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Geotextile Reinforcement of Low-Bearing-Capacity Soils

Comparison of Two Design Methods Applicable to Thawing Soils

Karen S. Henry

June 1999

Abstract: Thawing fine-grained soils are often saturated and have extremely low bearing capacity. Geosynthetics are used to reinforce unsurfaced roads on weak, saturated soils and therefore are good candidates for use in stabilization of thawing soils. To stabilize the soil, a geotextile is placed on it, then the geotextile is covered with aggregate. Design involves selection of aggregate thickness and geotextile. There are two commonly used design techniques for geotextile reinforcement of low-volume roads, and the Army uses one of them. The theory and use of the two design methods for static loading (i.e., up to 100 vehicle passes) are presented and compared in this report. The design method not used by the Army offers the potential to reduce aggregate thickness over the geotextile because it accounts for the fact that the geotextile helps support the traffic load (when in tension) and confines the soil between the wheels and the subgrade. However, this alternative method appears to be unconservative with respect to stresses estimated at the subgrade surface. Thus, the current Army design technique should be used until more research is conducted. In the meantime, straightforward design curves for Army 10- and 20-ton trucks as well as vehicle loading and tire pressure information for a number of other vehicles are included in this report to help make the current design method easy to use.

Future work should consider adopting a hybrid design method that provides realistic estimates of stresses at the subgrade and accounts for the tensile properties of geotextiles. In addition, aggregates other than the high-quality crushed rock that is inherently assumed by each design method should be accounted for in new design development.

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June 1999

Prepared for
OFFICE OF THE CHIEF OF ENGINEERS

PREFACE

This report was written by Dr. Karen Henry, Research Civil Engineer of the Civil Engineering Research Division, Research and Engineering Directorate, U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire.

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The author thanks Professor Mark Evans of West Point Military Academy for technical review of the manuscript. His review questions required responses that added significantly to the breadth and depth of this work. She also thanks George Blaisdell of CRREL for technical review as well as for numerous discussions of this topic and encouragement. Finally, the contributions of Professor Robert D. Holtz of the University of Washington are acknowledged. This work was initiated as an independent study project while the author was a graduate student under his guidance.

CONTENTS

ILLUSTRATIONS

TABLES

Geotextile Reinforcement of Low-Bearing-Capacity Soils
Comparison of Two Design Methods Applicable to Thawing Soils

KAREN S. HENRY

INTRODUCTION

Thawing fine-grained soils are often saturated or even supersaturated and thus have extremely low bearing capacity. Geotextiles have been used in the construction of low-volume, unsurfaced roads on weak and saturated soils to reinforce the base course–subgrade interface and therefore are good candidates for use in stabilization of thawing soils. To stabilize weak soil with a geotextile for trafficking, the geotextile is placed directly on the soil and then covered with aggregate. The design involves selecting aggregate thickness and the geotextile. There are two commonly used techniques for designing for soil reinforcement using geotextiles, one of which is prescribed in U.S. Army guidance.

The current Army design technique for static loading of low-volume roads on low-bearing-capacity soils was examined for ease of use and applicability to the reinforcement of thawing soil, specifically for Army vehicles. Static loading is defined as up to 100 passes of a vehicle at the maximum wheel load and a minimum rut depth of 0.10 m (4 in.). Information about Army vehicle loading and design curves for specific Army vehicles is provided in this report to help make the design technique easier to use.

Another design method that offers the potential to reduce required aggregate thickness over the geotextile (and thus cost) was compared with the Army method. Theory and results from both design methods are presented in this report. Although both design methods include traffic loading for up to 1000 vehicle passes, here we deal only with design for static loading, which involves a maximum of 100 vehicle passes.

METHOD CURRENTLY USED BY THE ARMY (TM 5-818-8)

The design method currently used by the U.S. Army for the stabilization of low-bearing-capacity soils for low-volume roads and trails with geotextile and aggregate was developed by the U.S. Forest Service (Steward et al. 1977) based on theory presented by Barenberg et al. (1975).* The design method is presented in TM5-818-8 (1995) as a series of soil strength vs. aggregate thickness design curves for various wheel loads (defined below), with a tire pressure of 552 kPa (80 psi) (Fig. 1 through 3). The design procedure includes

1) Converting soil strength to an equivalent cohesion, *c*.
2) Selecting a maximum wheel load.
3) Selecting a value for a bearing capacity factor, N_c. N_c values used with design curves for static loading are 6.0 with geotextile and 3.3 without geotextile.
4) Using the product cN_c in the appropriate design chart (e.g., Fig. 1), and determining the depth of aggregate required with and without a geotextile.
5) Determining which section is less costly to build.
6) If use of a geotextile is advantageous, specifying one according to geotextile construction survivability requirements.

Although TM5-818-8 does not specify the aggregate properties required for low-volume roads,

*The low-bearing-capacity soils were assumed to be soft, cohesive soils by Barenberg et al. (1975).

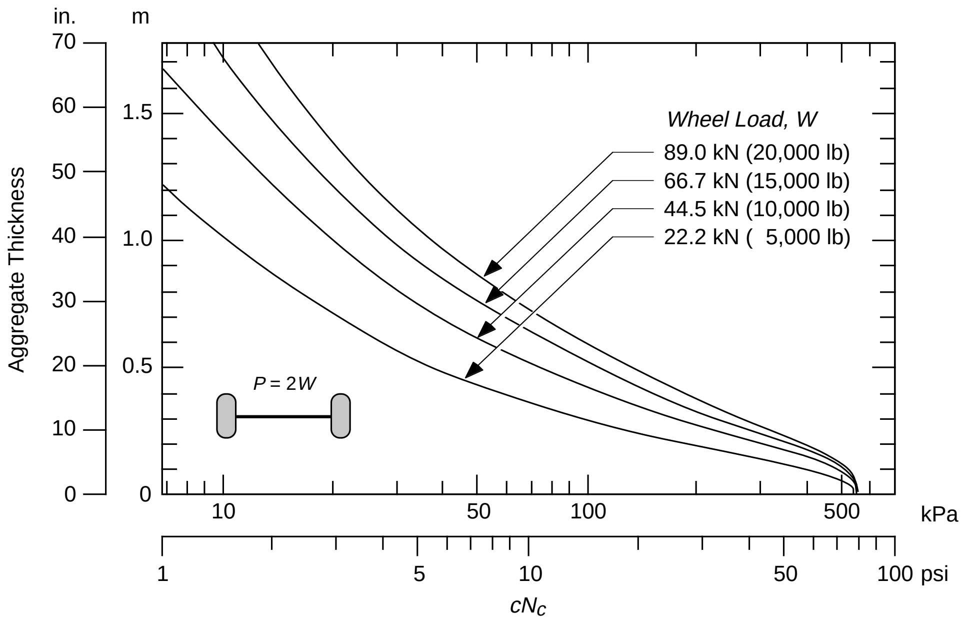

Figure 1. Aggregate thickness design curve for single-wheel load on gravel-surface roads. (From TM 5-818-8.)

aggregate that meets base course requirements is presumably required. Field Manual FM5-430-001 (1994) recommends that base course material have minimum CBR values of 80 to 100. Soils that yield these values include crushed rock, mechanically stabilized aggregates, and well-graded gravel (e.g., FM5-430-001; Holtz and Kovacs 1981). Further evidence for the requirement of high-quality aggregate is the fact that the design technique is based on experiments that used crushed-rock aggregate (Barenberg et al. 1975).

Guidance for selecting wheel loads and contact pressures to use with this design is not given in TM5-818-8; thus, it is now provided. For single and

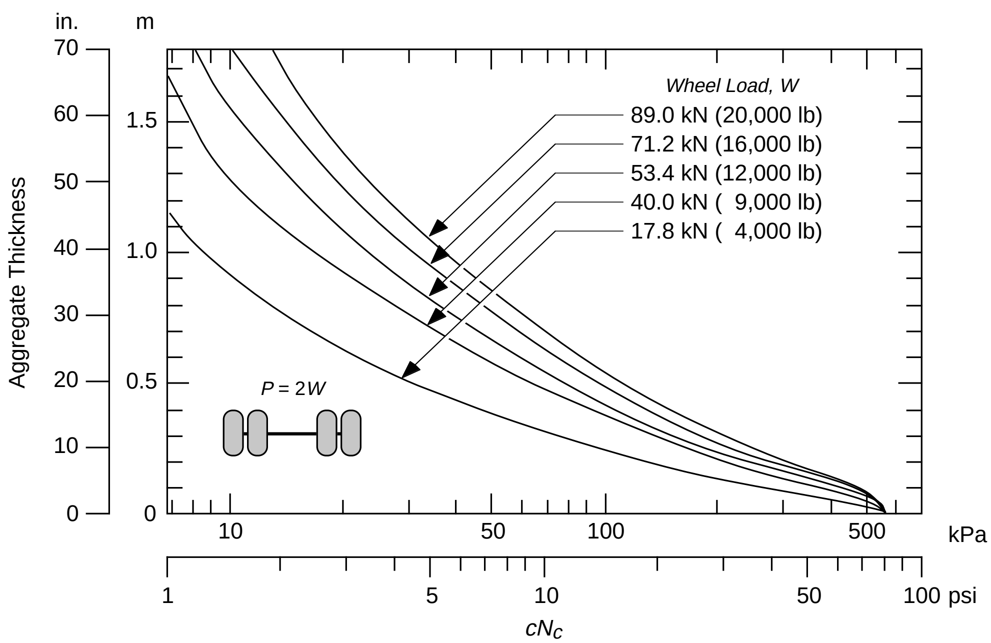

Figure 2. Aggregate thickness design curve for dual-wheel load on gravel-surface roads. (From TM 5-818-8.)

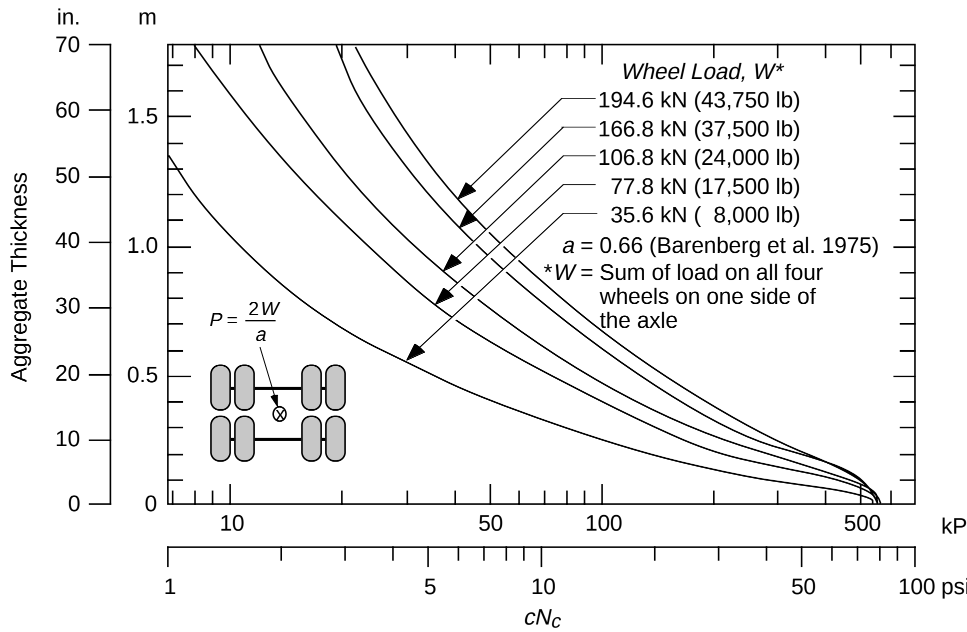

Figure 3. Aggregate thickness design curve for tandem (dual)-wheel load on gravel-surface roads. (From TM 5-818-8.)

dual wheels on a single axle (Fig. 4a and 4b), the wheel load, *W*, is defined as the total load on either the left or right side of the axle. The axle load, *P*, a quantity used in other design methods, is defined as the total load on the axle, or 2*W*. For tandem axles (Fig. 4c and 4d), estimations of wheel and axle loads vary. Barenberg et al. (1975) use a wheel load of 0.66 times the total load on one side of both of the tandem axles. Giroud and Noiray (1981) obtained the design axle load by multiplying the sum of the two axle loads by 0.60. Most U.S. states allow maximum loads on each pair of tandem axles equal to 0.563 times the maximum allowable single axle load (Yoder and Witczak 1975). Contact pressures for use in design are approximately 0.9 to 1.0 times the tire inflation pressure for single-tired vehicles and 0.70 to 0.75 times the tire inflation pressure for dual tires (Barenberg et al. 1975). However, for the design method presented in TM5-818-8, there is negligible difference in the aggregate thickness design curves for actual tire inflation pressure vs. contact pressure (e.g., Fig. 5).

Design example

Given an 80-kN (18,000-lb) maximum expected single axle load (40-kN or 9,000-lb wheel load) on a dual-tired vehicle, determine the aggregate thickness required with and without geotextile for a soil cohesion of 52 kPa (7.5 psi) or CBR of 2.*,† There will be approximately 100 passes of this vehicle, and a rut depth of 0.3 m (12 in.) can be tolerated. The tire inflation / contact pressure is equal to 552 kPa (80 psi).

1. Calculate cN_c as (52)(3.3) = 172 kPa (25 psi) without geotextile, and (52)(6) = 312 kPa (45 psi) with geotextile.
2. Enter Figure 2 with a cN_c value of 172 kPa (25 psi) for a wheel load of 40 kN (9,000 lb) to obtain a value of 0.32 m (13 in.) of aggregate required without geotextile. Using a cN_c value of 312 kPa (45 psi) for the same wheel load, 0.20 m (8 in.) of aggregate is required with geotextile.
3. Determine whether the cost of the geotextile exceeds the cost of 0.12 m (5 in.) aggregate, which would be saved by using the geotextile.
4. If it is advantageous to use a geotextile, specify one using Tables 2-2 through 2-4 in TM5-818-8 (based on the need for the geotextile to survive

*The relationship between shear strength (cohesion) or CBR and Cone Index is given in Figure 2-3 of TM5-818-8 (1995).

†Saturated silts and clays are likely to have CBR values of this order of magnitude. Thawing frost-susceptible soils may have even lower CBR values.

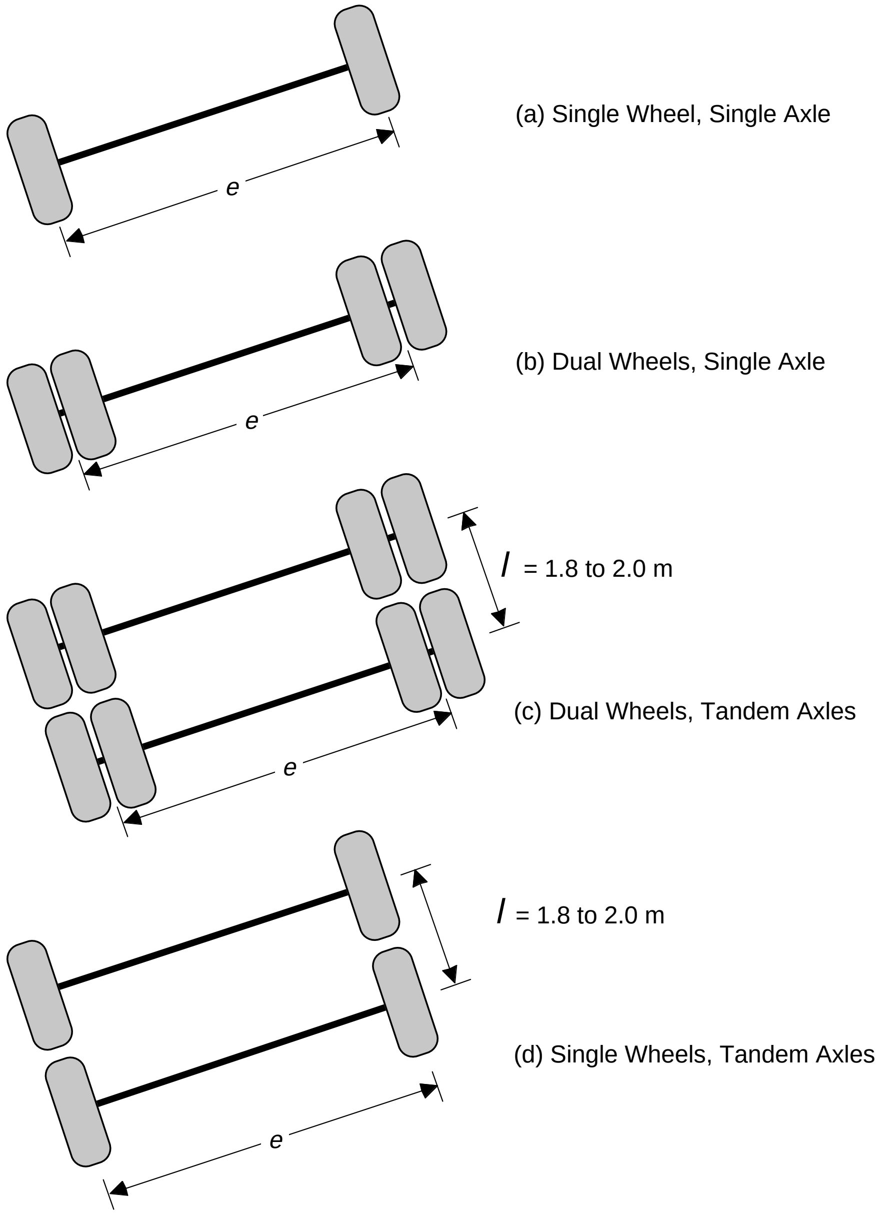

Figure 4. Wheel and axle configurations.

construction). Alternatively, the guidance presented in Appendix A, which was developed more recently than that listed in TM5-818-8 (1995), can be used to select a geotextile.

Theory

Bender and Barenberg (1978) summarize the theory and tests that led to the design method described above. Using the Mohr–Coulomb failure criteria for soils, the shear strength is

$$s = c + \overline{p} \tan \phi \tag{1}$$

where s is the shear strength of the soil, c is the cohesion, $\overline{p}$ is the effective stress, and ϕ is the angle of internal friction. For soft clay subgrades at or near saturation, moving wheel loads are transient, meaning that undrained loading applies. Thus, the angle of internal friction is zero and the undrained shear strength of the soil is equal to its cohesion. Based on the theory of plastic equilibrium, the ultimate bearing capacity, q_d, for soil in this condition is

$$q_d = (2 + \pi)c \,. \tag{2}$$

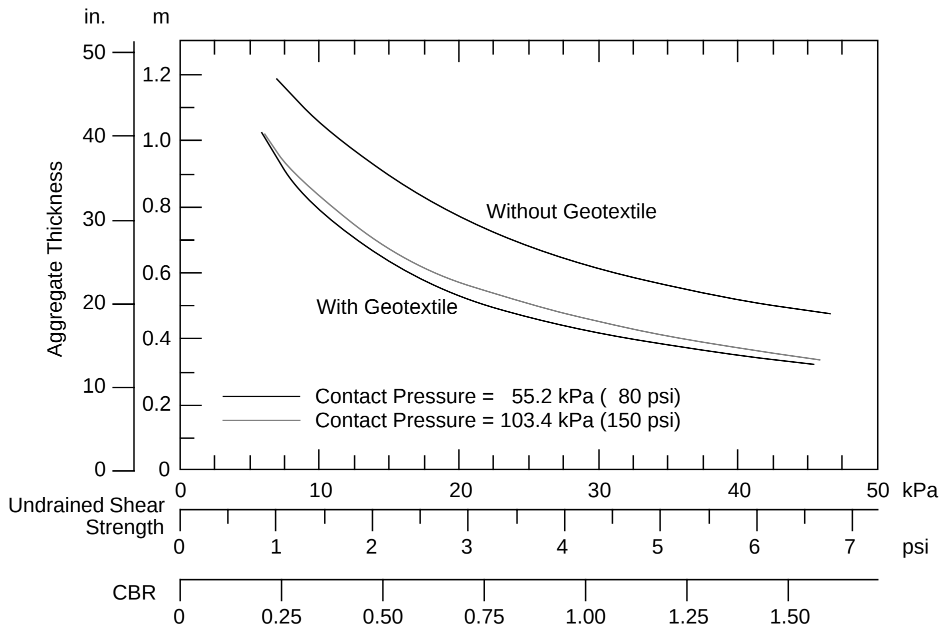

Figure 5. Design aggregate thickness chart for a 88.95-kN (20,000-lb) wheel load. (From Barenberg et al. 1975.)

However, localized plastic strains that can cause localized shear failure begin at

$$q \approx \pi c \ . \tag{3}$$

Barenberg et al. (1975) conducted laboratory tests (two dimensional, cyclic loading) with a geotextile (Mirafi 140) placed between crushed-rock aggregate and a saturated clay subgrade. Stress levels on the subgrade were estimated by using a Boussinesq stress distribution beneath a circularly loaded area (e.g., Newmark 1942), and ratios between the calculated subgrade stress and measured soil strength were developed. The allowable stress with geotextile on the subgrade surface was

$$\sigma_{zallowable} = 6c \ . \tag{4}$$

However, without geotextile, this relationship was

$$\sigma_{zallowable} = 3.3c \ . \tag{5}$$

These numbers are very close to the theoretical values of general and local bearing capacity failure (eq 2 and 3). In addition to the change in failure mode from local to general bearing capacity failure, the soil systems that contained geotextile reached a level of permanent deformation so that further loading of the same magnitude caused negligible additional deformation. The unreinforced systems deformed progressively with repeated loading.

Barenberg et al. (1975) constructed design charts for aggregate thickness vs. soil strength by assuming that the allowable pressure at the subgrade is 3.3c without geotextile and 6c with geotextile (Fig. 5). Stress at the subgrade was calculated by using Boussinesq stress distribution beneath a circularly loaded area (Newmark 1942). The contact area, A, was determined by dividing the wheel load by the contact pressure. The radius, r, needed for determination of the stress at the subgrade surface, was obtained from $A = \pi r^2$.

Barenberg et al. (1975) did not consider tensile modulus or strength (or any mechanical property) of the geotextile in developing their design method. Furthermore, even though Bender and Barenberg (1978) note that “a layer of aggregate material is always needed on top of the fabric to anchor it so that the necessary tensile forces can be developed in the fabric” (p. 66), neither the minimum depth for anchorage nor the mechanical properties of the aggregate layer are specified in either Barenberg et al. (1975) or Bender and Barenberg (1978). The effects of traffic loading when vehicle passes exceed 100 were accounted for by Steward et al. (1977) by reducing the N_c values.

Applicability for Army use on thawing soils

The design curves supplied in TM5-818-8 (Fig. 1 through 3) apply to vehicles with a 552 kPa (80 psi) tire pressure, for single and dual wheel loads varying from 22.24 to 88.96 kN (5,000 to 20,000 lb). Since TM5-818-8 does not list typical wheel loads for Army vehicles (nor does U.S. Army Field Manual FM5-430-00-1), some are provided for vehicles most likely to be used on low-volume roads or trails (Table 1). The maximum wheel load for tandem axles listed in Table 1 was determined by multiplying the total load on the rear axles by 0.60 (e.g., Giroud and Noiray 1981) then dividing by two. For the 20-ton dump truck (three rear axles), the maximum wheel load was estimated by multiplying the total load on the rear axles by 0.70 then dividing it by two.

The maximum wheel loads listed in Table 1 for Army vehicles are reasonably well-represented in Figures 1 through 3. Even though the tire pressures in the figures are higher than the tire pressures listed in Table 1, Barenberg et al. (1975) demonstrated negligible difference in the design curves due to variation in the contact pressures ranging from 552 to 1034 kPa (80 to 150 psi). This theory and design method assume that the subgrade is uniform and that full plastic failure zones can develop, the depth of which depends on the geometry and magnitude of the loading. Thus, this method was not intended for shallow thaw layers, and it would be conservative to use this method for shallow thawed layers. Bounds on the depth of the thawed layer for full development of the plastic zone are discussed in the following section.

ALTERNATIVE METHOD PRESENTED BY GIROUD AND NOIRAY

Giroud and Noiray (1981) developed a design method for geotextile placement between the aggregate and subgrade of unpaved roads based on bearing capacity theory for static loading. The method accounts for the load support and soil confinement provided by the geotextile itself and is presented as a set of curves for dual wheels on a single axle, with an axle load of 80 kN (18,000) at various rut depths (Fig. 6 and 7). The curves are used to determine aggregate thickness without geotextile (h_o') and reduction of aggregate thickness with geotextile (Δh) for geotextiles with different values of tensile modulus, K. The Giroud and Noiray (1981) design method was chosen for comparison with the method now used by the U.S. Army because it is widely used (e.g., Holtz et al. 1995) and because of its potential for cost savings by allowing thinner aggregate layers over the geotextile because it takes into account the tensile properties of the geotextile.

Design example

Given the same vehicle and soil conditions described in the previous design example for the method currently used by the Army, determine the aggregate thickness required with a geotextile of modulus K of 100 kN / m (570 lb / in.) and without a geotextile. Conditions: 80 kN load on a dual-wheel single axle, soil cohesion of 52 kPa (7.5 psi), approximately 100 passes of the vehicle, tire inflation pressure of 552 kPa (80 psi), and a tolerable rut depth of 0.3 m (12 in.). Although Figure 6 is constructed for a tire inflation pressure of 480 kPa (70 psi), little difference in aggregate thickness is

Table 1. Traffic loading data for Army vehicles (Foss 1983).

Vehicle/axle type	*Gross vehicle weight (kN/lb)*	*Load on rear axles (kN/lb)*	*Maximum wheel load (kN/lb)*	*Tire pressure (kPa/psi)*	*Ground clearance (m/in.)*
HMMWV	33.31 / 7,489	21.65* / 4,869	10.83 / 2,434	241 / 35	0.41 / 16
M939 (6 × 6) 5-ton cargo truck	146.80 / 33,000	138.59 / 31,156	41.58 / 9,346	345 / 50†	0.30 / 12
HEMTT, M985	302.48 / 68,000	169.0 / 38,000	50.7 / 11,400	483 / 70†	0.30 / 12
M125 10-ton truck	289.14 / 65,000	187.94* / 4,225	56.4 / 12,680	Not available	0.52 / 20
Articulated 8 × 8	Not available	258.0 / 58,000	77.4 / 17,400	414 / 60	0.30 / 12
M917 20-ton dump truck	324.30 / 72,906	324.30 / 72,906	113.51 / 25,520	414 / 60	Not available

Notes:

*No rear axle load was given; it was assumed that 65% of the gross vehicle load is applied on the rear axles.

†From Jeffrey Stark (personal communication, CRREL, 1997).

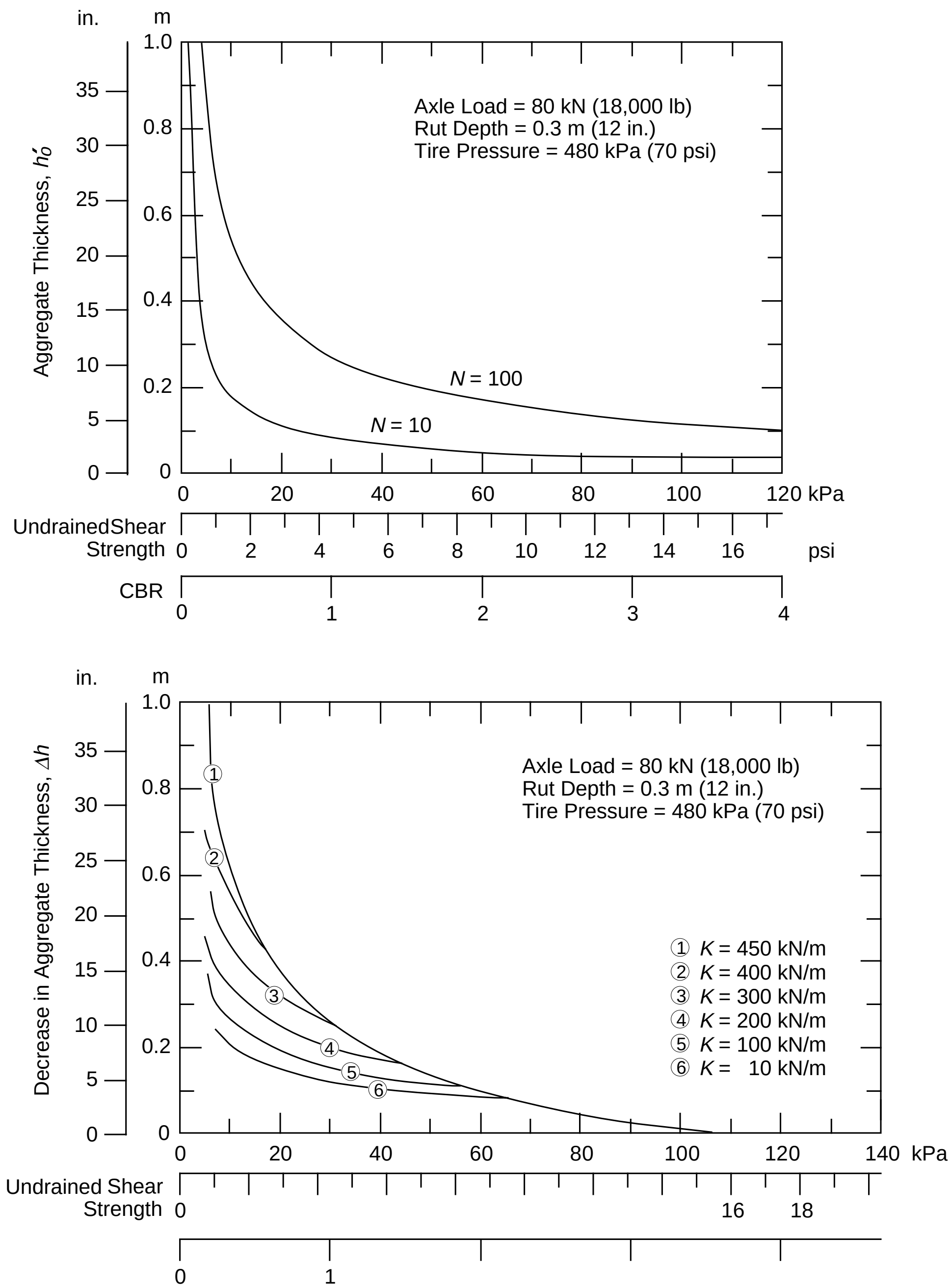

Figure 6. Aggregate thickness, h_o' *(top), and reduction of aggregate thickness,* Δh *(bottom), resulting from use of geotextile as a function of soil cohesion for 12-in. rut depth.* N *is number of vehicle passes and* K *is tensile modulus of geotextile. (From Giroud and Noiray 1981.)*

expected (e.g., Barenberg et al. 1975, Giroud and Noiray 1981).

1. Enter the top chart in Figure 6 with a soil cohesion value, *c*, of 52 kPa, for 100 vehicle passes and determine that h_o' is 0.20 m (8 in.) of aggregate required without a geotextile.

2. Enter the bottom chart in Figure 6 with a soil cohesion value of 52 kPa, for 100 vehicle passes, and for a geotextile with $K = 100$ kN / m and determine that the reduction in aggregate thickness Δh allowed is 0.12 m (5 in.). Thus, the aggregate thickness theoretically allowed when a geotextile is used is 0.06 m (3 in.). However, the

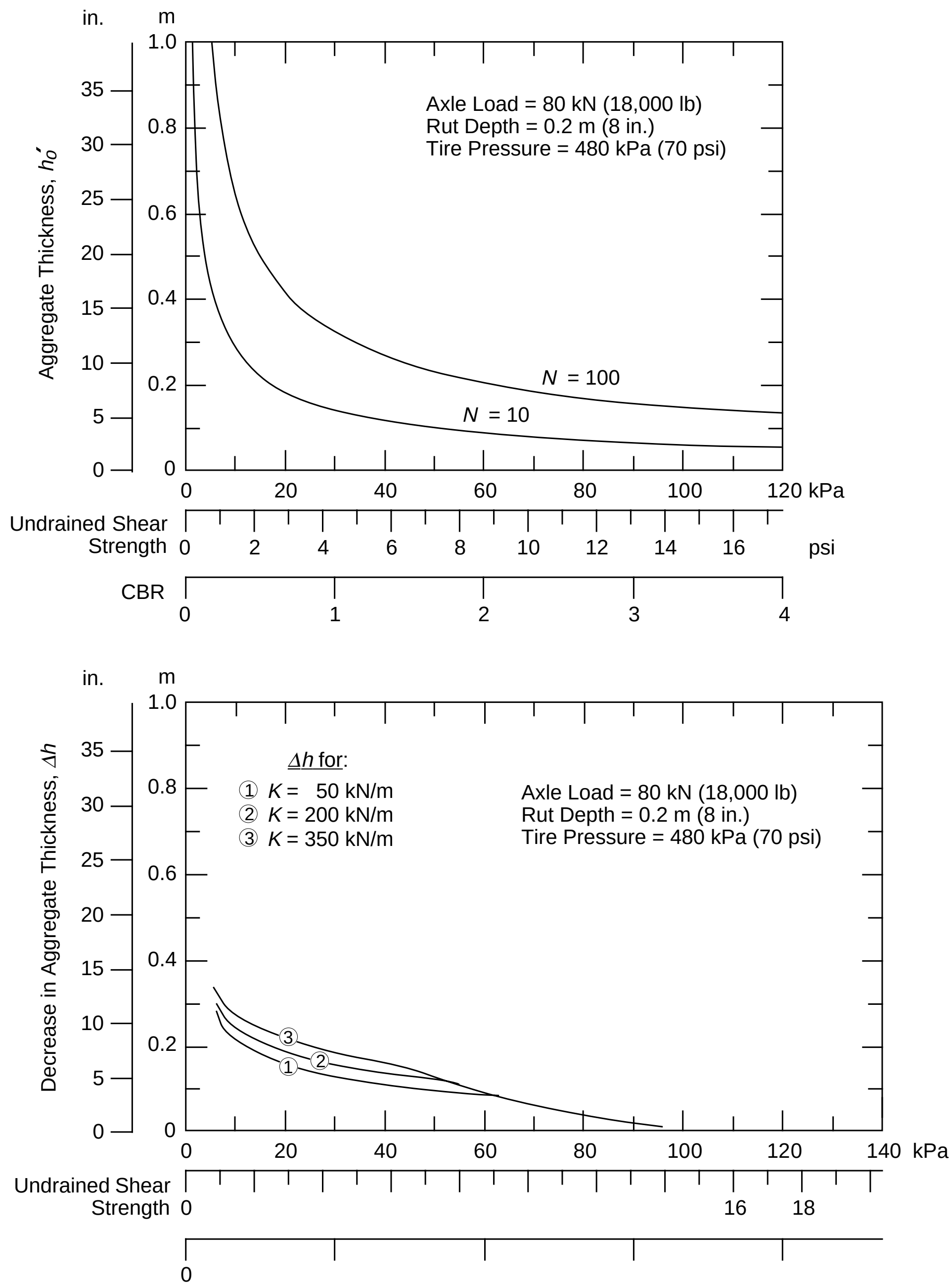

Figure 7. Aggregate thickness, h_o' *(top), and reduction of aggregate thickness,* Δh *(bottom), resulting from use of geotextile as a function of soil cohesion for 8-in. rut depth.* N *is number of vehicle passes and* K *is tensile modulus of geotextile. (From Holtz and Sivakugan 1987.)*

aggregate layer should probably be a minimum of 0.10 to 0.15 m (4-6 in.) to prevent damage to the geotextile as discussed below.

Notice that for the same conditions without geotextile, less aggregate is recommended by the Giroud and Noiray design method than by the Army method—0.2 m (8 in.) compared with 0.25 m (10 in.). The difference results from how each method estimates stress at the subgrade; this is discussed in *Theory*, below, and in *Stress Distribution Through the Aggregate Layer* in the comparison of the Giroud and Noiray and Army methods. Although the total amount of aggregate required by the two design methods is different, approximately the same aggregate savings are realized by using the Giroud and Noiray method with a 100 kN / m modulus geotextile and the Army method (0.12 m vs. 0.13 m, or essentially 5 in. for each case).

If a geotextile with a modulus of 300 kN / m (1,715 lb / in.) is used, however, Figure 6 indicates that no aggregate is required since Δh = 0.2 m (8 in.). However, a minimum 0.10 to 0.15 m (4 to 6 in.) aggregate cover over a geotextile is recommended at all times. The aggregate protects the geotextile from damage imposed by construction traffic as well as degradation due to exposure to ultraviolet light (sunlight). It also helps anchor the geotextile to allow it to develop the required tension.

Theory

Giroud and Noiray (1981) assumed a soft, saturated cohesive subgrade in undrained loading, and that the effect of the geotextile placed between the aggregate and the subgrade will change the bearing capacity failure from local (near the elastic limit, see e.g., Whitman and Hoeg 1965) to general (plastic). Thus, they applied the same soil mechanics principles as did Barenberg et al. (1975). However, Giroud and Noiray (1981) extended this concept to account for the "membrane effect" of the geotextile. Membrane effect refers to the fact that the material contained by the concave side of a stretched, flexible membrane is at a higher pressure that the material on the outside of it. As bearing capacity failure deforms the subgrade, the geotextile undergoes deformation that puts it in tension. The tensile strength of the geotextile then helps to both support the load and confine the soil above the geotextile, making it stronger (Fig. 8). The modulus, K, of the geotextile is increasingly influential as the rut depths increase (membrane action occurs at large strains).

Like that of Barenberg et al. (1975), this theory and the design technique is based on the assump-

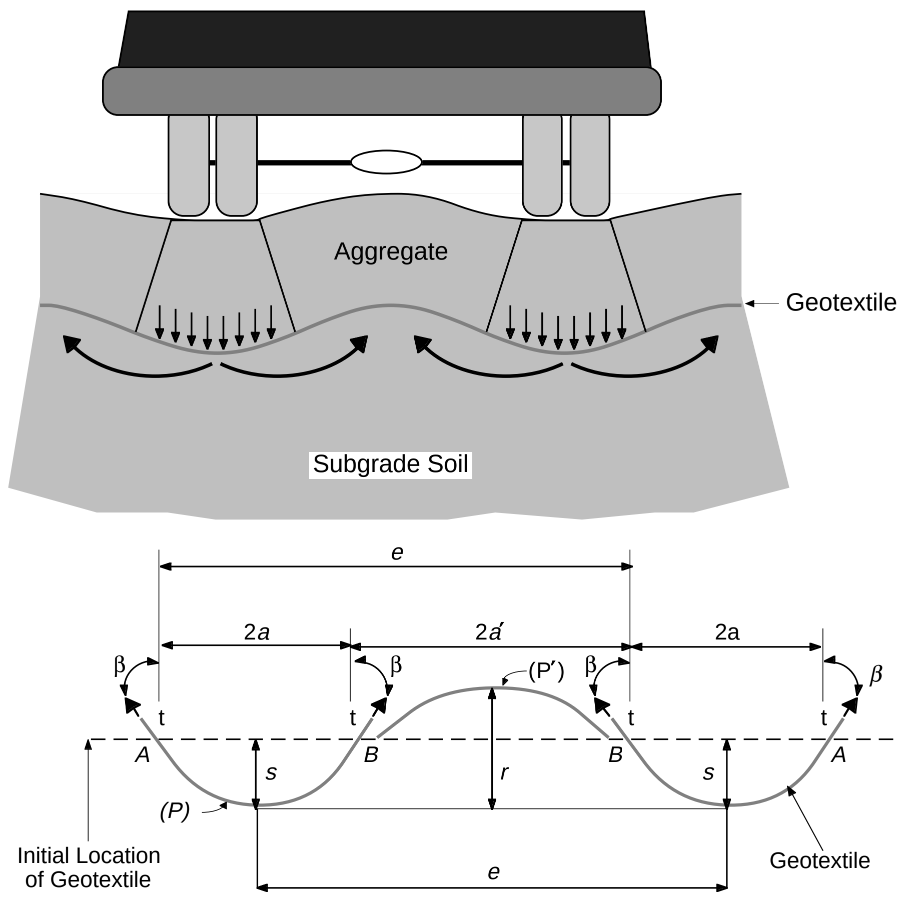

Figure 8. Diagram of "membrane effect" of geotextile reinforcement of thawing soil (top), and shape of deformed geotextile (bottom). (After Giroud and Noiray 1981.)

tion that the subgrade soil is of a sufficient depth, H_{min}, to allow the plastic zones associated with ultimate bearing capacity to develop. For the stress distribution assumed by Giroud and Noiray (1981), this amounts to

$$H_{min} = \frac{B + 2h\tan\alpha}{\sqrt{2}} \qquad (6)$$

where B is the width of the loaded area at the soil surface (Fig. 9b), h is the thickness of the aggregate layer, and $\alpha = (\pi/4) - (\phi/2)$, where ϕ is the friction angle of the base course expressed as radians (Fig. 10). Giroud and Noiray (1981) assumed the value of $\tan\alpha$ to be 0.6. Assuming a dual-tired truck with an axle load, P, having tire pressures, P_c, the width, B (m), of the wheel load is given by (Giroud and Noiray 1981)

$$B = \sqrt{\frac{P}{P_c}} \quad \text{for off-highway trucks and}$$

$$B = \sqrt{\frac{P\sqrt{2}}{P_c}} \quad \text{for on-highway trucks.} \qquad (7)$$

Table 2 presents values for H_{min} for a minimal aggregate cover of 0.15 m (6 in.).

Thus, the Giroud and Noiray (1981) technique is not generally applicable to thawed (or weak) layers less than 0.4 m (16 in.) thick, and the same is assumed for the current U.S. Army design technique. If a geotextile is used and the full plastic zones do not develop, the tension in the geotextile will not be fully mobilized. However, assuming that the subgrade soil underlying the thawed soil is stronger than the thawing soil, the support required of the geotextile will also be less than if the subgrade were uniformly weak; and, therefore (as mentioned above), designing for reinforcement with geotextile would be conservative. When the thawed layer is so thin that full plastic zones cannot develop, the geotextile may provide important separation between the thawing soil and the aggregate that will likely lead to longer use of the road without maintenance.

Other assumptions pertaining to the geosynthetic include that

1. The geotextile does not fail,
2. The shape of the deformed geotextile consists of parabolas (Fig. 8, bottom),
3. The aggregate will not slide along the geotextile surface,
4. The elongation, or strain, is uniform along the entire length of the geotextile, and
5. The modulus of the geotextile, K, used in de-

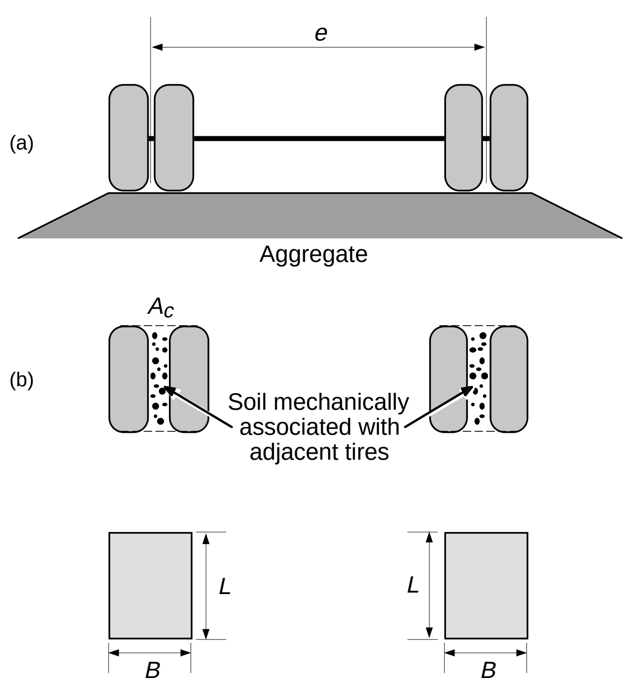

Figure 9. Definition of tire contact area for dual tires (top), and equivalent contact area used in analysis (Giroud and Noiray 1981) (bottom). (For single tires, L *and* B *refer to length and width of single tire print, respectively.)*

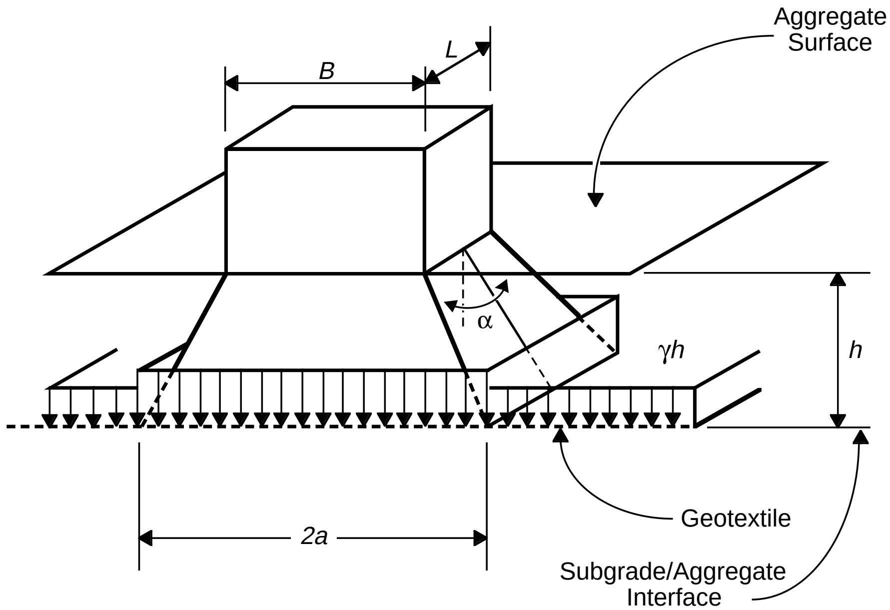

Figure 10. Wheel load distribution by aggregate layer to subgrade (Giroud and Noiray 1981).

sign is the secant modulus obtained from tensile tests.*

The first assumption is reasonable for geotextiles that meet survivability requirements. Measurements of test sections that were carefully trafficked to minimize the wander of wheels in lanes indicated that the shape of the deformed geotextile is approximately parabolic (e.g., Kinney and Barenberg 1979, Fannin and Sigurdsson 1996). If the geotextile–aggregate friction is inadequate, the aggregate can (and most likely does) slide along the surface of the geotextile (e.g., Kinney and Barenberg 1979, Kinney 1982); however, use of a high-quality aggregate (as assumed by this method) would probably prevent this from occurring. Tests have also shown that the strain is not completely uniform along the length of the geotextile (e.g., Kinney and Barenberg 1979, Fannin and Sigurdsson 1996). For example, Fannin and Sigurdsson (1996) measured the strain in three geotextiles placed beneath a 0.25-m-thick layer of sandy gravel and over a subgrade of average strength of 40kPa. The strain in the three geotextiles at 80 passes of a standard axle load of 80 kN and 620 kPa tire pressure averaged 0.9, 2.1, and 5.2%, while the maximum strain in each of the geotextiles was 2.6, 3.3, and 7.4%, respectively. The influence of the inaccuracy of this assumption on the validity of the design technique is not yet clear.

It is also not clear what modulus value should be used with this design method. Giroud and Noiray (1981) recommend the use of a biaxial tensile test, where the lateral deformation of the geotextile is prevented during testing, and that the secant modulus in the transverse direction of the road be used. Biaxial testing would lead to estimates of modulus values that are higher than those determined in uniaxial tests by about 1.1 to 1.35 times (e.g., Giroud 1992, Soderman and Giroud 1995). However, Kinney (1982, 1998, personal com-

Table 2. Thickness of plastic zone in the subgrade for dual-tired truck loading and aggregate layer of thickness 0.15 m (6 in.).

Axle load kN (lb)/ tire pressure kPa (psi)	*Plastic zone thickness, on-highway truck H_{min} (m/in.)*	*Plastic zone thickness, off-highway truck H_{min} (m/in.)*
80 kN (18,000 lb) / 480 kPa (70 psi)	0.42 / 16.5	0.47 / 18.5
60 kN (13,500 lb) 480 kPa (70 psi)	0.38 / 15	0.43 / 17

*The tensile modules of geotextiles can now be obtained from ASTM D 4595 (1998) Standard test method for tensile properties of geotextiles by the wide-width strip method.

munication) claimed that repeated loading of the only geotextile that he tested resulted in an effective modulus that was "many times lower" than that determined from monotonically loaded uniaxial and biaxial tests. Furthermore, in typical stress–strain relations for tensile loading of needle-punched geotextiles, the slope of the stress–strain curve is initially quite low, resulting in low modulus values at low strains. Therefore, research to determine the effective modulus values when geotextiles are being repeatedly loaded or trafficked in-situ would be useful.

In addition to including the tensile support provided by the geotextile, there are two more ways in which the Giroud and Noiray theory differs from that presented by Barenberg et al. (1977). Most significant is the shape of the stress distribution through the aggregate layer to the subgrade. Giroud and Noiray (1981) used a trapezoidal distribution of the stress beneath a loaded rectangle (Fig. 10) as opposed to the Boussinesq distribution beneath a circular plate used by Barenberg et al. (1975). The assumed shape of the load and the assumed stress distribution through the aggregate layer to the subgrade results in significant differences in the estimated stresses at the subgrade for certain loading and soil conditions. The difference is especially significant for relatively thin aggregate layers (less than approximately 0.3 m or 12 in.), as will be demonstrated in the next section. Giroud and Noiray (1981) also assumed a minimum CBR value of 80 for the overlying aggregate, but Barenberg et al. (1975) did not discuss the mechanical properties of aggregate, although the tests they performed utilized crushed-rock aggregate.

In addition to eq 8, the design equations from Giroud and Noiray are

a) $L = \dfrac{B}{2}$ for off-highway trucks and

$$L = \frac{B}{\sqrt{2}} \quad \text{for on-highway trucks} \qquad (9)$$

where L is the length of the rectangle formed by a set of dual wheels (Fig. 9).

b) $2a = B + 2h \tan \alpha$ (10)

where a is defined in Figure 8b.

c) $2a' = e - B - 2h \tan \alpha$ (11)

where e and a' are defined in Figure 8 (bottom).

d) $s = \dfrac{ra'}{a + a'}$ for $a > a'$ and

$$s = \frac{2ra^2}{2a^2 + 3aa' - a'^2} \quad \text{for } a > a' \qquad (12)$$

where s and r are defined in Figure 8 (bottom).

Applicability for use by the Army

The design curves based on the Giroud and Noiray (1981) method currently published are for standard 80-kN (18,000-lb) axle loads for on-highway trucks with tire inflation pressures of 480 and 620 kPa (70 and 90 psi) (e.g., Fig. 6 and 7). Estimated axle loads for U.S. Army vehicles range up to 324 kN (73,000 lb), and tire pressures can be as low as 241 kPa (35 psi) (Table 1). In addition, consideration should be given to the shape of the wheel load applied on the surface. Giroud and Noiray (1981) assumed a single-axle dual-wheel configuration, whereas many Army vehicles have single tires on tandem axles (e.g., the HEMTT). Thus, if this design technique were to be adopted by the Army, design curves for Army vehicles should be developed for higher axle / wheel loads and for variations in the shape of the applied loading.

The method published by Giroud and Noiray (1981) uses geotextile tensile modulus values ranging from 10 to 450 kN / m. Geotextile modulus values at 5% strain, provided by the manufacturers for a variety of geotextiles, are presented in Table 3. Based on limited field experiments, 5% strain appears to be a reasonable estimate for static loading of geotextiles performing reinforcement over low-bearing-capacity soils (e.g., Fannin and Sigurdsson 1996). Table 3 indicates that the tensile modulus values in the cross-machine direction, the direction that would be transverse to traffic, of some products commercially available today are significantly greater than those for which Giroud and Noiray (1981) provided design curves. This suggests that the design method should include higher modulus values. However, recall from the above discussion that modulus values are higher in biaxial tension, but possibly far lower for repeated loading than for monotonically loaded uniaxial tests. In reinforcing low-bearing-capacity soil, the geotextile is expected to undergo both biaxial tension and repeated loading. Therefore, field or other experimental work is needed to help establish the effective modulus values of

Table 3. Tensile modulus values of geotextiles at 5% strain and at failure based on information in Geotechnical Fabrics Report (1996).

Product	*Construction, mass/area (g/m²)/ (oz/yd²)*	K *at 5% strain (kN/m)/(lb/in.)*		K *at failure (kN/m)/(lb/in.)*	
		MD	*XD*	*MD*	*XD*
Amoco 2044	W-PP, na	420 / 2400	760 / 4340	700 / 4000	875 / 5000
Carthage FX-400MF	W-PP, 427 / 12	386 / 2206	456 / 2606	542 / 3098	783 / 4475
Contech C-300	W / S-PP, 200 / 6	174 / 994	210 / 1200	306 / 1749	383 / 2186
Huesker Comtrac 800	W-PET, 1430 / 42	7200 / 41150	800 / 4572	7910 / 45206	667 / 3810
Linq GTF 550T	W-PET, na	404 / 2309	404 / 2309	876 / 5006	876 / 5006
Linq GTF 1000T	W-PET, na	1050 / 6000	1050 / 6000	1402 / 8012	1402 / 8012
Synthetic Industries Gtx. 200ST	W / S-PP, 150 / 4	174 / 994	192 / 1097	233 / 1333	300 / 1715
Synthetic Industries Gtx. 4 × 4	W / C-PP, 440 / 13	384 / 2195	454 / 2595	500 / 2858	583 / 3334
TNS W300	W-PP, 203 / 6	100 / 570	280 / 1600	290 / 1657	310 / 1772
USA Spantex 5710	K-PET, 2566 / 76	8000 / 45720	4000 / 22860	10000 / 57150	4167 / 23814
Webtec, TTHPG-50	W-PP, na	200 / 1143	220 / 1257	267 / 1524	260 / 1486
Webtec, TTHPG-57	W-PP	700 / 4000	700 / 4000	538.5 / 3078	487.5 / 2786

Notes: na = not available, W = woven, K = knitted, PP = polypropylene, PET = polyester, MD = machine direction, XD = cross-machine direction.

the geotextiles when they are being used and to relate them to the values measured in tensile tests.

COMPARISON OF GIROUD AND NOIRAY METHOD WITH ARMY METHOD

The tensile reinforcement advantages offered by high-strength geotextiles may offset the increased cost. Therefore, the currently used Army design technique is compared with the design technique of Giroud and Noiray (1981) in this section. Design curves provided in Barenberg et al. (1975) and Giroud and Noiray (1981) for static loading were reconstructed to verify that the calculation techniques used for this work are accurate. Design curves for the loading imposed by typical military vehicles using each design method are also presented to demonstrate potential aggregate savings by use of the Giroud and Noiray (1981) method.

Validation of calculation techniques

Design equations were programmed using Mathcad 6.0 (Mathsoft 1995) to generate design curves. Details are given in Appendix B. Figure 11 shows the static load design curves from Barenberg et al. (1975) and points calculated for this work to verify the calculations. Similarly, Figures 12 are 13 are static load design curves from Giroud and Noiray (1981) and from calculations performed for this work. There is a difference between the curves generated for Figure 13 and those from Giroud and Noiray (1981) for the 450 kN / m geotextile being used for the 480 kPa tire pressure. This difference is estimated to be about 10% at the very lowest values of aggregate thickness. The reason for this discrepancy is unknown.

Stress distribution through the aggregate layer

Figure 14 shows the soil strength vs. aggregate thickness curves for both design techniques without geotextiles for dual wheels on a single axle with wheel loads of 60 and 115 kN (13,500 and 25,850 lb) and tire pressures of 414 kPa (60 psi). These represent 10-ton and 20-ton trucks (e.g., Table 1). The Barenberg et al. (1975) method is more conservative at these loading conditions, and this stems from the load distribution assumptions pertaining to the spreading of the load beneath the wheels. Table 4 shows the maximum vertical stress at various depths below the load for a wheel load of 115 kN and contact pressure of 414 kPa using the Boussinesq stress distribution beneath a circularly loaded area (i.e., Newmark 1942) and the trapezoidal stress distribution beneath a rectangular load used by Giroud and Noiray (1981).

Barenberg et al. (1975) used the Boussinesq stress distribution because experimental and field work of others show that stress distribution

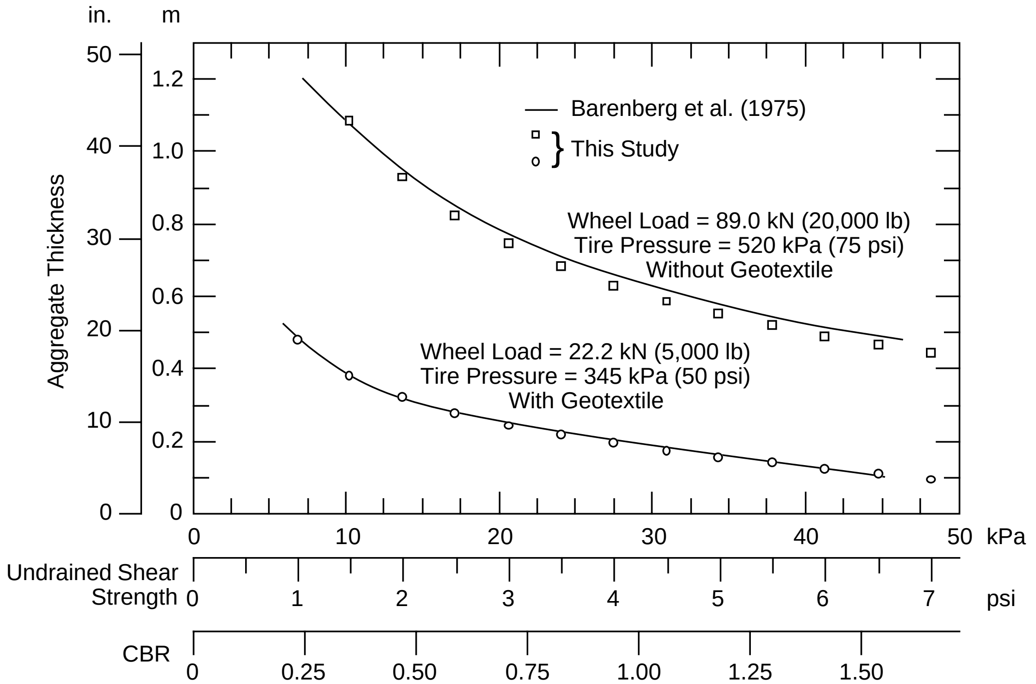

Figure 11. Static loading design curves from Barenberg et al. (1975) and design points generated for this report according to method documented in Appendix B.

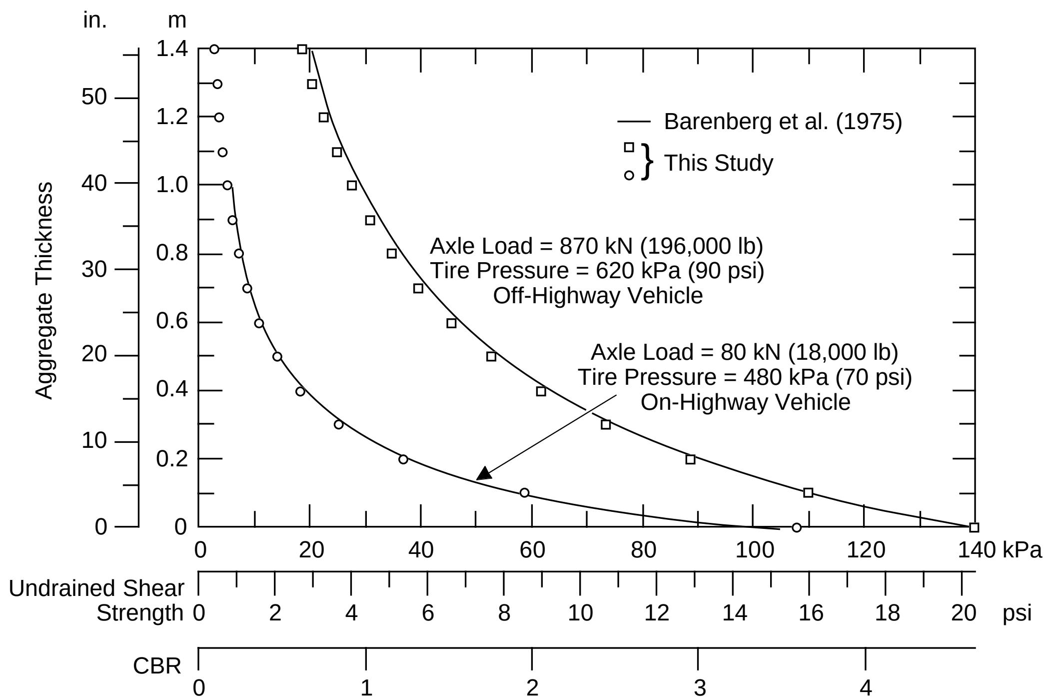

Figure 12. Static loading design curves without geotextile from Giroud and Noiray (1981) and points generated for this report according to method documented in Appendix B.

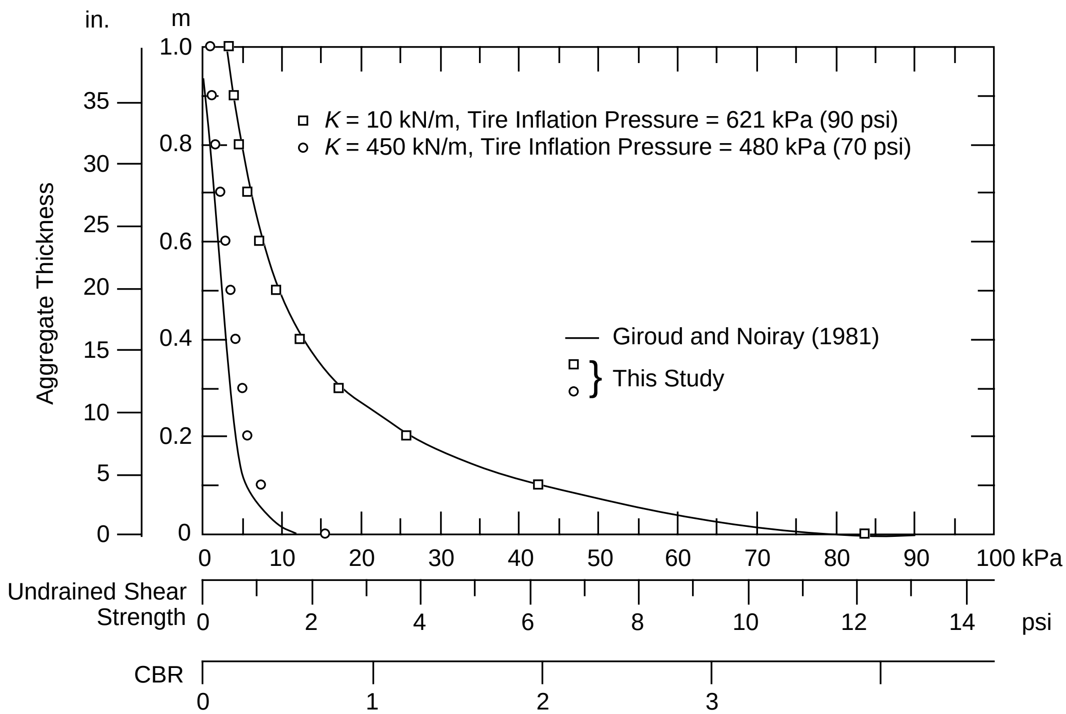

Figure 13. Static loading design curves with geotextiles from Giroud and Noiray (1981) and points generated for this report according to method documented in Appendix B.

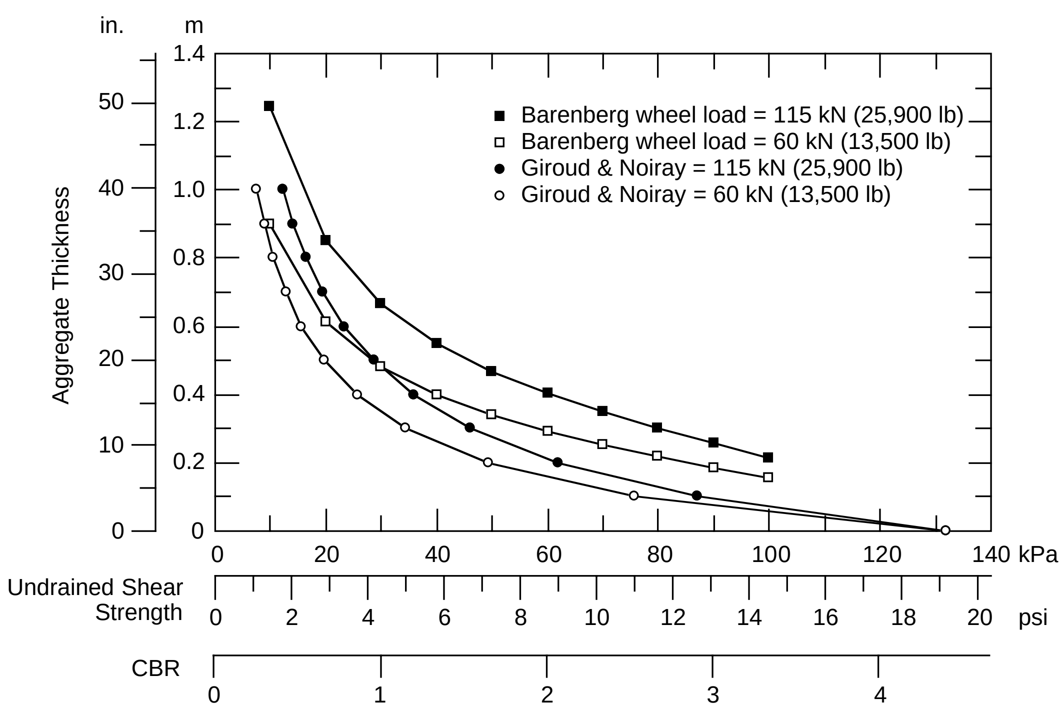

Figure 14. Static loading design curves adapted from Giroud and Noiray (1981) and Barenberg et al. (1975) for 10- and 20-ton trucks with tire pressures of 414 kPa (60 psi).

Table 4. Maximum vertical stress at various depths below applied wheel load of 115 kN and contact pressure of 414 kPa according to Newmark (1942) and trapezoidal stress distribution used by Giroud and Noiray (1981).*

Depth below applied stress, z (m)	*Stress according to trapezoidal stress distribution (kPa/psi)*	*Stress according to Boussinesq (Newmark) method (kPa/psi)*	*Ratio of the trapezoidal stress to the Boussinesq stress*
0.1	275.4 / 39.9	400.1 / 58.0	0.69
0.2	198.1 / 28.7	342.1 / 49.6	0.58
0.3	151.0 / 21.9	265.7 / 38.5	0.57
0.4	120.4 / 17.5	210.0 / 30.5	0.60
0.5	99.7 / 14.5	151.1 / 21.9	0.66
0.6	85.2 / 12.4	116.2 / 16.9	0.73
0.7	74.8 / 10.8	91.2 / 13.2	0.82
0.8	67.3 / 9.8	73.0 / 10.6	0.92
0.9	61.9 / 9.0	59.6 / 8.6	1.04
1.0	57.9 / 8.4	49.4 / 7.2	1.17

*The Boussinesq method used to generate results in this report did not add the pressure due to the weight of the overburden (= γz) whereas the trapezoidal method used did. The calculations were carried out in this manner to be consistent with how the original researchers presented them. If the weight of the overburden were added to the stresses estimated by the Boussinesq method, the differences in stresses at depths of up to 1 m would be even greater than those listed in Table 5.

through a granular layer to the subgrade follows the same pattern as that given by the Boussinesq theory. Yoder and Witzak (1975) also refer to the use of a Boussinesq distribution of stresses below traffic loading for the purposes of pavement design. Indeed, mobility models also incorporate Boussinesq stress distributions.* Although trapezoidal stress distribution below rectangular-shaped loads is commonly used in shallow foundation design (e.g., Perloff 1975), Giroud and Noiray (1981) did not cite other work that uses trapezoidal stress distribution to estimate traffic loading stresses through aggregate.

The significant difference in estimation of stresses at the surface used by the two methods warrants further investigation. There is limited evidence suggesting that the Giroud and Noiray (1981) method is unconservative for static loading conditions in both reinforced and unreinforced test sections when the aggregate layers are 0.25 to 0.50 m thick and the subgrade strength ranges from 30 to 40 kPa (CBR of about 1.5) (e.g., Fannin and Sigurdsson 1996; Fig. 15). The ratio of the trapezoidal stress below a rectangle to the Boussinesq stress below a circular plate for these loading conditions ranges from 0.61 for a 0.25-m- (10-in.-) thick aggregate to 0.78 for the 0.5-m- (20-in.-) thick aggregate (Giroud and Noiray 1981, Newmark 1942). Thus, until further investigation, use of the guidance in TM5-818-8, which incorporates the Boussinesq stress distribution through the aggregate, is recommended.

The aggregate quality significantly influences the stress distribution through it (Herner 1955), and this should not be discounted as a potential factor in the observed unconservative design for static loading by the Giroud and Noiray method described above. For example, when a 45-kN (10-kip) load was applied by an airplane tire at 690 kPa (100 psi), the vertical stress reaching the subgrade through a 0.6-m- (24-in.-) thick layer of sand was about twice that of the stress reaching the subgrade through a layer of crushed limestone (Herner 1955). McMahon and Yoder (1960) demonstrated that, for compacted, crushed limestone base rock layers ranging in thickness from 0.1 to 0.3 m (4 to 12 in.) and loaded with circular plates,

*Personal communication, G.L. Blaisdell, Research Civil Engineer, US Army Cold Regions Research and Engineering Laboratory, Hanover, N.H., 1997.

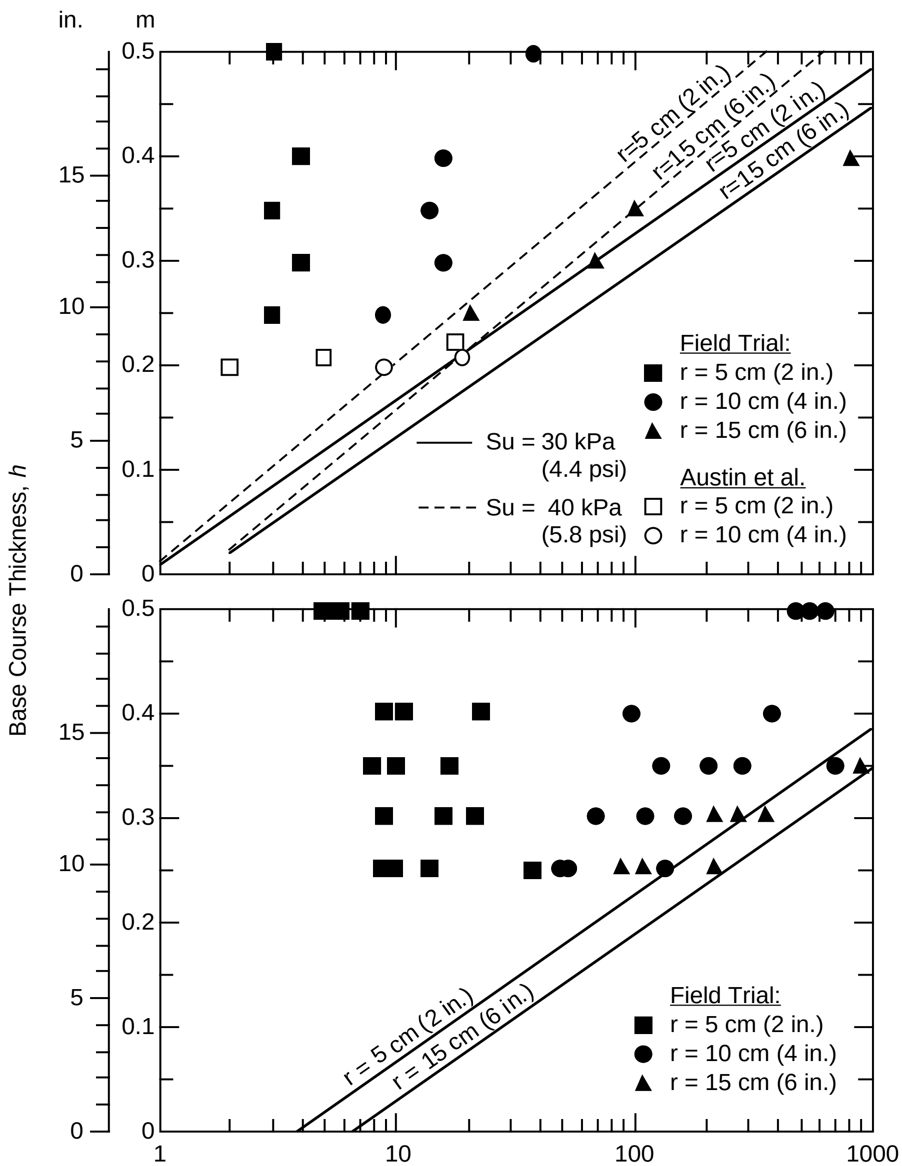

Figure 15. Field performance vs. theoretical prediction by Giroud and Noiray (1981) for unreinforced test sections (top) and reinforced test sections (bottom). (From Fannin and Sigurdsson 1996.)

the pressure measured in compacted clay-soil below the rock layers was reasonably approximated by a Boussinesq distribution beneath circular plates. Barenberg et al. (1975) based their theory on tests that utilized "crushed stone aggregate," and, based on the work of McMahon and Yoder (1960), a Boussinesq stress distribution through it is a reasonable assumption for such an aggregate. Unfortunately, Steward et al. (1977) did not describe the aggregate that was used in tests to validate the Barenberg et al. (1975) design method.

The possibility of using Boussinesq stress distribution through the aggregate layer could be added to the Giroud and Noiray (1981) design technique. In addition, shapes other than a circularly loaded area should be considered, and work that examines stress distributions through aggregates other than crushed rock should also proceed. This would allow the confident use of design techniques for relatively low-quality aggregate that might be the only option for theater of operations military construction.

Design curves for Army vehicles

Because the potential for aggregate and cost savings is of interest to the U.S. Army, and the Giroud and Noiray (1981) method shows promise for large savings over the current Army design method, design curves for Army vehicles were developed according to both methods for comparison of aggregate thickness required. Design curves for the U.S. Army's 10- and 20-ton trucks are presented in Figures 16 and 17, respectively. A geotextile tensile modulus of 200 kN / m (1143 lb / in.) was used for Figures 16 and 17 (bottom) because this value is easily obtained for commercially available products (Table 3).

Considerable aggregate savings can be realized if the Giroud and Noiray (1981) method is used. For the 10-ton truck, with a soil strength of 30 kPa (4.4 psi), the aggregate savings for the Giroud and Noiray (1981) method over the current Army method is about 0.2 m (8 in.) with geotextile. For the 20-ton truck at a soil strength of 40 kPa (5.8 psi), the aggregate savings for the Giroud and Noiray (1981) method over the current Army method is about 0.2 m (8 in.) with geotextile. Thus, accounting for the tensile support provided by the geotextile provides considerable advantages of aggregate savings. It is important to remember that the aggregate used with this method should have a minimum CBR of 80 (Giroud and Noiray 1981).

RECOMMENDATIONS FOR FUTURE WORK

Using the Giroud and Noiray (1981) method as it is presented herein may lead to unconservative design and construction results because the stress distribution through the aggregate layer to the subgrade is less than the Boussinesq method, which is widely accepted and well-supported. However, it should be further investigated because it promises large aggregate savings compared with the current design method, due solely to its ability to account for tensile properties of the geotextile reinforcement at large rut depths, a situation that can be tolerated by military vehicles on thawing soils. Depending on the outcome of an investigation of the stress distribution through the aggregate layer to the subgrade, it may be worthwhile to develop a hybrid design method that uses a Boussinesq stress distribution through the subgrade with a membrane support mechanism as presented by Giroud and Noiray (1981). For use of a Boussinesq stress distribution, the load at the surface is not necessarily best modeled as a rigid circular area (as it is now). For example, the length to width ratio for a HEMTT is estimated as $L = 1.6B$ (e.g., Richmond et al. 1990). Thus, other wheel-load geometries should be considered.

The Giroud and Noiray (1981) design method indicates that the geotextile may be able to provide reinforcement with no aggregate on the surface. As discussed earlier, this is not a currently recommended practice because of the increased risk of damage to the geotextile due to trafficking and because of deterioration when exposed to sunlight (e.g., Holtz et al. 1993). However, it is potentially of great interest to the U.S. Army for reinforcement of thawing soils, especially for expedient, temporary operations where ultraviolet degradation due to exposure to sunlight is not a consideration (e.g., less than 10 days of exposure) and aggregate is not available. For this concept to be implemented, the geotextiles would likely have to be anchored in some way in order for the tensile properties to fully develop and provide the necessary reinforcement. (Even though the geotextile is in a state of tension between the wheels, the portion on the outside of each set of wheels could easily slip into ruts formed by the vehicles.)

An important factor in the adoption of the Giroud and Noiray (1981) design method for use is knowledge of the appropriate geotextile modulus values. Geotextile modulus values at 5% strain are readily available. Based on limited field experiments, this appears to be a reasonable strain estimate for static loading of geotextiles performing reinforcement over low-bearing-capacity soils. However, modulus values are higher in biaxial tension, but possibly far lower for repeated loading than for monotonically loaded uniaxial tests—the tests that are now performed to determine geotextile modulus values. In reinforcing low-bearing-capacity soil, the geotextile is expected to undergo both biaxial tension and repeated loading. Therefore, field or other experimental work is needed to help establish the effective modulus values of the geotextiles when they are being used and to related them to the values measured in tensile tests. Finally, the tensile modulus values of some commercially available geotextiles far exceed those used in Figures 16 and 17 (bottom). Future work should consider the use of available products with appropriately high modulus values. This could result in substantial aggregate savings.

Regardless of whether the Giroud and Noiray

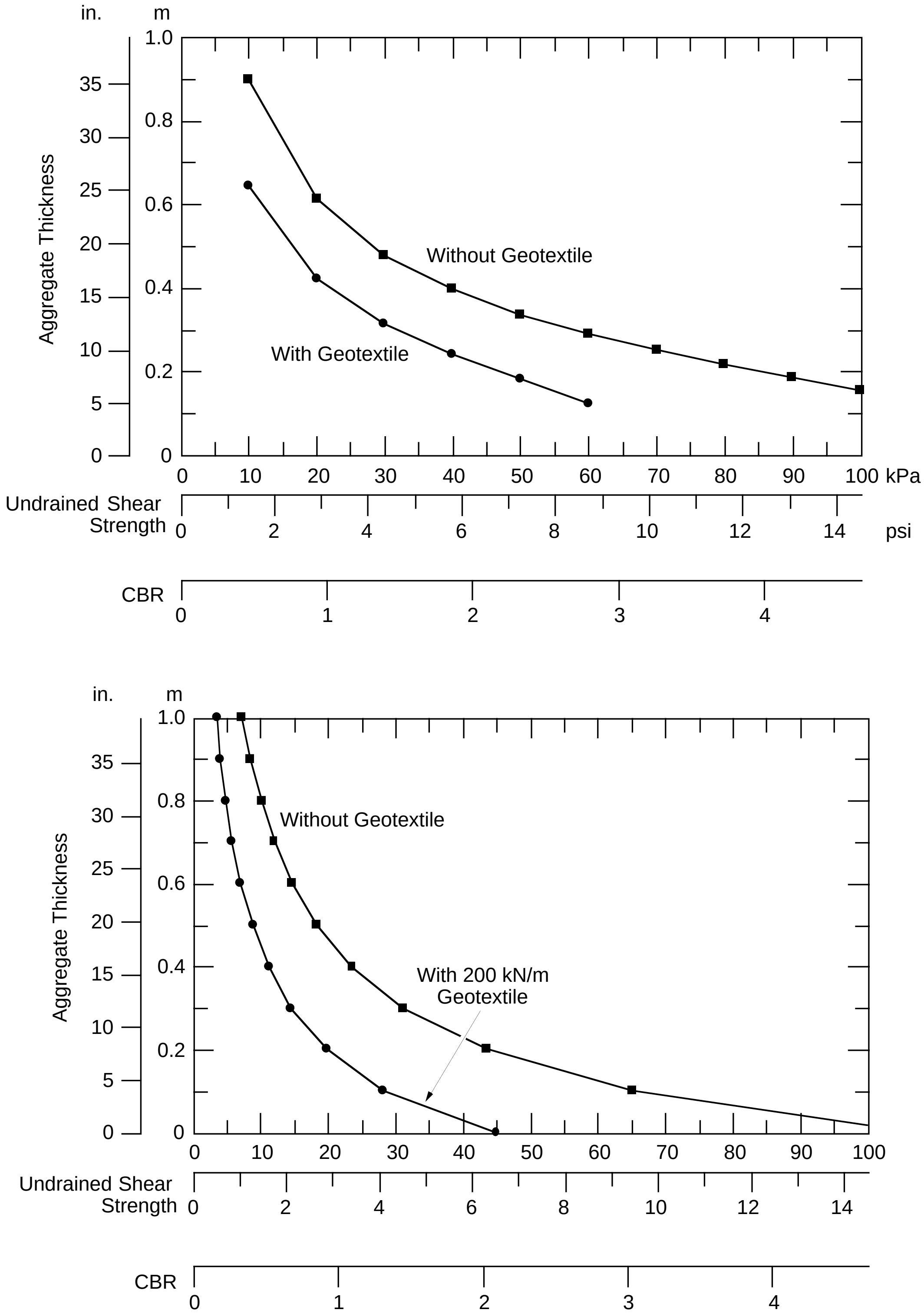

Figure 16. Design curves for static loading (up to 100 passes) for 10-ton dump truck, according to Barenberg (1975) method (top) and Giroud and Noiray (1981) method (bottom). Use of upper figure is recommended until further research is conducted.

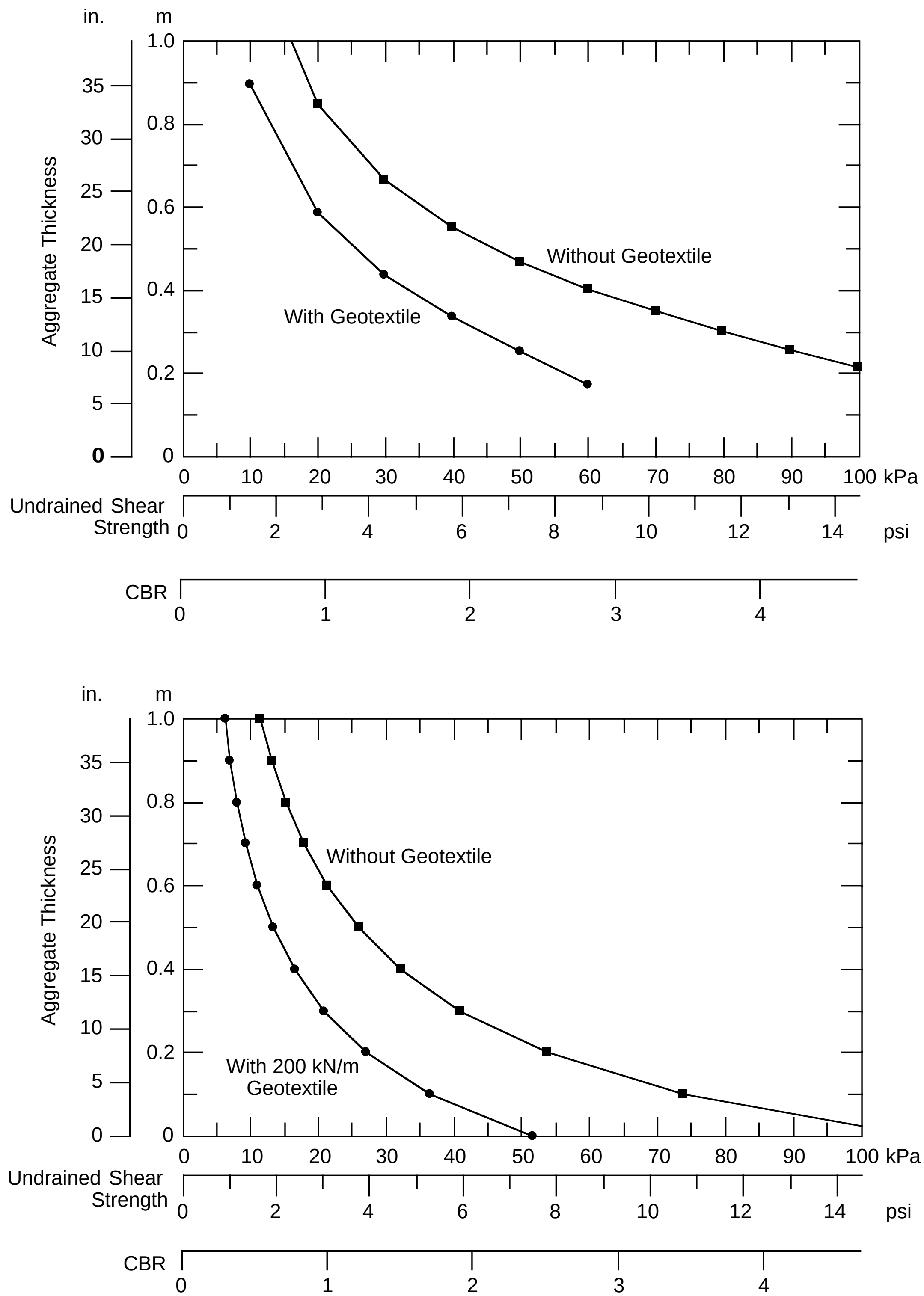

Figure 17. Design curves for static loading (up to 100 passes) for 20-ton dump truck, according to Barenberg (1975) method (top) and Giroud and Noiray (1981) method (bottom). Use of upper figure is recommended until further research is conducted.

(1981) design method is selected for use by the U.S. Army, estimates of stress through a variety of aggregates, not just crushed rock, should also be completed so the design method can be adjusted accordingly. More rounded material such as sand and gravel has been shown to concentrate stresses over a significantly smaller area on the subgrade than crushed rock. Due to the likelihood that theater of operations construction will be completed with limited sources of high-quality aggregate, this would be an important addition to the current design method.

Finally, even though soils are usually only temporarily in a weakened state when they thaw, they will sometimes have to carry more than 100 vehicles during thawing. Thus, a method that accounts for repeated traffic loading is desirable, and this should also be included in future development efforts.

SUMMARY AND CONCLUSIONS

Table 1 may be used with the design curves presented in TM5-818-2 for convenience in using the current Army geotextile reinforcement design method. However, if 10- or 20-ton trucks are expected to exert the maximum wheel loads on thawing or other low-bearing-capacity subgrade soils, Figure 16 or 17 (top) may be used, respectively. These methods both require the use of high quality aggregate. If the thawed layer is less than 0.4 m (16 in.) thick, these methods are likely to be conservative. However, a geotextile separator will probably still provide benefit to lengthen times between maintenance of the gravel surface.

If further research proves that the Giroud and Noiray (1981) design method is adequate, considerable aggregate savings for the U.S. Army would be realized by using it. However, since the Giroud and Noiray (1981) design method may be unconservative, it should not be used by the U.S. Army until further study is completed. A hybrid method, combining a Boussinesq stress distribution through the aggregate layer with a membrane support mechanism as presented by Giroud and Noiray (1981) might be an optimum design technique. When this approach is further developed, it should also include determination of representative modulus values, the use of a variety of aggregates, the shape of the wheel load, and repeated traffic loading.

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APPENDIX A: GEOTEXTILE SURVIVABILITY REQUIREMENTS

This appendix is provided as a convenience to readers, so that geotextiles meeting survivability requirements may be specified for acquisition without referring to TM5-818-8 (1995). It can be used in place of Tables 2-2, 2-3, and 2-4 in U.S. Army TM5-818-8. The guidance provided here is taken from AASHTO-AGC-ARTBA Joint Committee: Subcommittee on New Highway Materials, Task Force 25 (1990) and is more recent guidance than that provided in TM5-818-8.

Table A-1. Construction survivability ratings.

Site soil CBR	*<1*		*1 to 2*		*>3*	
Equipment ground contact pressure, kPa (psi)	>350(50)	<350(50)	>350(50)	<350(50)	>350(50)	<350(50)
Cover thickness, compacted (mm / in.)[a]						
100 / 4[b,c]	NR	NR	H	H	M	M
150 / 6	NR	NR	H	H	M	M
300 / 12	NR	H	M	M	M	M
450 / 18	H	M	M	M	M	M

[a]Maximum aggregate size not to exceed one half of compacted cover thickness.
[b]For low-volume, unpaved roads (average daily traffic less than 200 vehicles).
[c]Minimum cover thickness is limited to existing road bases and is not intended for use in new construction.
H = high, M = medium, NR = not recommended.

Table A-2. Geotextile physical property requirements for survivability[a]

	<50% elongation/>50% elongation[b,c]		
Required survivability level	*Grab strength (kN/lb) ASTM D 4632*	*Puncture resistance (kN/lb) ASTM D 4833*	*Trapezoidal tear strength (kN/lb) ASTM D 4533*
H	270 / 180	100 / 75	100 / 75
M	180 / 115	70 / 40	70 / 40

Additional requirements	*Test method*
Apparent opening size 1. <50% soil finer than 0.075 mm, AOS <0.6 mm 2. >50% soil finer than 0.075 mm, AOS > 0.3 mm	ASTM D 4751
Permeability $K_{geotextile} > K_{soil}$ $K_{geotextile}$ = permittivity × nominal geotextile thickness	ASTM D 4491
Ultraviolet degradation At 150 hr exposure, 70% of strength retained for all cases	ASTM D 4355
Geotextile acceptance	ASTM D 4759

[a]Values shown are minimum roll average values.
[b]Elongation as determined by ASTM D 4632.
[c]Values of geotextile elongation do not imply the allowable consolidation properties of the subgrade soil. These must be determined by a separate investigation.

Table A-3. Recommended overlaps.

Soil strength (CBR)	*Unsewn overlap (mm/in.)*	*Sewn overlap (mm/in.)*
<1	NR	229 / 9
1–2	965 / 38	203 / 8
2–3	762 / 30	76 / 3
>3	610 / 24	—

NR = Not recommended.

APPENDIX B: METHODS USED TO RECALCULATE DESIGN CURVES FROM BARENBERG ET AL. AND GIROUD AND NOIRAY.

The design method presented by Barenberg et al. (1975) necessitated the solving of equations for the aggregate thickness required, given a known range of soil cohesion. For Giroud and Noiray (1981), equations for cohesion were solved for a given range of aggregate thickness. Mathcad 6.0 (Mathsoft 1995) was used to solve these equations and generate design curves as well as to produce a symbolic equation for the aggregate thickness required for the Barenberg et al. (1975) approach.

SOLUTION FOR DEPTH OF AGGREGATE FOR BARENBERG DESIGN METHOD

Barenberg et al. (1975) assumed that the stress transmitted to the subgrade surface through the aggregate layer can be approximated by a Boussinesq stress distribution through an elastic, homogeneous, isotropic half-space. The ratio of vertical stress at depth, z, to the stress on a uniformly loaded circular area in an elastic, homogeneous, isotropic solid bounded by a plane horizontal surface is (Newmark 1942)

$$\sigma = 1 - \cos^3 \alpha \tag{B.1}$$

where $\alpha = a\tan(r / z)$, and r is the radius of the circle and z is the depth at which the stress determination is desired (z is located directly below the center of the circularly loaded area).

The expression for α is substituted into eq B.1 and is rearranged to yield

$$a\tan\left(\frac{r}{z}\right) = a\cos(1-\sigma)^{\frac{1}{3}}. \tag{B.2}$$

This equation is then solved for z (using Mathcad 6.0) to yield

$$z = r\left(\frac{(1-\sigma)^{\frac{1}{3}}}{\sqrt{1-(1-\sigma)^{\frac{2}{3}}}}\right).$$

For a range of soil cohesion, c, and a known applied stress, the stress that can be tolerated at the subgrade is given by either 3.3 c (without geotextile) or 6.0 c with geotextile. Thus, the ratio of soil strength to applied pressure, e.g., σ = (3.3c / contact pressure) is used to determine the thickness of the aggregate layer needed. Example work sheets from the Mathcad 6.0 software used to make these calculations follow.

EXAMPLE MATHCAD 6.0 WORKSHEET FOR BARENBERG DESIGN METHOD

Solving of Boussinesq equation for *z*, the depth of aggregate for wheel loads of 60 and 115 kN, applied contact pressure is 414 kPa. With geotextile.

Wload = 115 000 newton

ConPress = 414 000 Pa

$$r = \sqrt{\frac{\text{Wload}}{\text{ConPress} \cdot \pi}}$$

r = 0.29735 m

c = 10 000 Pa, 20 000 Pa … 60 000 Pa

$$\sigma(c) = \frac{6.0c}{\text{ConPress}}$$

Stuff in English units: $\frac{c}{\text{psi}}$ $\frac{r}{\text{ft}} = 0.97557$

1.45038
2.90075
4.35113
5.80151
7.25189
8.70226

$$z(c,r) = r \frac{[1-\sigma](c)^{\frac{1}{3}}}{\sqrt{1-[1-\sigma(c)]^{\frac{2}{3}}}}$$

z(c,r)
0.89647 m
0.58732 m
0.43709 m
0.33620 m
0.25473 m
0.17497 m

c
$1.000\,000\ 10^4$ kg m^{-1} sec^{-2}
$2.000\,000\ 10^4$ kg m^{-1} sec^{-2}
$3.000\,000\ 10^4$ kg m^{-1} sec^{-2}
$4.000\,000\ 10^4$ kg m^{-1} sec^{-2}
$5.000\,000\ 10^4$ kg m^{-1} sec^{-2}
$6.000\,000\ 10^4$ kg m^{-1} sec^{-2}

This part of the sheet creates a database and makes a .prn file out of it for importing to a spreadsheet later.

WRITEPRN (Baretwent) = z(c,r)

EXAMPLE MATHCAD 6.0 WORKSHEET FOR GIROUD AND NOIRAY DESIGN METHOD

This is a file to calculate the aggregate depth needed for different K values of the geotextile. It uses eq 43, 33, 35, 36, and 37 (a' > a, meaning that the parabola between wheels is wider than the sum of the widths of the parabolas under the wheels), 30 and 31 as well as 5 and 7 (on-highway trucks). The original reference is Giroud and Noiray (1981).

$K = 200\,000\ \mathrm{N\ m^{-1}}$	$e = 2.0\ \mathrm{m}$
$\tan\alpha = 0.6$	$P = 230\,000$ newton
$Pc = 414\,000\ \mathrm{P}$	$r = 0.3\ \mathrm{m}$

Width of wheel load (on road), 5: $B = \sqrt{\dfrac{P}{Pc}}$ $\quad B = 0.745\ \mathrm{m}$

$h = 0, 0.1 \ldots 1.0\ \mathrm{m}$

Length of wheel load(on road), 7: $L = \dfrac{B}{\sqrt{2}}$ $\quad L = 0.527\ \mathrm{m}$

Width of parabola under wheel, 30:
$a(h) = 0.5\,(B + 2 \cdot h \cdot \tan\alpha)$

Width of parabola between wheels, 31:
$aprime(h) = 0.5(e - B - 2 \cdot h \cdot \tan\alpha)$

Settlement of geotextile from original position, 33:

$$s(h) = \frac{r \cdot aprime(h)}{a(h) + aprime(h)}$$

Equation for half length of parabola under the wheel, 36:

$$b(h) = a(h)\left[1 + 0.5\left(\sqrt{1 + \left[\frac{2 \cdot s(h)}{a(h)}\right]^2} + \frac{a(h)}{2 \cdot s(h)} \cdot \ln\left\{\frac{2 \cdot s(h)}{} + \sqrt{1 + \left[\frac{2 \cdot s(h)}{a(h)}\right]^2}\right\} - 2\right)\right]$$

Equation for half length of parabola between the wheels, 37:

$$bprime(h) = aprime(h) \cdot \left(1 + 0.5\left\{\sqrt{1 + \left(\left\{\frac{2 \cdot [r - s(h)]}{aprime(h)}\right\}^2\right)} + \frac{aprime(h)}{2 \cdot [r - s(h)]} \cdot \ln\left[\left(2 \cdot \frac{r - s(h)}{aprime(h)} + \sqrt{1 + \left\{\frac{2 \cdot [r - s(h)]}{aprime(h)}\right\}^2}\right)\right]\right\} - 2\right)$$

The elongation of the geotextile, 35: $e(h) = \left[\dfrac{b(h) + bprime(h)}{a(h) + aprime(h)}\right] - 1$

And the equation of thickness of aggregate vs. soil strength, 43:

$$Cu(K,P,h) = \frac{1}{\pi+2}\cdot\left[\frac{P}{2\cdot(B+2\cdot h\cdot \tan\alpha)\cdot(L+2\cdot h\cdot \tan\alpha)} - \frac{K\cdot\varepsilon(h)}{a(h)\cdot\sqrt{1+\left[\frac{a(h)}{2\cdot s(h)}\right]^2}}\right]$$

Cu (K, P, h)	h
$5.181\ 10^{4}$ kg m^{-1} sec^{-2}	0 m
$3.659\ 10^{4}$ kg m^{-1} sec^{-2}	0.1 m
$2.722\ 10^{4}$ kg m^{-1} sec^{-2}	0.2 m
$2.099\ 10^{4}$ kg m^{-1} sec^{-2}	0.3 m
$1.663\ 10^{4}$ kg m^{-1} sec^{-2}	0.4 m
$1.346\ 10^{4}$ kg m^{-1} sec^{-2}	0.5 m
$1.111\ 10^{4}$ kg m^{-1} sec^{-2}	0.6 m
$9.36\ 10^{3}$ kg m^{-1} sec^{-2}	0.7 m
$8.058\ 10^{3}$ kg m^{-1} sec^{-2}	0.8 m
$7.116\ 10^{3}$ kg m^{-1} sec^{-2}	0.9 m
$6.477\ 10^{3}$ kg m^{-1} sec^{-2}	1 m

WRITEPRN(Gir20t) = Cu (K, P, h)

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13. ABSTRACT (*Maximum 200 words*)

Thawing fine-grained soils are often saturated and have extremely low bearing capacity. Geosynthetics are used to reinforce unsurfaced roads on weak, saturated soils and therefore are good candidates for use in stabilization of thawing soils. To stabilize the soil, a geotextile is placed on it, then the geotextile is covered with aggregate. Design involves selection of aggregate thickness and geotextile. There are two commonly used design techniques for geotextile reinforcement of low-volume roads, and the Army uses one of them. The theory and use of the two design methods for static loading (i.e., up to 100 vehicle passes) are presented and compared in this report. The design method not used by the Army offers the potential to reduce aggregate thickness over the geotextile because it accounts for the fact that the geotextile helps support the traffic load (when in tension) and confines the soil between the wheels and the subgrade. However, this alternative method appears to be unconservative with respect to stresses estimated at the subgrade surface. Thus, the current Army design technique should be used until more research is conducted. In the meantime, straightforward design curves for Army 10- and 20-ton trucks as well as vehicle loading and tire pressure information for a number of other vehicles are included in this report to help make the current design method easy to use.

Future work should consider adopting a hybrid design method that provides realistic estimates of stresses at the subgrade and accounts for the tensile properties of geotextiles. In addition, aggregates other than the high-quality crushed rock that is inherently assumed by each design method should be accounted for in new design development.

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