{all} = 3.2Ac_u \tag{3}$$

where c_u = subgrade undrained shear strength

A = coefficient related to confining effects of fabric

Recommended Values for Coefficient A	
Fabric	Coefficient A
None	1.0
Mirafi® 140	1.9
Mirafi® 500X	2.0

3. Estimate required aggregate layer thickness.

Use 70-75% of aggregate thickness determined for unreinforced system.

4. Establish the rut geometry including width and maximum rut depth both on the aggregate surface and at the interface.

Subgrade rut depth, d , can for practical purposes be taken as the surface rut depth.

Subgrade rut width, W , can be estimated by the following equation:

$$W = B + 2X \tag{4}$$

where B = aggregate surface rut width

X = spreading effect of granular layer with and without fabric

Note: X varies from 0 inches up to 7-20 inches, increasing with an increase in the amount the geotextile spreads out the rut.

5. Using assumed rut geometry, calculate the strain in the geotextile.

$$\theta = 2 \tan^{-1} \left(\frac{5d}{3W} \right) \quad (5)$$

$$R = \frac{3W}{8 \sin \theta} \quad (6)$$

$$\text{Percent Strain in Fabric, } \varepsilon_f = \left(\frac{4\pi R\theta}{135W} - 2 \right) \times 100\% \quad (7)$$

6. Taking the product of the fabric strain, ε_f , times the fabric modulus, K , calculate the tension in the fabric, t_f .

$$t_f = K\varepsilon_f \quad (8)$$

7. Calculate the differential normal stress across the fabric due to the uplifting effect of the fabric.

The differential normal stress is the normal stress difference between the top and bottom side of the fabric. The summation of the differential normal stress over the fabric in the loaded region is the portion of the applied load that is carried by the fabric. For practical purposes, only the differential stress need be calculated and added to the permissible stress on the subgrade to determine the permissible stress on the surface of the fabric.

$$\Delta\sigma_{z-f} = \frac{t_f}{R} \quad (9)$$

where $\Delta\sigma_{z-f}$ = differential normal stress across the fabric

t_f = tension in fabric

R = radius of circular deflected shape

8. Calculate the permissible vertical stress on the top of the fabric by summing the maximum permissible stress on the subgrade plus the differential normal stress due to the uplift by the fabric tension:

$$\sigma_{p-f} = \Delta\sigma_{z-f} + 3.2Ac_u \quad (10)$$

where $3.2Ac_u$ is the allowable stress given in Equation 3

$\Delta\sigma_{z-f}$ is the differential stress across the fabric given in Equation 9

9. Calculate the maximum vertical stress, σ_z , on the fabric using the Boussinesq theory. If the maximum vertical stress on the fabric is greater than the permissible vertical stress on the fabric, increase the thickness of the granular layer and return to step 4.

$$\sigma_z = p \left[1 - \left(\frac{1}{1 + \left(\frac{a}{z} \right)^2} \right)^{\frac{3}{2}} \right] \quad (11)$$

where P = total applied load

p = average contact pressure

z = aggregate layer thickness

a = radius of loaded area = $\sqrt{\frac{P}{\pi p}}$

2.1.4 Giroud and Noiray's Analytical Design Procedure

Giroud and Noiray (1981) developed a method that enables the engineer to calculate the required thickness of the aggregate layer and make the proper selection of the geotextile to be used. The results were presented in the form of charts and are applicable only to purely cohesive subgrade soils and roads subjected to light - medium traffic (1-10,000 cycles).

The aggregate was assumed to have a CBR greater than 80. The subgrade soil was assumed to be saturated and to have low permeability and therefore, under quick loading, behave in an undrained manner. This means that the subgrade soil is incompressible, and in terms of total stresses, its friction angle is zero and shear strength is

simply a constant, c_u , the undrained shear strength. The geotextiles mechanical properties were described by the elongation behavior in a biaxial tensile test. For a woven geotextile, the tension-elongation curves obtained from a uniaxial test and a biaxial test are not very different. For a nonwoven geotextile, the difference is usually important and the biaxial test was strongly recommended. This study assumed that the geotextile is rough enough to prevent sliding of the aggregate layer along the geotextile surface (that is, a *no slip* condition).

The failure of an unpaved road can occur at different locations: the aggregate layer (although this was ignored), the foundation soil, and the geotextile. It was assumed that the friction coefficient of the aggregate layer was large enough to ensure the mechanical stability, and that the friction of the geotextile in contact with the aggregate layer under the wheels was large enough to prevent sliding of the aggregate layer on the geotextile. The aggregate layer was assumed to provide a pyramidal load distribution, and based on this assumption, equations for uniform normal stress at the base of the aggregate layer were developed as a function of the vertical angle of the load distribution pyramid. Experiments suggested that the tangent of this angle was between 0.5 and 0.7, while theoretical solutions yielded values for the tangent of the angle between 0.47 and 0.64. Preliminary calculations have shown that the value of the calculated aggregate thickness is not significantly influenced by the angle of load distribution if the tangent is between 0.5 and 0.7.

The design of an unpaved road without a geotextile was considered by comparing the maximum normal stress on the subgrade soil to the bearing capacity. Based on this analysis, charts were generated for the determination of aggregate thickness when the subgrade soil strength and the axle load are known. However, the charts developed should be restricted to very light traffic since they were established using quasi-static analysis.

Giroud and Noiray (1981) introduced the concept of membrane support for an unpaved road reinforced with a geotextile. Membrane support can be described as follows:

In the case of wheel loadings on a geotextile reinforced unpaved road, the normal stress applied through the aggregate and onto the geotextile is greater than the normal stress applied through the geotextile and onto the subgrade. The stress against its aggregate face is higher than the stress against its subgrade face. This stress difference is attributed to membrane support (Giroud and Noiray, 1981).

The geotextile takes on a wavy shape when deformed under the wheel loadings due to the relative incompressibility of the subgrade soil. Therefore, the volume of subgrade soil displaced downwards by the wheel loads equals the volume of soil displaced upwards. Giroud and Noiray (1981) assumed that the shape of the deformed geotextile consisted of parabolas and developed relationships between the settlement and the rut depth, which characterized the shape of the geotextile.

For unpaved roads without a geotextile, Giroud and Noiray (1981) presented two design approaches: (1) a quasi-static analysis leading to the required aggregate thickness and (2) an empirical method taking traffic into account. For unpaved roads with a geotextile, only a quasi-static analysis was presented for determining the required aggregate layer thickness.

Giroud and Noiray (1981) found that for typical geotextiles, the reduction of aggregate thickness resulting from the use of a geotextile generally ranged from 20-60% for a subgrade soil with a CBR value of one and a number of passages between 1,000 and 10,000. An increase in the allowable rut depth caused a decrease in the required thickness of the aggregate layer with and without a geotextile. Furthermore, the smaller the geotextile tensile modulus, the larger the required thickness of the aggregate layer.

Procedure

1. Determine axle load, P , and contact area, A_c :

$$P = 4A_c p_c \quad (12)$$

where p_c = tire contact pressure.

For dual wheel axle configurations shown in Fig. 2:

contact area $\approx 2A_c$

$$BL \approx 2A_c \sqrt{2} \quad (13)$$

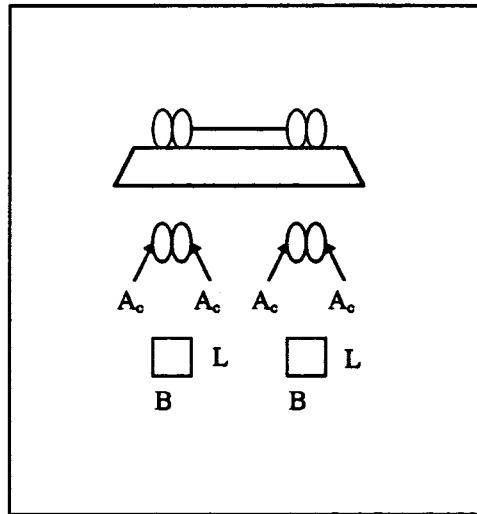


Figure 2. Dual wheel axle configuration.

2. Set $p_c = p_{ec}$, where p_{ec} is the “equivalent contact pressure” between area $B \times L$ and the aggregate layer:

$$P = 2LBp_{ec} \quad (14)$$

$$p_{ec} = \frac{P_c}{\sqrt{2}} \quad (15)$$

3. Typical dual tire print geometry is such that for

$$\text{on-highway trucks: } L = \frac{B}{\sqrt{2}} \quad (16)$$

$$\text{off-highway trucks: } L = \frac{B}{2} \quad (17)$$

Note: We will look at on-highway trucks; the derivation for off-highway trucks follows the same procedure.

4. Using Equations (15) and (16), L and p_{ec} can be eliminated from Equation (14):

$$B = \sqrt{\frac{P}{p_c}} \quad (18)$$

5. The relationship between normal stress on top of the aggregate, p_{ec} , with normal stress a distance h or h_o below is assumed to be pyramidal (Fig. 3):

$$\text{without geotextile: } p_{ec}LB = (B + 2h_o \tan \alpha_o)(L + 2h_o \tan \alpha_o)(p_o - \gamma h_o) \quad (19)$$

$$\text{with geotextile: } p_{ec}LB = (B + 2h \tan \alpha)(L + 2h \tan \alpha)(p - \gamma h) \quad (20)$$

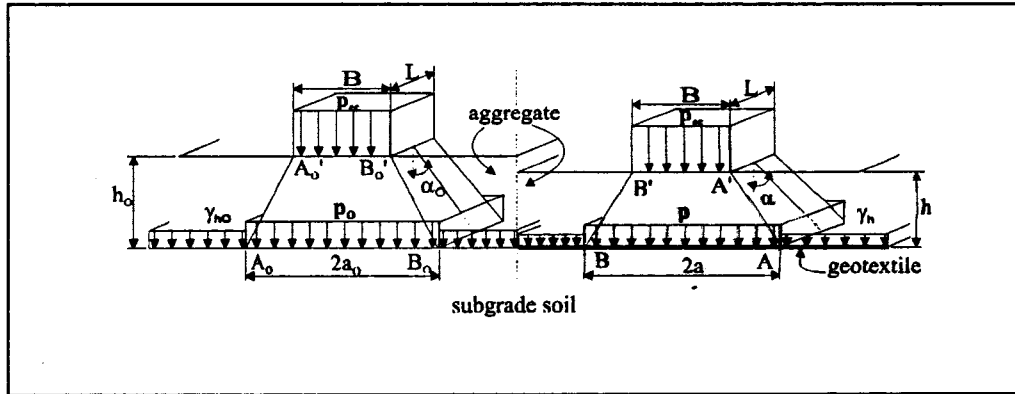


Figure 3. Pyramidal load distribution.

6. An expression for stress on top of the geotextile can be generated using Equations (14) and (20):

$$p = \frac{P}{2(B + 2h \tan \alpha)(L + 2h \tan \alpha)} + \gamma h \quad (21)$$

Note: $\tan \alpha$ is typically taken as 0.5-0.7.

7. The stress at the plastic limit, q_p , was assumed to be

$$q_p = (\pi + 2)c_u \quad (22)$$

where c_u = undrained shear strength of the subgrade.

8. Due to the assumed mechanism with 45° angles in the plastic zone, the depth of the plastic zone, H_p , (Fig. 4) can be expressed as:

$$H_p = a\sqrt{2} \quad (23)$$

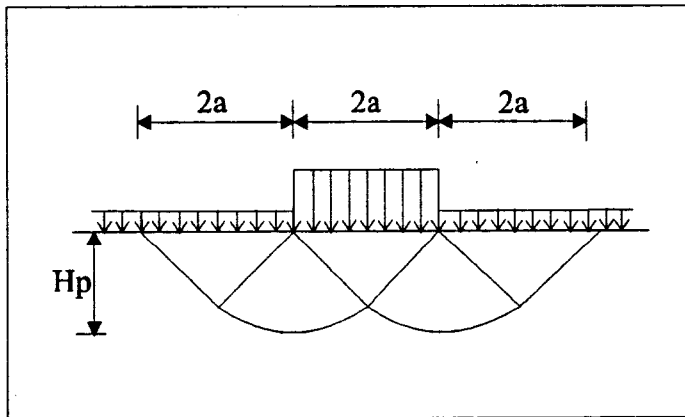


Figure 4. Plastic zone.

From Fig. 2, $2a = B + 2h \tan \alpha$, so the plastic zone height can be rewritten as follows :

$$H_p = \frac{2a + 2h \tan \alpha}{2} \sqrt{2} = \frac{B + 2h \tan \alpha}{\sqrt{2}} \quad (24)$$

9. The actual stress on the subgrade, p^* , and the stress reduction due to the geotextile, p_g , is given by

$$p^* = p - p_g \quad (25)$$

At the plastic limit $p^* = q_p$:

$$p - p_g = (\pi + 2)c_u \quad (26)$$

Note: p_g is a function of the tension in the geotextile.

10. Determination of p_g :

Assume that the deformed shape of the geotextile consists of parabolas (Fig. 5) and that the subgrade is incompressible; thus, the volume of the subgrade displaced downwards equals the volume heaved upwards.

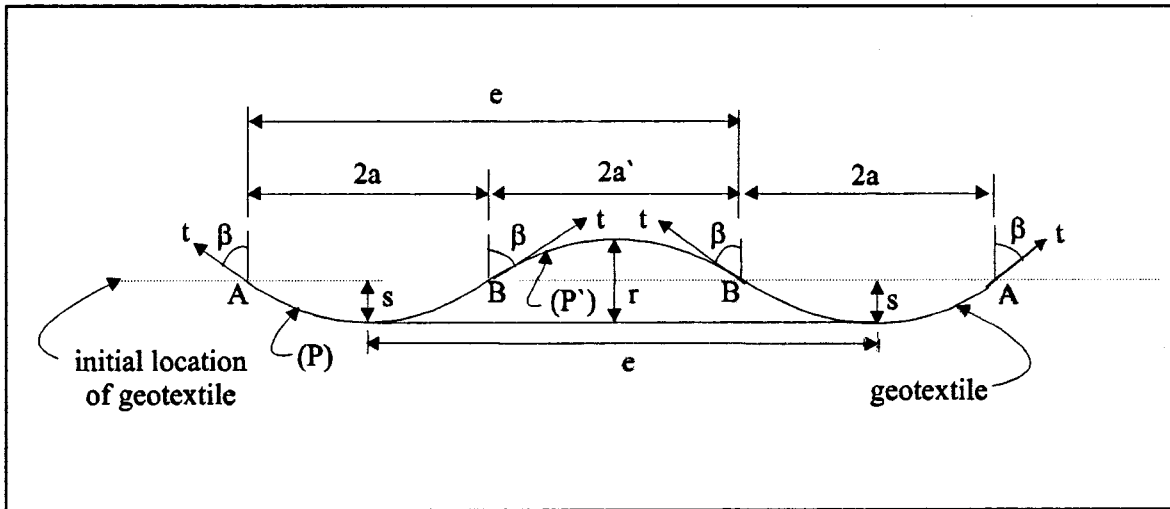


Figure 5. Geotextile deformed shape.

There are two cases: (1) the parabola horizontal distance between the wheel loads is greater than that beneath the wheel load (i.e. $a' > a$); (2) the parabola horizontal distance underneath the wheel load is greater than that between the wheel load (i.e. $a > a'$). From the geometry of a curved geotextile:

$$2a = B + 2h \tan \alpha \quad (27)$$

$$2a' = e - B - 2h \tan \alpha \quad (28)$$

$$a' > a: \quad s = \frac{ra'}{a+a'} \quad (29)$$

$$a > a': \quad s = \frac{2ra^2}{2a^2 + 3aa' - a'^2} \quad (30)$$

Geotextile Elongation

$a' > a$: Equation (29) becomes $\frac{s}{a} > \frac{r-s}{a'}$ and tension in P is greater than in P' .

P' is pulled towards P and uniform elongation, ε , is assumed:

$$\varepsilon = \frac{b+b'}{a+a'} - 1 \quad (31)$$

where a, a' = relative chord half-lengths

b, b' = half length of P and P'

$a > a'$: Equation (30) becomes $\frac{r-s}{a'} > \frac{s}{a}$ and tension in P' greater than in P .

Geotextile does not move because higher normal stresses on P and elongation, ε , is expressed as:

$$\varepsilon = \frac{b}{a} - 1 \quad (32)$$

Elongation, ε , calculated using the following relationships between an arc of a parabola and subtended chord:

$$\frac{b}{a} - 1 = \frac{1}{2} \left[\frac{a}{2s} \ln \left(\frac{2s}{a} + \sqrt{1 + \left(\frac{2s}{a} \right)^2} \right) - 2 + \sqrt{1 + \left(\frac{2s}{a} \right)^2} \right] \quad (33)$$

$$\frac{b'}{a'} - 1 = \frac{1}{2} \left[\frac{a'}{2(r-s)} \ln \left(\frac{2(r-s)}{a'} + \sqrt{1 + \left(\frac{2(r-s)}{a'} \right)^2} \right) - 2 + \sqrt{1 + \left(\frac{2(r-s)}{a'} \right)^2} \right] \quad (34)$$

From Figs. 4 and 5, the following two expression can be generated:

$$ap_g = t \cos \beta \quad (35)$$

$$\tan \beta = \frac{a}{2s} \quad (36)$$

where p_g = pressure reduction on the soil

Knowing the relationship between load, t , and strain, ε , for a geotextile:

$$t = K\varepsilon$$

the following expression for p_g can be derived:

$$p_g = \frac{K\varepsilon}{a\sqrt{1 + \left(\frac{a}{2s}\right)^2}} \quad (37)$$

11. Determination of aggregate layer thickness, h , using Equations 20, 28, and 37:

$$\boxed{(\pi + 2)c_u = \frac{P}{2(B + 2h \tan \alpha)(L + 2h \tan \alpha)} + \frac{K\varepsilon}{a\sqrt{1 + \left(\frac{a}{2s}\right)^2}} \quad (38)}$$

By using Equation (38), design charts can be generated for the required aggregate thickness, h , of a geotextile reinforced unpaved road.

Giroud et al. (1990) expanded the design procedure for unpaved roads by combining the geotextile tension membrane theory with a soil arching theory, **thereby** providing a more complete design approach. A soil-geotextile system will deflect over a void and either fail, undergo limited deflection and bridge the void, or deflect until it comes in contact with the void bottom. A two-step approach was used. First, the behavior of the soil layer was analyzed using classical arching theory. This step gives the stress at the base of the soil layer on a portion of geotextile located above the void. Second, tension membrane theory was used to establish a relationship between the stress on the geotextile, the tension and strain in the geotextile, and the deflection of the

geotextile. It was assumed that the soil deformation required to generate the soil arch was compatible with the tensile strain required to mobilize the geotextile tension. The equations presented for the tensioned membrane theory were based on the assumption that the strain in the portion of the geotextile overlying the void was uniformly distributed and the strain in the portion of the geotextile outside the void area was zero; therefore, that portion of the geotextile does not move.

Holtz and Sivakugan (1987) used the Giroud and Noiray (1981) design procedure to develop design charts for rut depths of 75, 100, 150, 200 and 300 mm (3, 4, 6, 8, and 12 in.). The modulus of the geotextile was found to be not as important for smaller rut depths as it was for a rut depth of 300 mm (12 in.). This finding was attributed to the small strains induced in the geotextile at smaller rut depths. It was explained that at smaller rut depths the membrane resistance assumed by Giroud and Noiray (1981) is not developed. So, for smaller rut depths, the geotextile was claimed to act primarily as a separation layer.

Finally, the tire pressure was shown to have no influence on the required aggregate thickness when the undrained shear strength of the subgrade was less than about 50 kPa (0.5 tsf). Holtz and Sivakugan (1987) found that for very low values of shear strength, a rapid increase in required aggregate thickness occurred, irrespective of the rut depth and the geotextile modulus. Thus, a slight increase in the value of shear strength used for design could result in a significant reduction in the required aggregate thickness.

2.2 Shear Reinforcement

A general expression for evaluating the increase of the bearing capacity due to membrane action based on limit equilibrium conditions was presented by Espinoza (1994). In order to provide closed form solutions for design purposes, an average membrane action was defined. Particular expressions for the average membrane action were obtained assuming constant and variable strain of the membrane. Assuming the geometry of the deformed geotextile and a *no slip* condition, an expression was given for the horizontal and vertical

forces. The shear stresses along the upper and lower sides of the interface were related by $\tau_{lower} = k \tau_{upper}$. For a subgrade composed of a soft soil and a base of cohesionless material, it was noted that the friction along the base-geotextile interface could be higher than the friction along the subgrade-geotextile interface. In such cases, it was found acceptable to assume $k = 0$, yielding $\tau_{lower} = 0$, and $\tau_{upper} = \tau$. Espinoza (1994) derived expressions based on equilibrium equations for a differential element, dl , of geotextile supporting applied vertical stresses. Sellmeijer et al. (1982) developed an expression relating tensile force to the extensional strain along the fabric. Based on these expressions, Espinoza (1994) formulated a simple expression for the additional bearing capacity due to the membrane effect assuming constant strain along the deformed geotextile.

It was shown that in general, for small rutting factors, α ($\alpha = r/L$, where r is the rut depth and L is the effective horizontal length of the geotextile providing support), no significant differences for membrane support values were obtained when assuming a circular versus parabolic rut geometry. Also, for small α values, no significant differences were calculated for the additional bearing capacity assuming constant and variable strain. On the contrary, for large α values the choice of membrane support model significantly influenced the additional bearing capacity calculated. Espinoza (1994) compared his model to other existing models, including Giroud and Noiray's (1981) model and found that Giroud and Noiray's (1981) model renders the most conservative results. An outline of the development of Espinoza's (1994) expressions for determining additional bearing capacity for geotextile reinforced unpaved roads follows.

Outline

1. Development of expressions for the horizontal and vertical components of tension

(Fig. 6):

$$\tan \beta(x) = \frac{dy}{dx} \quad (39a)$$

$$T_h = T(x) \cos \beta(x) \quad (39b)$$

$$T_v = T(x) \sin \beta(x) \quad (39c)$$

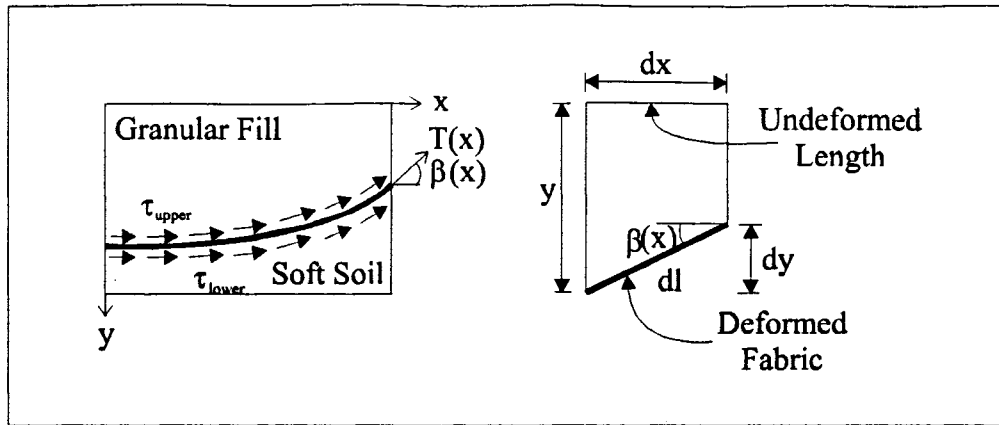


Figure 6. Section of stretched geotextile.

2. Relationship between shear stresses on the upper and lower sides of the geotextile:

$$\tau_{lower} = k\tau_{upper} \quad (40)$$

Assuming that the friction is significantly larger on the geotextile's upper side:

$$k = 0$$

$$\tau_{lower} = 0$$

$$\tau_{upper} = \tau$$

3. Force equilibrium in the horizontal direction:

$$dT_h + \tau dl \cos \beta = dT_h + \tau dx = 0 \quad (41)$$

which yields:

$$\tau = -\frac{dT_h}{dx} \quad (42)$$

4. Force equilibrium in the vertical direction:

$$dT_v - (q_{ap} - q_s)dx + 2\tau dl \sin \beta = 0 \quad (43)$$

where q_{ap} = load intensity or stress at depth d

q_s = soil reaction

5. Dividing Equation (43) by dx and regrouping:

$$q_{ap} = \frac{dT_v}{dx} + q_s + \tau \tan \beta \quad (44)$$

6. Dividing Equation (39b) by (39c) and taking the first derivative of the resulting expression:

$$\frac{dT_v}{dx} = \frac{dT_h}{dx} y'(x) + T_h y''(x) \quad (45)$$

7. Substituting Equations (45) and (43) into Equation (44) and using $q_{ap} = q_s + q_g$ where q_g is the additional bearing capacity due to the membrane effect, we get

$$q_g = T_h(x) \frac{d^2 y(x)}{dx^2} \quad (46)$$

8. A similar expression was developed by Sellmeijer et al. (1982) using the theory of linear elasticity:

$$T(x) = E\varepsilon(x) \quad (47)$$

where E = fabric stiffness (Young's modulus)

$\varepsilon(x)$ = fabric strain

9. Substituting Equation (47) into (39b):

$$T_h = \varepsilon E \cos \beta \quad (48)$$

β can be written in terms of the first derivative of the fabric deflection:

$$\cos \beta = \frac{1}{\sqrt{1 + \left(\frac{dy}{dx}\right)^2}} \quad (49)$$

10. Substituting Equation (49) into Equation (48) and the resulting expression into Equation (46), an equation for the additional bearing capacity due to membrane effect is obtained:

$$q_g(x) = \frac{E\varepsilon(x)y''(x)}{\sqrt{1 + (y'(x))^2}} \quad (50)$$

11. It is convenient to define an average membrane effect, q_{avg} , as follows:

$$q_{avg} = \frac{\int_{-\frac{L}{2}}^{\frac{L}{2}} q_g(x) dx}{L} = \frac{1}{L} \int_{-\frac{L}{2}}^{\frac{L}{2}} T_h(x) \frac{d^2 y(x)}{dx^2} dx \quad (51)$$

$$q_{avg} = \frac{2E}{L} \int_0^{\frac{L}{2}} \frac{\varepsilon(x)y''(x)}{\sqrt{1+(y'(x))^2}} dx \quad (52)$$

Note: This expression holds true if *no slip* occurs along the soil-geotextile interface (i.e. tensile failure occurs first).

12. The fabric can be assumed to have *constant strain* or *variable strain*:

CONSTANT STRAIN:

Integrating Equation (52) assuming $\varepsilon(x)$ is constant:

$$\frac{q_{avg}}{2E} = \varepsilon \ln \left(\tan \beta_o + \sqrt{1 + \tan^2 \beta_o} \right) \quad (53)$$

where β_o = deflection angle at $L/2$

Geotextile strain for an assumed *parabolic deformation*:

$$\varepsilon = \frac{1}{2} \left[\sec \beta_o + \frac{\ln(\tan \beta_o + \sec \beta_o)}{\tan \beta_o} \right] \quad (54a)$$

where $\tan \beta_o = 4\alpha$, $\sec \beta_o = \sqrt{1 + (4\alpha)^2}$, and α is the rutting factor defined as r/L . (54b)

Geotextile strain for an assumed *circular deformation*:

$$\varepsilon = \frac{1 + (2\alpha)^2}{4\alpha} \tan^{-1} \beta_o - 1 \quad (55a)$$

where $\tan \beta_o = \frac{4\alpha}{1 - (2\alpha)^2}$, $\sec \beta_o = \frac{1 + (2\alpha)^2}{1 - (2\alpha)^2}$ (55b)

Note: Equations (54) and (55) can be substituted into Equation (52) to obtain an expression for average membrane support for a parabolic or circular geotextile deformation under constant strain.

VARIABLE STRAIN:

Assuming that any point on the geotextile remains on the same vertical plane prior to deflecting:

$$\varepsilon = \frac{dl - dx}{dx} = -1 + \sqrt{\left(\frac{dy}{dx}\right)^2 + 1} \quad (56)$$

Substituting Equation (56) into Equation (52) and integrating an expression for average membrane support for a geotextile experiencing variable strain:

$$\frac{q_{avg}}{2E} = \tan \beta_o - \ln\left(\tan \beta_o + \sqrt{1 + \tan^2 \beta_o}\right) \quad (57)$$
$$L$$

If it is assumed that the geotextile is experiencing constant strain then, depending on the assumed geotextile deformed shape, either Equation (54) or (55) can be substituted into Equation (52) to determine the average membrane support. If it assumed that the geotextile is experiencing variable strain, then Equation (57) can be used to calculate the average membrane support (Equation 56 has already been substituted into Equation 52).

A procedure for evaluating the load capacity of single-layer reinforced soils was presented by Espinoza and Bray (1995). *Slip between the reinforcement and the granular fill was considered*. The analysis incorporated two important membrane support contributions, namely *normal stress* and *interfacial shear stress* membrane support. It was demonstrated that a significant membrane contribution can be developed even for cases in which proper anchorage was not provided.

Bearing capacity considerations often govern the design of these systems. The inclusion of a single geotextile within a two-layer soil system increases the bearing capacity due to three important soil-structure interaction effects:

1. Membrane support due to the deformed geotextile sustaining normal stresses.
2. Increased shear resistance due to the geotextile sustaining shear stresses.
3. Subgrade bearing capacity improvement due to reduction of shear stresses on the subgrade surface.

Many of the design procedures mentioned rely heavily upon the benefit of only one of these three effects, and this has often led to contradictory findings. Espinoza and Bray (1995) introduced a bearing capacity equation that satisfies both vertical and horizontal force equilibrium and incorporates all three of the above mentioned effects. It was shown that the development of tensile forces in the membrane due to the transmission of shear stresses from the deforming fill to the underlying membrane (effect 2) is the main component of the membrane support contribution, and this effect does not require anchorage. In addition, the subgrade shear stress reduction (effect 3) is often the most significant benefit when including a single geotextile within a two-layer system.

A vertical surface traction, p , induces additional stresses to those created by the self-weight of the granular fill. The additional forces per unit area above the geotextile, q_{ap} , are balanced by the vertical soil reaction, q_s , and the support provided by the geotextile, q_g . From vertical force equilibrium of a differential geotextile element of unit area, it follows that the forces per unit area, $q_{ap}(x)$, $q_s(x)$ and $q_g(x)$ are related by $q_{ap}(x) = q_s(x) + q_g(x)$. The actual distribution of $q_s(x)$, $q_{ap}(x)$ and $q_g(x)$ depends on the geometric characteristics and material properties of the granular fill, subgrade soil and geotextile, and their interrelations. A number of design procedures have been proposed that are based on simplified deformation mechanisms that do not capture the interrelation between the different material components, that employ some empiricism, or that do not strictly satisfy equilibrium. The proposed design procedure was based on a closed form solution of the equilibrium equation that has less restrictive assumptions than existing designs, and incorporates all three primary benefits due to the inclusion of the geotextile (Espinoza, 1995).

The modified bearing capacity equation can be used to reconcile some contradictory findings that have been reported regarding the contribution of geotextile stiffness to single-layer reinforcement of granular road bases. It is generally believed that soils reinforced with high modulus geotextiles perform better. Yet, in a series of full-scale road section tests performed by Douglas and Kelly (1986), results indicated that despite the large range in geotextile modulus, nearly identical performances were obtained for geotextiles that were fully anchored or completely free at the outside edges. Thus, a

higher geotextile modulus was found to not necessarily result in higher road stiffness and, regardless of whether the geotextile edges are fixed or free, the geotextile should have similar performance. Furthermore, interfacial shear stress support depends on the applied load itself, q_{ap} , and the mobilized interface friction.

The interfacial shear stress was found to be independent of the conditions at the geotextile edges and the geotextile stiffness for typical fill and geotextile properties (Douglas and Kelly, 1986). So, for the tests performed by Douglas and Kelly (1986) it was likely that most of the bearing capacity improvement was due to interfacial shear stress support (effect 2) and the membrane induced reduction of applied shear stresses on the surface of the cohesive subgrade (effect 3). Moreover, the only tests that gave some differences in performance were those obtained using a polyethylene film separator. This type of material is likely to have smaller interface friction than woven and nonwoven materials and therefore, a difference on performance should be expected.

Similarly, Bakker (1977) discussed the reinforcement of embankments and gravel roads and presented a model for the calculation of the bearing capacity of the subsoil underneath a layer of aggregate and membrane. The bearing capacity equation was modified to take into account a membrane. The friction at the membrane-aggregate interface and at the membrane-subsoil was neglected. Based on the modified bearing capacity equation and the geometry of the deformed subgrade, a relationship between the modulus of elasticity of the membrane and the rut-depth for a set of given soil properties was calculated.

Modified Bearing Capacity Equation

1. The expression for average membrane support (Equation 52) was developed earlier by Espinoza (1994):

$$q_{avg} = \frac{\int_{-\frac{L}{2}}^{\frac{L}{2}} q_g(x) dx}{L} = \frac{1}{L} \int_{-\frac{L}{2}}^{\frac{L}{2}} T_h(x) \frac{d^2 y(x)}{dx^2} dx \quad (58)$$

This expression satisfies vertical and horizontal force equilibrium at the global level.

2. Derivation of an expression for the subgrade bearing capacity, \bar{q}_s : the geotextile outside of the effective length (i.e. AB and DE in Figure 7) exerts a vertical stress, \bar{q}_{lat} , due to membrane support, reducing the heave of the subgrade soil.

$$\bar{q}_s = cN_c + \gamma h + \bar{q}_{lat} \quad (59)$$

$$\bar{q}_{lat} = \frac{1}{L} \int_{\frac{L}{2}}^{L_c + \frac{L}{2}} q_g(x) dx \quad (60)$$

$$N_c = 1 + \frac{\pi}{2} + a + \sin a \quad (61)$$

where $a = \cos^{-1}\left(\frac{\tau_c}{c_u}\right)$

τ_c = shear applied to the soft soil surface

c_u = undrained shear strength

N_c = bearing capacity factor

h = granular fill thickness

γ = aggregate unit weight

L_c = length of geotextile preventing heave

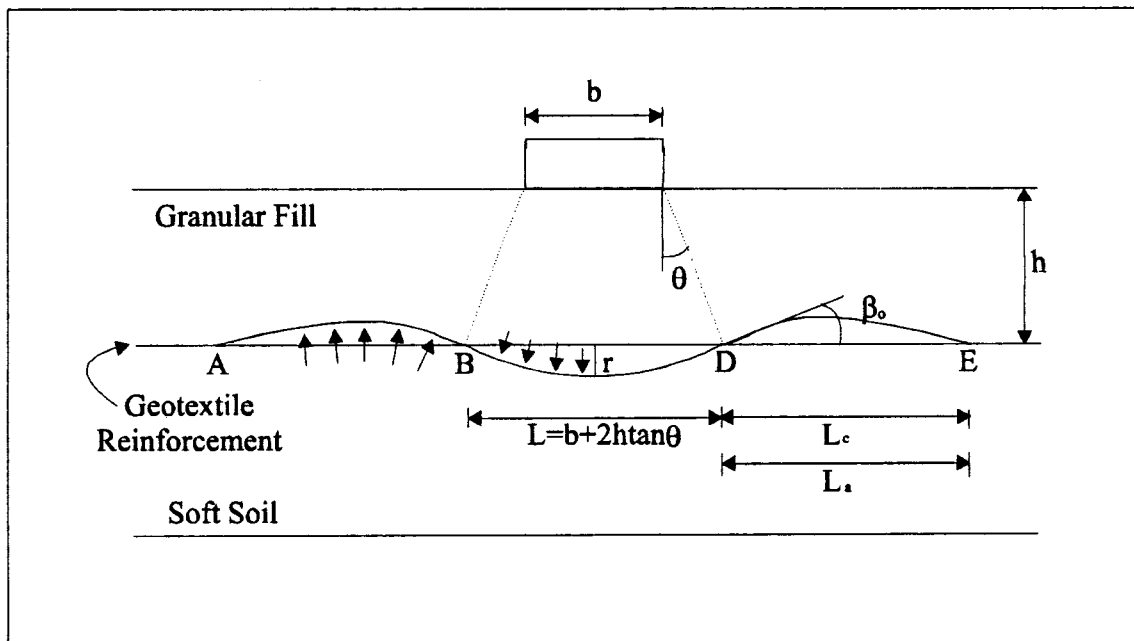


Figure 7. Geometry of deformed geotextile.

3. An expression for the stress applied to the geotextile, \bar{q}_{ap} , can be developed by looking at the vertical stress within the aggregate layer, $\sigma_v(z)$:

$$\sigma_v(z) = \gamma z + \frac{pb}{b + 2z \tan \theta} \quad (62)$$

According to Equation (62), the pressure applied to the geotextile can be expressed as:

$$\bar{q}_{ap} = \gamma h + a_b p \quad (63)$$

where $a_b = b/L$

$$L = b + 2h \tan \theta$$

4. By substituting equations (58), (59), and (63) into Equation (46):

$$a_b = c_u N_c + \bar{q}_t \quad (64)$$

where \bar{q}_t = total membrane support, including both normal stress membrane support (effect A) and interfacial shear stress support (effect B).

$$\bar{q}_t = \frac{2}{L} \int_0^{\frac{L}{2}} q_s(x) dx + \frac{1}{L} \int_{\frac{L}{2}}^{L_c + \frac{L}{2}} q_s(x) dx \quad (65)$$

5. Integrating Equation (65) and knowing that the shear stress along the interface, τ_x , equals $-\frac{dT_h}{dx}$; the extreme points A and E (Fig. 7) were chosen such that the geotextile is horizontal (i.e. $y'(L_c + L/2) = 0$), and assuming the shear stress, τ_x , is constant along BD, Equation (65) becomes:

$$\bar{q}_t = \frac{1}{L} \left[T_o \sin \beta_o + 2 \int_{y(0)}^{y\left(\frac{L}{2}\right)} \tau(x) dy \right] \quad (66)$$

$$\text{for } T_o \leq \min(\gamma h L_a \tan \psi, T_{\max}) \text{ and } \tau(x) \leq q_{ap} \tan \psi \quad (67)$$

where T_o = geotextile tensile force

β_o = angle of inclination at point D (Fig. 7)

ψ = interface friction angle

L_a = geotextile anchorage length

T_{\max} = geotextile maximum tension

Note: Equation 66 allows for slip to be taken into consideration when the upper limits set by Equations (67) are exceeded. The first term in Equation (67) represents the membrane contribution obtained from outside the effective length (normal stress membrane support, effect A) and depends on proper anchorage outside the effective length. The second term in Equation (67) represents the interfacial shear stress support (effect B) within the effective length and depends upon the applied load and the mobilized interface friction.

6. From limit equilibrium with a Mohr-Coulomb failure criterion, the shear stress along the effective length can be estimated as

$$\tau(x) = \bar{q}_{ap} \tan \psi_m = (\gamma h + a_b p) \tan \psi_m \quad (68)$$

where ψ_m = mobilized interface friction angle

7. Assuming that ψ_m is constant along L , Equation (67) reduces to

$$\bar{q}_t = \frac{1}{L} \left[T_o \sin \beta_o + 2r(\gamma h + a_b p) \tan \psi_m \right] \quad (69)$$

where r = rut depth

8. Substituting Equation (69) into Equation (64), an expression for the admissible surface stress, p_{adm} , can be estimated:

$$P_{adm} = \frac{(c_u N_c + T_o \sin \beta_o / L + 2\alpha_r \gamma h \tan \psi_m)}{\alpha_b (1 - 2\alpha_r \tan \psi_m)} \quad (70)$$

where $\alpha_r = \alpha_r$ (from earlier) = r/L = rutting factor

Note: In many practical situations, proper anchorage cannot be ensured at all times during construction. In such cases, $T_o = 0$ should be used to estimate the admissible stress. Even in cases where proper anchorage is provided, its effect will not be felt until large deformations are induced.

9. Using Equation (62) to represent the vertical stresses and assuming that the horizontal stresses are proportional to the vertical stress ($\sigma_h = K\sigma_v$), Houlsby et al. (1989) derived the following expression to compute the total horizontal force exerted on face HC in Fig. 8:

$$P_{HC} = \int_0^h \sigma_h dz = K \left(\frac{\gamma h^2}{2} + \frac{\eta p b}{2} \right) \quad \eta = \begin{cases} \ln(L/b)/\tan \theta & \theta \neq 0 \\ 2h/b & \theta = 0 \end{cases} \quad (71)$$

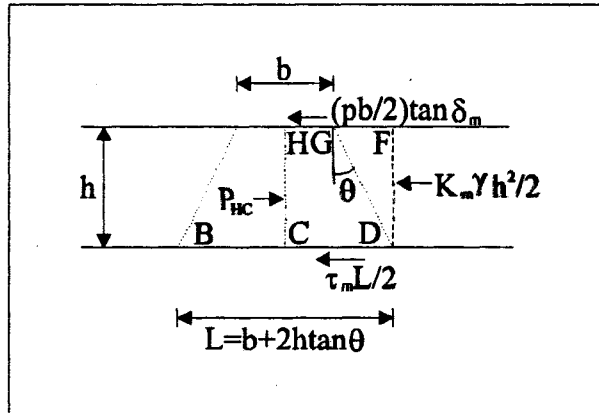


Figure 8. Load distribution through aggregate.

10. Assuming that the applied load pushes the fill outward (Fig. 8), a passive pressure is developed on face DF. The corresponding force is computed as

$$\gamma K_{pm} h^2 / 2$$

where K_{pm} = the passive earth pressure coefficient

$$K_{pm} = \tan^2(\pi/4 + \phi_m/2)$$

ϕ_m = the soil friction angle

Allowing for friction force $(pb/2)\tan \delta_m$ on the footing base, where δ_m is the mobilized interface friction angle, the horizontal force equilibrium of the block CDFH yields:

$$\left(\gamma h + \frac{pb}{L}\right) \tan \psi_m \frac{L}{2} + \frac{1}{2} K_{pm} \gamma h^2 + \frac{pb}{2} \tan \delta_m = K \left[\frac{1}{2} \gamma h^2 + \eta \frac{pb}{2} \right] \quad (72)$$

Solving Equation (72) for the interface mobilized friction, ψ_m gives

$$\tan \psi_m = \frac{\left[\alpha_h (K - K_{pm}) + \frac{pb}{\gamma h L} (\eta K - \tan \delta_m) \right]}{\left(1 + \frac{pb}{\gamma h L} \right)} \quad (73)$$

where $\alpha_h = h/L$ = dimensionless parameter

11. The mobilized friction angle on the base of the footing, δ_m , is expected to be quite small even for rough footings (Houlsby et al., 1989) and, thus, it can be omitted from Equation (73). Assuming that the fill tends to move outward from beneath the footing, the value of K in equation (71) can be assumed to be equal to the active earth pressure coefficient (i.e. $K_{am} = \tan^2(\pi/4 - \phi/2)$). Replacing p in Equation (73) with p_{adm} from 71, an expression of the mobilized interface friction, ψ_m , can be obtained:

$$\tan \psi_m = \frac{[a_h(K - K_{pm}) + M_c(\eta K - \tan \delta_m)]}{(1 + M_c + 2a_r(a_h(K - K_{pm}) - \eta K + \tan \delta_m))} \quad (74)$$

where $M_c =$ dimensionless factor

$$M_c = \frac{c_u N_c + T_o \sin \beta_o / L}{\gamma h}$$

Note: Equations (72) and (74) have been derived based on equilibrium between the membrane and the sliding block above it, and are only valid for the situation shown in Figure 8. The reinforcement should be checked for tension failure:

$$\frac{q_{ap} L \tan \psi_m}{2} < T_u$$

3. FACTORS AFFECTING REINFORCED UNPAVED ROADS

Many factors affect how a geotextile performs in an SFA system:

- fabric stiffness
- fabric creep
- fabric location (placement somewhere other than the interface)
- fabric anchorage
- fabric texture
- aggregate type and shape
- subgrade properties

How the SFA system performs is greatly affected by so-called interface friction, which can be seen as a function of many of the above listed factors, including fabric stiffness and texture, aggregate type and geometry, and the subgrade properties. However, the interface properties are often expressed as a relationship between the aggregate and geotextile in reference to how the load is transferred. The information presented in this section pertains to research that has been performed to look specifically at one or more factors affecting the performance of an SFA system.

The effects of fabric stiffness, texture, creep, and location have been investigated. Some of the findings of the research pertaining to these issues are included in the following sections. The effects of fabric texture and aggregate type and shape on the interface friction properties and the overall effect of interface friction properties on the performance of the SFA system have not been closely examined. Subgrade properties are of course important in the performance of the SFA system: for example, it is generally recommended to use geotextiles for reinforcement in unpaved roads when constructing on a subgrade soil of $CBR < 3$ (Holtz et al., 1995).

3.1 Stiffness

A small displacement finite element study was used by Burd and Brocklehurst (1990) to investigate the mechanisms of reinforcement in a geotextile reinforced unpaved road where large rutting *is not* permitted. The geotextile stiffness was varied. Results of the modeling showed that fabric stiffness has a modest effect on the load-deformation response of the road, but a substantial effect on the magnitude and nature of the shear stresses acting at the reinforcement-soil interface.

Work performed by Housby et al. (1989) and Milligan et al. (1989) involved looking at the horizontal stresses that develop in the fill layer due to a vertical load. These horizontal stresses were found to generate shear stresses at the base of the fill layer. If reinforcement was absent, then these shear stresses were sustained directly by the subgrade and in some instances were found to have a detrimental effect on the subgrade's bearing capacity. In the reinforced case, the shear stresses were sustained by the reinforcement. The presence of the reinforcement caused a reduction in the outward acting shear stresses on the surface of the subgrade and increased the bearing capacity of the subgrade.

Finite element studies of Zeevaert (1981), Burd (1986), and Burd and Housby (1989) were based on calculations in which the only parameter that was varied was the reinforcement stiffness. It was assumed that *no slip* occurred at the soil/reinforcement interface. From these studies, it was clear that the reinforcement stiffness had little effect on the initial slope of the load-displacement response. But, in each case an approximate limit load was reached that increased with an increasing value of reinforcement stiffness (increased at most by 15%). It was shown that for each value of reinforcement stiffness, the shear stresses at the base of the fill reached a peak at a distance of about $0.7B$ from the footing centerline, where B is the footing width, and that as the reinforcement stiffness increased the magnitude of the shear stresses also increased. The shear stresses applied to the surface of the clay immediately beneath the reinforcement followed a similar pattern to

the shear stresses at the fill base. Outward shear stresses were found to act underneath the footing and reached a peak value at a distance of about $0.7B$ from the footing centerline. The outward shear stresses decayed to zero at about $2.5B$ from the footing centerline. For the stiffest reinforcement, the direction of the shear stresses reversed and acted inwards. Lateral movement of the clay was thought to be restrained by the reinforcement.

The presence of outward shear stresses can reduce a soil's bearing capacity to as little as one-half of the value for purely vertical loading. If reinforcement is introduced, these shear stresses are picked up by the reinforcement, which is put into tension, and purely vertical forces are transmitted to the clay below, allowing the capacity of the clay to be realized. This helps to explain why reinforcement is able to provide an improvement in road performance even at small rut depths.

The problem of soft subgrades, extremely heavy design axle loads, very low traffic volumes, and very low cost tolerances associated with access roads have led engineers to adopt designs using geotextiles. A number of design methods have been put forward, where the basis of the method is the relationship between rut depth, traffic and geosynthetic characteristics. Douglas and Valsangkar (1992) pointed out that overall transportation costs and efficiencies are linked to the relationship between roads and vehicles. Because of the impact that road stiffness has on fuel consumption and therefore vehicle operating costs, it was contended that stiffness rather than permanent rut depth should be adopted as the key design criterion for resource access roads. The range of road stiffness to be expected and how it is affected by geotextile inclusions was investigated by cycled-load testing of a large-scale model of a permanent structure consisting of different granular bases with various geotextiles placed on a peat subgrade. Surprisingly, the improvement in model pavement stiffness over that for the subgrade itself was not great.

Douglas (1993) states that geotextile research in unpaved roads has evolved over the last few decades. Early studies were concerned with the effect the geotextile inclusion would have on the bearing capacity of such roads. Later, attention was focused on how geotextiles might slow the development of ruts. Research has dealt mainly with the relationship between rut depth and traffic. It was contended that this design method is

based upon the wrong criterion for unpaved road behavior because roads are built for vehicles. Continuing to use rut depth as the design criterion is a carry-over from the design of paved roads into the design of unpaved low-volume roads. The implication is that the adoption of rut depth as the design criterion for unpaved access roads is inappropriate: it attempts to solve a problem that does not exist.

The scope of Douglas' test program included an investigation of the possible variation in pavement stiffness with cycled loading and a simulation of the low-frequency load cycling appropriate to slow-speed travel of heavy vehicles on low-standard roads. The tests were carried out in a 3 m x 4 m x 2 m pit with the loads applied through a 300 mm diameter plate. The loads were cycled at a frequency of 0.5 Hz, corresponding to a steady stream of axles spaced at 4 m, traveling at about 7 km/h. The subgrade CBR was much less than one; the gravel was a pit run gravel and crushed greywacke. Two geotextiles were used: a nonwoven geotextile at the interface in all cases and in some cases a geogrid was placed at the mid-depth of the gravel base. Sinusoidal loading was applied with peak to trough loads of 4.5 kN (1012 lbs) and 0.1 kN (22.5 lbs). The test ran up to 30,000 cycles and it was shown that the stiffness continued to gradually increase with cycled loading. The stiffness for a compacted, crushed rock model road with a depth ratio $H/D=0.5$ and a geogrid inclusion at the mid-depth, was approximately 3.6 times that of the peat subgrade itself for the same number of cycles at a peak stress of 62 kPa (9 psi).

3.2 Creep

Bonaparte and Berg (1987) described a methodology for the selection of long-term allowable reinforcement tension taking into account reinforcement creep, in-ground material degradation and construction-induced material damage. The selection of product-specific long-term reinforcement properties relied on the use of constant load creep tests as described by McGown et al. (1984) and on interpretation of the test results in terms of isochronous (constant-time) parameters. Furthermore, the long-term, in-ground durability of the reinforcement and the effect of construction-induced material

damage were required. The end result was a product specific long-term reinforcement tension that could be used in limit equilibrium calculations. Both reinforcement rupture and reinforcement load-elongation response were considered.

3.3 Fabric Location

Haliburton and Barron (1983) showed that placement of an optimum aggregate thickness on the fabric, when related to the width of the loaded area and independent of subgrade strength and wheel load, increased the strength and deformation resistance of the aggregate cover and produced significant Burmister-type stress reductions (decrease in vertical stress within a soft layer when a stiff layer is above) at the subgrade surface. It was noted that, although the use of fabric for separation does not in itself strengthen the road system, it does allow dissipation of excess subgrade pore pressures and subgrade consolidation, which could cause long-term subgrade strength improvement. This factor would allow the road to improve, rather than degrade, with time and number of load repetitions.

Research by Haliburton and Barron (1983) determined that maximum reinforcement of cohesionless soil placed above a geotextile would occur when the geotextile was located so as to interfere with the normal cover soil (base) shear failure patterns. It was concluded that the actual stress immediately beneath the fabric was approximately 50 percent of that predicted by the Boussinesq theory. The optimum placement depth was approximately one third the width of the loaded area. Even at large deformations, the amount of membrane-type support obtained from the fabrics tested, including the very high modulus fabric, was small compared to the total load capacity of the model systems.

Ishmail and Raymond (1995) investigated the placement of a single layer of fabric on bearing capacity and settlement. They found that the aggregate-subgrade interface may not be the best location for geotextile reinforcement. Laboratory model tests and finite

element analysis for the bearing capacity of a strip footing were performed on both single and two-layer geotextile SFA systems with the following results:

1. For a uniform granular soil layer, the bearing capacity increased as the geosynthetic placement depth decreased. For a two-layered granular soil deposit, the bearing capacity was highest in the $D/B = 0.3$ to 0.5 range, when a thin, stronger layer overlaid a weak layer.
2. As the thickness of the upper stronger layer was increased, the bearing capacity became highest when the geosynthetic was located closest to the footing. As the thickness of the weaker layer increased, the optimum reinforcement depth was more likely to be below the interface.
3. When the bearing capacity for the geosynthetic reinforced single layer was compared to two layers, the two-layer results were higher for the same geosynthetic reinforcement positions.
4. For frictional materials, no significant benefit was observed in placing the geosynthetic reinforcement at the interface. In determining the best position to place the geosynthetic in a two-layered granular soil, the thickness of both the upper and lower layers must be taken into account. Where little information was available, placement close to the footing base was found to approach that of the optimum location depth. Repeated loadings did not influence the optimum placement depth of the geosynthetic reinforcement.

Barksdale et al. (1982) developed a general finite element program for analyzing a plane strain or axisymmetric fabric reinforced soil system. An extensive series of model tests were also conducted. The model tests were performed to aid in the validation of the program. Haul-road type systems were modeled in two circular test tanks. One tank was 0.9 m (3 ft) and the other 2.5 m (8 ft) in diameter. The geotextile was cut into circles larger than the diameter of the tank and folded up at the edges.

Systems reinforced with fabric showed a 20% reduction in vertical stress in the clay along the centerline. SFA systems reinforced with this specific fabric exhibited up to three times more vertical plastic strain in the aggregate at failure than did nonreinforced systems with similar rutting. In the SFA system, the greatest amount of plastic vertical

strain occurred in the aggregate layer, while in the soil-aggregate system the greatest amount occurred in the subgrade. Both the model tests and the program showed that the presence of fabric was beneficial in haul road design due to the fabric's ability to alter the stress in the SFA system.

3.4 Anchorage

Fatigue tests were performed using an SFA system and a soil-aggregate (SA) system (Douglas et al. 1986). The reinforcement was placed at the base-subgrade interface and in other cases at both the interface and the base middepth. The lateral anchorage was also varied between a long lateral anchorage (7.0 m geotextile width) and a short lateral anchorage (4.5 m geotextile width). The clay subgrade had an undrained shear strength of between 30 and 50 kPa (0.3 and 0.5 tsf). The subbase used was a crushed limestone with zero cohesion and a friction angle of 47°. Four types of geotextiles were used: Bidim[®] U 24, Bidim[®] U 34, Scotlay[®] (100 g) and the Scotlay[®] (200 g). The loading system consisted of a truck with an axle weight of 130 kN (29,000 lbs) moving at 20 km/hr (12.5 miles/hr).

The rut evolution confirmed the following: (1) the rutting was less for the stiffer geotextile, (2) an anchorage of 7 m resulted in better conditions than an anchorage of 4.5 m and (3) the use of two geotextiles-one at the interface the other at the base middepth-for unsurfaced roads with small strains provided better behavior than the use of one geotextile.

4. SEPARATION IN UNPAVED ROADS

The function of separation can be defined as the introduction of a flexible synthetic barrier placed between dissimilar materials so that the integrity and function of both materials can remain intact or be improved. A great deal of research has gone into the more glamorous geosynthetic role of reinforcement, with minor credit given to the separation function. It has been shown, however, that it takes only 10-20% percent by weight of a subgrade soil mixed into a base material to reduce a pavement's bearing capacity to that of the subgrade soil. Many times the focus has been on the immediate aggregate savings afforded by geotextile reinforcement while overlooking the long-term benefits that geotextile separators provide over the life of the pavement structure.

4.1 Pumping of Fines

There are many factors that affect the amount of pumping at the subgrade-subbase interface of unpaved roads. These factors include the number of cycles, mean stress, stress amplitude, standing water, frequency, aggregate thickness, over-consolidation ratio, confining pressure, and size of subbase particles. A laboratory assessment of the ability of fabrics to prevent soil migration under conditions of dynamic loading was performed by Hoare (1982). It was concluded that the amount of soil migration measured as the SCV (soil contamination value taken as the weight of subgrade soil passing the fabric per unit area of fabric) was quite variable when 20 mm single sized crushed rock aggregate was used as the sub-base, especially at high applied stress levels. The use of spherical glass balls for the sub-base gave much more repeatable SCVs and the use of this type of material was recommended as a basis for future comparative tests of the performance of different fabrics. The frequency of the applied loading was found to have little effect on the SCV, when the frequency was in the range 2.5 to 10 Hz.

The effects of varying the volume of water used to simulate the effects of the ponded water on the subgrade surface seemed to have little effect on the SCV, provided there was sufficient moisture available for the soil. The SCV was found to increase linearly with the log of the number of applied cycles. The soil was found to migrate through the fabric at the points of contact of the subbase particles with the subgrade. The moisture content of the soil at the points of contact with a subbase particle increased to a value close to its liquid limit. This was thought to possibly be a result of soil suction and local shearing caused by the high stresses at the points of contact. The phenomenon of clay pumping was thought to be caused by the wet (and hence soft) soil on the surface of the subgrade squeezing through the fabric.

The SCV may be reduced by decreasing the stresses at the points of contact of the sub-base particles on the subgrade. This may be achieved by either reducing the applied stress levels or by using a subbase with a finer grading or higher fines content, which will increase the number of contact points and hence reduce the stresses at each point of contact. The SCV was found to be insignificantly reduced by the use of a heavier needle-punched fabric rather than a light melt-bonded fabric.

Alobaidi and Hoare (1994) performed a laboratory investigation into the factors that have an important influence on the pumping phenomenon in an SFA system. One specific subgrade soil and geotextile were chosen for the tests. Alobaidi and Hoare (1994) found that the amount of pumping was directly related to the cyclic deformation of the system and it was suggested that this finding will be of great significance in the design and development of materials to minimize pumping

Tests showed that the rate of pumping can be determined from the rate of permanent deformation. Tests also indicated that the rate of pumping decreased with the number of cycles. Factors which may have caused this decrease in the pumping rate included the formation of a self-induced filter at the back of the geotextile, decreased contact stress, clogging of the geotextile, and increased viscosity of the slurry. Tests were also performed where the stress amplitude remained constant and the mean stress was varied. Tests showed that the increase in mean stress led to a decrease in cyclic deformation.

Tests also showed that an increase in the stress amplitude led to (1) an increase in the rate of permanent deformation, (2) an increase in the cyclic deformation of the system, and (3) a consequent rise in the amount of pumping at every stage in the pumping test. The amount of pumped fines increased with the amount of standing water on top of the subgrade. The effect of frequency was more pronounced in the series with the greater amount of standing water. The ELT (equivalent loss of thickness from the subgrade surface arising from pumping) due to pumping was affected to a large extent by the concentration of fines in the standing water, while the cyclic history seemed to have no effect (this conclusion must be restricted to the particular materials used in this test). The ELT decreased significantly with increase of the over-consolidation ratio. Finally, the permanent deformation of the system decreased with an increase in the confining pressure during all stages of a pumping test and the amount of pumped fines was less for small-size particles.

4.2 Subbase Protection

Bell et al. (1981) suggested that thick, relatively incompressible geotextiles with small pore sizes would be effective in limiting subbase contamination to an acceptable level. Tests were performed on both geotextile and granular filters. The SFA system was placed in a 355 mm diameter steel mold. Single size aggregate was used for the subbase. Water was poured into the mold and left to stand to ensure equilibration of pore water pressures caused by the preparation technique. Tests indicated a clear dependence of contamination on the initial subgrade moisture content for nonwoven geotextiles. High pore water pressure dissipation rates were associated with high amounts of contamination, and low dissipation rates with low amounts of contamination.

A close inspection of the recovered fabric revealed that the soil had passed through the fabric only at points where the subbase particles had been in contact with the fabric. Nonwoven fabric was found to be relatively ineffective in preventing clayey fines

contamination, but was more successful in preventing penetration of the subbase into the subgrade.

Contamination of the base layer not only lowers the permeability of the base layer and restricts drainage, but also acts as a lubricant which significantly reduces the shear strength of the stone sub-base aggregates. Snaith and Bell (1978) showed under laboratory-simulated conditions that for filter membranes beneath road sub-bases, normal filter principles (for unidirectional flow conditions) did not seem to apply.

The main objectives of the research program of Bell and Cochrane (1987) were to assess the dynamic filtration performance and the separation function effectiveness of a range of geotextiles when used as separating membranes between a clay subgrade and stone subbase. They also investigated the various parameters that influenced the geotextiles. A related objective was to study subgrade-subbase interaction in order to gain a better understanding of the physical processes involved in clay pumping under cyclic loading. Measurements of the SCV were made.

Clay pumping under dynamic cyclic loading on a model stone subbase was found to be dependent mainly on stress level, soil moisture content, and clay content of the subgrade soil. Geotextiles with effective opening sizes of less than 100 mm reduced clay pumping significantly when used as separating membranes between the clay subgrade and stone subbase. The mechanism of clay pumping was found to arise mainly from slurry filled depressions formed by stone penetration into the clay subgrade surface. The thick geotextiles were most successful in reducing clay pumping over a wide range of moisture contents and soils. The cushioning effect of the geotextiles thickness seemed to reduce stone point penetration into the subgrade. Slurry penetration through the fabric was found to only occur between the stone-point depressions, where the fabric was relatively uncompressed. The thin geotextiles were found to perform not as well as the thicker geotextiles in reducing clay pumping. However, all the geotextiles tested performed a useful separating function in reducing the stone penetration down into soft clay subgrades and in preventing plastic flow of the clay up into the stone interstices.

4.3 Clogging

Lafleur and Rollin (1990) noted that geotextiles must satisfy two criteria. The first relates to its punching resistance: it must be able to contain the individual coarse particles without tearing, in the range of the applied stresses. The second criterion bears on its filtering capacity: it must be able to prevent fine subgrade particles from migrating upward into the subbase. Under dynamic stresses, the filtering conditions are more severe since alternating flow can take place at the subgrade-geotextile interface, preventing the development of self-filtration and arching of the particles.

Bell et al. (1982) tested the separation capability of various geotextiles laid between 20 mm (0.75 in.) aggregates and compacted silts of low plasticity; the stresses ranged between 25 and 75 kPa (3.6 and 11 psi) and were applied at a frequency of 5 Hz over 24 hours, amounting to 432,000 loading cycles. They found that the subbase contamination and geotextile clogging were related to the ratio between the geotextile equivalent opening size, O_{95} , and the subgrade particle size, D_{85} , and their results confirmed the current piping criterion of $O_{95} < D_{85}$, since the contamination was minimum when this criterion was satisfied.

Nishida and Nishigata (1994) investigated the relationship between the reinforcement and separation functions in road construction. A distinct relationship was found between the opening size of woven geotextiles and the total weight of the soil mass through the geotextile. However, for the nonwovens, this relationship was unclear. The excess pore pressure in each case increased as the number of loading increased.

To differentiate between reinforcement and separation, Nishida and Nishigata (1994) used the concept that when loading a system with a small circular piece of geotextile below the loaded area, no tensile forces can develop in the geotextile. Therefore only separation takes place. Then when reinforcing an entire system tensile forces can develop in the geotextile along with separation. Separation became the primary function when $\sigma/c_u < 8$ and reinforcement became the primary function when $\sigma/c_u > 8$,

where σ is the vertical stress on the subgrade and c_u is the subgrade undrained shear strength.

5. CASE STUDIES

5.1 Haul Roads Over Soft Foundation Soils

Austin and Coleman (1993) presented the results of a full-scale field study conducted to evaluate the effectiveness of various geosynthetics as the primary reinforcement in aggregate layers placed over very soft subgrades. A test road containing several test sections was constructed. Each test section contained a different geosynthetic with three unreinforced sections as controls.

A two axle dump truck was used with a rear axle of 80 kN (18,000 lbs). The tires were inflated to 550 kPa (80 psi). The test road was cycled with traffic until a 75 mm (3 in.) rut occurred. This rut depth was considered failure. All failed sections exhibited similar characteristics of severe rutting with adjacent upheaval. Contamination of the subbase was apparent where geotextile separators were not used. The results of the field evaluation were compared with the theoretical design procedures proposed by Giroud et al. (1984) and the results correlated fairly well. The number of axle load passes actually achieved for a given thickness of a geosynthetic-reinforced aggregate subbase over soft subgrade soils was less than the expected number of passes calculated. It was believed that the difference was due to the fact that higher in-place aggregate strengths are assumed in the theoretical design procedure than those actually achieved during the construction of haul roads.

5.2 Long-Term Performance

The long-term separation and drainage performance of 22 geotextile separators installed between 1978 and 1991 and exhumed from permanent roadways in Washington state was evaluated (Metcalf et al. 1995). The geotextiles consisted of heat-bonded nonwovens, needle-punched nonwovens and woven slit films. The exhumed pieces of geotextiles

showed that the short-term separation and drainage functions were found to be more critical to the pavement system than their long-term performance, due to consolidation and subsequent strength gain of the subgrade soils. It also appeared that geotextile retention may not be as critical in separation applications as previously thought.

Several of the sites upon installation of the geotextile had very soft subgrades. However, upon exhuming the geotextiles, the subgrade was found to be consolidated and strong with no free water, where if today the subgrade were being built on, no geotextile would be necessary; the soil was fully competent. It was noted that even if the geotextile separators become somewhat blinded or clogged in the short-term but were still capable of separating materials and dissipating excess pore pressures until the subgrade consolidated, there was little need for a functioning separator for the remainder of the roadway design life.

The primary purpose of the geotextile separator is to prevent mixing of aggregate and subgrade materials. However, in order for the geotextile to be an effective separator during the life of the pavement system, it is generally recognized that the geotextile must also provide secondary functions of filtration and drainage at the soil/geotextile interface. A geotextile can prevent migration, but it can be adversely affected by soil particles blocking and clogging/caking pore openings.

5.3 Separators

Page (1990) reported on eight sites that involved exhuming geotextiles from existing roadways. It was noted that the required strength needed to maintain separation over the design life of a project was relatively small compared to the strength required to resist damage during construction. The following four index tests were described: Grab Tensile Strength (ASTM D 4632), Burst Strength (ASTM D 3786), Puncture Resistance (ASTM D 4833) and Tear Strength (ASTM D 4533). Other characteristics such as minimum lift thickness, apparent opening size (AOS) and permeability were mentioned.

Of the eight sites chosen for the investigation, five were considered useful for further study. Four sites utilized a woven slit film material with mass per area of approximately 135 g/m^2 (4 oz/yd^2) and one site utilized a nonwoven needle punched material with a mass per area of 180 g/m^2 (5.3 oz/yd^2). Minor puncture damage was found at all locations. This damage, however, did not seem to affect the geotextile's performance as a separator. Grain size distributions were determined for the base material at different heights above the geotextiles, showing slightly higher fines content directly above the geotextile. Index strength tests were performed on the exhumed geotextile and indicated that all the geotextiles appeared to have survived reasonably well with an average percent of strength retained for the grab tensile test, trapezoidal tear test, puncture test and burst test between 72-99%.

It was pointed out that the increase in the amount of fines in the base course was so small in all cases that it was hard to say with certainty that the fines migrated up from the subgrade. Blinding was observed in varying degrees, with the worst blinding observed in the woven slit film. It was apparent from the tests performed that, for the woven slit film geotextiles, only a small amount of contamination of the material by fine-grained soil particles was required to cause a significant drop in permittivity.

Slit film woven geotextiles would be adequate for separation applications over most subgrade soils. However, they tend to become blinded more readily than the nonwovens when used over clayey-slit subgrades. The use of a heavier geotextile with large grab elongation would help to minimize the damage during installation. Lightweight (3.5 oz/yd^2) nonwoven geotextile should not be used for any separation application regardless of the subgrade type or initial base course lift thickness. Puncture holes were present in many of the woven slit film geotextiles, where gravel-sized particles were present on the subgrade surface. The use of a heavier geotextile (8 oz/yd^2), which meets the high survivability strength criteria and has a high grab elongation, would help to minimize damage during installation. However, the heavier geotextiles are more expensive and there was no evidence from this study that the presence of moderate construction damage to the geotextile separator significantly affected the performance of the roadway.

5.4 Geogrid Reinforcement

Unpaved roads and trafficked areas consisting of a base layer placed on a subgrade soil could be reinforced by geogrids placed within the base layer and/or, more often, at the base layer/subgrade interface. Design methods developed for geotextile-reinforced unpaved structures are not suitable because they do not take into account the mechanism of geogrid/base layer material interlocking. However, certain other mechanisms by which geogrids and geotextiles reinforce unpaved structures are similar, and consequently design methods for geogrid-reinforced unpaved structures may utilize features from existing geotextile design methods. Starting from the design method of Giroud and Noiray (1981) for fabric-reinforced unpaved roads, Giroud et al. (1984) developed a design method for geogrid-reinforced unpaved roads.

In comparison to paved structures where only small deformations can be accepted, rather large deformations can be acceptable in some unpaved roads. A road or trafficked area is considered failed when surface deformations are such that conditions for traffic become unacceptable. Unacceptable deformations can occur after one axle passage if the axle load exceeds the bearing capacity of the structure. Unacceptable deformations can also occur after several passages as a result of (1) an accumulation of small permanent deformations occurring at each axle passage and accelerated by progressive deterioration of the structure and/or (2) large deformations associated with shear failure of the structure, which occurs when the bearing capacity becomes smaller than the axle load as a result of progressive deterioration of the structure.

According to McGown and Andrawes (1977), optimum reinforcement depth is approximately 0.3 times the width of the load. Raymond and Hayden (1983) found the optimum depth to be in the range of 0.3 and 0.6 times the load width. It has also been found that a smooth slippery surface at a depth smaller than the width of the load can decrease the shear strength of the base layer. This would be the case of a smooth surface geotextile placed at a depth less than approximately 0.3 m under a typical dual wheel.

In addition to the tensile behavior, it is necessary to address the friction characteristics of the geogrid/base layer aggregate interface. Available data leads to the reasonable assumption that the frictional resistance at the geogrid-aggregate interface is approximately equal to the frictional resistance of the aggregate alone.

Following the work of Hammit (1970), Giroud et al. (1984) extended the method of design for unpaved roads with standard axle loads to that for any axle load, and hence to the development of a design method for geogrid reinforced unpaved roads. The design method presented takes into account three mechanisms: confinement of the subgrade soil, improved load distribution and tensioned membrane effect. The influence that the geogrid had on load distribution was an increase in the angle at which the load was distributed through the base to the subgrade. The influence of the tensioned membrane effect resulted in a difference in the normal stresses on both sides of the reinforcing element. Based on these effects, an equation was developed for the thickness ratio, h/h_0 , where h and h_0 are the thickness of the base layer with and without reinforcement.

In conclusion, Giroud et al. (1984) stated that the calculations presented in this paper show that the improvement of load distribution capability of the base layer was one of the two mechanisms that provided the most improvement to unpaved structures. Their work showed that approximately half of the aggregate savings resulted from the geogrid reinforcement confinement and the other half from the improved load distribution resulting from geogrid-base layer interlocking.

Love et al. (1987) studied the use of geogrids for reinforcement. It was noted that with the advent of stiffer geotextiles and geogrids, it seemed possible that additional benefits might be obtained, not only from an increased membrane action but also from a reinforcing action of the fill, particularly with geogrids that have the ability to interlock mechanically with the aggregate particles in the fill. There is little doubt that some reinforcing materials also improve the performance of an unpaved road by acting as a separator between the clay and granular fill. However, Love et al. (1987) thought this benefit difficult to quantify.

Theoretical studies have shown that the geogrid reinforcement tends to reduce the shear stresses transmitted to the surface of the clay subgrade. The amount of reduction

has been found to depend primarily on the strength of the clay and the thickness and stiffness of the granular layer. The failure mechanisms in the clay mobilized at quite small deformations of the fill, and large deformations were not necessary for the benefits of reinforcement to be felt. At large deformations, additional benefit was obtained from the membrane action of the reinforcement.

To have the desired effect, the reinforcement had to be stiff enough and strong enough to take the tension induced by the shear stresses from the granular layer above. For the shear to be transferred to the reinforcement, it was necessary for the granular layer to interact efficiently with the reinforcement. This was true of grids provided they remained in contact with granular material that interlocked with the grid structure. There was a danger with soft clay subgrades, as evidenced by both model and full-scale tests, of clay being extruded through the grid breaking the bond between the grid and granular layer. This problem could probably be overcome by good construction techniques. On soft clay subgrades, when separation of fill and clay and the contribution of membrane forces are more significant than the reinforcing action, a geotextile would probably be more appropriate.

6. SUMMARY

A summary table has been prepared from the literature review of geotextiles for reinforcement and separation in unpaved roads. Included in the summary table are condensed procedures, key findings, and assumptions

Further research will include performing interface tests, either through direct shear or pullout, to determine stiffness and friction properties of a select number of geotextiles. Small-scale performance tests will also be performed so as to develop a better understanding of the mechanics that are involved in the performance of the system. Using the information gained by performing these tests and supplementing it with the information included in the summary table, it is believed that guidelines for designing unpaved roads using geotextiles for reinforcement can be developed.

Author(s) and Year of Publication	Overview of Work and Findings	Key Assumptions
<i>Barenberg (1975)</i>	<ul style="list-style-type: none"> • Developed reinforced unpaved road design criteria • Found geotextile properties have a significant effect on behavior of SFA system—specifically, the geotextile modulus • Permissible values for allowable subgrade stress were found to be: <i>with geotextile: $\sigma=6c_u$</i> <i>without geotextile: $\sigma=3.2c_u$</i> 	<ul style="list-style-type: none"> • Deflected shape of geotextile was a circular arc • No slip
<i>Kinney (1975)</i>	<ul style="list-style-type: none"> • Developed the fabric tension model (FTM) • Included methods to determine geotextile shear and normal stresses and geotextile strain energy • FTM was basis for Barenberg's (1980) revised SFA design procedure 	<ul style="list-style-type: none"> • No slip

<p>Barenberg (1980)</p>	<p>Developed procedure using Kinney's (1979) FTM <i>Procedure:</i></p> <ol style="list-style-type: none"> 1. $\sigma_{all} = 3.2Ac_u$ $A = 1.0-2.0$ 2. Estimate gravel thickness 3. Establish rut geometry: depth, d, chosen width, W: $W=B+2X$ B = gravel rut width X = spreading effect 4. Geotextile strain: $\varepsilon_f = \left(\frac{4\pi R\theta}{135W} - 2 \right) \times 100\%$ $R = \frac{3}{8} \frac{W}{\sin \theta}, \theta = 2 \tan^{-1} \left(\frac{5}{3} \frac{d}{W} \right)$ 5. Fabric tension, t_f: $t_f = K\varepsilon_f$ 6. Differential stress across fabric, $\Delta\sigma_{z-f}$: $\Delta\sigma_{z-f} = t_f / R$ 7. Permissible stress on fabric: $\sigma_{p-f} = \Delta\sigma_{z-f} + 3.2Ac_u$ 8. Using Boussinesq equation, calculate vertical stress on fabric and check to see if less than permissible. 	<ul style="list-style-type: none"> • Developed for Mirafi® 140 and 500X • Deflected shape of geotextile was a circular arc • No slip
<p>Bell et al. (1981)</p>	<ul style="list-style-type: none"> • Performed laboratory tests to investigate subbase contamination • Found relatively thick geotextiles with small pore sizes to be effective in limiting contamination • High pore water pressure dissipation rates were associated with high contamination and low dissipation rates with low contamination • Nonwoven geotextiles found to be ineffective in preventing clayey fines contamination but more successful in preventing subbase penetration • Geotextiles with effective opening sizes less than 100 mm reduced clay pumping significantly • Cushioning effect of thick geotextiles reduced stone penetration into soil • All geotextiles tested performed a useful separation function 	<ul style="list-style-type: none"> • Separation function only

<p><i>Giroud and Noiray (1981)</i></p>	<ul style="list-style-type: none"> • Developed an equation to determine the required aggregate layer thickness, h, as a function of loading, subgrade undrained shear strength, and geotextile properties. <p><i>Analytic expression:</i></p> $(\pi + 2)c_u = \frac{P}{2(B + 2h \tan \alpha)(L + 2h \tan \alpha)} + \frac{K \varepsilon}{a \sqrt{1 + \left(\frac{\sigma}{2s}\right)^2}}$	<ul style="list-style-type: none"> • No slip of geotextile relative to aggregate and subgrade • <10,000 cycles • Aggregate CBR of 80 • Purely cohesive subgrade soils • Membrane action
<p><i>Hoare (1982)</i></p>	<ul style="list-style-type: none"> • Performed lab tests to investigate subgrade pumping • Used soil contamination value, SCV (weight of subgrade soil passing the fabric per unit area of fabric) to measure pumping • Soil found to migrate through fabric at contact points between subbase and subgrade • SCV found to be insignificantly reduced by the use of a heavier needle-punched fabric rather than a light melt-bonded fabric 	<ul style="list-style-type: none"> • Separation function only
<p><i>Haliburton and Barron (1983)</i></p>	<ul style="list-style-type: none"> • Investigated effect of fabric location on SFA performance • Optimum placement depth found to be 1/3 the width of the loaded area • At large deformations, membrane support (even for high modulus fabrics) was small compared to total load capacity • Separation seen as allowing pore pressure dissipation and consolidation 	<ul style="list-style-type: none"> • Observations from field and lab tests
<p><i>Douglas et al. (1986)</i></p>	<ul style="list-style-type: none"> • Performed fatigue tests on SFA systems • Rutting found to be less for stiffer geotextiles • More anchorage resulted in better conditions • Use of two geotextiles-one at the interface the other at the base middepth-performed better than one geotextile at interface 	<ul style="list-style-type: none"> • Based on performance tests

<p>Page (1990)</p>	<ul style="list-style-type: none"> • Reported on 8 sites where geotextiles were exhumed • Worst blinding was observed in the woven slit films • It was recommended that lightweight (3.5 oz/yd²) nonwoven geotextiles not be used for separation 	<ul style="list-style-type: none"> • Findings based on tests and observations
<p>Espinoza (1994)</p>	<ul style="list-style-type: none"> • Developed an expression for evaluating the increase in bearing capacity due to membrane action. • Allow. applied stress, q_{ap}: $q_{ap} = q_s + q_{avg}$ $q_s = \text{soil reaction}$ $q_{avg} = \text{additional capacity due to geotextile}$ • $q_{avg} = \frac{2E}{L} \varepsilon \ln(\tan \beta_o + \sqrt{1 + \tan^2 \beta_o})$ <p><i>Parabolic Deformation:</i></p> $\varepsilon = \frac{1}{2} \left[\sec \beta_o + \frac{\ln(\tan \beta_o + \sec \beta_o)}{\tan \beta_o} \right]$ <p><i>Circular Deformation:</i></p> $\varepsilon = \frac{1 + (2\alpha)^2}{4\alpha} \tan^{-1} \beta_o - 1$ 	<ul style="list-style-type: none"> • No slip • $\tau_{lower} = 0, \tau_{upper} = \tau$ • Constant strain
<p>Nishida and Nishigata (1995)</p>	<ul style="list-style-type: none"> • Investigated relationship between reinforcement and separation in geotextile reinforced unpaved roads • Separation found to be primary function when $\sigma/c_u < 8$ and reinforcement found to be primary function when $\sigma/c_u > 8$ $\sigma = \text{vertical stress on subgrade}$ $c_u = \text{subgrade undrained shear strength}$ 	<ul style="list-style-type: none"> • Results not applicable to nonwovens

<p>Espinoza and Bray (1995)</p>	<ul style="list-style-type: none"> • Developed procedure for evaluating the load capacity of reinforced soils • Analysis incorporated two important membrane support contributions: normal stress and interfacial shear stress support • An expression for admissible surface stress, p_{adm}, was developed based on a modified bearing capacity equation $p_{adm} = \frac{(c_u N_c + T_o \sin \beta_o / L + 2a_s \gamma h \tan \psi_m)}{a_s (1 - 2a_s \tan \psi_m)}$ where: $\tan \psi_m = \frac{[\alpha_s (K - K_{pm}) + M_c (\gamma K - \tan \delta_s)]}{(1 + M_c + 2\alpha_s [\alpha_h (K - K_{pm}) - \gamma K + \tan \delta_h])}$ 	<ul style="list-style-type: none"> • slip considered
<p>Metcalf et al. (1995)</p>	<ul style="list-style-type: none"> • Investigated the performance of 22 geotextile separators installed between 1978 and 1991 • Short-term separation and drainage functions were found to be more critical than their long-term performance due to subgrade strength gain due to consolidation • Subgrades of excavated sites found to be consolidated and strong • Even if geotextiles became blinded or clogged in short-term still able to perform while subgrade consolidated • Found to be little need for long-term separation 	<ul style="list-style-type: none"> • Separation function only

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8. APPENDIX

Product Name	Physical Properties		Filtration/Hydraulic Properties			Mechanical Properties					Creep Limited Strength ASTM D 5262 T1 [5] kN/m (lb/ft)	Manufact. Suggested Applic. [6]		
	Structure [1]	Polymer Type [2]	Mass Per Unit Area ASTM D 5261 g/m ² (oz/yd ²)	Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec ⁻¹ /Flow Rate ¹ FH or CH [3] l/min/m ² (gal/min/ft ²)	Puncture ASTM D 4833 kN (lb)	Mullen Burst ASTM D 3786 kPa (psi)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile /Elongation ASTM D 4632 kN (lb)%			Wide Width Tensile Properties [4]	
													Strength @ 5% Strain	Ultimate Strength %
4000	NW	PP	NA	NA	0.212 (70)	(150)	(55)	(195)	(40)	(95)	NP	NP	NP	S/S
4420	NW	PP	NA	NA	0.212 (70)	(140)	(60)	(210)	(45)	(105)	NP	NP	NP	S/S
6600	NW	PP	NA	NA	0.180 (80)	(110)	(90)	(305)	(65)	(165)	NP	NP	NP	S/S
8800	NW	PP	NA	NA	0.180 (80) 0.150	(110)	(115)	(355)	(85)	(210)	NP	NP	NP	S/S
1020	NW	PP	NA	NA	(100) 0.150	(70)	(149)	(440)	(100)	(285)	NP	NP	NP	S/S
1220	NW	PP	NA	NA	(100)	(60)	(170)	(550)	(114)	(325)	NP	NP	NP	S/S

Advanced Drainage Systems Inc.

Contact: Steve Helmrich
Advanced Drainage Systems, Inc.
3300 Riverside Drive
Columbus, OH 43221
fax: 614.459.0169

* Information sent included Construction Specs published by the Construction Specifications Institute (CSI).

- (1) W = Woven NW = Nonwoven
- (2) PP = Polypropylene
- (3) FH = Test is run by the falling head method
CH = Test is run by the constant head method
- (4) Reinforcement applications only (contact manufacturer for information on how the test is performed)

- (5) Test per ASTM D 5262, for a minimum of 10,000 hours and extrapolated to a 75 year time period.
- (6) S/S = Separation/stabilization
NA = Not applicable, per manufacturer
NP = Not provided

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

GEOTEXTILES

Product Name	Physical Properties			Filtration/Hydraulic Properties			Mechanical Properties					Creep Limited Strength ASTM D 5262 TI [5] kN/m (lb/ft) Manufact. Suggested Applic. [6]					
	Structure [1]	Polymer Type [2]	Mass Per Unit Area ASTM D 5261 g/m ² (oz/yd ²)	Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec ⁻¹ /Flow Rate ¹ FH or CH [3] l/min/m ² (gal/min/ft ²)	Puncture ASTM D 4833 kN (lb)	Mullen Burst ASTM D 3786 kPa (psi)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile /Elongation ASTM D 4632 kN (lb)%	Wide Width Tensile Properties [4]						
											Strength @ 5% Strain		Ultimate Strength %				
							MD	XD	MD	XD	MD	XD					
Amoco 2000	W	PP	NA	NA	0.600 (30)	0.05/160 (4), FH	0.285 (65)	2240 (325)	0.200 (45)	0.620 (140)/15	NA	NA	NA	NA	NA	NA	S/S
Amoco 2002	W	PP	NA	NA	0.300 (50)	0.05/200 (5), FH	0.400 (90)	2750 (400)	0.330 (75)	0.890 (200)/15	NA	NA	NA	NA	NA	NA	S/S
Amoco 2006	W	PP	NA	NA	0.425 (40)	0.05/160 (4), FH	0.530 (120)	4130 (600)	0.530 (120)	1.40 (315)/15	NA	NA	30.6 (175)/ (175)/	30.6 (175)/ (175)/	NA	NA	S/S
Amoco 2016	W	PP	NA	NA	0.425 (40)	0.55/1625 (40), FH	0.530 (120)	5510 (800)	0.530 (120)	1.40 (315)/15	NA	NA	35.0 (200)/ (200)/	35.0 (200)/ (200)/	NA	NA	S/S
Amoco 2019	W	PP	NA	NA	0.212 (70)	0.28/730 (18), FH	0.620 (140)	3510 (510)	.420(95) warp fill	1.55x1.11 (320x250) /15	NA	NA	NA	NA	NA	NA	S/S
Amoco 2044	W	PP	NA	NA	0.600 (30)	0.15/405 (10), FH	0.755 (170)	9300 (1350)	1.11 (250)	2.67x2.22 (600x500) /15	21 (9121)	38 (217)	70.0 (400)/ (400)/	70.0 (400)/ (400)/	14.8 (1020)	NA	S/S
Amoco 4510	NW	PP	335 (10.0)	NA	0.150 (100)	1.2/3460 (85), FH	0.730 (165)	3790 (550)	0.445 (100)	1.11 (250)	NA	NA	NA	NA	NA	NA	S/S
Amoco 4512	NW	PP	405 (12.0)	NA	0.150 (100)	0.90/2850 (70), FH	0.865 (195)	4480 (650)	0.510 (115)	1.33 (300)/50	NA	NA	NA	NA	NA	NA	S/S
Amoco 4514	NW	PP	470 (14.0)	NA	0.150 (100)	0.80/2440 (60), FH	1.02 (230)	5170 (750)	0.575 (130)	1.60 (360)/50	NA	NA	NA	NA	NA	NA	S/S
Amoco 4516	NW	PP	540 (16.0)	NA	0.150 (100)	0.70/2035 (50), FH	1.11 (250)	5510 (800)	0.645 (145)	1.78 (400)/50	NA	NA	NA	NA	NA	NA	S/S
Amoco 4546	NW	PP	NA	NA	0.212 (70)	2.0/5700 (140), FH	0.285 (65)	1550 (225)	0.200 (45)	0.445 (100)/50	NA	NA	NA	NA	NA	NA	S/S
Amoco 4547	NW	PP	NA	NA	0.212 (70)	1.8/4885 (120), FH	0.310 (70)	1650 (240)	0.220 (50)	0.530 (120)/50	NA	NA	NA	NA	NA	NA	S/S
Amoco 4550	NW	PP	NA	NA	0.212 (70)	1.7/4685 (115), FH	0.355 (80)	1820 (265)	0.240 (55)	0.575 (130)/50	NA	NA	NA	NA	NA	NA	S/S

Amoco Fabrics & Fibers Co.

Amoco Fabrics & Fibers Co. (cont.)

Amoco 4551	NW	PP	NA	NA	0.150 (100)	1.7/4480 (110), FH	0.400 (90)	2170 (315)	0.285 (65)	0.665 (150)/50	NA	NA	NA	NA	S/S
Amoco 4552	NW	PP	NA	NA	0.150 (100)	1.5/4275 (105), FH	0.465 (105)	2410 (350)	0.330 (75)	0.800 (180)/50	NA	NA	NA	NA	S/S
Amoco 4553	NW	PP	NA	NA	0.150 (100)	1.5/3255 (80), FH	0.575 (130)	2750 (400)	0.355 (80)	0.900 (203)/50	NA	NA	NA	NA	S/S

Contact: Rachel Wallace or Winfred Brown
Amoco Fabrics & Fibers Co.
900 Circle 75 Pkwy. Suite 550
Atlanta, GA 30339
fax: 770.956.2430

* Extensive information was received on product line construction specifications. Technical information was included pertaining to filter design, chemical and UV degradation, and construction techniques. No specific design procedure was recommended.

Belton Industries Inc.

113	W	PP	250	NA	0.425 (40)	0.09/300	0.91	5170	0.56x0.3	2.0x1.7/1	NP	NP	NP	NP	S/S
776	W	PP	125	NA	0.425 (40)	0.17/530	0.37	2665	0.23x0.4	0.82x.94/	NP	NP	NP	NP	S/S
866	W	PP	135	NA	0.425 (40)	0.25/815	0.42	3045	0.35	1.1x.75/1	NP	NP	NP	NP	S/S
777	W	PP	180	NA	0.355 (45)	0.02/120 (3)	0.56	4065	0.41	1.2x1.3/1	NP	NP	NP	NP	S/S

Contact: Bob Moran
Belton Industries Inc.
P.O. Box 127
Belton, SC 29627
fax: 864.338.5594

Information pertaining to erosion control and silt fences were received after contacting manufacturer.

- (1) W = Woven NW = Nonwoven
- (2) PP = Polypropylene
- (3) FH = Test is run by the falling head method
CH = Test is run by the constant head method
- (4) Reinforcement applications only (contact manufacturer for information on how the test is performed)

(5) Test per ASTM D 5262, for a minimum of 10,000 hours and extrapolated to a 75 year time period.

(6) S/S = Separation/stabilization

NA = Not applicable, per manufacturer

NP = Not provided

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

GEOTEXTILES

Product Name	Physical Properties			Filtration/Hydraulic Properties			Mechanical Properties					Creep Limited Strength ASTM D 5262 TI [5] kN/m (lb/ft)	Manufact. Suggested Applic. [6]	
	Structure [1]	Mass Per Unit Area ASTM D 5261 g/m ² (oz/yd ²)	Polymer Type [2]	Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec ⁻¹ /Flow Rate ¹ FH or CH [3] l/min/m ² (gal/min/ft ²)	Puncture Strength ASTM D 4833 kN (lb)	Mullen Burst Strength ASTM D 3786 kPa (psi)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile /Elongation ASTM D 4632 kN (lb)%	Wide Width Tensile Properties [4]			
											Strength @ 5% Strain			Ultimate Strength %
Phoenix B-060	NW needlepunch	200 (6.0)	PP	NA	0.18 (80)	1.2 (100)	(90)	(325)	(70)	(155)/50	NP	NP	NP	S/S
Phoenix B-080	NW needlepunch	275 (8.0)	PP	NA	0.18 (80)	0.95 (85)	(140)	(425)	(95)	(225)/50	NP	NP	NP	S/S
Phoenix B-100	NW needlepunch	340 (10.0)	PP	NA	0.15 (100)	0.90 (75)	(160)	(500)	(110)	(250)/50	NP	NP	NP	S/S
Phoenix SCS-1	NW needlepunch	(6.0)	PET	NA	0.210 (7.0)	1.4 (110)	(90)	(350)	(70)	(180)/60	NP	NP	NP	S/S
Phoenix (4-60)	NW needlepunch	(4.0-6.0)	PET	NA	NP	NP	NP	NP	NP	NP	NP	NP	NP	S/S
Phoenix To spec.	W	(4-20)	PP	NP	NP	NP	NP	NP	NP	NP	NP	NP	NP	S/S

Bradley Industrial Textiles Inc.

Contact: Shepherd Bradley
 Bradley Industrial Textiles Inc.
 P.O. Box 254
 Valparaiso, FL 32580
 fax: 904.729.1052

* Contacted, no response

Carthage Mills

Carthage 6%	W calendered	PP	197 (5.8)	5-6	0.212 (70)	0.29/773 (19), CH	0.664 (145)	3376 (490)	.449x.289 (110x65)	1.69x1.16 (380x260) /15	NA	NA	37.65 (215)/ 18 24.52 (9140)/ 14	NA	S/S
Carthage 10%	W calendered	PP	180 (5.3)	10-12	.425-.3 (40-50)	1.2/2848 (70), CH	0.578 (130)	3514 (510)	.356x.333 (80x75)	1.22x1.16 (275x260) /16	NA	NA	27.15 (155)/ 19 28.9 (165)/ 17	NA	S/S
Carthage 15%	W	PP	183 (5.4)	12-15	.425-.3 (40-50)	1.5/4678 (115), CH	0.533 (120)	3514 (510)	.511x.356 (115x80)	1.64x1.04 (370x235) /15	NA	NA	35.9 (205)/ 19 28.9 (165)/ 11	NA	S/S
Carthage 20%	W	PP	217 (6.4)	15-20	0.212 (70)	0.55/1505 (37), CH	0.667 (150)	4031 (585)	.467x.644 (105x145)	1.24x1.73 (280x390) /16	NA	NA	32.4 (185)/ 10 27 (185)/ 11	NA	S/S
Carthage 30%	W palmered	PP	203 (6.0)	20-30	.425-.30 (40-50)	2.1/6143 (151), CH	0.533 (120)	2859 (415)	.422x.244 (95x55)	1.47x.93 (330x210) /15	NA	NA	31.52 (180)/ 20 20.14 (115)/ 15	NA	S/S
Carthage 8%-HD	W calendered	PP	264 (7.8)	6-8	.30-.25 (50-60)	0.824/2441 (60), CH	0.933 (210)	5925 (860)	.578x.667 (130x150)	1.78x2.0 (400x450) /17	NA	NA	NP	NP	S/S
Carthage 10%-HD	W calendered	PP	264 (7.7)	8-12	(40-50)	1.05/73	(170)	(700)	(150x140) /23	(430x360)	NA	NA	NP	NP	S/S
FX-22	W	PP	92 (2.7)	<1	0.300 (50)	NA	0.267 (60)	1550 (225)	0.178 (40)	0.444 (100)/15	NA	NA	NA	NA	S/S
FX-33	W	PP	105 (3.1)	<1	0.30 (50)	NA	0.289 (65)	1860 (270)	0.222 (50)	0.533 (120)/20	NA	NA	NA	NA	S/S
FX-44	W	PP	119 (3.5)	<1	0.425 (40)	NA	0.311 (70)	2067 (300)	0.267 (60)	0.644 (145)/20	NA	NA	NA	NA	S/S
FX-55	W	PP	169 (5.0)	<1	0.425 (40)	NA	0.467 (105)	3169 (460)	0.333 (75)	0.889 (200)/15	8.8 (603)	9.7 (665)	21.02 (120)	21.02 (120)	S/S
FX-66	W	PP	203 (6.0)	<1	.425-.30 (40-50)	NA	0.556 (125)	4134 (600)	0.533 (120)	1.33 (300)/15	8.8 (603)	10.6 (726)	30.65 (175)	30.65 (175)	S/S
FX-77	W	PP	264 (7.8)	<1	0.355 (45)	0.16/407 (10), CH	1.0 (225)	5443 (790)	.62x.42 (140x95)	2.11x1.78 (475x400) /20	12.61 (72)	9.46 (54)	48.16 (275)	38.53 (220)	S/S

(1) W = Woven
 (2) PP = Polypropylene
 (3) FH = Test is run by the falling head method
 CH = Test is run by the constant head method
 (4) Reinforcement applications only (contact manufacturer for information on how the test is performed)
 (5) Test per ASTM D 5262, for a minimum of 10,000 hours and extrapolated to a 75 year time period.
 (6) S/S = Separation/stabilization
 NA = Not applicable, per manufacturer
 NP = Not provided

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

GEOTEXTILES

Product Name	Physical Properties			Filtration/Hydraulic Properties			Mechanical Properties					Creep Limited Strength ASTM D 5262 T ₁ [5] kN/m (lb/ft)	Manufact. Suggested Applic. [6]		
	Structure [1]	Polymer Type [2]	Mass Per Unit Area ASTM D 5261 g/m ² (oz/yd ²)	Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec ⁻¹ /Flow Rate ¹ FH or CH [3]	Puncture ASTM D 4833 kN (lb)	Mullen Burst ASTM D 3786 kPa (psi)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile /Elongation ASTM D 4632 kN (lb)%	Wide Width Tensile Properties [4] ASTM D 4595 kN/m (lb/in)				
											Strength @ 5% Strain			Ultimate Strength %	MD
Carthage Mills (cont.)															
FX-400MF	W	PP	427 (12.6)	<1	0.60 (30)	0.5/1525 (40), CH	0.733 (165)	8371 (1215)	.822x.822 (185x185)	NA	19.3 (1322)	22.8 (1562)	70.5 (400)/9	70.5 (400)/9	S/S
FX-60HS	NW	PP	203 (6.0)	NA	0.180 (80)	1.5/4475 (110), CH	0.422 (95)	2239 (325)	0.289 (65)	0.756 (170)/50	NA	NA	NA	NA	S/S
FX-70HS	NW	PP	7.0 (7.0)	NP	(80)	1.3/NP	(100)	(330)	(75)	(270)/50	NA	NA	NA	NA	S/S
FX-80HS	NW	PP	271 (8.0)	NA	0.180 (80)	1.4/4181 (105), CH	0.578 (130)	2756 (400)	0.378 (85)	0.933 (210)/50	NA	NA	NA	NA	S/S
FX-100HS	NW	PP	339 (10.0)	NA	0.150 (100)	1.2/3460 (85), CH	0.711 (160)	3445 (500)	0.444 (100)	1.2 (270)/50	NA	NA	NA	NA	S/S
FX-120HS	NW	PP	12.0 (12.0)	NP	(80)	1.0/NP	(180)	(600)	(115)	(325)/50	NA	NA	NA	NA	S/S
FX-160HS	NW	PP	542 (16.0)	NA	0.150 (100)	0.7/2035 (50), CH	1.089 (245)	5512 (800)	0.689 (155)	1.778 (400)	NA	NA	NA	NA	S/S

Contact: Alan Ossege

Carthage Mills

4243 Hunt Road

Cincinnati, OH 45242

fax: 513.794.3434

Additional information was sent concerning woven high performance geotextiles not included in the above list. Information on Research on Fabric Clogging was also sent.

Comparative Hydraulic Performance Evaluation of Geotechnical Fabrics, 1980

Philip D. Wood, Cecilia Hayes Shappee, T. Allan Haliburton

Haliburton Associates, Stillwater, Oklahoma, U.S.A.

Evaluation of the U.S. Army Corps of Engineers Gradient Ratio Test for Geotextile Performance - Proceedings of Second International Conference on Geotextiles, Las Vegas, Nevada, U.S.A. 1982

Haliburton, T.A.

Haliburton Associates, Stillwater, Oklahoma, U.S.A.

Wood, P.D.

McClelland Engineers, St. Louis, Missouri, U.S.A.

Contech Construction Products Inc.

Contech		155	1.4/4685	0.375	1890	0.250	0.600/50	NA	NA	NP	NP	NA	S/S
C-50 NW	NW	PP (4.7)	NA	0.212 (70)	1.4/4685 (115), CH	0.375 (85)	1890 (275)	0.250 (570)	0.600/50 (135)	NA	NP	NA	S/S
Contech	NW	PP (5.6)	NA	0.212 (70)	1.3/4480 (110), CH	0.420 (95)	2240 (3250)	0.265 (60)	0.665/50 (150)	NA	NP	NA	S/S
C-60 NW	NW	PP (6.0)	NA	0.212 (70)	1.3/4480 (110), CH	0.485 (110)	2340 (340)	0.285 (85)	0.710/50 (160)	NA	NP	NA	S/S
Contech	NW	PP (6.5)	NA	0.212 (70)	1.5/4880 (110), CH	0.485 (110)	2410 (350)	0.330 (75)	0.800/50 (180)	NA	NP	NA	S/S
C-70 NW	NW	PP (7.3)	NA	0.180 (80)	1.5/4480 (110), CH	0.575 (130)	2750 (400)	0.375 (85)	0.890/50 (200)	NA	NP	NA	S/S
Contech	NW	PP (9.25)	NA	0.150 (100)	1.2/3460 (85), CH	0.710 (160)	3510 (510)	0.445 (100)	1.110/50 (250)	NA	NP	NA	S/S
10 NW	NW	PP (11.2)	NA	0.150 (100)	1.0/3055 (75), CH	0.800 (180)	4130 (600)	0.510 (115)	1.335/50 (300)	NA	NP	NA	S/S
Contec C-	NW	PP (15.1)	NA	0.150 (100)	0.7/2035 (50), CH	1.065 (240)	5510 (800)	0.645 (195)	1.690/50 (380)	NA	NP	NA	S/S
16 NW	NW	PP (16.6)	NA	0.150 (100)	0.7/2035 (50), CH	1.155 (260)	6030 (875)	0.665 (150)	1.735/50 (390)	NA	NP	NA	S/S
Contec C-	W/S	PP (4.5)	<1 ^A	0.425 (40)	0.07/240 (6), FH	0.445 (100)	3100 (450)	0.330x.33	.890x.890 (200x200)/11	8.7 (50)	21/9 (120)	21/7 (120)	S/S
200	W/S	PP (6.0)	<1 ^A	0.425 (40)	0.06/200 (5), FH	0.530 (120)	4130 (600)	0.30	1.335x1.335 (300x300)/11	8.7 (50)	30.6/1 (175)	30.6/8 (175)	S/S
Contec C-	W/M	PP (6.1)	4 ^A	0.212 (70)	0.28/730 (18), CH	0.665 (150)	3300 (480)	0.420x.24	1.755x1.155 (395x260)/12	NA	NA	NA	S/S

Contact: Thomas Baas
 Contech Construction Products Inc.
 1001 Grove Street
 Middletown, OH 45044
 fax: 513.425.5993

Contech Construction Products Inc. distributes Tensar. Generally, only Contech nonwoven products could be used for S/S purposes. Information sent included Literature Review on Geotextiles to Improve Pavements for General Aviation Airports (D. White, USAE Waterways Experiment Station, Vicksburg, MS, Feb. 1991). Review pertains to flexible pavements, although chart pertains to unpaved roads.

- (1) W = Woven
- (2) PP = Polypropylene
- (3) FH = Test is run by the falling head method
- (4) Reinforcement applications only (contact manufacturer for information on how the test is performed)

- (5) Test per ASTM D 5282, for a minimum of 10,000 hours and extrapolated to a 75 year time period.
- (6) S/S = Separation/stabilization
- NA = Not applicable, per manufacturer
- NP = Not provided

(A) Area of openings / total x 100% determined from a light projection test.

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

GEOTEXTILES

Product Name	Physical Properties			Filtration/Hydraulic Properties			Mechanical Properties					Creep Limited Strength ASTM D 5262 T, [5] kN/m (lb/ft) Manufact. Suggested Applic. [6]				
	Structure [1]	Polymer Type [2]	Mass Per Unit Area ASTM D 5261 g/m ² (oz/yd ²)	Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec ¹ /Flow Rate ¹ FH or CH [3] l/min/m ² (gal/min/ft ²)	Puncture ASTM D 4633 kN (lb)	Mullen Burst ASTM D 3786 kPa (psi)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile /Elongation ASTM D 4632 kN (lb)%	Wide Width Tensile Properties [4]					
											ASTM D 4595 kN/m (lb/in)		Strength @ 5% Strain	Ultimate Strength %		
Hoechst Celanese Corp.																
Trevira 011/200	NW	PET	193 (5.7)	NA	0.210 (70)	1.74/88 (130)	0.356 (80)	1965 (285)	0.267 (60)	0.711 (160)/60	NA	NA	NA	NA	NA	S/S
Trevira 011/250	NW	PET	241 (7.1)	NA	0.210 (70)	1.47/75 (110)	0.423 (95)	2484 (360)	0.334 (75)	0.933 (210)/60	NA	NA	NA	NA	NA	S/S
Trevira 011/280	NW	PET	271 (8.0)	NA	0.210 (70)	1.20/61 (90)	0.444 (100)	2622 (380)	0.356 (80)	1.023 (230)/60	NA	NA	NA	NA	NA	S/S
Trevira 011/350	NW	PET	339 (10.0)	NA	0.210 (70)	1.07/54 (80)	0.578 (130)	3519 (510)	0.444 (100)	1.356 (305)/60	NA	NA	NA	NA	NA	S/S
Trevira 011/420	NW	PET	406 (12.0)	NA	0.210 (70)	0.87/44 (65)	0.667 (150)	3795 (550)	0.534 (120)	1.556 (350)/60	NA	NA	NA	NA	NA	S/S
Trevira 011/550	NW	PET	541 (16.0)	NA	0.149 (100)	0.53/27 (40)	0.867 (195)	5382 (780)	0.667 (150)	2.224 (500)/70	NA	NA	NA	NA	NA	S/S
* Contacted, no response																
Hoesker Inc.																
Comtrac 70.70	W (5 m wide)	PET	240 (7)	NA	0.212 (70)	NA	NA	NA	NA	NA	31.5 (180)	28 (160)	70/11 (400)	42 (2900)	NA	S/S
Comtrac 90.90	W (5 m wide)	PET	270 (8)	NA	0.212 (70)	NA	NA	NA	NA	NA	40.5 (230)	36 (205)	90/11 (510)	54 (3700)	NA	S/S
Contact: Tom Collins Hoesker Inc. 11107A S. Commerce Blvd. Charlotte, NC 28273 fax: 704.588.5988																
Contact: Dennis Wedding Hoechst Celanese Corp. P.O. Box 5650 Spartanburg, SC 29304 fax: 864.579.5930																
Contact: Tom Collins Hoesker Inc. 11107A S. Commerce Blvd. Charlotte, NC 28273 fax: 704.588.5988																
Contact: Tom Collins Hoesker Inc. 11107A S. Commerce Blvd. Charlotte, NC 28273 fax: 704.588.5988																
Information sent pertained to Fortrac Geogrids. Also included was a Geogrid Specification and Installation Guide Booklet. The design procedure was based on the work of Giroud and Noiray (1981).																

LINQ Industrial Fabrics

GTF 110	W	PP	NA	NA	0.60	0.01	223	2067	245	490/15	NP	NP	NP	NP	NP	NP	S/S
GTF 200S	W	PP	NA	NA	0.30 (50)	0.07, FH	334 (80)	2068 (300)	312 (70)	800 (180)/15	NP	NP	NP	NP	NP	NP	S/S
GTF 200	W	PP	NA	NA	0.425 (40)	0.07, FH	445 (100)	3102 (400)	334 (75)	890 (200)/15	NP	NP	(150)	NP	(150)	NP	S/S
GTF 300	W	PP	NA	NA	0.425 (40)	0.02, FH	645 (145)	4136 (600)	512 (115)	1335 (300)/15	NP	NP	(180)	NP	(180)	NP	S/S
GTF 570	W	PP	NA	NA	0.60 (30)	0.4, FH	712 (160)	8273 (1200)	801 (180)	1847x1825 (415x110)/	NP	NP	70.1 (400)	NP	70.1 (400)	NP	S/S
Typar 3401	NW	PP	NA	NA	0.21 (70)	0.7, FH	178 (40)	965 (140)	267 (60)	579 (130)/60	NP	NP	NP	NP	NP	NP	S/S
Typar 3601	NW	PP	NP	NP	0.10 (140)	0.1, FH	312 (70)	1448 (210)	401 (90)	1068 (240)/60	NP	NP	NP	NP	NP	NP	S/S
Typar 3631	NW	PP	NP	NP	0.7 (200)	0.08, FH	401 (90)	1724 (250)	401 (90)	1446 (325)/60	NP	NP	NP	NP	NP	NP	S/S
Typar 3801	NW	PP	NP	NP	0.07	NA	401	1724	401	1446/60	NP	NP	NP	NP	NP	NP	S/S
150 EX	NW	PP	NP	NP	0.18 (80)	1.3, FH	401 (90)	2137 (310)	289 (65)	734 (165)/50	NP	NP	NP	NP	NP	NP	S/S
160 EX	NW	PP	NP	NP	0.18 (80)	1.2, FH	402 (90)	2206 (320)	312 (70)	801 (180)/50	NP	NP	NP	NP	NP	NP	S/S
180 EX	NW	PP	NP	NP	0.18 (80)	1.2, FH	445 (100)	2275 (330)	334 (75)	890 (200)/50	NP	NP	NP	NP	NP	NP	S/S
225 EX	NW	PP	NP	NP	0.18 (80)	1.0, FH	512 (115)	2482 (360)	378 (85)	957 (215)/50	NP	NP	NP	NP	NP	NP	S/S
250 EX	NW	PP	NP	NP	0.15 (100)	0.8, FH	668 (150)	3102 (450)	445 (100)	1268 (285)/50	NP	NP	NP	NP	NP	NP	S/S

(1) W = Woven NW = Nonwoven

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CH = Test is run by the constant head method

(4) Reinforcement applications only (contact manufacturer for information on how the test is performed)

(5) Test per ASTM D 5262, for a minimum of 10,000 hours and extrapolated to a 75 year time period.

(6) S/S = Separation/stabilization

NA = Not applicable, per manufacturer

NP = Not provided

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

Information obtained from Geotextiles Fabrics Report 1997 Specifier's Guide, Dec. 1996, 14 (9).

GEOTEXTILES

Product Name	Physical Properties			Filtration/Hydraulic Properties			Mechanical Properties					Creep Limited Strength ASTM D 5262 T ₁ [5] kN/m (lb/ft) Manufact. Suggested Applic. [6]		
	Structure [1]	Polymer Type [2]	Mass Per Unit Area ASTM D 5261 g/m ² (oz/yd ²)	Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec ¹ /Flow Rate ¹ FH or CH [3] l/min/m ² (gal/min/ft ²)	Puncture ASTM D 4833 kN (lb)	Mullen Burst ASTM D 3786 kPa (psi)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile /Elongation ASTM D 4632 kN (lb)%	Wide Width Tensile Properties [4]			
											Strength @ 5% Strain		Ultimate Strength %	
275 EX	NW	PP	NP	NP	0.15 (100)	0.7, FH	757 (170)	3792 (550)	512 (115)	1446 (325)/50	NP	NP	NP	S/S
350 EX	NW	PP	NP	NP	0.15 (100)	0.5, FH	1113 (250)	5171 (750)	734 (165)	1891 (425)/50	NP	NP	NP	S/S

LINQ Industrial Fabrics (cont.)

Contact: Sean Kiniry

LINQ Industrial Fabrics

2550 W. Fifth North St.

Summerville, SC 29483-9669

fax: 803.875.8276

Information was received after contacting manufacturer. Geotextile Design Manual For Paved and Unpaved Roads (provided by Exxon Chemical Co.) was sent. The design methods included in the manual are consistent with FHWA design procedures as established in their publication entitled Geotextile Engineering Manual. Unpaved road design references are Steward et al. (1977) and Giroud and Noiray (1981).

Woven Geotextiles

GTF-110

Designed for light duty stabilization applications where separation is the primary function and strength is a minimum requirement.

A medium strength fabric designed for applications where separation is the major function and moderate strength is desired.

GTF200S meets the AASHTO M288-92 requirements for medium survivability.

GTF-200

A general use, high-strength, high-modulus fabric designed to meet demanding strength and separation requirements. GTF 200 meets the AASHTO M288-92 requirements for moderate survivability.

A specialized, heavy-duty fabric with superior strength, modulus, and other physical properties. GTF 300 meets the AASHTO M288-92 requirements for high survivability.

Typar (Non Woven) Geotextiles

Typar 3401, 3601

General-use, durable, nonwoven geotextiles where separation, stabilization, drainage and erosion control are primary functions.

Typar 3401 and Typar 3601 satisfy AASHTO M288-92 requirements for protected drainage, erosion control applications, and medium survivability separation applications

Typar 3631, 3801

High-strength, nonwoven geotextiles designed to meet demanding separation requirements. These rugged, heavy-duty fabrics add a margin of safety to your design. Typar 3631 and Typar 3801 satisfy AASHTO M288-92 requirements for high survivability separation (stabilization) applications.

Mirafi														
00CX	W	PP	NA	NA	0.595 (30)	NAV1019 (25), CH	0.27 (60)	2067 (300)	0.27x0.27 (60x60)	0.53x0.45 (120x100) /15	NA	NA	NA	S/S
500XL	W	PP	NA	1	0.595 (30)	0.040/163 (4), CH	0.31 (70)	2412 (9350)	0.20x0.20 (45x45)	0.62 (140)/15	NA	NA	NA	S/S
500X	W	PP	NA	1	0.595 (30)	0.030/163 (4), CH	0.40 (90)	2756 (400)	0.33x0.33 (75x75)	0.89x0.89 (200x200) /15x15	NA	NA	NA	S/S
600X	W	PP	NA	1	0.420 (40)	0.020/81 (2), CH	0.53 (120)	4134 (600)	0.53x0.53 (120x120)	1.34x1.34 (300x300) /15x15	NA	NA	NA	S/S
700XG	W	PP	NA	10	0.420 (40)	1.360/4074 (100), CH	0.47 (105)	3307 (480)	0.51x0.33 (115x75)	1.62x0.89 (365x200) /24x10	NA	NA	NA	S/S
HP800X	W	PP	NA	1	0.210 (70)	0.100/326 (8), CH	0.73 (165)	4479 (650)	0.53x1.11 (120x250)	1.49x1.89 (335x425) /20x15	NA	29 (1980)	46 (3120)	S/S
HP500	W	PP	NA	8	0.595 (30)	1.500/4685 (115), CH	0.65 (145)	4995 (725)	0.65x0.56 (145x125)	1.78x1.49 (400x335) /20x15	10 (660)	10 (660)	39 (2700)	S/S
HP550	W	PP	NA	6	0.420 (40)	0.960/2852 (70), CH	0.73 (165)	4789 (695)	0.65x0.56 (145x125)	1.89x1.56 (425x350) /21x21	11 (720)	11 (720)	39 (2700)	S/S
HP565	W	PP	NA	NA	0.420 (40)	0.070/204 (5), CH	0.89 (200)	8268 (1200)	0.78x0.96 (175x215)	2.23x2.23 (500x500) /15x10	23 (1560)	35 (2400)	61 (4200)	S/S
HP570	W	PP	NA	NA	0.595 (30)	0.4090/1222 (30), CH	0.71 (160)	8269 (1200)	0.80x0.80 (180x180)	1.85x1.82 (415x410) /12x8	35 (2400)	36 (2400)	70 (4800)	S/S

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CH = Test is run by the constant head method
(4) Reinforcement applications only (contact manufacturer for information on how the test is performed)

(5) Test per ASTM D 5262, for a minimum of 10,000 hours and extrapolated to a 75 year time period.
(6) S/S = Separation/stabilization
NA = Not applicable, per manufacturer
NP = Not provided

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

GEOTEXTILES

Product Name	Physical Properties			Filtration/Hydraulic Properties			Mechanical Properties						Creep Limited Strength ASTM D 5262 T ₁ [5] kN/m (lb/ft)	Manufact. Suggested Applic. [6]		
	Structure [1]	Polymer Type [2]	Mass Per Unit Area ASTM D 5261 g/m ² (oz/yd ²)	Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec ⁻¹ /Flow Rate ¹ FH or CH [3]	Puncture ASTM D 4833 kN (lb)	Mullen Burst ASTM D 3786 kPa (psi)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile /Elongation ASTM D 4632 kN (lb)/%	Wide Width Tensile Properties [4]					
											Strength @ 5% Strain	Ultimate Strength %			MD	XD
Mirafi (cont.)																
HP600	W	PP	NA	NA	0.300 (50)	0.216/652 (16), CH	0.98 (220)	9302 (1350)	1.05x1.05 (235x235)	2.36 (530)/16x16 (1620)	24 (1620)	44 (3000)	78 (5340)	70 (4800)	16 (1068)	S/S
140N	NW	PP	NP	NA	0.210 (70)	1.800/5500 (135), CH	0.31 (70)	1654 (240)	0.22 (50)	0.53 (120)/50	NA	NA	NA	NA	NA	S/S
160N	NW	PP	NP	NA	0.210 (70)	1.500/4889 (120), CH	0.42 (95)	2239 (325)	0.27 (60)	0.71 (160)/50	NA	NA	NA	NA	NA	S/S
180N	NW	PP	NP	NA	0.177 (80)	1.300/4481 (110), CH	0.58 (130)	2756 (400)	0.36 (80)	0.90 (205)/50	NA	NA	NA	NA	NA	S/S
1100N	NW	PP	NP	NA	0.149 (100)	1.100/3056 (75), CH	0.69 (155)	3514 (510)	0.45 (100)	1.11 (250)/50	NA	NA	NA	NA	NA	S/S
1120N	NW	PP	NP	NA	0.149 (100)	0.900/2648 (65), CH	0.78 (175)	4134 (600)	0.51 (115)	1.34 (300)/50	NA	NA	NA	NA	NA	S/S
1160N	NW	PP	NP	NA	0.149 (100)	0.700/2037 (50), CH	1.05 (235)	4892 (710)	0.62 (140)	1.69 (380)/50	NA	NA	NA	NA	NA	S/S

Contact: Greg Roscoe

Mirafi

365 S. Holland Drive
Pendergrass, GA 30567
fax: 706.693.4400

Information was received after contacting manufacturer. Mirafi recommends using their heavy duty woven products for reinforcing/stabilizing unpaved roads. Use of non woven products is not recommended. The Mirafi design procedure is based on Kinney's (1979) fabric tension model. Design examples are included in the procedure. Mirafi 500X, and 600X are recommended products for reinforcing unpaved roads.

GEOTEXTILES

Product Name	Physical Properties			Filtration/Hydraulic Properties				Mechanical Properties						Creep Limited Strength ASTM D 5262 T ₁ [5] kN/m (lb/ft)	Manufact. Suggested Applic. [6]		
	Structure [1]	Polymer Type [2]	Mass Per Unit Area ASTM D 5261 g/m ² (oz/yd ²)	Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec ² /Flow Rate ¹ FH or CH [3] l/m ² (gal/min/ft ²)	Puncture ASTM D 4833 kN (lb)	Mullen Burst ASTM D 3786 kPa (psi)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile /Elongation ASTM D 4632 kN (lb)%	Wide Width Tensile Properties [4]		XD			MD	XD
											ASTM D 4595 kN/m (lb/in)	Strength @ 5% Strain					
Geotex 701	NW	PP	220 (6.5)	NA	0.212 (70)	1.5/4480 (110), CH	0.485 (110)	2410 (350)	0.330 (75)	0.800 (180)/50	NA	NP	NP	NA	S/S		
Geotex 801	NW	PP	245 (7.3)	NA	0.180 (80)	1.5/4480 (110), CH	0.575 (130)	2750 (400)	0.375 (85)	0.890 (200)/50	NA	NP	NP	NA	S/S		
Geotex 1001	NW	PP	310 (9.25)	NA	0.150 (100)	1.2/3460 (85), CH	0.710 (160)	3510 (510)	0.445 (100)	1.110 (250)/50	NA	NP	NP	NA	S/S		
Geotex 1201	NW	PP	375 (11.2)	NA	0.150 (100)	1.0/3055 (75), CH	0.800 (180)	4130 (600)	0.510 (115)	1.335 (300)/50	NA	NP	NP	NA	S/S		
Geotex 1601	NW	PP	510 (15.1)	NA	0.150 (100)	0.7/2035 (50), CH	1.065 (240)	5510 (800)	0.645 (145)	1.690 (380)/50	NA	NP	NP	NA	S/S		
Geotex 1751	NW	PP	560 (16.6)	NA	0.150 (100)	0.7/2035 (50), CH	1.155 (260)	6030 (875)	0.665 (150)	1.735 (390)/50	NA	NP	NP	NA	S/S		

Synthetic Industries Inc. (cont.)

Contact: Marc S. Theisen

Synthetic Industries Inc.

4019 Industry Dr.

Chattanooga, TN 37416

fax: 423.899.7617

* Contacted, no response

TNS Advanced Technologies

E040	NW	PP	135 (4.0)	NA	0.212 (70)	2.0 (160)	0.29 (65)	1585 (230)	0.20 (45)	0.47 (105)	NA	NA	NA	NA	S/S
E060	NW	PP	203 (6.0)	NA	0.18 (80)	1.63 (125)	0.42 (95)	2412 (350)	0.29 (55)	0.71 (160)	NA	NA	NA	NA	S/S
E070	NW	PP	237 (7.0)	NA	0.18 (80)	1.41/4470 (110)	0.51 (115)	2756 (400)	0.33 (75)	0.89 (200)	NA	NA	NA	NA	S/S
E080	NW	PP	271 (8.0)	NA	0.18 (80)	1.26/4064 (100)	0.58 (130)	3101 (450)	0.40 (90)	1.00 (225)	NA	NA	NA	NA	S/S
E100	NW	PP	339 (10.0)	NA	0.15 (100)	0.94/3055 (75)	0.73 (165)	3858 (560)	0.45 (100)	1.20 (270)	NA	NA	NA	NA	S/S
E120	NW	PP	407 (12.0)	NA	0.15 (100)	0.90/2544 (70)	0.85 (190)	4479 (650)	0.56 (125)	1.56 (350)	NA	NA	NA	NA	S/S
E140	NW	PP	474 (14.0)	NA	0.15 (100)	0.64/2037 (50)	0.93 (210)	4995 (725)	0.60 (135)	1.74 (390)	NA	NA	NA	NA	S/S
E160	NW	PP	542 (16.0)	NA	0.15 (100)	0.57/1833 (45)	1.07 (240)	5512 (800)	0.67 (150)	1.89 (425)	NA	NA	NA	NA	S/S
R060	NW	PP	NP	NA	0.18 (80)	1.6/4480 (110)	0.401 (90)	2170 (315)	0.29 (65)	0.70 (160)	NA	NA	NA	NA	S/S
R070	NW	PP	NP	NA	0.18 (80)	1.5/4074 (100)	0.467 (105)	2411 (350)	0.33 (75)	0.80 (180)	NA	NA	NA	NA	S/S
R080	NW	PP	NP	NA	0.18 (80)	1.4/3657 (90)	0.575 (130)	2750 (400)	0.375 (85)	0.89 (200)	NA	NA	NA	NA	S/S
R100	NW	PP	NP	NA	0.18 (80)	1.2/3251 (80)	0.71 (160)	3545 (520)	0.445 (100)	1.11 (250)	NA	NA	NA	NA	S/S
R120	NW	PP	NP	NA	0.15 (100)	1.0/3055 (75)	0.80 (180)	4130 (600)	0.51 (115)	1.335 (300)	NA	NA	NA	NA	S/S
R160	NW	PP	NP	NA	0.15 (100)	0.7/2035 (50)	1.065 (240)	5510 (800)	0.645 (145)	1.69 (380)	NA	NA	NA	NA	S/S

(1) W = Woven NW = Nonwoven

(2) PP = Polypropylene

(3) FH = Test is run by the falling head method

CH = Test is run by the constant head method

(4) Reinforcement applications only (contact manufacturer for information on how the test is performed)

(5) Test per ASTM D 5262, for a minimum of 10,000 hours and extrapolated to a 75 year time period.

(6) S/S = Separation/stabilization

NA = Not applicable, per manufacturer

NP = Not provided

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

Information obtained from Geotextiles Fabrics Report 1997 Specifier's Guide, Dec. 1996, 14 (9).

GEOTEXTILES

Product Name	Physical Properties			Filtration/Hydraulic Properties				Mechanical Properties					Creep Limited Strength ASTM D 5262 T ₁ [5] kN/m (lb/ft) Manufact. Suggested Applic. [6]
	Structure [1]	Polymer Type [2]	Mass Per Unit Area ASTM D 5261 g/m ² (oz/yd ²)	Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec ⁻¹ /Flow Rate ¹ FH or CH [3] l/min/m ² (gal/min/ft ²)	Puncture ASTM D 4833 kN (lb)	Mullen Burst ASTM D 3786 kPa (psi)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile /Elongation ASTM D 4632 kN (lb)%	Wide Width Tensile Properties [4] ASTM D 4595 kN/m (lb/in)		
											Strength @ 5% Strain	Ultimate Strength %	

TNS Advanced Technologies (cont.)

W200	W	PP	136 (4)	1	0.300 (50)	0.04/NP	0.44 (100)	2756 (400)	0.31 (70)	0.89 (200)	4 (300)	12 (840)	18 (1200)/10 (1440)/10	21 (1440)/10	NP	S/S
W300	W	PP	203 (6)	1	0.355 (45)	0.02/NP	0.44 (100)	4134 (100)	0.44 (100)	1.33 (300)	5 (336)	14 (960)	29 (1980)/10 (2100)/10	31 (2100)/10	6 (396)	S/S

Contact: Monte Thomas
* Contacted, no response

TNS Advanced Technologies
681 Deyoung Road
Greer, SC 29651
fax: 864.879.4639

USA Spantex

Spantex 571	K	PET	314 (9.62)	NA	NA	5	NA	NA	NA	NA	40 (228)	20 (114)	100 (571)/10 (285)/12	50 (285)/12	<1	S/S
Spantex 1086	K	PET	436 (12.86)	NA	NA	6	NA	NA	NA	NA	76 (434)	34 (9194)	190 (1085)/10 (486)/12	85 (486)/12	<1	S/S
Spantex 2543	K	PET	1072 (31.62)	NA	NA	7	NA	NA	NA	NA	178 (1017)	366 (366)	445 (2543)/10 (914)/12	160 (914)/12	<1	S/S
Spantex 5710	K	PET	2566 (75.69)	NA	NA	8	NA	NA	NA	NA	400 (2284)	200 (1142)	1000 (5710)/10 (2855)/12	500 (2855)/12	<1	S/S

Contact: F.P. Flynn
* Contacted, no response

USA Spandex Inc.
132 Franklin Street
Norwich, CT 06360
fax: 860.889.3532

WEBTEC Inc.

TerraTex	W	PP	NA	NA	0.60 (30)	1.3/400 (10), FH	0.27 (60)	2060 (300)	0.29 (65)	0.53 (120)/15	NA	NA	NA	NA	NA	NA	NA	S/S
TerraTex SC	W	PP	NA	NA	0.60 (30)	1.3/400 (10), FH	0.27 (60)	2060 (300)	0.29 (65)	0.53 (120)/15	NA	NA	NA	NA	NA	NA	NA	S/S
TerraTex GS-150	W	PP	NA	NA	0.60 (30)	0.04/163 (4), FH	0.29 (65)	2239 (325)	0.20 (45)	0.62 (140)/15	NA	NA	NA	NA	NA	NA	NA	S/S
TerraTex GS	W	PP	NA	NA	0.30 (50)	0.04/163 (4), FH	0.40 (90)	2756 (400)	0.33 (75)	0.89 (200)/15	NP	NP	NP	NP	NP	NP	NP	S/S
TerraTex HD	W	PP	NA	NA	0.42 (40)	0.02/81 (2), FH	0.53 (120)	4134 (600)	0.53 (120)	1.34 (300)/15	NP	NP	NP	NP	NP	NP	NP	S/S
TerraTex EP	W	PP	NA	4	0.21 (70)	0.28/730 (18), FH	0.60 (135)	3300 (480)	0.45x0.27 (100x60)	1.65x1.11 (375x250) /16x15	NP	NP	NP	NP	NP	NP	NP	S/S
TerraTex HPG-50	W	PP	NA	NA	0.595 (30)	1.5/4685 (115), CH	0.65 (145)	4995 (725)	0.65x0.56 (145x125)	1.78x1.49 (400x335) /20x15	10	11	11	11	15	15	8 (552)	S/S
TerraTex HPG-55	W	PP	NA	NA	0.42 940)	0.96/2852 (70), CH	0.73 (165)	4789 (695)	0.65x0.56 (145x125)	1.89x1.56 (425x350) /21x21	11	12	12	12	18	18	9 (648)	S/S
TerraTex HPG-57	W	PP	NA	NA	0.595 (30)	0.4/1222 (30), CH	0.71 (160)	8628 (1200)	0.80x0.80 (180x180)	1.85x1.82 (415x410) /12x8	35	35	35	35	400)	21 (1440)	S/S	
TerraTex N03	NW	PP	NA	NA	0.21 (70)	2.1/6110 (150), CH	0.22 (50)	1171 (170)	0.11 (25)	0.36 (80)/50	NA	NA	NA	NA	NA	NA	NA	S/S
TerraTex N04	NW	PP	NA	NA	0.21 (70)	2.2/6110 (150), CH	0.24 (55)	1379 (200)	0.18 (40)	0.42 (95)/50	NA	NA	NA	NA	NA	NA	NA	S/S
TerraTex OL	NW	PP	NA	NA	0.21 (70)	NA	0.27 (60)	1378 (200)	0.16 (35)	0.40 (90)/50	NA	NA	NA	NA	NA	NA	NA	S/S
TerraTex SD	NW	PP	NA	NA	0.21 (70)	2.0/5700 (140), CH	0.29 (65)	1480 (215)	0.2 (45)	0.47 (105)/50	NA	NA	NA	NA	NA	NA	NA	S/S

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 (5) Test per ASTM D 5262, for a minimum of 10,000 hours and extrapolated to a 75 year time period.
 (6) S/S = Separation/stabilization
 NA = Not applicable, per manufacturer
 NP = Not provided
 Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

GEOTEXTILES

Product Name	Physical Properties			Filtration/Hydraulic Properties			Mechanical Properties					Creep Limited Strength ASTM D 5262 T ₁ [5] kN/m (lb/ft)	Manufact. Suggested Applic. [6]		
	Structure [1]	Polymer Type [2]	Mass Per Unit Area ASTM D 5261 g/m ² (oz/yd ²)	Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec ⁻¹ /Flow Rate ¹ FH or CH [3] l/min/m ² (gal/min/ft ²)	Puncture ASTM D 4833 kN (lb)	Mullen Burst ASTM D 3786 kPa (psi)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile /Elongation ASTM D 4632 kN (lb)%	Wide Width Tensile Properties [4]				
											Strength @ 5% Strain			MD	XD
TerraTex S04	NW	PP	NA	NA	0.21 (70)	0.70/1629 (40), FH	0.18 (40)	965 (140)	0.26 (60)	0.58 (130)/60	NA	NA	NA	NA	S/S
TerraTex N05	NW	PP	NA	NA	0.21 (70)	1.6/4481 (110), FH	0.36 (80)	1860 (270)	0.24 (55)	0.60 (135)/50	NA	NA	NA	NA	S/S
TerraTex N06	NW	PP	NA	NA	0.18 (80)	1.5/4074 (100), FH	0.40 (95)	2239 (325)	0.29 (65)	0.73 (165)/50	NA	NA	NA	NA	S/S
TerraTex N07	NW	PP	NA	NA	0.18 (80)	1.3/3666 (90), FH	0.44 (110)	2618 (380)	0.33 (75)	0.89 (85)	NA	NA	NA	NA	S/S
TerraTex N08	NW	PP	NA	NA	0.18 (80)	1.2/3259 (80), FH	0.51 (130)	2756 (400)	0.39 (85)	0.96 (215)/50	NA	NA	NA	NA	S/S
TerraTex N10	NW	PP	NA	NA	0.15 (100)	1.1/2852 (70), FH	0.67 (160)	3617 (525)	0.44 (100)	1.27 (285)/50	NA	NA	NA	NA	S/S
TerraTex N12	NW	PP	NA	NA	0.15 (100)	0.8/2444 (60), FH	0.76 (180)	4306 (625)	0.51 (115)	1.45 (325)/50	NA	NA	NA	NA	S/S
TerraTex N16	NW	PP	NA	NA	0.12 (120)	0.6/1833 (45), FH	1.1 (250)	5512 (800)	0.73 (165)	1.89 (425)/50	NA	NA	NA	NA	S/S

Information was received on strip drains after contacting manufacturer.

Contact: Keith Harris
 WEBTEC Inc.
 P.O. Box 240302
 Charlotte, NC 28224
 fax: 704.394.7946

TerraTex GS is woven from isotactic polypropylene filaments. It is nonbiodegradable, resistant to most soil chemicals and unaffected by acids and alkalines within a pH range of 3 to 12. TerraTex GS has been treated to withstand prolonged exposure to ultraviolet degradation.

TerraTex HD is a high strength fabric woven from isotactic polypropylene filaments. It is nonbiodegradable, resistant to most soil chemicals and unaffected by acids and alkalines within a pH range of 3 to 12. TerraTex HD has been treated to withstand prolonged exposure to ultraviolet degradation.

TerraTex Nonwoven Construction Fabrics (TerraTex N08, N10, N12, N16)
 Heavyweight fabrics for multiple applications: embankment protection, erosion control, ground stabilization, geomembrane underliner, railroad bed stabilization, heavy duty drainage protection, separation.

Manufacturer's Product Equivalents

		<u>Nonwoven Fabrics</u>					
<u>Carthage Mills</u>	<u>Mirafi</u>	<u>Webtec</u>	<u>Nicolon</u>	<u>Amoco</u>	<u>Synthetic</u>	<u>LINQ</u>	<u>Trevira (Hoechst)</u>
FX-30HS	N/A	NO3	N/A	4535	311	3301	N/A
FX-35HS	140NL	NO4	N/A	4545	351	3341/125EX	N/A
FX-38OL	Mirapave	OL	N/A	Amopave	381	N/A	011/140
FX-40HS	140N	SD	S400	4547	401	3401/130EX	011/140
FX-60HS	160N	NO6	S600	4551	601	3601/150EX	011/200
FX-70HS	170N	NO7	S700	4553	801	3631/180EX	011/250
FX-80HS	180N	NO8	S800	4553	801	225EX	011/280
FX-100HS	1100N	NO10	S1000	4555	1001	250EX	011/350
FX-120HS	1120N	NO12	S1200	4557	1201	275EX	011/420
FX-160HS	1160N	NO16	S1600	4561	1601	350EX	011/550

		<u>Woven Slitfilm Fabrics</u>					
<u>Carthage Mills</u>	<u>Mirafi</u>	<u>Webtec</u>	<u>Nicolon</u>	<u>Amoco</u>	<u>Synthetic</u>	<u>LINQ</u>	<u>Trevira (Hoechst)</u>
FX-33	N/A	N/A	N/A	N/A	22Tex	N/A	N/A
FX-44	500XL	GS-150	N/A	2000	82Tex	GTF200F	N/A
FX-55	500X	GS	T200	2002	200ST	GTF200	N/A
FX-66	600X	HD	T300	2006	300ST	GTF300	N/A

		<u>Woven Monofilament Fabrics</u>					
<u>Carthage Mills</u>	<u>Mirafi</u>	<u>Webtec</u>	<u>Nicolon</u>	<u>Amoco</u>	<u>Synthetic</u>	<u>LINQ</u>	<u>Trevira (Hoechst)</u>
Carthage 6%	700X	EP	70/06	1199	Erosion I	GTF400E	N/A
Carthage 10%	40/10	N/A	40/10	N/A	Erosion V	N/A	N/A
Carthage 15%	700XG	N/A	G400	1198	Erosion X	N/A	N/A
Carthage 30%	40/30A	N/A	40/30A	N/A	N/A	N/A	N/A

*This information is to be used as a guideline only.

