TENSILE REINFORCEMENT OF ROAD EMBANKMENTS ON POLYGONAL GROUND
BY GEOTEXTILES OR RELATED MATERIALS

INTERIM REPORT

by

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ABSTRACT

This study is a continuation of a previous study. Together they include the development of the theory needed to design a road supported over a void by a geotextile or related material, and the performance of a series of full-scale field tests to verify the theory. The previous study included the basic development of the theory and the performance of 12 test sections to verify the theory. The field tests in that study were not conclusive. The current study includes some refinement of the theory and the performance of additional testing. The tests in this study were conclusive. Design guidelines are presented for use on a trial basis on a production project.

The theory has been presented in detail in several other publications and only the computer program is presented herein. The theory indicates that it would be possible to span voids of 10 feet or more with currently available materials.

The field test results from the previous study are presented in detail in the final report for that study. The field tests from this study are presented in an abbreviated form in accordance with the level of funding. The results of the tests prove that it is practical to span a six-foot-wide void with a geotextile or related material, and indicates that the design procedure is adequately conservative in those situations where the geology of the subgrade is such that the design technique is applicable and the design parameters can be reliably determined.

Additional field testing will be needed to establish methods to develop design parameters from field data under various conditions and to assess the practicality of the usage.
INTRODUCTION

A research project was recently completed (Kinney, 1985) to study the possibility of using a geotextile or related material to fully support an unpaved road over a void. That project included developing a theory to design the geotextile and performing a full-scale load test to verify the theory. The theory indicated that geotextiles and related materials could be used to support roads across voids on the order of ten feet or more in width. The full-scale test substantiated the theory. However, due to an unfortunate choice of materials, none of the twelve test sections succeeded in carrying the road fill and the loaded truck across the voids.

Since none of the test sections was completely successful, it was not prudent to develop design criteria. This study includes a reproduction of a portion of the previous field test using different materials and different techniques to verify that geotextiles or related materials can support roads over voids, thus allowing design criteria to be developed.

This report is primarily to present design criteria based on the theory previously developed and the test data from both tests. The test data from this study are presented only in the detail necessary to substantiate the conclusions that were not substantiated in the previous test. A complete description of the current test facility is beyond the scope of the presently funded project.

ACKNOWLEDGMENTS

There were five financial sponsors of the project: the Alaska Department of Transportation and Public Facilities, Research Section (DOT&PF) using Federal Highway Administration funds; Shannon and Wilson, Inc.; Signode Corporation; EXXON Corporation; and Mirafi Corporation on behalf of ICI Limited of England. These organizations supplied materials, funding and personnel at the time of the test, and have been instrumental in continuing data collection and analysis.
Tensar Corporation supplied materials and assistance during the test. In particular, I appreciate the efforts of: Joe Fluet from Geosciences in Boca Raton, Florida; Art Slaters and Don Graffan from Signode in Glenview, Illinois; Billy Connor and Bob McHattie from ADOT&PF in Fairbanks, Alaska; John Boone of EXXON in Summerville, South Carolina; and Tom Stephens of Mirafi in Houston Texas. David Baun of Tensar in Pleasant Hill, California, also helped with the test.

THEORY

The theory developed was described in detail in Kinney and Abbott (1984), and Kinney (1985). The theory was developed into a computer program which was presented in Kinney (1986). The flow chart for the program is shown in Figure 1, and the program is presented in Table 1. A brief description of the assumptions made in the theory is presented herein for completeness.

It is assumed that the full weight of the road fill and traffic is carried one dimensionally across the trench by the geotextile or related material in hoop tension. The reinforcing material derives its tension from friction with the soils outside the void. The reinforcing material is assumed to stretch into a segment of a circular arc; the extra length comes from stretch of the material throughout the void and back to the point of zero tension in the reinforcing material, plus any slippage that may occur at the ends of the material. The program presented herein is based on the assumption that the tension in the reinforcing material is constant across the void and varies linearly throughout the embedded length, and that the modulus of elasticity of the reinforcing material is constant, although neither of these restrictions is necessary.

FIELD TESTS

The test sections were constructed by digging trenches through a sandy silt subgrade, stretching the reinforcing materials across the trenches, and covering the reinforcing material with three or four feet
of loose sand and gravel, starting at the ends and working toward the center. A layer of Fibertex 150 (a lightweight nonwoven material) was placed over the geogrid which in turn spanned the void. Photograph 1 shows the test sections under construction.

**Figure 1 - Design Flow Chart**
1 REM TABLE #1 - BASIC PROGRAM TO SOLVE MATHEMATICAL MODEL
2 REM
3 REM
10 INPUT "WIDTH OF VOID (FT.)": W
20 INPUT "EMBEDMENT LENGTH (FT.)": EL
30 INPUT "APPLIED PRESSURE (PSF.)": P
40 INPUT "SHEAR STRESS ON GEOTEXTILE (PSF.)": S
50 INPUT "EFFECTIVE SECANT MODULUS OF GEOTEXTILE (LBS/FT.)": E
60 INPUT "ULTIMATE TENSILE STRENGTH OF GEOTEXTILE (LBS/FT.)": TMAX
70 PRINT
80 REM START WITH R = W/2
90 R = W/2
100 T = P*R
110 IF T < TMAX THEN GOTO 140
120 PRINT "MAXIMUM TENSION EXCEEDED WITH R = W/2  T = " : T : "LB/FT" 
130 GOTO 510
140 L = T/S
150 IF L <= EL THEN GOTO 180
160 PRINT "INSUFFICIENT EMBEDMENT WITH R = W/2  REQUIRED L = " : L : "FT" 
170 GOTO 510
180 A = T+W/E + T*L/E
190 IF A < 3.1416*R - W THEN GOTO 240
200 PRINT "VERTICAL DISPLACEMENT > W/2 AND ENDS DO NOT SLIP" 
210 D = A/2 + 0.2146*W
220 DX = 0
230 GOTO 420
240 REM EQUILIBRIUM R > W/2
250 R = R + .01
260 T = P*R
270 IF T <= TMAX GOTO 300
280 PRINT "MAXIMUM TENSION EXCEEDED WITH R > W/2" 
290 GOTO 510
300 L = T/S 
310 A = T+W/E + T*L/E
320 TH = 2*ATN(1/((2*R/W)^2 - 1))
330 IF L >= EL THEN GOTO 370
340 IF A < TH*R - W THEN GOTO 250
350 DX = 0
360 GOTO 400
370 PRINT "R > W/2 AND ENDS SLIP" 
371 L = EL
372 T = L*S
373 R = T/P
374 TH = 2*ATN(1/((2*R/W)^2 - 1))
375 A = T+W/E + T*L/E
380 DX = (R*TH - W - A)/2
390 GOTO 410
400 PRINT "R > W/2 AND ENDS DO NOT SLIP" 
410 D = R - R*COS(TH/2)
420 REM PRINT OUTPUT
430 PRINT "MAXIMUM TENSION (LBS/FT.) = " : T 
440 PRINT "FACTOR OF SAFETY WITH RESPECT TO TENSION = " : TMAX/T 
450 PRINT "MAXIMUM STRAIN (R) = " : 100*T/E 
460 PRINT "CENTERLINE DISPLACEMENT (IN.) = " : D*12 
470 PRINT "RADIUS (FT.) = " : R 
480 PRINT "FACTOR OF SAFETY WITH RESPECT TO SLIPAGE = " : EL/(T/S) 
490 PRINT "END SLIPAGE (IN.) = " : DX*12 
500 PRINT "REQUIRED EMBEDMENT (FT.) = " : L
510 END
Photograph 1. Test sections under construction.

The fill was spread with a Caterpillar 930 front end loader weighing a measured 23,960 pounds empty and 33,860 pounds loaded. An estimated 28,000 pounds was on the front axle when the loader crossed each test section during construction.

The trenches were three- and six-feet wide, and four-feet deep. The sides were braced with 3x12 timber lagging to keep them from caving. The top of the lagging was placed about six inches below the original plane of the reinforcing material. The trenches were open at the ends so that measurements could be made from inside the trench throughout the test.

The reinforcing materials were six-feet wide and 30- to 50-feet long with the exception of one which was anchored at a distance of six feet from the edge of the void. All but one of the 12 different reinforcing materials carried the full load as expected. The materials in Table 2 represent the range of material properties that were used and thus provide a test of the procedure.

The measured and calculated maximum centerline displacements of the reinforcing material just after construction are shown in Table 3. Photographs 2, 3 and 4 show the deformation shape of the EXXON GTF-800, the ParaGrid 50S and the Signode Grid, respectively, just after construction.

Following the construction of the test section, they were trafficked with 200 passes of a dump truck with dual wheels on tandem
TABLE 2. Reinforcing materials.

<table>
<thead>
<tr>
<th>Material</th>
<th>Tensile strength lbs/ft</th>
<th>Failure strain %</th>
<th>Effective modulus lbs/ft</th>
<th>Trench width ft</th>
<th>Material length ft</th>
<th>Fill depth ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>EXXON GTF-800</td>
<td>6,600</td>
<td>20.0</td>
<td>29,500</td>
<td>4.0±(1)</td>
<td>30</td>
<td>4.0</td>
</tr>
<tr>
<td>ParaGrid 50S</td>
<td>3,420</td>
<td>12.0</td>
<td>28,000</td>
<td>6.0</td>
<td>40</td>
<td>3.0</td>
</tr>
<tr>
<td>Signode Grid</td>
<td>7,000</td>
<td>7.5</td>
<td>110,000</td>
<td>6.0</td>
<td>50</td>
<td>3.0</td>
</tr>
<tr>
<td>Signode Grid</td>
<td>7,000</td>
<td>7.5</td>
<td>110,000</td>
<td>6.0</td>
<td>18(2)</td>
<td>3.0</td>
</tr>
</tbody>
</table>

(1) Edges of 3' trench sloughed to form a 4.0± foot trench.

(2) Ends anchored by burying them in a trench 12-inches wide and 12-inches deep in the subgrade.

Photograph 2. EXXON GTF-800 after construction.
Photograph 3. ParaGrid 50S after construction.

Photograph 4. Signode Grid after construction.
TABLE 3. Maximum centerline displacement of reinforcing material.

<table>
<thead>
<tr>
<th>Material</th>
<th>Effective load (psf)</th>
<th>Shear stress (psf)</th>
<th>Displacement (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fill</td>
<td>Loader</td>
<td></td>
</tr>
<tr>
<td>EXXON GTF-800</td>
<td>570</td>
<td>280</td>
<td>200</td>
</tr>
<tr>
<td>ParaGrid 50S</td>
<td>500</td>
<td>290</td>
<td>230</td>
</tr>
<tr>
<td>Signode Grid</td>
<td>450</td>
<td>290</td>
<td>230</td>
</tr>
<tr>
<td>Signode Grid</td>
<td>470</td>
<td>290</td>
<td>230</td>
</tr>
</tbody>
</table>

axles loaded to the full legal limit of 32,000 pounds on the rear axles as shown in Photograph 5. Each test section was loaded with wheels on one side of the truck during trafficking.

Photograph 5. Trafficking the Signode test section.

During trafficking there was no visible deformation of the reinforcing materials upon each pass (including the first one) or any visual creep over the 200 passes. The displacement between passes was measured photographically at several times during trafficking using the
relative location of the washers and styrofoam balls shown in Photographs 2, 3 and 4. There was no movement within the 0.5 inch accuracy of the measurements.

After trafficking, the displacement under traffic loading was measured on some of the sections using a falling weight deflectometer (Photograph 6), and on all of the sections using a rocker arm (Photograph 7).

Photograph 6. Dynatest Falling Weight Deflectometer.

The Dynatest Falling Weight Deflectometer data were interpreted at a 9,650 pound peak load on a 30 cm diameter plate. The eight-foot-long rocker arm was placed between the dual wheels of a single axle dump truck loaded to 19,300 pounds on the rear axle. Rebound measurements were made as the truck was removed. Measurements were made with a dial gage from within the trench on the bottom of the reinforcing material, along with the rocker arm measurements as shown in Photograph 8. The results of the post traffic loading tests are shown in Table 4.
Photograph 7. Rocker arm.

Photograph 8. Dial gage measuring rebound deflection on reinforcing material.
TABLE 4. Post trafficking test results.

<table>
<thead>
<tr>
<th>Material</th>
<th>Rocker arm deflections (in)</th>
<th>Falling weight deflection (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Surface</td>
<td>Reinforcement</td>
</tr>
<tr>
<td>EXXON GTF-800</td>
<td>0.049</td>
<td>0.020</td>
</tr>
<tr>
<td>ParaGrid 50S</td>
<td>0.042</td>
<td>0.034</td>
</tr>
<tr>
<td>Signode Grid</td>
<td>0.080</td>
<td>0.065</td>
</tr>
<tr>
<td>Signode Grid</td>
<td>0.076</td>
<td>0.052</td>
</tr>
<tr>
<td>Control Section</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(not over void)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

DESIGN METHODOLOGY

The theory and the field test results indicate that it is possible and practical to use geotextiles and/or related materials under roads constructed across polygonal ground in discontinuous permafrost zones to limit the catastrophic settlement that occurs when the ice melts. The use of these materials should decrease the abruptness of settlement of roads constructed over ice-rich permafrost and massive ice formations. However, the results are not expected to be as dramatic, and these applications are not considered directly herein.

Design Geometry

Ice wedges vary greatly in size, shape, orientation and depth below the ground surface. It must be assumed that they will have a random orientation in any situation, so a detailed geotechnical investigation does not appear to be warranted. It appears reasonable to pick some design void width and spacing, use an appropriate factor of safety, and accept some finite probability that there will be some failures. Photograph 9 shows a typical ice wedge. A design width of ten feet and a factor of safety of two appears reasonable unless there is some local reason to use something else.
Photograph 9. Typical ice wedge.

MATERIAL PROPERTIES

Soils

Ice wedges generally grow in soils that are primarily silts. In general, the surrounding soil has a high moisture content and may contain ice lenses and massive ice features as shown in Photograph 9. Thawing of the surrounding ground may cause uneven settlement with large deflection basins that the reinforcing material will do little to even out.

The thawed soil will probably be soft, and the edges of the voids may tend to collapse. Edge collapse would create a wider void than would otherwise be expected, however, the wide void would require more side support causing more collapse. It seems likely that the thawed permafrost will redistribute itself until equilibrium is reached between the collapsing soil and the reinforcing material. The road surface would probably settle more if the edges collapse than if the sides do not collapse but less than if the reinforcing material had not been there, assuming the reinforcing material does not fail in tension. If the reinforcing material fails in tension, the road performance should be similar to that experienced if the reinforcing had not been there.
There will probably be a thick organic mat over the area. It appears prudent to leave the organic mat in place in most areas where voids are expected to occur. The beneficial effects (including construction expedience, insulation, separation, reinforcement and load distribution) should outweigh the detrimental effects of the low modulus material and the possibility of long-term settlement.

Geotextiles and/or Related Materials

Some of the ice wedges will run at odd angles to the axis of the roads, and others will be near the toe of the embankment and perhaps parallel to the edge of the road. It will, therefore, be desirable to use a reinforcing material that is not highly directional in its tension-strain characteristics. In addition, the material should be relatively insensitive to creep over a period of several years and insensitive to cold down to about -10°F.

The wide width tensile test being developed by the American Society of Testing Materials (ASTM) appears to be the best method for determining the short-term tension-strain characteristic of a geotextile in the high modulus directions. Similar tests can be used on geogrids and other such materials, but it is not as important to keep the wide width tensile test dimensions. It is necessary to test the joints in such materials since they are usually the weak spots in the material and change the tension-strain characteristics of the material. The joints may also behave differently to time and the environment than the webbing material. Currently there is no generally accepted test method for considering long-term effects, the effects of soil impregnation and confinement, or the effects of temperature on the mechanical properties of either geotextiles or related materials.

Creep and temperature are known to effect some materials more than others. It seems prudent to avoid those materials that are sensitive to creep and temperature unless detailed testing is done and it is established that the materials will satisfy the design criteria. The wide width type of tensile test can be used to study creep and temperature effects, but the details of the loading and environment
must approach those in the field, and not those in the test procedure as proposed at this time.

There is some evidence that confinement and soil impregnation greatly increase the tension-strain properties of some geotextiles and related products, particularly on the bias. ASTM is currently considering tests that consider these effects. But until such time as additional information becomes available, it seems reasonable and conservative to use the wide width tensile test on the warp, woof and bias.

Interaction Between Soils and Geotextiles or Related Materials

The interaction between soils and foreign materials has been the subject of many studies. There is no widely accepted test method for determining the stress displacement characteristics of the interface between geotextiles and related material and soils. The studies that have been made on the contact between various mineral soils and various reinforcing materials (using a variety of equipment and methods) indicate the following trends.

- If the holes in the reinforcing material are about the same size or larger than the soil grains, then the failure will be through the soil, and the shear strength will be that of the soil or slightly higher.

- If the holes in the reinforcing material are significantly smaller than the soil grains, then the failure may be at the interface, and the shear strength may be significantly lower than the strength of the soil alone. The controlling factors are the shape of the reinforcing material, the roughness of the surface, the type of material and the soil properties. Strengths as low as one half of the soil strength are common.

- If the soil under the reinforcement is soft, the reinforcing material is drappable, and the soil over the reinforcement is large grained, the plane of the reinforcing material may be
deformed to the point where the failure will be forced through the soil. The shear strength will be that of the soil.

- A movement of the reinforcing material of 0.1 to 0.2 inches appears to be enough to develop nearly the full shear strength between most materials.

To the author's knowledge, there is no information in the literature on the shearing behavior between geotextiles or related materials and organic mats. Until such information is obtained, it appears reasonable to consider the relationship frictional with a coefficient of friction of about 0.35.

It is not clear whether the shear stress acts on the top and bottom of the reinforcing materials or just on the bottom. If the material on top of the reinforcing material moves with the reinforcing material as it moves, there will not be relative motion between the two. Therefore, no shear stress will develop. This will happen unless there is resistance built up in the void area to inhibit the motion of the fill over the reinforcing material. Resistance will be built up if there is a relatively small deformation into the void relative to the amount of movement of the reinforcing material outside the void. This effect has not been quantified. Until it has, it appears reasonable to take the conservative approach in design and assume that there is shear stress only on the bottom of the reinforcing material.

**DESIGN TECHNIQUE**

The field data indicate that the theory can be used for design. The major uncertainties lie in developing the design criteria from field exploration data. At this stage in the development of the design techniques, it seems reasonable to gather as much field information as practical, to develop the design criteria from this information, to use the available theory to choose appropriate reinforcing materials, and then to monitor the performance following construction. These steps will be addressed separately.
Field Exploration

The field exploration should establish the extent and character of the ice formations, the magnitude of anticipated thaw settlement not associated with ice formations of limited extent, the competency of the thawed permafrost and the character of the active zone material.

Aerial photographs should be analyzed to determine the extent of ice wedge formation and the extent of other thermokarst features. A surface reconnaissance should be performed looking in detail for surface expressions of the subsurface conditions. It may be appropriate to do some geophysical testing in search of the size and shape of massive ice formations.

Borings should be made in areas where massive ice is expected and in areas between the massive ice formations where massive ice is not expected. Undisturbed samples should be taken of the material without massive ice and of any mineral soil in the active layer for laboratory testing. Laboratory testing on the frozen soil should consist of at least water content tests, in situ density tests, thaw strain tests and thawed strength tests. Laboratory tests on the active layer should consist of at least water content tests and a measure of the thawed strength which is appropriate for the material encountered.

Establishing Design Criteria

Based on the field and laboratory tests and on experience, the character of the subsurface materials should be established. The required parameters are:

- Extent of thaw settlement excluding that caused by ice masses of limited lateral extent.
- Width of voids to be spanned.
- Minimum distance between anticipated voids.
- Stability of the edges of the void.

The design methodology presented herein is most applicable when there are widely spaced ice masses of limited lateral extent, a minimum
amount of other thaw settlement, and a relatively firm material
remaining after the permafrost thaws. If these conditions do not exist,
it does not mean that the use of geotextiles will not improve the
performance of the road. But it does mean that the results will not be
as dramatic, and the design procedure presented herein should not be
used without modification.

The estimation of the void width must take into account the
distribution of the void as it passes through the active zone. There is
virtually no factual information to guide the engineer in making this
projection. Thermokarstts are typically smaller at the surface when they
first form. This may reverse itself as weathering and other forces cave
in the sides. The pressure from the reinforcing material on the edges
of the void will probably preclude any overhang but may or may not cause
additional caving of the sides.

Any real situation will be less than perfect, and it will be
necessary to make some reasonable assumptions. These assumptions will
undoubtedly be unconservative at some locations, so some reasonable
probability of failure must be accepted. Assuming that conditions
permit using this technique, it will probably be adequate to use a
design width of 10 feet and a design spacing of 20 feet in most areas.

Choosing the Proper Reinforcing Material and Embankment Cross Section

The embankment must be thick enough to distribute the traffic
loading sufficiently on the subgrade material, and to provide any
insulation or frost heave protection that is included in the design.
The reinforcing material must support the weight of the embankment plus
the traffic loading. The traffic loading decreases but the embankment
load increases as the embankment height increases. Therefore, there is
some optimum embankment thickness that is ultimately determined by
economics.

The design procedure contained herein can be used directly to
determine the optimum conditions. Parametric runs of the computer
program have been included in Figure 2 to give the designer a place to
Centerline Depression
0.2*120*W

Traffic Loading
Fill
3*120

Pressure Over Void (P) - psf
800 400

Effective Modulus of Reinforcement 50 100

Width of Void (W) - ft
8

Tension (T) - k/ft
4

Notes:
* Height of fill (H) = 3 ft.
* Unit weight of fill (D) = 120 pcf
* Shear stress on reinforcement (S) = 250 psf

Example:
Given:
W = 8 ft
E = 50 k/ft

Calculated:
P = 795 psf
T = 4.40 k/ft
L = 17.6 ft
d = 20.5 in

Required Anchor Length (L) - ft
10 20 30

Centerline Displacement (d) - in
-40

Effective Modulus of Reinforcement

FIGURE #2 - PARAMETRIC DESIGN CALCULATIONS
start in the analysis. The following assumptions were made in the parametric runs:

- Embankment fill height = 3 feet at 120 pcf.
- Shear stress on reinforcing material = 250 psf.
- Traffic loading = dual wheels on tandem axles with a total load of 32,000 pounds.
- Ends of reinforcement are buried enough so that they do not slip.
- The reinforcing material is linearly elastic.

If the reinforcing material used has holes that are large enough to let the subgrade material mix with the fill material or to let the fill material fall through, it will be necessary to add a separator over the reinforcing material. Any lightweight geotextile with the appropriate opening size would work.

CONSTRUCTION TECHNIQUES

Site Preparation

The use of reinforcing material considered herein requires that the material be smoothed when the fill is placed and that it be anchored at the ends. Anchoring may be done by artificial means such as burying the end in a trench or by burying enough length of material under the embankment. These conditions require that the grade be relatively smooth when the reinforcing material is placed. This does not mean that there cannot be holes in the subgrade or that the subgrade must be graded. It merely means that it must be possible to lay the reinforcing materials relatively flat in relatively good contact with the subgrade. If the tundra is left in place, it must be walked down with construction equipment prior to construction. All stumps and other protrusions must be cut off as close as practical to the ground surface. If practical, it would be preferable to remove them.
In most applications, a fill depth on the order of 2.5 to 4.0 feet will probably be the optimum, considering the design procedure presented herein and economics. There may be other considerations that require a higher fill, for instance, to avoid flooding or to cross a low spot in the grade. The reinforcing material will be most beneficial if it is placed as low as practical in the profile, but the strength and stiffness criteria increase as the fill height increases above about three feet. The problem is complicated by the shape of the void width as it passes up through the fill material if the reinforcing material is placed higher in the road profile. There is virtually no solid information to guide the engineer, so it appears reasonable to assume that the void width remains constant through the fill. Using this assumption, the design procedure can be used as though the reinforcing material were directly over the void.

It is difficult to get sufficient anchorage along the outside edge of the road by merely burying the end of the reinforcing material under the road fill. Either the embankment must be extended far enough to give anchorage to the main part of the roadway, or some form of anchorage must be used. There is some evidence that an anchor trench might work. The present information indicates that, if an anchor trench is to be used, it should extend at least 18 inches into mineral soil and be at least 18-inches wide. It should be at a point in the embankment where there are at least 18 inches of fill on top of the outside of the trench.

Reinforcing Material Installation

The reinforcing material should be laid flat on the surface. All seams must be tied together in a manner that will take the full design tension. Overlapping seams will not carry the required tension. When practical, the stiff and strong direction should be orientated perpendicular to the direction of the expected void. Reinforcing materials with large opening sizes should be covered with a separation material if necessary. Overlapping the separation material a minimum of six inches at the seams will be acceptable if it is not considered to be part of the reinforcing material.
Embankment Construction

The embankment material must be placed in such a way as to avoid damaging the reinforcing or separating materials. Under most conditions, it would be appropriate to add the first two feet of material in a single lift by end dumping it and spreading it with a small bulldozer from the end that has been previously covered. Following the first lift, the fill should be compacted. All additional material should be placed in no more than nine-inch loose lifts and compacted.

Good compaction would be desirable in most installations, but it may be difficult to attain. It appears most appropriate to use a performance specification such as so many passes with a certain size piece of equipment instead of an acceptance criterion such as a certain minimum density. This would allow the owner to get the best project possible under the conditions with the contract disputes which might arise from the possibility that a given density might not be possible to attain because of the soft subgrade.

It is often desirable to do the construction in the winter when the ground is frozen and access is easier. Fill material cannot be compacted with any degree of success when it is frozen, regardless of the amount of compactive effort that is applied or the water content of the fill material (Kinney and Goetz, 1984). Winter construction is a viable option for at least the lower portions of the embankment as long as the reinforcing material can be worked in the cold weather. The fill should be spread and a minimal amount of compactive effort should be applied in the cold weather. After the fill is thawed in the spring, it should be compacted again. It would be desirable to leave at least the final one foot of fill for spring placement to allow better compaction.

Surfacing

There will be a significant amount of settlement on the surface of the road during thawing regardless of the strength or stiffness of the reinforcing material. Several inches to two or more feet of settlement may occur in a system that is functioning properly. Photograph 10 shows a two-foot-deep depression over a 5.5-foot-wide void in the 1984 test.
that was performing satisfactorily at that point. Depending upon the thermal regime, it will probably take one season for most of the settlement to take place. During the period when rapid thawing is taking place, it will be necessary to regrade the road frequently to keep it passible if that is to be attempted at all.

![Photograph 10. Depression over void in 1984 test prior to filling and releveling.](image)

Once the thaw has progressed to the point where voids have formed below the reinforcing material, the rate of settlement should taper off quickly and a high quality gravel surface could be applied. Once the maintenance records indicate that the settlement rate is negligible, consideration could be given to paving the surface.

The test results were unrealistically severe and still indicated that the void areas were marginally stiff enough for paving. It seems appropriate to test the surface with a falling weight deflectometer and run an analysis to determine the expected life of a pavement. The results may indicate that paving is economically viable.
CONCLUSIONS AND RECOMMENDATIONS

The test results and the theoretical analyses indicate that geotextiles and related products can be used to support roads over voids of over 10 feet across. Voids up to 15-feet wide have been spanned (Fluet et al., 1986). The design methodology presented herein can be used with reasonable reliability in those situations where it is applicable.

The most significant unknowns in the design procedure are in defining the design parameters. It appears reasonable at this point to establish some parameters that appear to be reasonable and to accept the possibility of some failures. Each installation should be monitored to establish an empirical basis for developing future design parameters.

IMPLEMENTATION

The design technique presented herein is ready for trial on a production project. An area of new construction should be identified where the design technique would be applicable. The area should be over polygonal ground in the discontinuous permafrost zone. The road should be one that will experience heavy traffic loads and does not necessarily ever have to be paved. There should not be areas of massive ice with large areal extent. The profile shown in Photograph 11 would not be acceptable.

The design procedure discussed herein should be used on one section of the road and the conventional design should be used on the sections on each end. The reinforced section should be on the order of 500- to 1,000-feet long and the end sections should not be less than 200-feet long. The surface settlement in the test area should be monitored for at least two years, and the performance of the reinforced section should be compared to the performance of the unreinforced sections. The design procedure should be modified if necessary based on the performance of the test sections.

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Photograph 11. Ice wedges with massive ice of large areal extent.

REFERENCES


