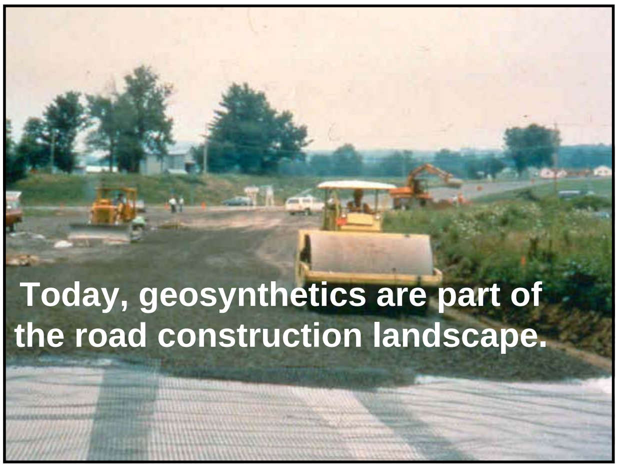
Romanian Chapter
International Geosynthetics Society

igs

GEOSYNTHETICS
IN
ROAD APPLICATIONS

J.P. GIROUD



This is because geosynthetics are used in many road applications where they perform a variety of functions.

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Let's start with the four classical functions:

- TRANSMISSION
- FILTRATION
- SEPARATION
- REINFORCEMENT

These functions were first identified for geotextiles.

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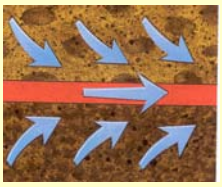
TRANSMISSION

The geosynthetic conveys water within its plane.



Types of geosynthetics: Needle-punched nonwoven geotextiles , geonets, geomats

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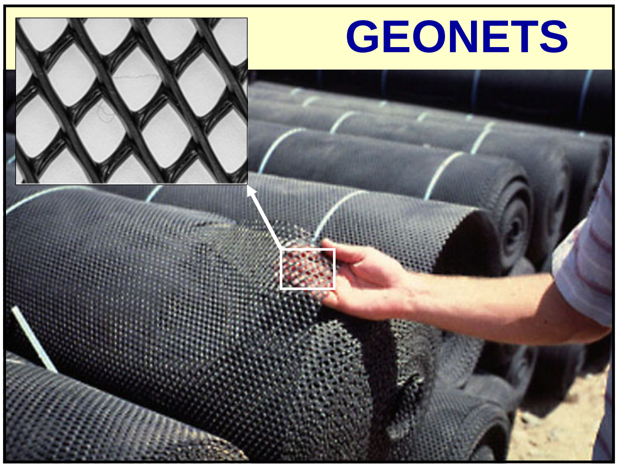
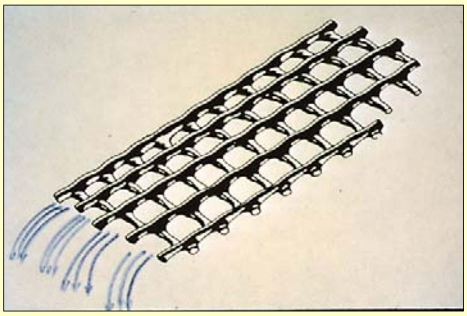
THICK NEEDLE-PUNCHED NONWOVEN GEOTEXTILE

GEONETS



Geonets can convey water within their channels.



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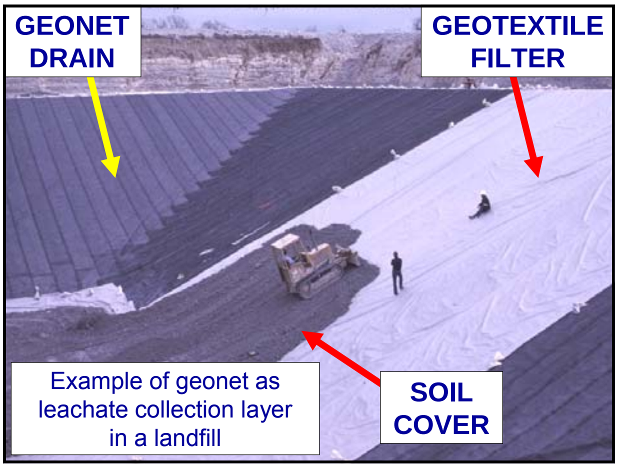
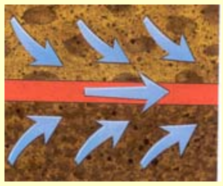


Example of geonet as leachate collection layer in a landfill

TRANSMISSION

The geosynthetic conveys water within its plane.



Types of geosynthetics: Needle-punched nonwoven geotextiles , geonets, geomats

Main relevant property: Hydraulic transmissivity (= thickness × permeability)

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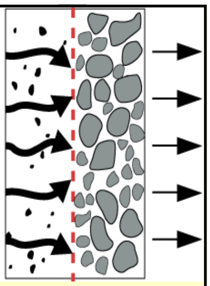
FILTRATION

The geosynthetic allows water to pass while retaining the sol.



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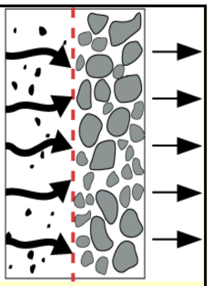
Example of geotextile filter between gravel drain and soil

FILTRATION

The geosynthetic allows water to pass while retaining the sol.



Types of geosynthetics:
geotextiles, but
some geotextiles are adequate, some are not.

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For filtration,
a needle-punched nonwoven geotextile is generally adequate,

a monofilament woven geotextile is often adequate,

a slit-film woven geotextile is not.

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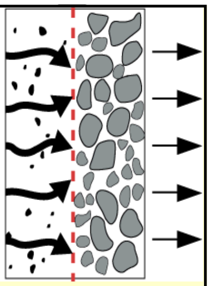
FILTRATION

The geosynthetic allows water to pass while retaining the sol.



Types of geosynthetics: some geotextiles

Main relevant properties:
permeability, retention

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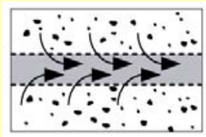
TRANSMISSION & FILTRATION

A geotextile filter is associated with a transmissive core to form a geocomposite.



Types of material for the core:
geonet, geomat, cuspated sheet

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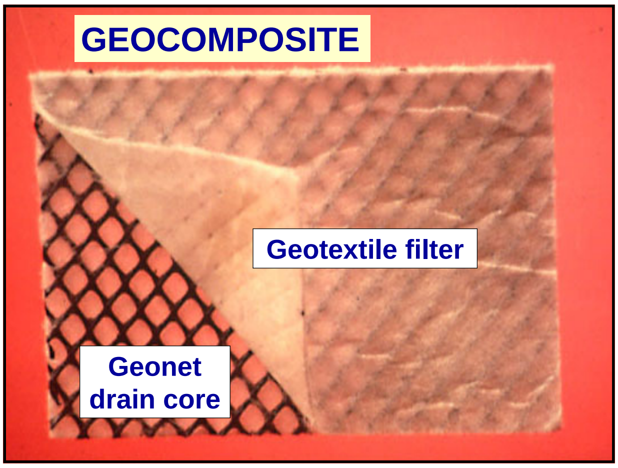
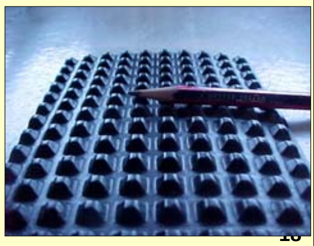


The drain core of a geocomposite can also be:

GEOMAT

CUSPATED SHEET



There are also very thick geocomposites made especially for road edge drains.

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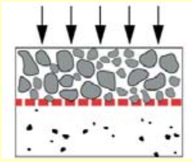
SEPARATION

The geosynthetic separates two materials that tend to mix when they are squeezed together by applied loads.



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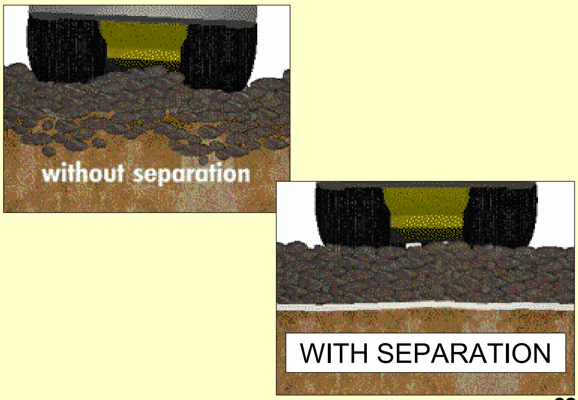

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SEPARATION

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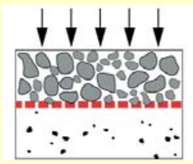

Types of geosynthetics: geotextiles

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SEPARATION

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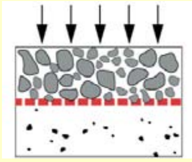

Types of geosynthetics: geotextiles

Main relevant properties:
permeability, retention,
resistance to concentrated stresses

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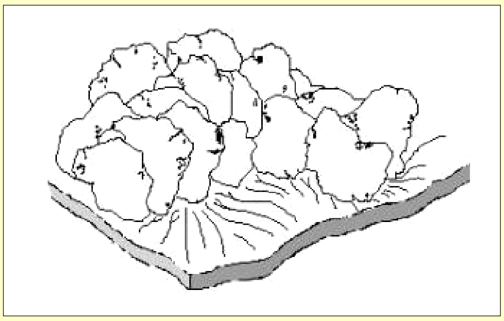
To resist concentrated stresses, the geotextile must have high strength and elongation.



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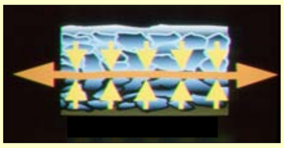
REINFORCEMENT

The geosynthetic carries tensile loads that the soil is unable to carry.



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REINFORCEMENT

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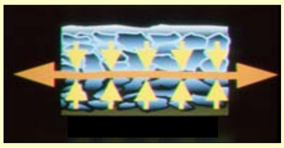

Types of geosynthetics:
high-strength geotextiles, geogrids

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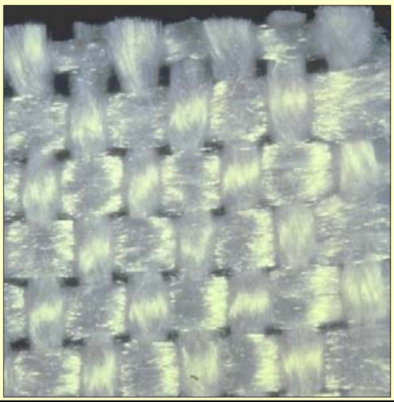
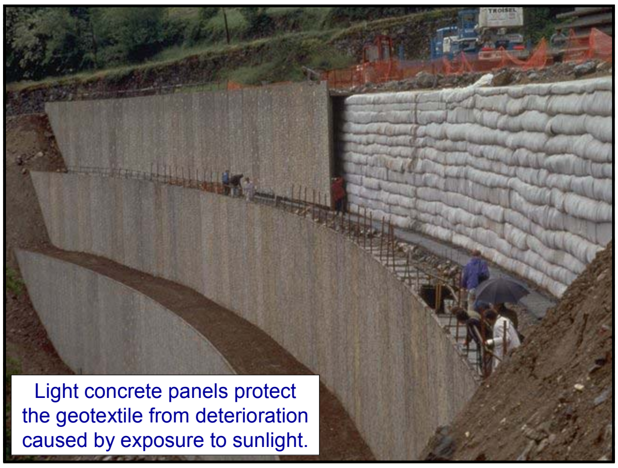
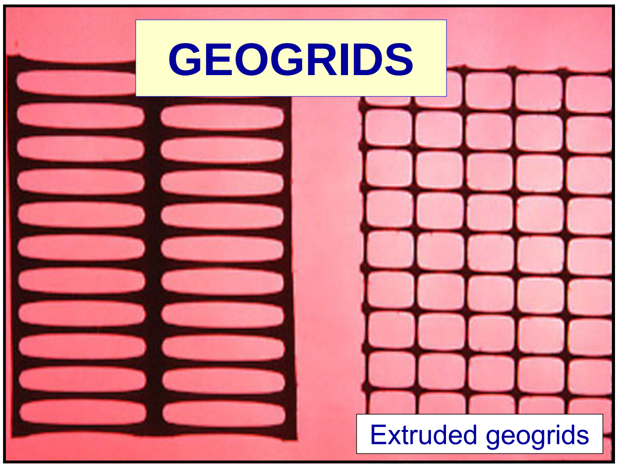
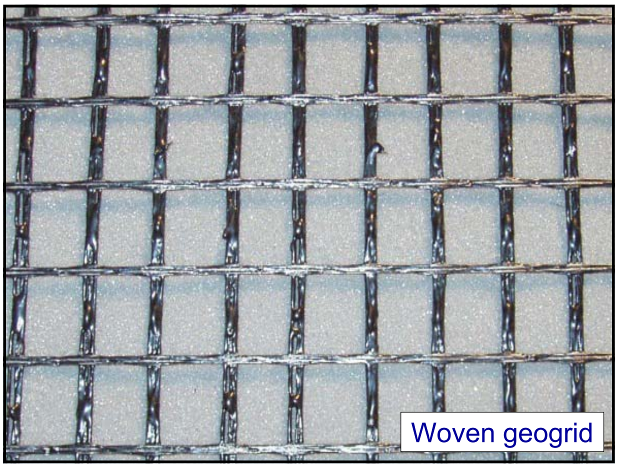
HIGH-STRENGTH WOVEN GEOTEXTILE

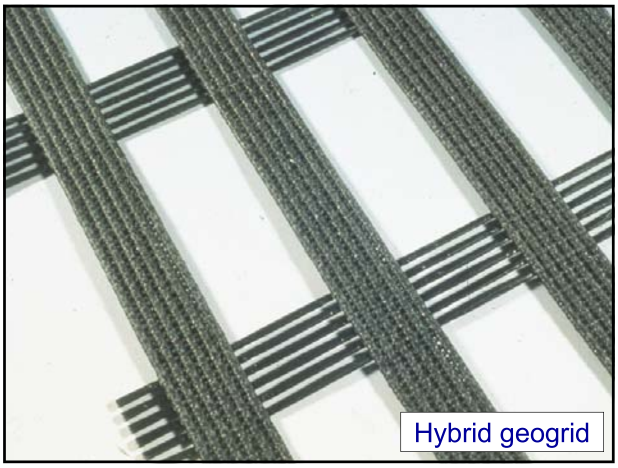


Multi-layer reinforced soil wall during construction

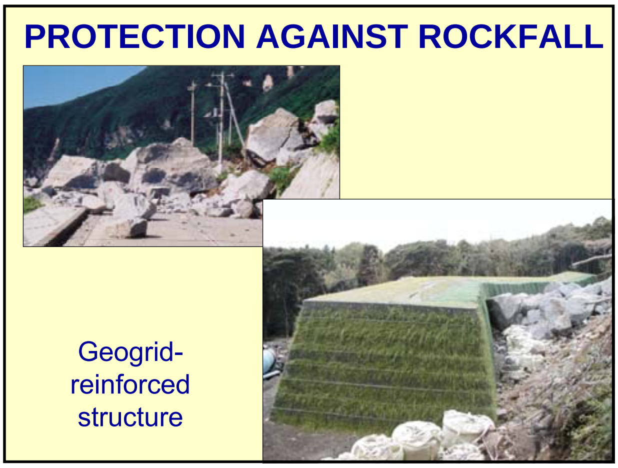

Light concrete panels protect the geotextile from deterioration caused by exposure to sunlight.


GEOGRIDS
Extruded geogrids

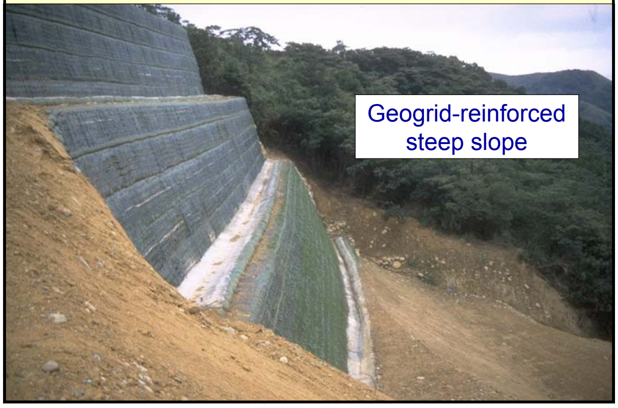

Woven geogrid


Hybrid geogrid


RETAINING WALL
Light concrete blocks used as facing
Geogrid reinforcement

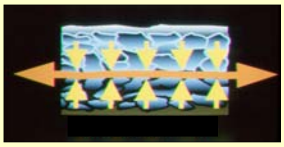

PROTECTION AGAINST ROCKFALL
Geogrid-reinforced structure

LANDSLIDE REPAIR



REINFORCEMENT

The geosynthetic carries tensile loads that the soil is unable to carry.



Types of geosynthetics:
high-strength geotextiles, geogrids

Main relevant properties:
tensile strength and modulus, interface shear strength

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INTERFACE SHEAR STRENGTH

- **Interface adhesion**
- **Interface friction**
- **Interlocking**

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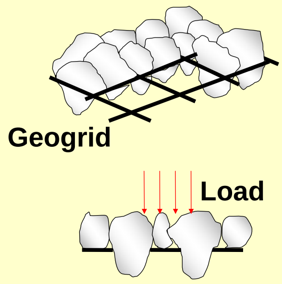
INTERLOCKING

Interlocking exists only with geogrids, and only if there is adequate relationship between the geogrid opening size and the soil particle size.



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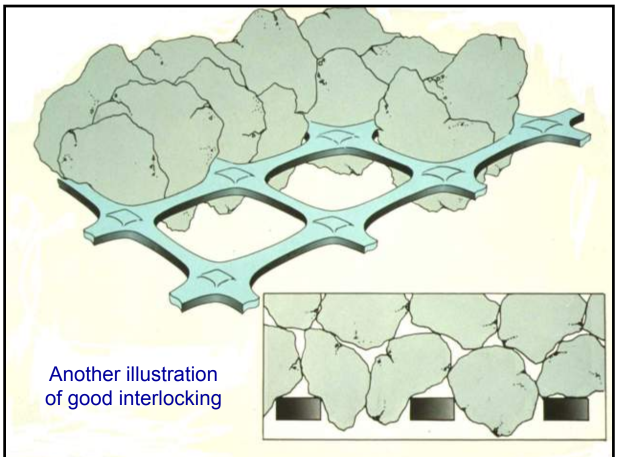
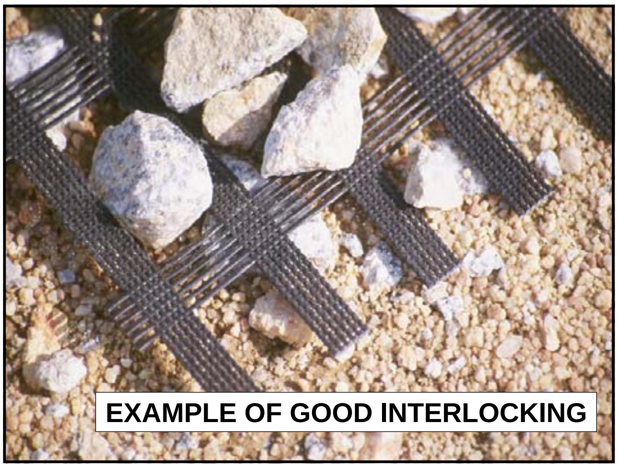
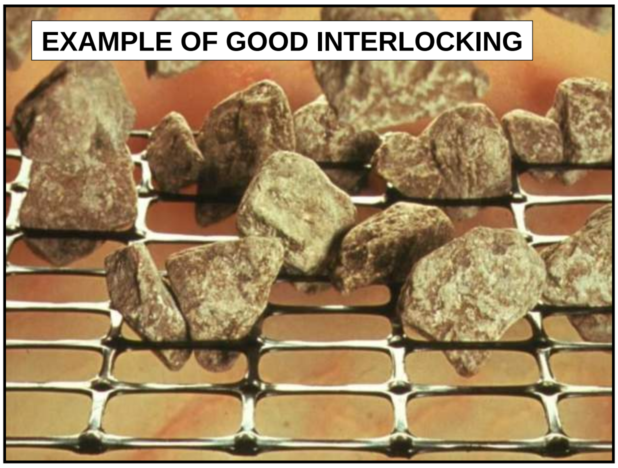


Another illustration of good interlocking





PARAMETERS OF INTERLOCKING

- **Geogrid aperture size relative to aggregate size.**
- **Shape and stiffness of transverse ribs.**
- **Strength of junction between perpendicular ribs.**

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The reinforcement function of a geosynthetic is more effective if there is less **relative displacement** between the geosynthetic and the soil to be reinforced.

If there is good interlocking between a geogrid and soil, it is believed that the relative displacement required to mobilize interlocking is less than the relative displacement required to mobilize interface friction (which is the other interface mechanism).

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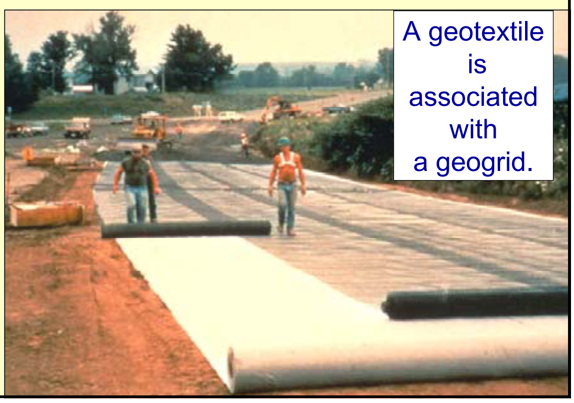
SEPARATION & REINFORCEMENT



SIMPLE RELATIONSHIP BETWEEN

Properties & FUNCTIONS

Thickness

TRANSMISSION

Permeability

FILTRATION

Retention

SEPARATION

Strength

REINFORCEMENT

Interface

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The functions reviewed so far do not include the fluid barrier function performed by geomembranes.

(This function will be discussed at the end.)

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After this review of functions, let's talk about applications of geosynthetics.

In a given application, a given geosynthetic will often perform several functions.

As a result, it is not rational to organize by functions.

Therefore, applications should be reviewed, and the functions identified

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APPLICATIONS OF GEOSYNTHETICS IN ROADS

THREE CATEGORIES:

- **Applications in road foundation**
- **Applications in road structure**
- **Applications in controlling water**

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APPLICATIONS OF GEOSYNTHETICS IN ROADS

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- **Applications in road structure**
- **Applications in controlling water**

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APPLICATIONS OF GEOSYNTHETICS IN ROAD FOUNDATION

- **Embankment on soft soil**
- **Cavity bridging**

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EMBANKMENT ON SOFT SOIL

The geotextile performs two functions:

- **SEPARATION**
- **REINFORCEMENT**

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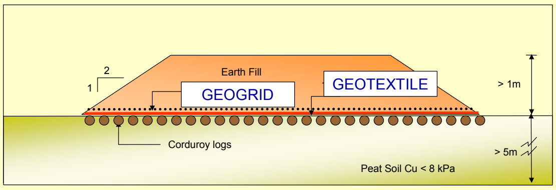
RAPP Airstrip Project in Indonesia



Here, the geotextile acts as a separator
between the soil
that could pass through the openings of the geogrid
and the logs.

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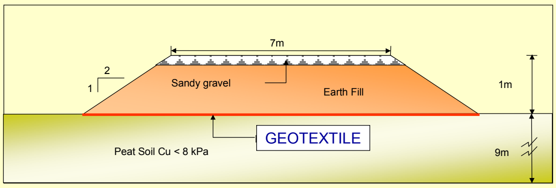
Here a geotextile will be used as a separator
between the embankment soil
and a very coarse material, the logs.

Road Construction on Peat Soil in Indonesia



Here, the geotextile may act as a separator
between the earth fill and the peat.
However, it certainly acts as reinforcement.

This is one of
the most typical uses of geotextiles.

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The reinforcement function
is often needed
as soon as
the beginning of construction.

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CONSTRUCTION ON SOFT SOIL

CONSTRUCTION ON SOFT SOIL

CONSTRUCTION ON SOFT SOIL

CONSTRUCTION ON SOFT SOIL

CONSTRUCTION ON SOFT SOIL

**In some other cases,
the field situation is better
and
reinforcement is only needed
for the long term.**

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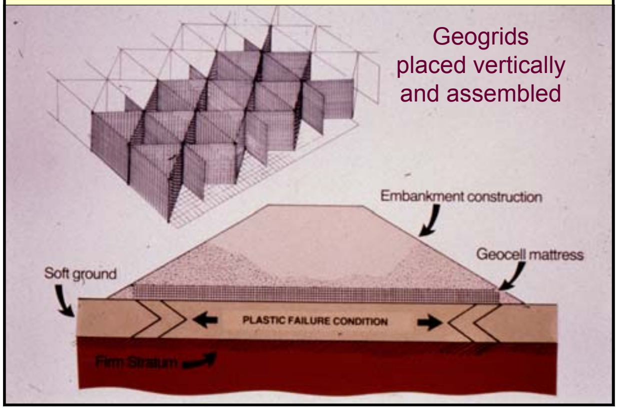
Another way of using geogrids at the base of an embankment

MATTRESS OF GEOGRID CELLS



The geogrids are positioned vertically and tensioned.

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Steel bodkins are inserted to form joints.

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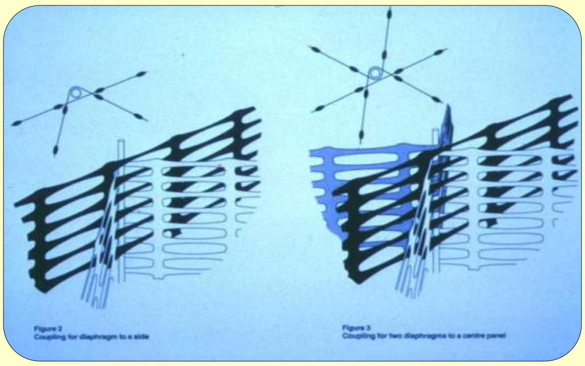
STEEL BODKIN JOINTS



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Geogrid cells ready for backfilling

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BACKFILLING EQUIPMENT SUPPORTED BY BACKFILLED CELLS

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DISCUSSION

OF THE

REINFORCEMENT FUNCTION

IN

EMBANKMENTS ON SOFT SOIL

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EMBANKMENT FAILURE

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An embankment on soft soil reinforced with geosynthetic is a complex composite structure with three components:

- **The embankment**
- **The geosynthetic reinforcement**
- **The foundation soil**

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Therefore, failure mechanisms can be complex.

However, simple failure mechanisms are traditionally considered.

We will review the simple mechanisms, and describe their use and limitations.

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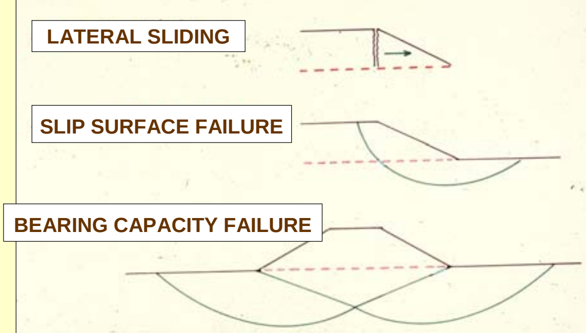
THE THREE FAILURE MECHANISMS



Bonaparte, Holtz, and Giroud (1987)

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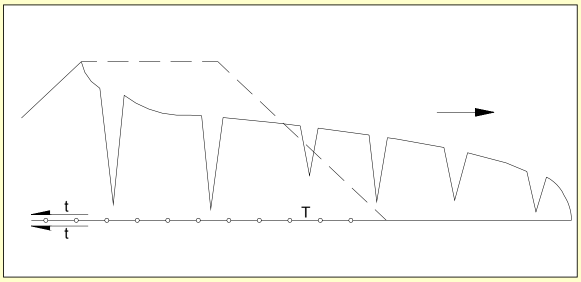
Lateral Sliding



The geosynthetic restrains the lateral movement of the embankment.

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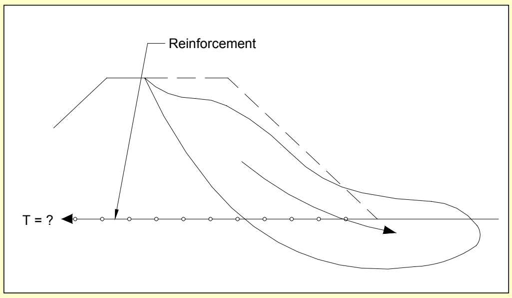
Slip Surface Failure



The geosynthetic provides a tensile force that contributes to stability.

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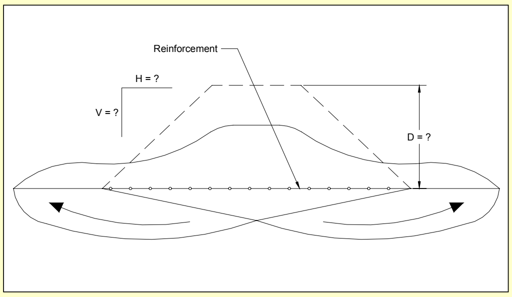
Bearing Capacity Failure



The geosynthetic keeps the embankment together, so it acts as a foundation.

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BEARING CAPACITY EVALUATION

- **The reinforcement properties are not involved in the evaluation.**
- **It is simply assumed that the embankment acts as a block having an equivalent width, *b*.**

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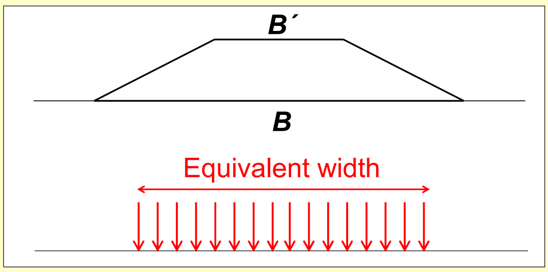
EQUIVALENT WIDTH

It is defined as the width of a uniform load that is “equivalent” to the non-uniform load applied by the embankment.



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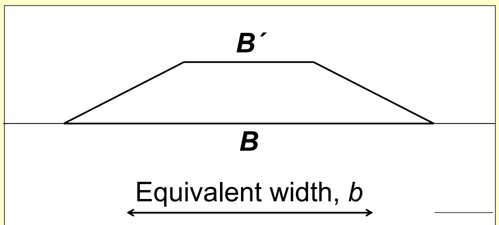
FIRST IDEA: AVERAGE



$$b = \frac{B + B'}{2} = B - \frac{H}{\tan\beta}$$

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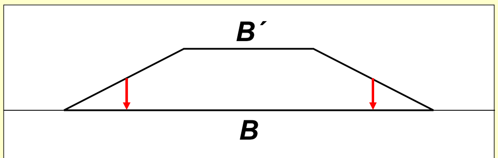
SECOND IDEA

Normal stress at edge is $(\pi + 2)c_o$



Equivalent width, b

$$b = B - \frac{2(\pi + 2)c_o}{\gamma \tan\beta}$$

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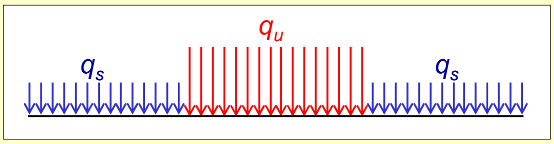
BEARING CAPACITY EQUATION



$q_u = c_o N_c + q_s$

where:

q_u = bearing capacity

q_s = lateral surcharge (usually zero)

c_o = cohesion (undrained shear strength) at surface

N_c = bearing capacity coefficient

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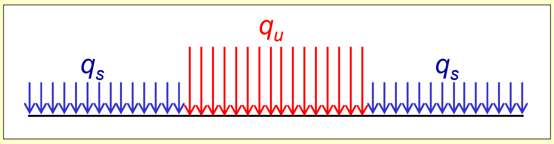
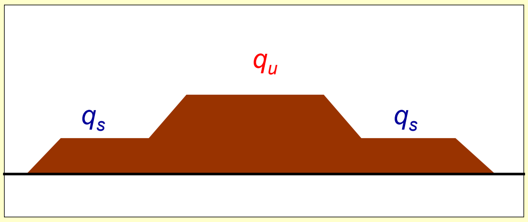
LATERAL SURCHARGE (BERM)





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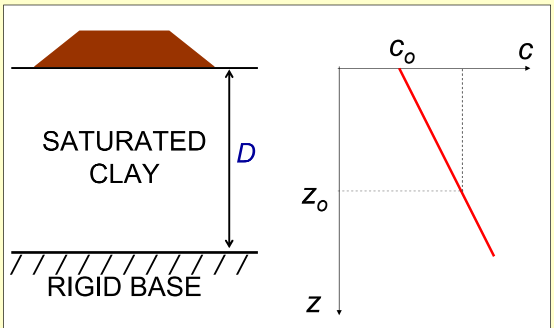

$$c = c_o + \lambda z = c_o\left(1 + \frac{z}{z_o}\right)$$

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BEARING CAPACITY COEFFICIENT

The values of N_c are used in the following equation to calculate the ultimate load, q_u, or "bearing capacity".

$$q_u = c_o N_c + q_s$$

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VALUE OF N_c

For uniform cohesion ($c = c_o$) over depth greater than 0.71 b, the classical value:

$$N_c = \pi + 2 = 5.14$$

used in the classical equation:

$$q_u = c N_c + q_s$$

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VALUE OF N_c

FOR COHESION THAT VARIES LINEARLY WITH DEPTH

If b/D ≤ 2 $\quad N_c = \pi + 2 = 5.14$

D = bedrock depth

If 2 ≤ b/D ≤ 14.32

$$4.14 + \left(\frac{1}{2}\right)\left(\frac{b}{D}\right) \le N_c \le 11.3 + 0.384\,\frac{b}{z_o}$$

If b/D ≥14.32 $\quad N_c \approx 11.3 + 0.384\,\dfrac{b}{z_o}$

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The preceding equations show that
(compared to the case of a uniform foundation soil)
the **bearing capacity**
is **significantly increased** when:

- the soft soil layer has a finite thickness and is underlain by a rigid base; and
- the undrained shear strength increases with depth.

We will see that these two conditions also increase the factor of safety against rotational failure.

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BEARING CAPACITY FACTOR OF SAFETY

$$FS_{BC} = \frac{q_u}{q_a}$$

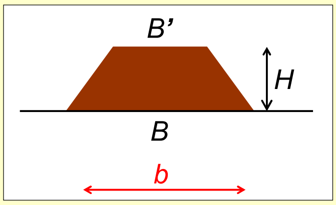
TYPICAL VALUE
***FS* = 1.5 TO 2**

$q_u = c_o\,N_c + q_s$

Often $q_s = 0$

$q_a = \dfrac{W}{b}$

$W = \gamma H \left(\dfrac{B + B'}{2}\right)$



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Remember:
The reinforcement properties are not involved in bearing capacity evaluation.

Therefore, if the factor of safety with respect to bearing capacity is not sufficient, increasing reinforcement strength will not solve the problem.

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If the factor of safety with respect to bearing capacity is not sufficient, solutions include:

- **DECREASE THE TOTAL LOAD (light weight embankment)**

101

Slip surface failure and bearing capacity failure can be avoided by using lightweight embankment.

If the factor of safety with respect to bearing capacity is not sufficient, solutions include:

- DECREASE THE TOTAL LOAD (light weight embankment)
- **INCREASE THE RATIO *b / D* (wider embankment)**

103

For cohesion that varies linearly with depth, **N_c increases as the ratio *b / D* increases**.

If b/D ≤ 2

$$N_c = \pi + 2 = 5.14$$

If 2 ≤ b/D ≤ 14.32

$$4.14 + \left(\frac{1}{2}\right)\left(\frac{b}{D}\right) \le N_c \le 11.3 + 0.384\frac{b}{z_o}$$

If b/D ≥14.32

$$N_c \approx 11.3 + 0.384\frac{b}{z_o}$$

104

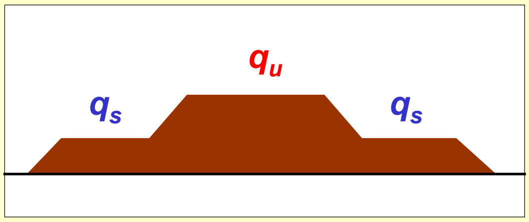
If the factor of safety with respect to bearing capacity is not sufficient, solutions include:

- DECREASE THE TOTAL LOAD (light weight embankment)
- INCREASE THE RATIO *b / D* (wider embankment)
- **ADD BERMS. Term q_s in equation.**

105

LATERAL SURCHARGE (BERM)

$$q_u = c_o\, N_c + q_s$$

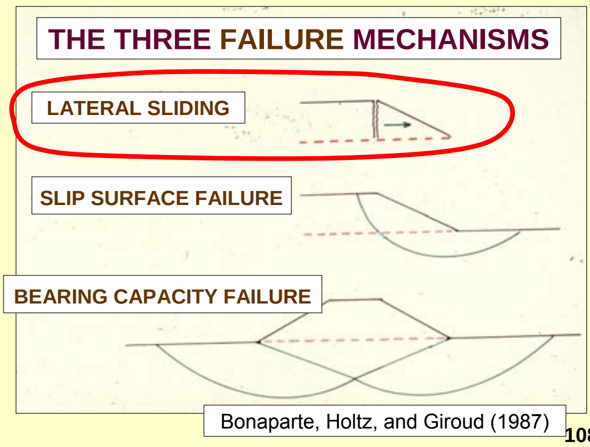


106

If the factor of safety with respect to bearing capacity is not sufficient, solutions include:

- DECREASE THE TOTAL LOAD (light weight embankment)
- INCREASE THE RATIO *b / D* (wider embankment)
- ADD BERMS. Term q_s in equation.
- **IMPROVE THE FOUNDATION SOIL (e.g. by consolidation, with low construction rate and/or vertical drains)**

107



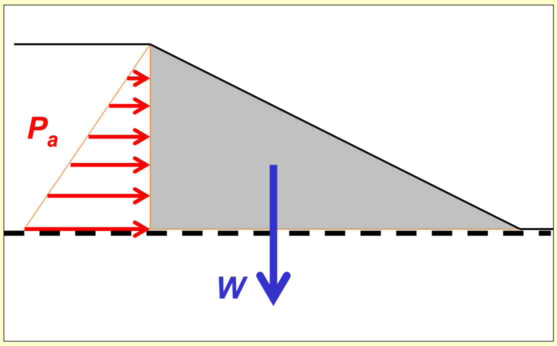
108

SLIDING EVALUATION

- **Sliding is caused by the active pressure exerted by one part of the embankment on the rest of the embankment.**
- **A parametric study shows that the worst location is at the embankment crest.**

109

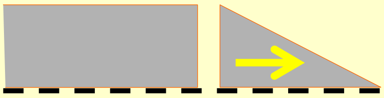
ACTIVE PRESSURE AND RESISTING WEIGHT



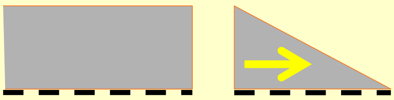
110

Two modes of sliding rupture are typically considered.

SLIDING OF FILL ON THE GEOSYNTHETIC



RUPTURE OF THE GEOSYNTHETIC
AND SLIDING OF THE GEOSYNTHETIC
ON THE FOUNDATION SOIL



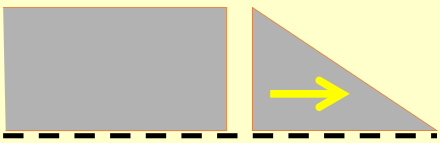
111

In fact, these two modes of sliding are related.

Therefore, they will be presented together.

112

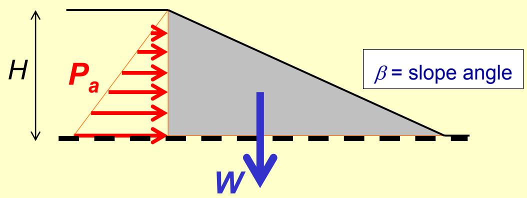
First, sliding between embankment and geosynthetic



Forces are analyzed on the next slide

113

First, sliding between embankment and geosynthetic



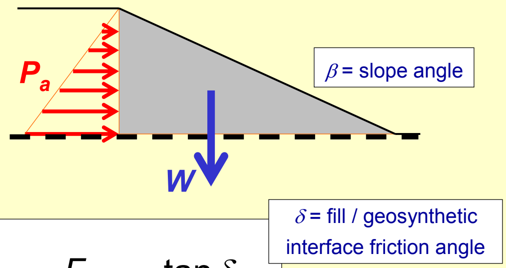
δ = fill / geosynthetic interface friction angle

Driving force: $P_a = \left(\frac{1}{2}\right) K_a \gamma H^2$

Resisting force: $F_r = W \tan\delta = \left(\frac{1}{2}\right) \gamma H^2 \frac{\tan\delta}{\tan\beta}$

114

Factor of safety for sliding between embankment and geosynthetic



$$FS_{Sup} = \frac{F_r}{P_a} = \frac{\tan\delta}{K_a \tan\beta}$$

115

$$FS = \frac{F_r}{P_a} = \frac{\tan\delta}{K_a \tan\beta}$$

Typical target value
FS = 1.5

DISCUSSION OF THE EQUATION

δ **CAN BE** close to the friction angle of the fill

β *is* generally less than the friction angle of the fill

K_a is generally small

$$K_a = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right) \approx 0.25 - 0.35$$

Therefore, ***FS* is generally large.**
(unless a slippery geosynthetic is used)

116

This explains why sliding of embankment on top of geosynthetic reinforcement is very rare.

117

$$FS = \frac{F_r}{P_a} = \frac{\tan\delta}{K_a \tan\beta}$$

If *FS* is not sufficient:

- Increase interface friction angle, δ
- Decrease slope angle, β
- Increase internal friction angle, ϕ, which decreases K_a since:

$$K_a = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

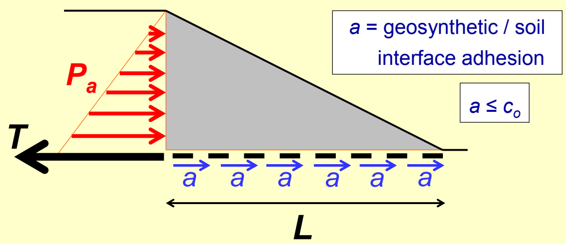
118

If *FS* with respect to sliding above the geosynthetic is sufficient, the driving force (due to lateral active pressure) is transmitted to the geosynthetic reinforcement.

As a result, the geosynthetic is under tension.

119

In this case, potential sliding is between geosynthetic and foundation soil



Driving force: $P_a = \left(\frac{1}{2}\right) K_a \gamma H^2$

Resisting force: $F_r = T + aL$

$$FS_{Sdown} = \frac{F_r}{P_a} = \frac{aL + T}{\left(\frac{1}{2}\right) K_a \gamma H^2}$$

120

The preceding equations can be used to determine the tension in the geosynthetic:

$$T = \left(\frac{1}{2}\right) K_a \gamma H^2 - aL$$

Then, it is possible to define a factor of safety with respect to the geosynthetic:

$$FS_G = \frac{T_{allow}}{T} = \frac{T_{allow}}{\left(\frac{1}{2}\right) K_a \gamma H^2 - aL}$$

This illustrates that there are several ways of defining the factor of safety.

121

Regardless of the factor of safety used, the factor of safety against sliding between the geosynthetic and the foundation soil

or the factor of safety with respect to the geosynthetic,

$$FS_{Sdown} = \frac{F_r}{P_a} = \frac{aL + T}{\left(\frac{1}{2}\right) K_a \gamma H^2}$$

$$FS_G = \frac{T_{allow}}{T} = \frac{T_{allow}}{\left(\frac{1}{2}\right) K_a \gamma H^2 - aL}$$

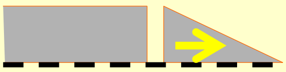
if the factor of safety is not sufficient, either the slope should be less steep or (more efficiently) the geosynthetic tension should be increased.

122

Two modes of sliding rupture are typically considered.

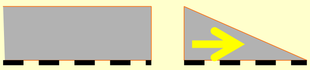
We started with this case

SLIDING OF FILL ON THE GEOSYNTHETIC



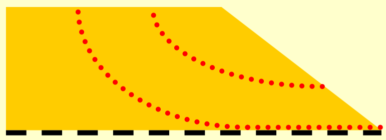
and we just analyzed this case

RUPTURE OF THE GEOSYNTHETIC AND SLIDING OF THE GEOSYNTHETIC ON THE FOUNDATION SOIL

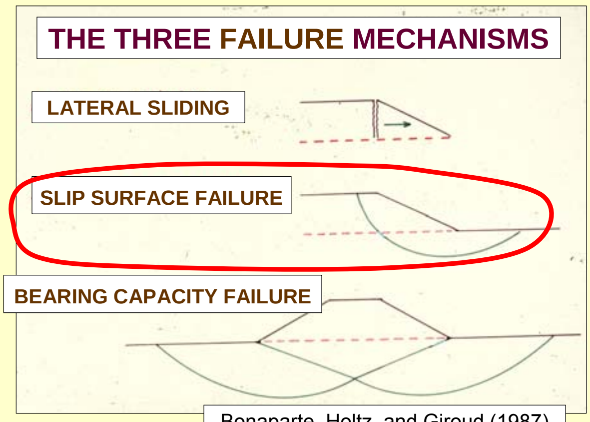


123

The two-step sliding analysis described in the preceding slides can be complemented by internal stability analysis using slope stability computer program with slip surfaces entirely in the fill or partly in the fill, partly along the reinforcement geosynthetic.



124



Bonaparte, Holtz, and Giroud (1987)

125

ROTATIONAL FAILURE EVALUATION

- **Rotational failure is caused by the weight of the embankment.**
- **The resisting moment is provided by the soil shear strength and the geosynthetic reinforcement strength.**
- **This is different from bearing capacity (where the reinforcement plays no role) and different from sliding analysis where the geosynthetic may be detrimental.**

126

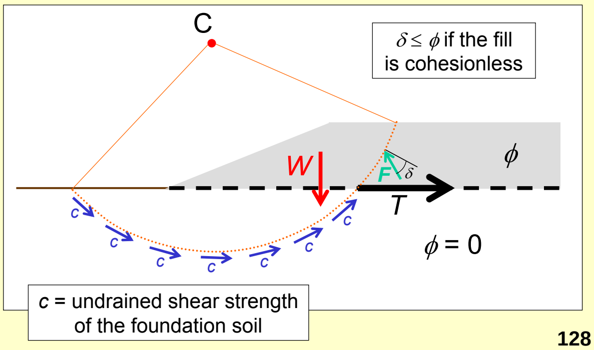
ROTATIONAL FAILURE EVALUATION

- **Rotational failure analysis is typically performed using a computer program for slope stability analysis.**
- **However, it is important to review the steps of the calculation to better understand and better use computer programs.**

127

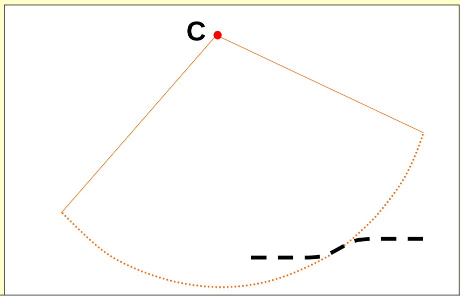
ROTATIONAL FAILURE

A circular slip surface is assumed.



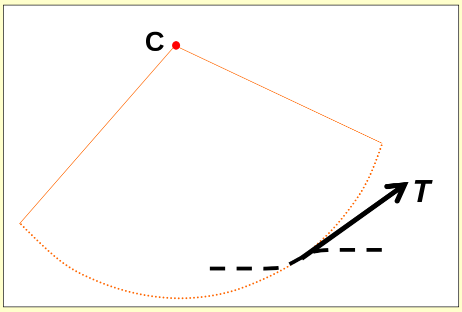
128

GEOSYNTHETIC TENSION ORIENTATION



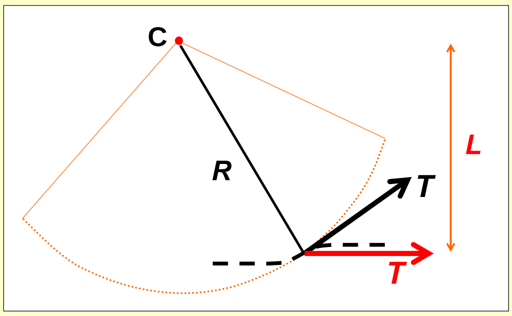
When the rotational movement starts, the geosynthetic deforms locally and tends to become tangent to the slip surface.

129



Therefore it is tempting to consider geosynthetic tension tangent to the slip surface.

130



However, this is unconservative ($TR > TL$). Furthermore, this may require significant embankment deformation.

131

FACTOR OF SAFETY WITH RESPECT TO ROTATIONAL FAILURE

$$FS = \frac{M_{RESISTING}}{M_{DRIVING}} = \frac{M_C + M_F + M_T}{M_W}$$

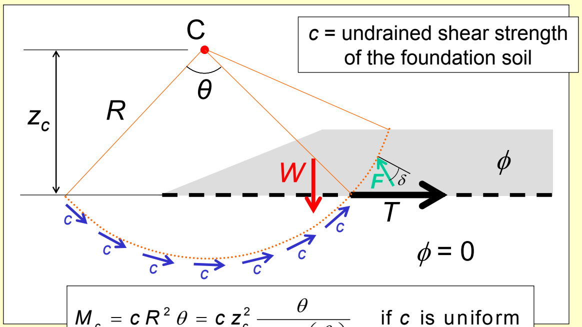
The moment, M_C, is due to the shear strength of the foundation soil.

The moment, M_F, due to the fill is small and can be neglected to account for the case of a cracked embankment.

The moment, M_T, due to the reinforcement is large if the slip surface is shallow.

132

The moment M_c can be obtained analytically in simple, but usual, cases.



$$M_c = c R^2 \theta = c z_c^2 \frac{\theta}{\cos^2\left(\frac{\theta}{2}\right)} \quad \text{if } c \text{ is uniform}$$

133

If the shear strength increases linearly with depth, the moment M_c can be calculated as follows:

$$M_c = c_o R^2 \theta \left\{ 1 + \frac{R}{z_o} \left[\frac{\sin\left(\frac{\theta}{2}\right)}{\frac{\theta}{2}} - \cos\left(\frac{\theta}{2}\right) \right] \right\}$$

$$= c_o z_c^2 \frac{\theta}{\cos^2\left(\frac{\theta}{2}\right)} \left\{ 1 + \frac{z_c}{z_o} \left[\frac{\tan\left(\frac{\theta}{2}\right)}{\frac{\theta}{2}} - 1 \right] \right\}$$

Adapted by Giroud from Rowe and Li

134

FACTOR OF SAFETY WITH RESPECT TO ROTATIONAL FAILURE

$$FS = \frac{M_{RESISTING}}{M_{DRIVING}} = \frac{M_C + M_F + M_T}{M_W}$$

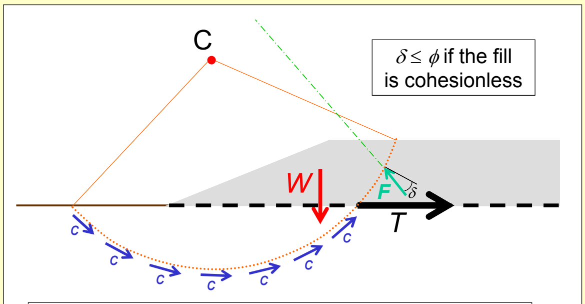
The moment, M_C, is due to the shear strength of the foundation soil.

The moment, M_F, due to the fill is small and can be neglected to account for the case of a cracked embankment.

The moment, M_T, due to the reinforcement is large if the slip surface is shallow.

135

The force, F, provided by the fill can be evaluated using active pressure theory .



The green line is close to the center, C, therefore, the moment of F with respect to the center O is small.

136

FACTOR OF SAFETY WITH RESPECT TO ROTATIONAL FAILURE

$$FS = \frac{M_{RESISTING}}{M_{DRIVING}} = \frac{M_C + M_F + M_T}{M_W}$$

The moment, M_C, is due to the shear strength of the foundation soil.

The moment, M_F, due to the fill is small and can be neglected to account for the case of a cracked embankment.

The moment, M_T, is due to the reinforcement tension.

137

The value of T to be used in the calculation of the moment is the minimum of the following values:

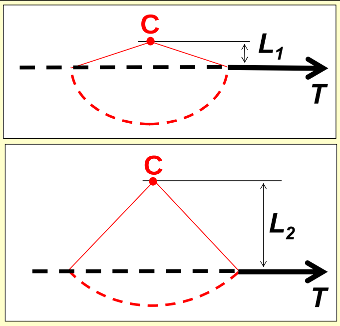
- An allowable value of the tension equal to the ultimate value divided by a partial factor of safety
- The value of the tension that corresponds to an allowable strain
- The value of the pullout force (from inside or outside the slip surface)

138

The moment, M_T,
due to the reinforcement
depends on the position
of the center of the slip surface.

The moment is large
if the center is high,
i.e. if the slip surface is shallow.

139



Deep slip surface
(low center)

Shallow slip surface
(high center)

$L_2 > L_1$ hence $L_2 T > L_1 T$

The moment is greater if the center is high.

140

Reinforcement is more effective
if the center of the circular slip surface
is high above the reinforcement,
i.e. if the slip surface is shallow.
This happens if:

- **the undrained shear strength of the foundation soil increases with depth; and/or**
- **if there is a rigid base at a depth relatively small compared to the embankment width.**

141

The influence of cohesion increasing with depth
is more important than the presence of a rigid base.

If the cohesion of the foundation soil is uniform
and if the depth of the rigid base
is greater than
b / 2 (half the equivalent width of the embankment)
or greater than
0.84 times the embankment crest width,
the reinforcement (even very strong) is not
effective. (Rowe & Soderman 1985, 1987).

This is because the slip surface is then very deep.

This is consistent with comments on bearing capacity.

142

Considering the fact that
embankments are usually wide,
there is a high probability
for having a foundation soil
with shear strength
that increases with depth
and /or
having a rigid base.

143

We already saw that bearing capacity
is increased by one or both of these two conditions
(rigid base and cohesion increasing with depth).

Indeed, there is a similarity
between bearing capacity and rotational failure:
both involve slip surfaces into the foundation soil.

However, there is an additional reason
in the case of rotational failure:
the factor of safety increase is due to the fact that
reinforcement is more effective
in presence of one or both of these two conditions
(rigid base and cohesion increasing with depth).

144

Therefore, in a preliminary phase of design of an embankment on soft soil, it is very important to identify and quantify the increase of shear strength with depth and to identify the presence of a rigid base.

145

FACTOR OF SAFETY WITH RESPECT TO ROTATIONAL FAILURE

$$FS = \frac{M_{RESISTING}}{M_{DRIVING}} = \frac{M_C + M_F + M_T}{M_W}$$

The target factor of safety with respect to rotational failure is typically 1.3 to 1.5.

All potential slip surfaces should be tried until the minimum factor of safety is found. The trials should not be limited to circular surfaces.

146

What if the factor of safety with respect to rotational failure is not sufficient ?

$$FS = \frac{M_{RESISTING}}{M_{DRIVING}} = \frac{M_C + M_F + M_T}{M_W}$$

The equation shows that we need to increase the reinforcement tension or decrease the embankment weight either by decreasing the embankment height or by using lightweight fill.

147

Increasing reinforcement strength can be achieved by using two or more layers of geosynthetics.

However, the strengths of two or more geosynthetics can be added only if their strains are similar.

The simplest way is to use two or more identical geosynthetics.

CAUTION: Risk of slippage between two geosynthetics

148

There is a limit beyond which it is useless to increase the reinforcement strength.

This limit is the bearing capacity of the foundation soil.

This is not surprising because, as discussed earlier, the bearing capacity of the foundation soil is independent of the magnitude of reinforcement.

149

We discussed embankment design based on **the three classical failure mechanisms:**

- **bearing capacity failure,**
- **sliding, and**
- **rotational failure.**

As a result of the above discussions, it may be more appropriate to organize the discussions in accordance with **three types of analyses:**

- **external stability,**
- **internal stability, and**
- **mixed stability.**

150

This would lead to an elegant summary:

- **EXTERNAL STABILITY.** Reinforcement plays no role.
- **INTERNAL STABILITY.** Reinforcement may be detrimental.
- **MIXED STABILITY.** Reinforcement is beneficial.

151

The analyses for bearing capacity, lateral sliding and rotational failure (or the ideal analyses for external stability, internal stability and mixed stability) are based on **limit equilibrium**. Therefore, **deformations are not considered** in these analyses.

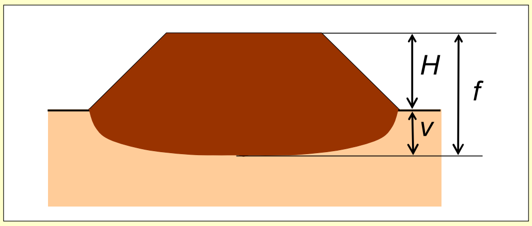
More sophisticated analyses involving quantification of deformations require extensive calculations and can only be done using numerical methods (finite elements or finite differences).

152

Numerical studies with quantification of deformations show that excessive vertical displacement may limit the height of the embankment at a value lower than that dictated by rotational failure.

153

CONCEPT OF NET HEIGHT



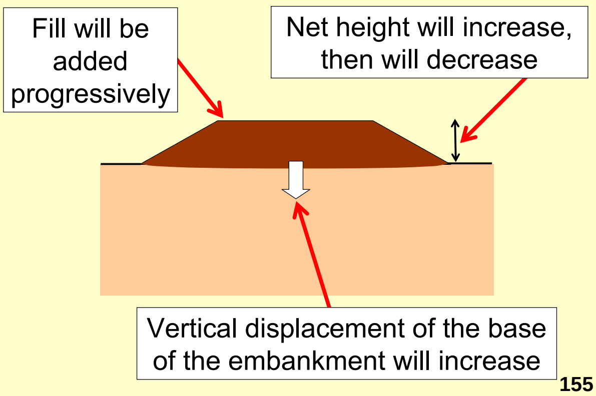
$$H = f - v$$

Net height = Fill height - Vertical displacement

After Rowe & Soderman, 1987

154

PROGRESSIVE INCREASE OF FILL THICKNESS



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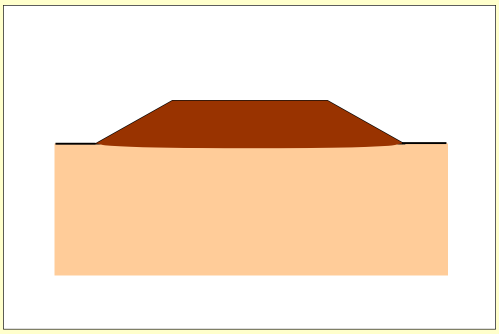
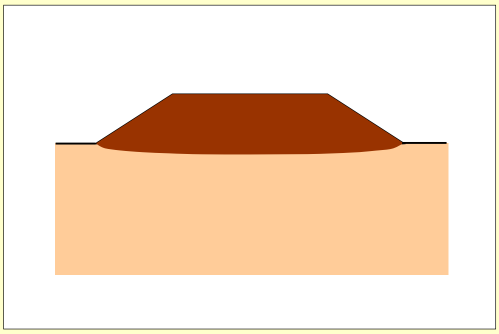
PROGRESSIVE INCREASE OF FILL THICKNESS



156

PROGRESSIVE INCREASE OF FILL THICKNESS



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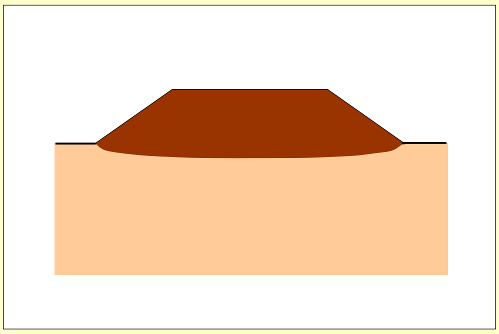
PROGRESSIVE INCREASE OF FILL THICKNESS



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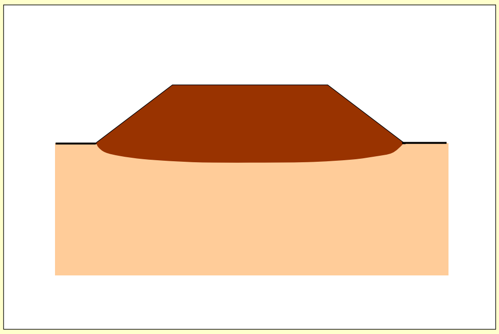
PROGRESSIVE INCREASE OF FILL THICKNESS



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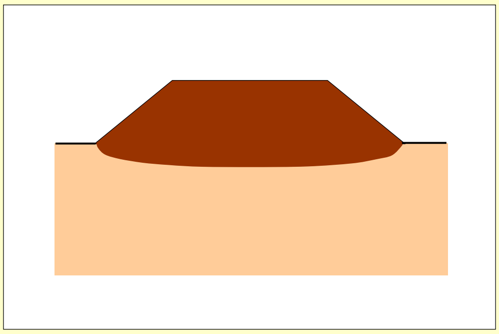
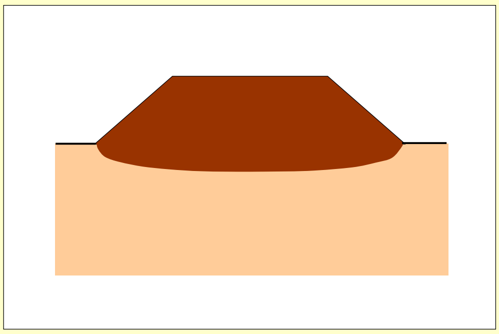
PROGRESSIVE INCREASE OF FILL THICKNESS



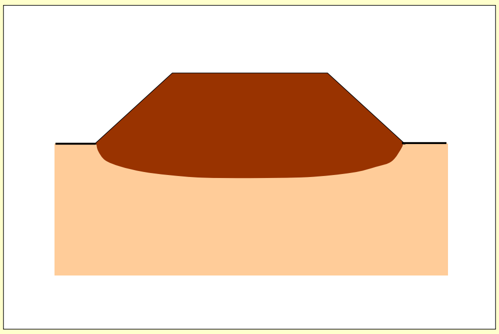
160

PROGRESSIVE INCREASE OF FILL THICKNESS



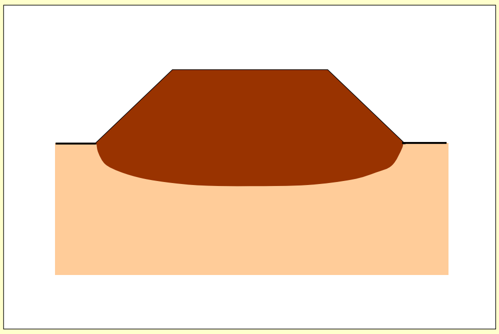
161

PROGRESSIVE INCREASE OF FILL THICKNESS



162

PROGRESSIVE INCREASE OF FILL THICKNESS



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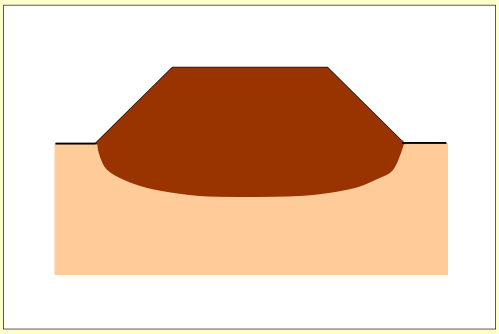
PROGRESSIVE INCREASE OF FILL THICKNESS



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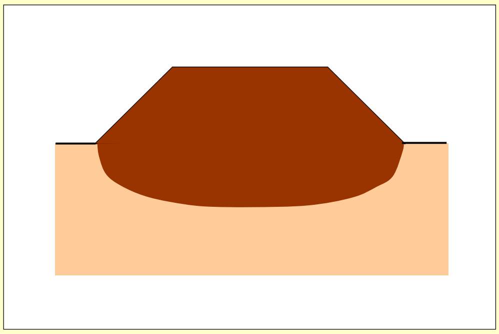
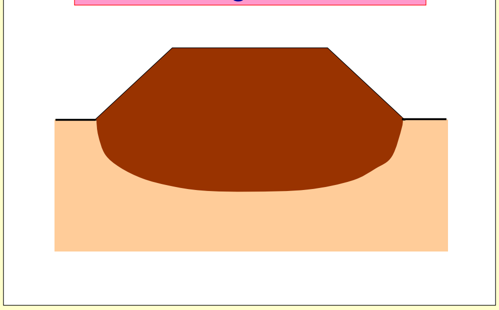
PROGRESSIVE INCREASE OF FILL THICKNESS



165

PROGRESSIVE INCREASE OF FILL THICKNESS

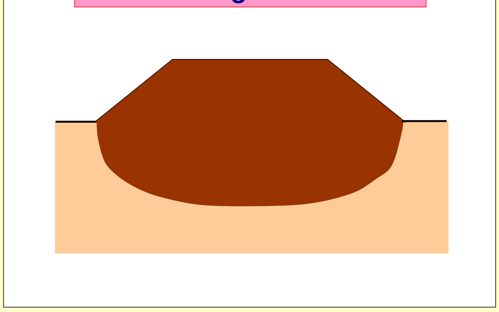
The net height decreases



166

PROGRESSIVE INCREASE OF FILL THICKNESS

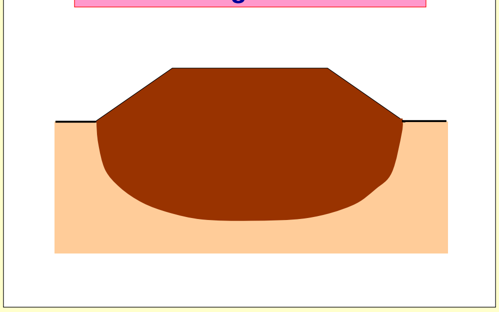
The net height decreases



167

PROGRESSIVE INCREASE OF FILL THICKNESS

The net height decreases



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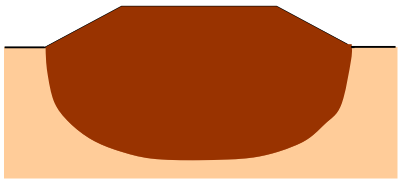
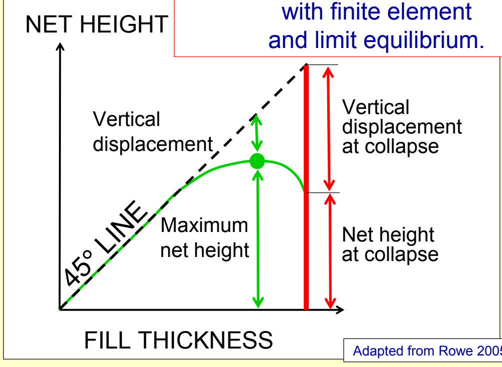
PROGRESSIVE INCREASE OF FILL THICKNESS

The net height decreases



169

Collapse of the embankment is obtained for the same fill thickness with finite element and limit equilibrium.



Adapted from Rowe 2005

170

It is satisfactory that the same value of embankment height at collapse is obtained with numerical analysis (finite element method) and with limit equilibrium analysis using the geosynthetic tension that correspond to the strain obtained with the finite element model.

171

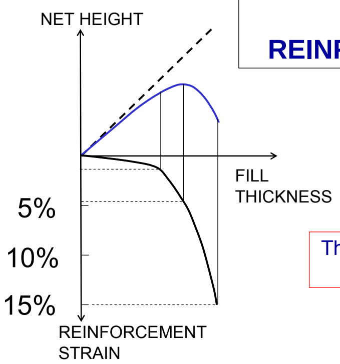
In conclusion, the geosynthetic tension used in rotational slip surface analysis should be selected by conducting a preliminary finite element calculation or field measurements.

This is very cumbersome. Unfortunately, it seems difficult to give general recommendations.

Strains in geosynthetic reinforcement in embankments on soft soil are typically from 0% to 8%.

172

STRAIN IN REINFORCEMENT



This is only an example, not a general result.

Adapted from Rowe 2005

173

COMPARISON THEORY/ FIELD MEASUREMENTS

Tensions and strains measured in geosynthetic reinforcement in the field are often smaller than predicted. One should not conclude that design methods are inadequate.

174

Potential reasons for measured tensions and strains in the field smaller than predicted are:

- Underestimation of undrained shear strength due to disturbance of samples
- Conservatism in design (low boundary of strength, high boundary of loads)
- Increase of undrained shear strength due to consolidation of foundation soil
- Confined geosynthetic in the field stronger than unconfined geosynthetic in laboratory

175

EXAMPLES OF STRAIN MEASUREMENT IN GEOGRID

- Polyester geogrid
  0.5% end of construction
  1.0% later (but 2.5% predicted)
- Polyethylene geogrid
  2.0% end of construction
  3.0% later
- These are only examples.

This leads to a comment on the effect of time.

176

EFFECT OF TIME ON BEHAVIOR OF REINFORCED EMBANKMENTS

- BENEFICIAL EFFECT
  Consolidation:
  dissipation of excess pore pressure
  increase in strength
- DETRIMENTAL EFFECT
  Creep of foundation soil and geosynthetic:
  decrease in strength and increase in strains

177

The smaller the allowable strain, the smaller the deformation of the embankment.

To have at the same time significant reinforcement and small deformation, a geosynthetic with high modulus should be used.

$$T = J\,\varepsilon$$

178

This leads to an important question.

Does reinforcement have an impact on embankment settlement ?

The answer depends on the definition of settlement.

The most general definition is considered herein:

"settlement is the vertical component of the displacement of the ground surface caused by load applied at the surface "

179

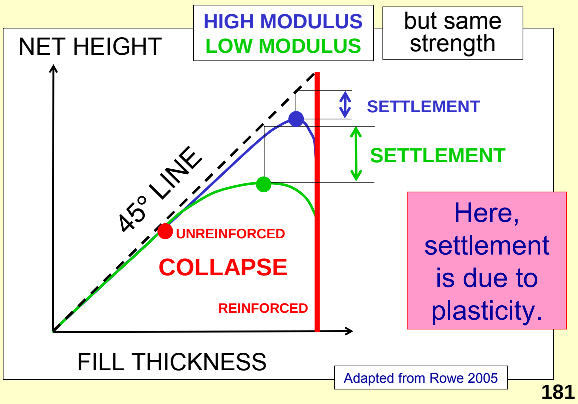
This broad definition includes two categories of settlement:

1. **Settlement related to stress-strain and stress-strain-time relationships.**
   This includes immediate settlement (without compression in low-permeability saturated soils and with compression in other soils) and deferred settlement (consolidation, creep)
2. **Settlement related to yielding of the soil due to the development of zones of plasticity in the soil, as the load is increased.**

Sometimes, the term "settlement" is restricted to the first category.

180

INFLUENCE OF GEOSYNTHETIC MODULUS



181

Clearly, geosynthetic reinforcement
has an impact on settlement,
and the impact is greater
(i.e. settlement is smaller)
if the geosynthetic has a higher modulus.
However, this impact is mostly limited
to settlement related to the development
of zones of plasticity in the foundation soil.

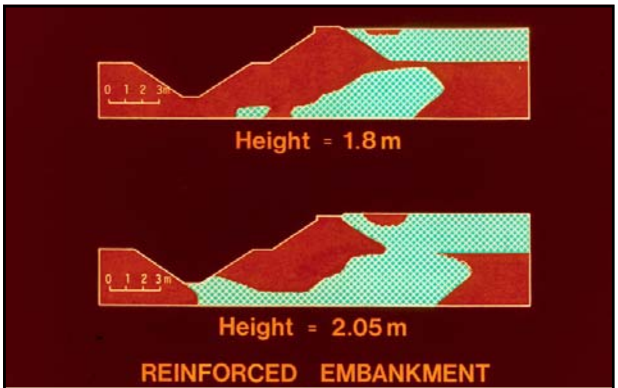
This is because the role of the geosynthetic,
as discussed in numerous preceding slides,
is to act as failure develops,
i.e. when zones of plasticity develop
in the foundation soil.

182

DEVELOPMENT OF PLASTIC ZONES

The plastic zone becomes continuous
(which causes collapse)
when the embankment height is 1.8 m.

183



A greater embankment height is achieved.

184

In the case of settlement
related to stress-strain
and stress-strain-time relationships,
geosynthetic reinforcement
has some impact on stress distribution.
Therefore, it has some impact
on settlement related to stress-strain
and stress-strain-time relationships.

However, this impact is not significant,
as the main function of the geosynthetic
is to act as failure develops
(see preceding slide).

185

If settlement related to stress-strain and stress-strain-time relationships
needs to be reduced,
lightweight fill can be considered.

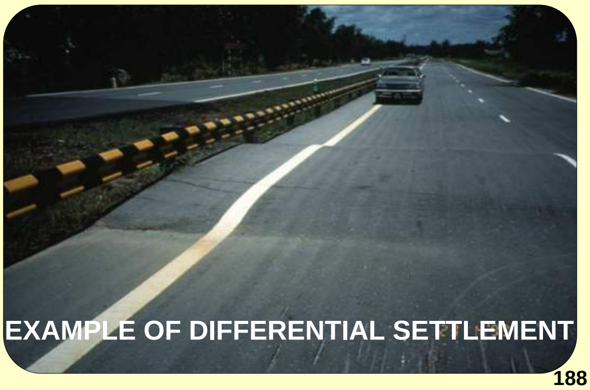
186

**More generally,
lightweight fill is beneficial
for all mechanisms
caused by the weight
of the embankment:**

- **Bearing capacity failure.**
- **Rotational failure.**
- **Settlement**

187

Now, what about differential settlement ?



188

**In several preceding slides,
we discussed
the influence
of geosynthetic reinforcement
on the magnitude of settlement.**

**Now, we will discuss
the influence
of geosynthetic reinforcement
on the distribution of
settlement.**

189

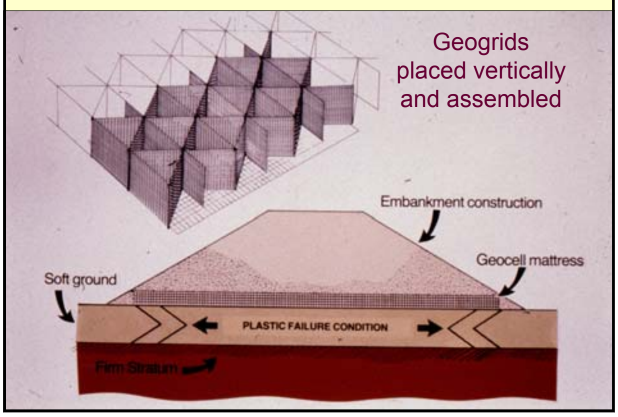
**Geosynthetics may have a significant impact
on differential settlement
when the foundation soil is not uniform.**

This is the case in particular
when geosynthetics stiffen the base of the embankment,
which makes the distribution of settlement more uniform.

**Examples:
geogrid cells or multiple layers of geogrids
at the base of embankments.**

190

MATTRESS OF GEOGRID CELLS



191

**CONCLUSION ON EFFECT OF REINFORCEMENT
ON VERTICAL DISPLACEMENT (SETTLEMENT)**

- **Reinforcement reduces displacements (including vertical displacement) that are due to large plastic deformations of the foundation soil that precede collapse.**
- **Reinforcement has minor effect on vertical displacement (i.e. settlement) related to stress-strain-time relationships, in particular consolidation settlement and secondary (creep) settlement.**
- **By stiffening the base of the embankment, reinforcement reduces differential settlement.**

192

EMBANKMENT ON PEAT

- Preceding discussions were related to foundation soil made of clay or clayey soils.
- Peat is another kind of weak foundation soil, but its characteristics and behavior are very different from those of clay.

193

TYPES OF PEAT

- Organic soils.
- TRUE PEATS. Characterized by
  high water content 100 - 1000%
  high porosity 0.75 - 0.95 (3 < e < 30)
  hence high compressibility
  fibrous structure with large voids
  hence high permeability
  (at least before compression)

194

With usual construction rates, excess pore pressure tends to dissipate during construction in the case of true peat.

As a result:

- Undrained failure mechanisms (rotational failure, bearing capacity failure) are rare with true peat.
- Undrained shear strength analysis is not adequate with true peat.

195

TYPICAL FAILURE OF EMBANKMENT ON PEAT

- No rotational failure with definite slip surface.
- No bearing capacity failure.
- Rapid collapse with excessive shear deformation resulting in large displacements (vertical and lateral movement).

196

DESIGN OF EMBANKMENTS ON PEAT

- Peat can be considered as purely frictional material with ϕ = 26 to 29°.
- Effective stress analysis with evaluation of pore pressure.
- Prediction of pore pressure is very difficult. Therefore, pore pressure measurements during construction are necessary to control the rate of construction. This requires good piezometers.

197

Peat deposits are often underlain by soft clay, which leads to complex failure mechanisms.

198

CONCLUSION

BENEFITS OF GEOSYNTHETIC REINFORCEMENT

- **Exhibits ductile and progressive failure rather than brittle and rapid failure**
- **Achieves higher embankment by increasing factor of safety**
- **Mobilizes bearing capacity by keeping embankment as a monolith**
- **Reduces plastic deformation of foundation soil (vertical and lateral)**
- **Reduces differential settlement**
- **May allow faster construction**

199

APPLICATIONS OF GEOSYNTHETICS

IN

ROAD FOUNDATION

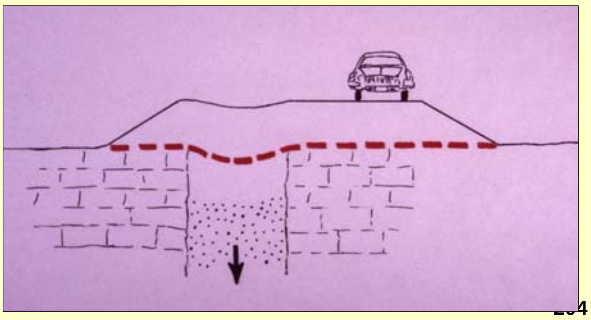
- Embankment on soft soil
- **Cavity bridging**

200

Courtesy M. Dobie

Courtesy J.P. Gourc

One or several layers of high-strength geosynthetic can be used to bridge a cavity.



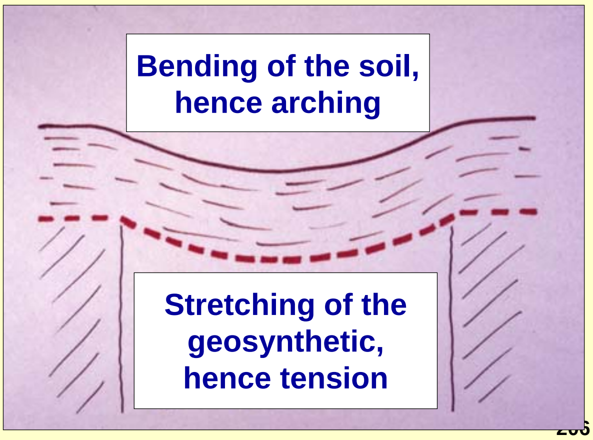
204

CAVITY BRIDGING

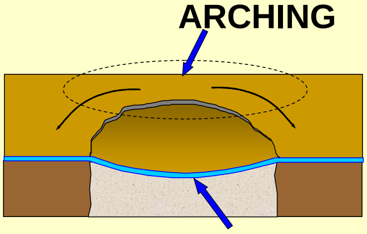
The geosynthetic performs

THE REINFORCEMENT FUNCTION

205

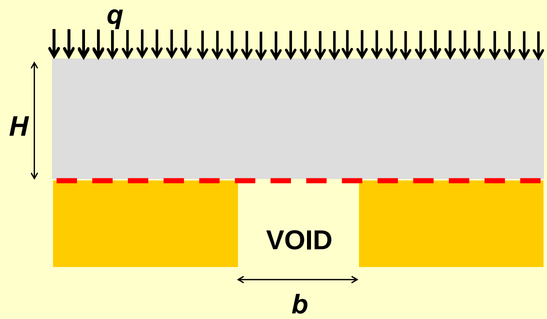


206



Thanks to arching,
only part of the overburden load
is applied to the geosynthetic.

207



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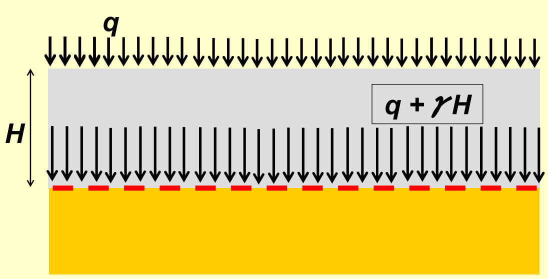
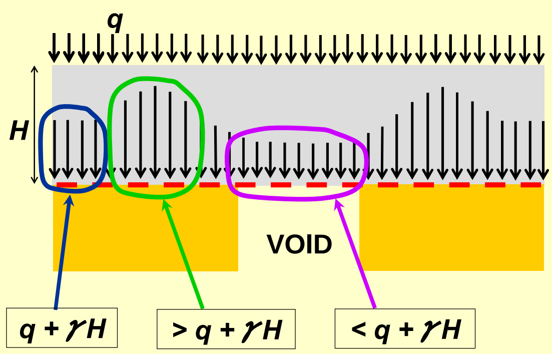


Case where there is no void

The pressure on the geosynthetic is uniform.

209



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EQUATIONS FOR VOID BRIDGING

The equations will be presented for the case of an infinitely long void.

The equations for the case of a circular void are very similar.

211

ARCHING EQUATIONS

Pressure on the geosynthetic:

$$p = \frac{\gamma b}{2K \tan\phi}\left[1 - e^{-2K \tan\phi\, H/b}\right] + q\, e^{-2K \tan\phi\, H/b}$$

It has been shown (Giroud et al. 1990) that:

$$K \tan\phi \approx 0.25 \quad \text{if}\ \phi \geq 20^\circ$$

hence:

$$p = 2\gamma b\left[1 - e^{-0.5\, H/b}\right] + q e^{-0.5\, H/b}$$

212

TENSIONED MEMBRANE EQUATIONS

Strain: $\varepsilon = 2\Omega \sin^{-1}\left[1/(2\Omega)\right] - 1$

Tension: $T = p b \Omega$

where: $\Omega = \left(\frac{1}{4}\right)\left(\frac{2y}{b} + \frac{b}{2y}\right)$

ε = geosynthetic strain

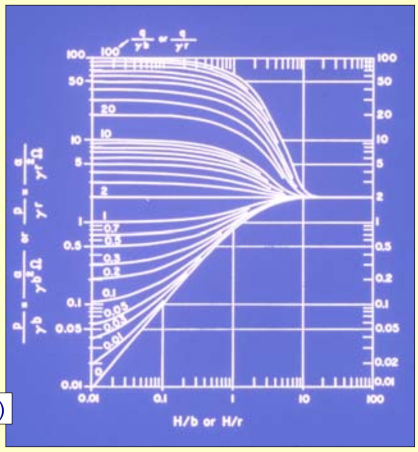
y = geosynthetic deflection

T = geosynthetic tension

Combining all equations
(arching and tension membrane)
leads to a chart.

213

DESIGN METHOD ACCOUNTING FOR BOTH ARCHING IN SOIL AND TENSIONED MEMBRANE IN GEOSYNTHETIC



(Giroud et al. 1990)

So far, the functions were easy to identify.

However, in road structures, functions are more complex.

215

APPLICATIONS OF GEOSYNTHETICS IN ROADS

THREE CATEGORIES:

- Applications in road foundation
- **Applications in road structure**
- Applications in controlling water

216

APPLICATIONS OF GEOSYNTHETICS

IN ROAD STRUCTURE

- Unpaved roads
- Paved roads
- Asphalt overlay

217

GEOSYNTHETICS

IN

UNPAVED ROADS

218

UNPAVED ROAD

WITHOUT GEOSYNTHETIC

220

UNPAVED ROAD

WITH GEOTEXTILE

221

UNPAVED

ROAD

WITH

GEOGRID

222

UNPAVED ROADS

The geosynthetic performs two functions:

- **SEPARATION**
- **REINFORCEMENT**

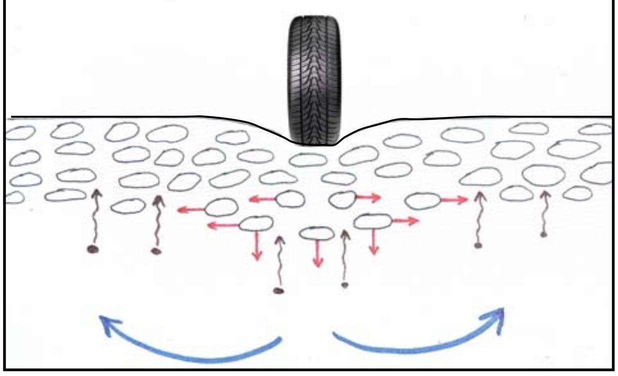
223

There is obviously a beneficial effect of separation when geotextiles are used in unpaved roads.

224

SEPARATION WITH GEOTEXTILE

MECHANISMS OF DETERIORATION



There is obviously a beneficial effect of separation when geotextiles are used in unpaved roads.

Also, there is obviously a beneficial effect of reinforcement, with both geotextiles and geogrids. But, how does it work?

227

REINFORCEMENT FUNCTION IN UNPAVED ROADS

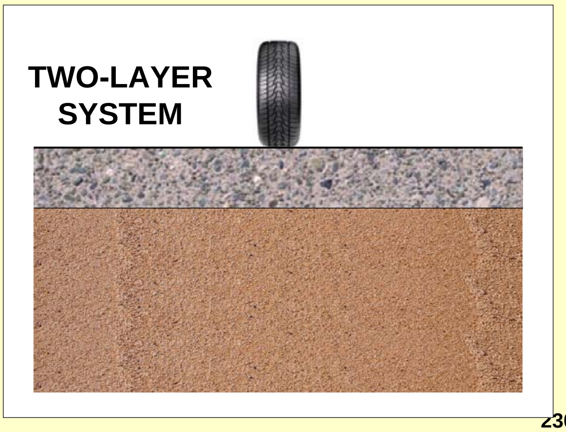
- **Load distribution**
- **Tensioned membrane**
- **Subgrade confinement**

228

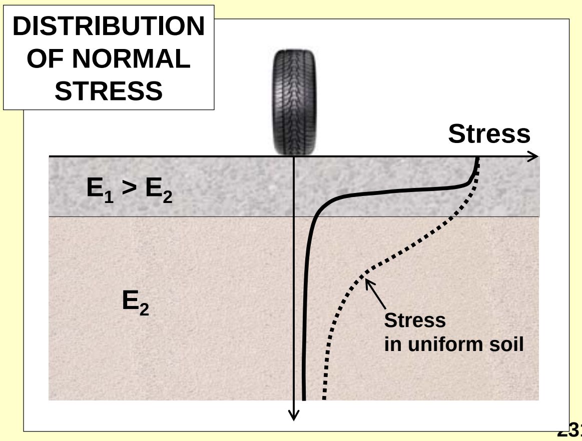
LOAD DISTRIBUTION

It is known from the theory of elasticity that,
in a two-layer system,
the **load distribution** on the lower layer
depends on the modulus of the upper layer.

229



230



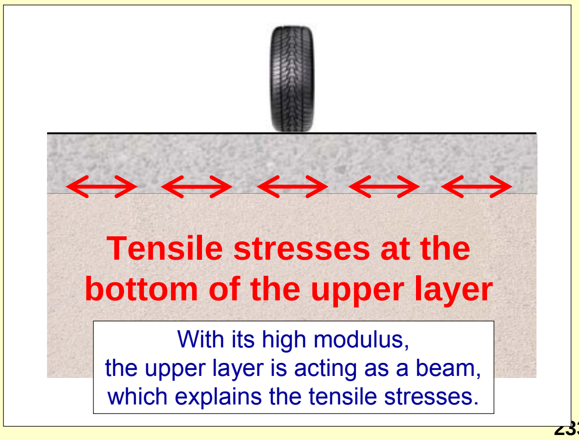
231

LOAD DISTRIBUTION

It is known from the theory of elasticity that,
in a two-layer system,
the distribution of load on the lower layer
depends on the modulus of the upper layer.

It is also known from the theory of elasticity
that there are **tensile stresses**
at the bottom of the upper layer,
which limits
the load distribution effectiveness.

232



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LOAD DISTRIBUTION (continued)

Therefore, the load distribution effectiveness
of the upper layer
can be increased
by **adding tensile stiffness**
at the bottom of the upper layer.

Hence the use of **reinforcement**
at the bottom of the upper layer,
which provides **lateral restraint**.

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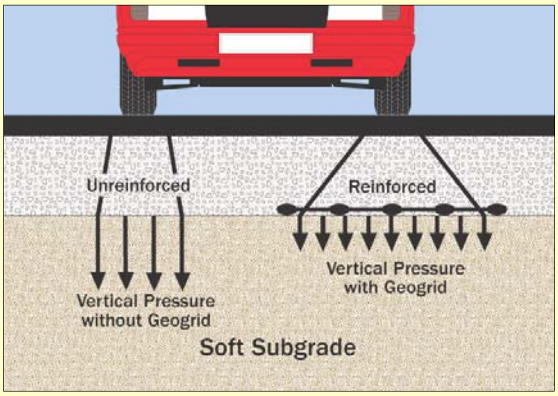
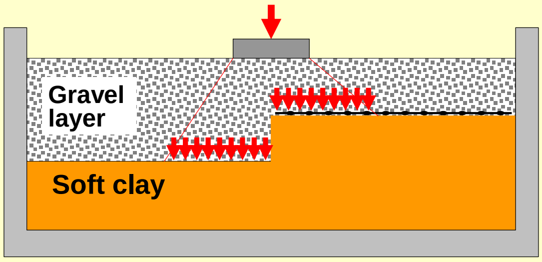


ILLUSTRATION OF LOAD DISTRIBUTION 235

LOAD DISTRIBUTION



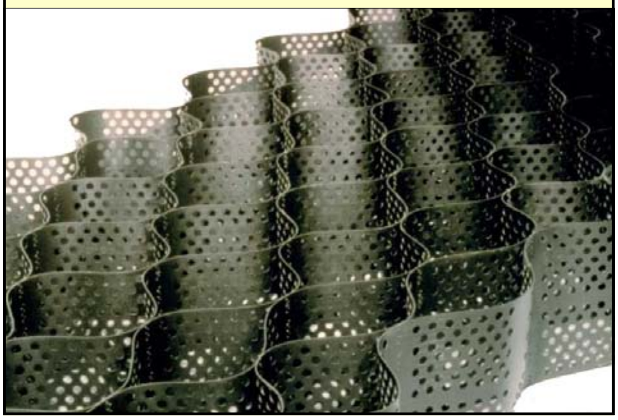
Thinner gravel layer with reinforcement results in same stress as thicker gravel layer without reinforcement .

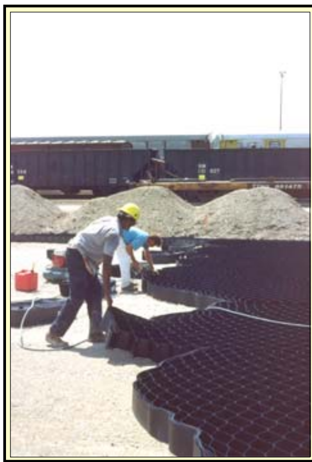
236

Load distribution can also be achieved by geocells.

237

GEOCELLS





GEOCELLS BEING DEPLOYED

239

GEOCELLS FILLED WITH AGGREGATE

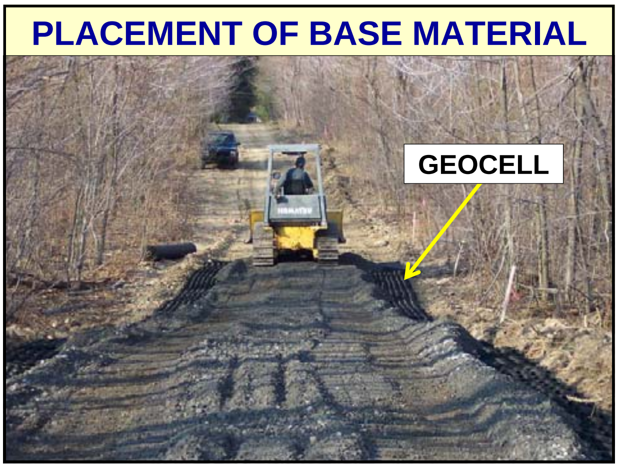
240


GEOCELL
MATTRESS
FILLED
WITH
AGGREGATE
AND
SUPPORTING
LOADS
241


UNPAVED ROAD
TO BE CONSTRUCTED
USING GEOCELLS


PLACEMENT OF GEOTEXTILE


PLACEMENT OF GEOCELL


PLACEMENT OF BASE MATERIAL
GEOCELL


UNPAVED ROAD COMPLETED

REINFORCEMENT FUNCTION IN UNPAVED ROADS

- Load distribution
- **Tensioned membrane**
- Subgrade confinement

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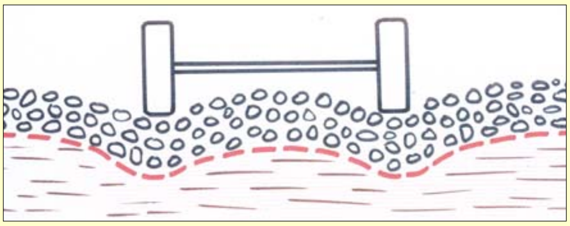
TENSIONED MEMBRANE EFFECT

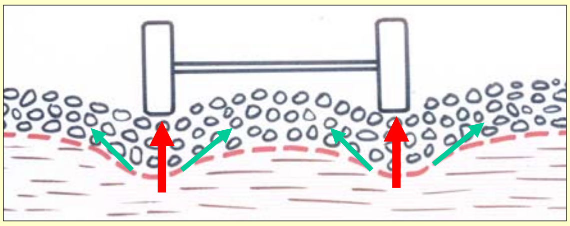


Due to the traffic loads, the geotextile is deformed and is, therefore, under tension.

Under the wheels, due to rutting, the geotextile has a concave shape.

248

TENSIONED MEMBRANE EFFECT



The geotextile tension on each side of the concave shape is shown in green.

The resultants of these tensions are shown in red. These resultants contribute to wheel support.

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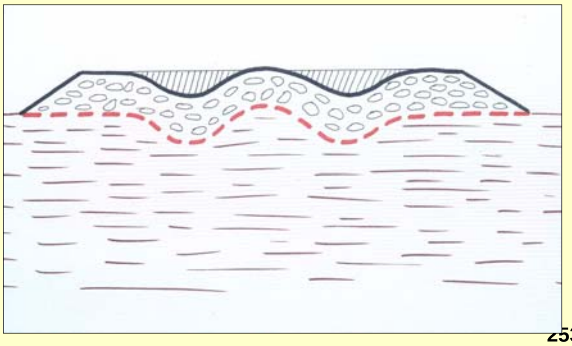
RUTTING IN THE FIELD

LIMITATIONS OF THE TENSIONED MEMBRANE EFFECT

- **The tensioned membrane effect is relatively small.**
- **The tensioned membrane effect works only with channelized traffic.**

252

Typically, ruts are periodically backfilled.



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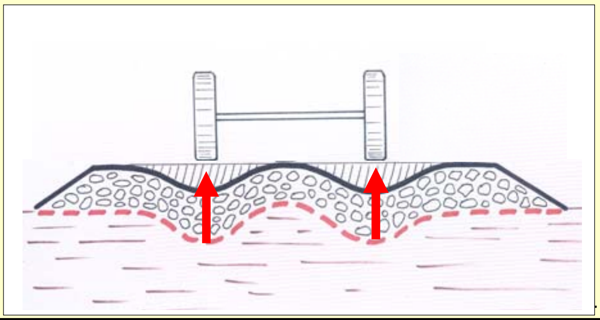
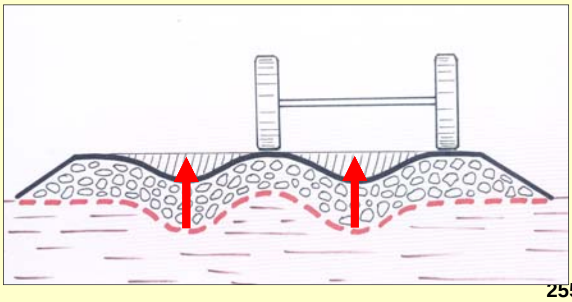
If the traffic continues to be channelized, the tensioned membrane effect continues to contribute to wheel support.



If the traffic is no longer channelized, the tensioned membrane effect does not contribute to wheel support.



255

Traffic is not channelized in the case of unpaved areas (area stabilization, log yards, etc.).

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AREA STABILIZATION

LOG YARD

Traffic may not be channelized in unpaved roads.

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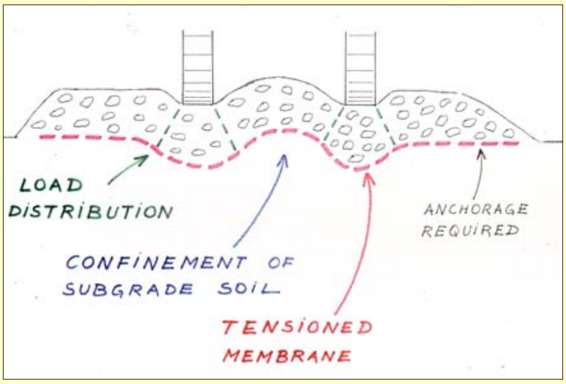
REINFORCEMENT FUNCTION
IN
UNPAVED ROADS

- Load distribution
- Tensioned membrane
- **Subgrade confinement**

260



261

Geotextile confining the subgrade soil

SUBGRADE CONFINEMENT

Thanks to the presence of the geosynthetic, the **deformations** of the soil are **limited**.

As a result, the soil can be loaded near its **ultimate bearing capacity**, and not only near its elastic limit.

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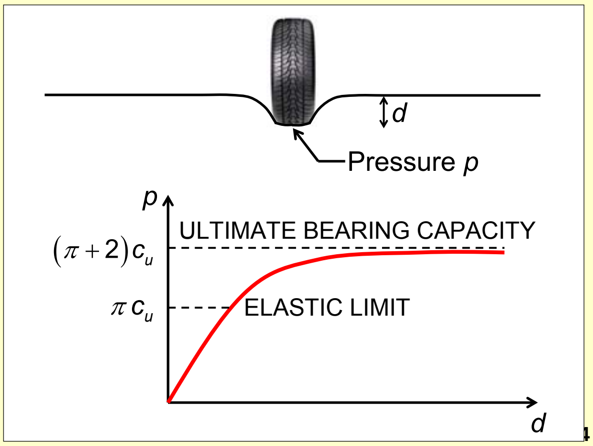


Without subgrade confinement, a load equal to the ultimate bearing capacity would cause **immediate failure**.
In other words, an unpaved road with no subgrade confinement by geosynthetic would fail at one axle pass if the load at the subgrade soil level is equal to the ultimate bearing capacity.

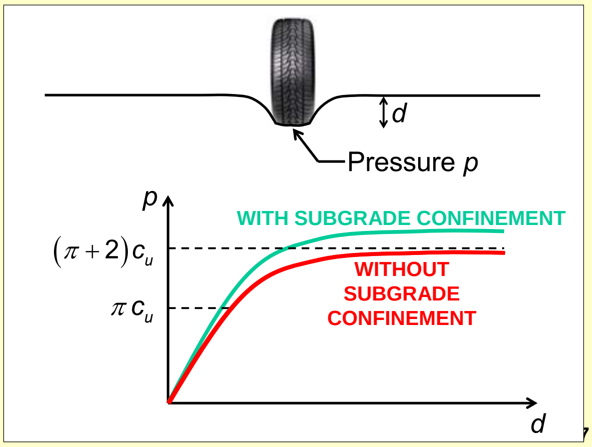
Therefore, unpaved roads without geosynthetic must be designed to avoid loads equal to the ultimate bearing capacity.
As a result, they **must be designed** for loads equal to the **elastic limit**.

265

Another approach to subgrade confinement is to consider that subgrade confinement is similar to tensioned membrane effect.

This results in a slight increase in bearing capacity of the subgrade soil, which allows the unpaved road to be safely designed with the ultimate bearing capacity.

266



SUBGRADE CONFINEMENT

Elastic limit: $p = \pi c_u$

Ultimate bearing capacity: (normal stress) $p = (\pi + 2) c_u$

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The usual equation for ultimate bearing capacity,

$$p = (\pi + 2) c_u$$

is applicable to the case of normal stress.
This is approximately the case of geotextile-reinforced unpaved roads.

In the case of geogrid-reinforced unpaved roads, the **stresses** at the base-subgrade interface are **inclined** (due to **lateral restraint**); as a result, the **bearing capacity** is slightly **increased**.

$$p = \left(\frac{3\pi}{2} + 1\right) c_u$$

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SUBGRADE CONFINEMENT

Elastic limit: $p = \pi c_u$

Ultimate bearing capacity: (normal stress) $p = (\pi + 2) c_u$

Ultimate bearing capacity: (inclined stress) $p = \left(\frac{3\pi}{2} + 1\right) c_u$

270

SUBGRADE CONFINEMENT

Elastic limit: $p = 3.14\,c_u$

Ultimate bearing capacity: (normal stress) $p = 5.14\,c_u$ +64%

Ultimate bearing capacity: (inclined stress) $p = 5.71\,c_u$ +82%

271

DEFORMATION ASSOCIATED WITH THE VARIOUS REINFORCEMENT MECHANISMS

Much less deformation (i.e. less rutting) is required to mobilize lateral restraint and load distribution than the tensioned membrane effect.

Consequence : lateral restraint will play an important role in paved roads.

272

GENERAL COMMENT ON ACTION OF REINFORCEMENT IN ROAD STRUCTURE

The mode of action of reinforcement in a road structure is complex because the working condition for the reinforcement is not ideal since the **load is vertical** while the **reinforcement is horizontal**.

This leads to a variety of modes of action such as lateral restraint, load distribution, tensioned membrane, subgrade confinement, etc.

273

IMPORTANT BENEFIT

Geosynthetic reinforcement in road applications (i.e. under embankments or in road structures) improves structure behavior by distributing stresses and bridging weak areas in the case on **non-uniform soils**.

This benefit is difficult to quantify, but it is real as it results from a combination of mechanisms such as cavity bridging, load distribution, tensioned membrane, subgrade confinement, etc.

This is an important benefit because non-uniform soils are frequent and unpredictable.

IMPORTANT BENEFIT IN THE CASE OF NON-UNIFORM SOILS

One aspect of this important benefit is the decrease of differential settlement in the case of non-uniform soils, as discussed earlier for embankments on soft soils.

275

There is a design method for unpaved roads that takes into account the mechanisms described above (except the tensioned membrane effect).

The Giroud – Han Design Method

6

Giroud – Han Design Method

- **Main features of design method**
  - **Theoretically based; uses bearing capacity theory and stress distribution**
  - **Calibrated using field data and cyclic plate load testing in the laboratory**

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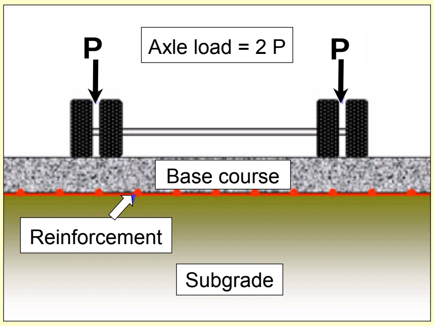

P is the load applied by a dual wheel
(as shown above)
or by a single wheel if the axis has only two wheels.

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GIROUD-HAN METHOD

$$h \geq \frac{r}{\tan\alpha}\left(\sqrt{\frac{P}{\pi r^2 m N_c c_u}} - 1\right)$$

h = required base thickness
α = load distribution angle
r = radius of load (equivalent to wheel or dual wheel)
m = bearing capacity mobilization coefficient

Unique equation
for unreinforced and reinforced unpaved roads.

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GIROUD-HAN METHOD

$$h \geq \frac{r}{\tan\alpha}\left(\sqrt{\frac{P}{\pi r^2 m N_c c_u}} - 1\right)$$

The bearing capacity coefficient, N_c,
depends on the type of reinforcement:

$N_c = 3.14$ no reinforcement
$N_c = 5.14$ geotextile reinforcement
$N_c = 5.71$ geogrid reinforcement

280

GIROUD-HAN METHOD

$$h \geq \frac{r}{\tan\alpha}\left(\sqrt{\frac{P}{\pi r^2 m N_c c_u}} - 1\right)$$

The bearing capacity mobilization coefficient, m,
quantifies the fact that only a fraction of the
bearing capacity of the subgrade soil is mobilized.

The bearing capacity mobilization coefficient
depends on the deformation of the subgrade soil.

281

The equation proposed for the
bearing capacity mobilization coefficient is:

$$m = \left(\frac{s}{f_s}\right)\left\{1 - \xi \exp\left[-\omega\left(\frac{r}{h}\right)^n\right]\right\}$$

s = allowable rut depth (at surface of base course)
$f_s = 75\,\text{mm}$
$\left.\begin{array}{l}\xi = \text{dimensionless coefficient}\\ \omega = \text{dimensionless coefficient}\\ n = \text{dimensionless coefficient}\end{array}\right\} \Leftarrow$ to be obtained by calibration

Limit case: $m = 1$ if $s = 75$ mm and $h = 0$

282

GIROUD-HAN METHOD

$$h \geq \frac{r}{\tan\alpha}\left(\sqrt{\frac{P}{\pi r^2 m N_c c_u}} - 1\right)$$

The stress distribution angle, α, depends on:

- **number of traffic passes**
- **subgrade and base properties**
- **geosynthetic properties**

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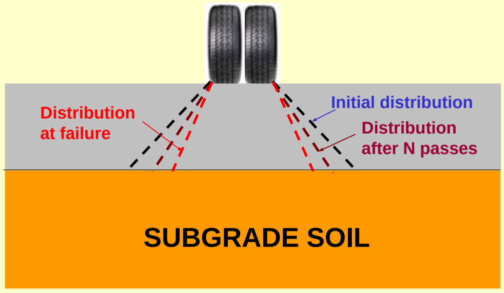
STRESS DISTRIBUTION ANGLE



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STRESS DISTRIBUTION ANGLE

$$\frac{1}{\tan\alpha} = \frac{0.868 + C \log N}{1 + 0.204(R_E - 1)}$$

Expression developed on the basis of stress distribution analysis and calibration with full-scale tests.

285

STRESS DISTRIBUTION ANGLE

$$\frac{1}{\tan\alpha} = \frac{0.868 + C \log N}{1 + 0.204(R_E - 1)}$$

C = term obtained through calibration

N = number of traffic passes

$$R_E = \min\left(\frac{E_{bc}}{E_{sg}}, 5\right) \quad \text{and} \quad \frac{E_{bc}}{E_{sg}} = \frac{3.48(CBR_{bc})^{0.3}}{CBR_{sg}}$$

E_{bc} = modulus of base course

E_{sg} = modulus of subgrade

CBR_{bc} = CBR of base course

CBR_{sg} = CBR of subgrade

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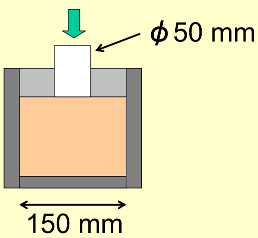
CBR Test



$$\text{CBR} = \frac{\text{Load at 2.5 mm displacement}}{\text{Standard load at 2.5 mm displacement In crushed stone}} 100\%$$

287

CALIBRATION FOR C

Example of calibration for a given geogrid:

$$C = (0.661 - 1.006 J^2)\left(\frac{r}{h}\right)^{1.5}$$

for geogrid reinforcement

where J = geogrid aperture stability modulus (m-N/°)

With J = 0 : $C = 0.661\left(\frac{r}{h}\right)^{1.5}$

for geotextile or for no reinforcement

288

GIROUD-HAN METHOD

$$h \geq \frac{r}{\tan\alpha}\left(\sqrt{\frac{P}{\pi r^2 m N_c c_u}} - 1\right)$$

The undrained cohesion of the subgrade soil, c_u, can be expressed as a function of the CBR of the subgrade soil:

$$c_u = f_c \; CBR_{sg}$$

with $f_c = 30$ kPa

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Combining all preceding equations, leads to the equation shown on the next slide.

It should be noted that some of the preceding equations are based on calibration performed for a given type of geogrid.

However, the Giroud-Han method is generic: it can be calibrated for other types of geosynthetics.

290

Giroud – Han Design Method

$$h = \frac{0.868 + (0.661 - 1.006 J^2)\left(\frac{r}{h}\right)^{1.5} \log N}{\left[1 + 0.204\left(\frac{3.48 CBR_{bc}^{0.3}}{CBR_{sg}} - 1\right)\right]} \left(\sqrt{\frac{\frac{P}{\pi r^2}}{\frac{s}{f_s}\left[1 - 0.9 e^{-\left\{\frac{r}{h}\right\}^2}\right] N_c \; f_c \; CBR_{sg}}} - 1\right) r$$

h = required thickness (m)
(on both sides of equation)

Input parameters:

- **P = wheel load (kN)**
- **r = radius of tire print (m)**
- **N = number of axle passes**
- **CBR_{bc} = base course CBR (%)**
- **CBR_{sg} = subgrade CBR (%)**

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Giroud – Han Design Method

$$h = \frac{0.868 + (0.661 - 1.006 J^2)\left(\frac{r}{h}\right)^{1.5} \log N}{\left[1 + 0.204\left(\frac{3.48 CBR_{bc}^{0.3}}{CBR_{sg}} - 1\right)\right]} \left(\sqrt{\frac{\frac{P}{\pi r^2}}{\frac{s}{f_s}\left[1 - 0.9 e^{-\left\{\frac{r}{h}\right\}^2}\right] N_c \; f_c \; CBR_{sg}}} - 1\right) r$$

- **s = maximum rut depth (mm)**
- **f_s = rut depth factor (= 75mm)**
- **J = geogrid aperture stability modulus (m-N/degree)**
- **N_c = bearing capacity factor**
  - **For unreinforced pavements, N_c = 3.14**
  - **For geotextile-reinforced pavements, N_c = 5.14**
  - **For geogrid-reinforced pavements, N_c = 5.71**
- **f_c = factor relating CBR of subgrade to equivalent c_u value (f_c = 30 i.e. c_u = 30 x CBR with c_u in kPa)**

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COMPARISON BETWEEN G-H AND G-N METHODS

G-H = Giroud and Han method (2003)
G-N = Giroud and Noiray method (1981)

BASE STRENGTH
G-H method: Taken into account through the stress distribution angle
G-N method: Not considered

STRESS DISTRIBUTION ANGLE
G-H method : Varies with base strength and number of traffic passes
G-N method: Fixed value

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COMPARISON BETWEEN G-H AND G-N METHODS

G-H = Giroud and Han method (2003)
G-N = Giroud and Noiray method (1981)

BASE THICKNESS REDUCTION
G-H method: Based on repeated loading
G-N method: Based on static loading

INFLUENCE OF RUT DEPTH
G-H method: Linked to stress-strain relationship
G-N method: Based on empirical relationship for paved roads

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COMPARISON BETWEEN G-H AND G-N METHODS

G-H = Giroud and Han method (2003)
G-N = Giroud and Noiray method (1981)

TENSIONED MEMBRANE EFFECT
G-H method: Not included
G-N method: Included

The tensioned membrane effect is significant only when the ruts are deeper than approximately 150 mm. (Giroud et al. 1985)

295

Also, it is important to note that the Giroud & Han method is expressed by a unique equation applicable to both unreinforced and reinforced unpaved roads.

In contrast, with the Giroud & Noiray method, two equations are necessary.

296

APPLICATIONS OF GEOSYNTHETICS

IN ROAD STRUCTURE

- **Unpaved roads**
- **Paved roads**
- **Asphalt overlay**

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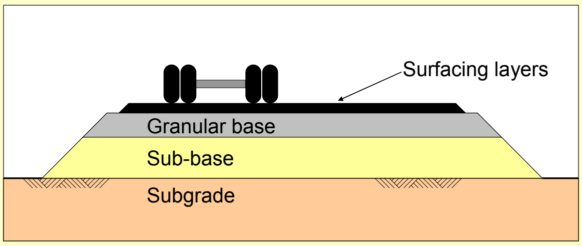
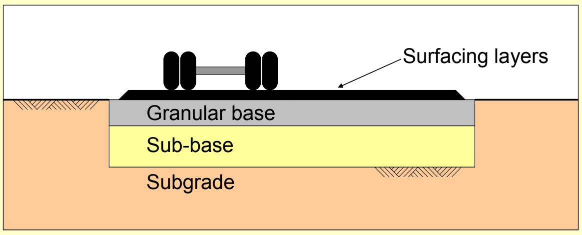
PAVED ROAD
IDEAL CROSS SECTION



Excellent lateral drainage

298

In many cases, the road structure is buried and lateral drainage is difficult.



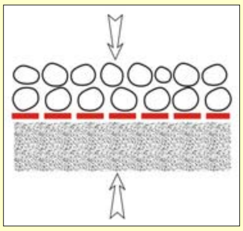
299

PAVED ROADS

The geotextile performs two functions:

- **SEPARATION**
- **REINFORCEMENT**

300



Geotextiles used for separation should resist damage by stones or other protruding objects.

They should have adequate tensile strength.

301

RESISTANCE TO DAMAGE BY STONES

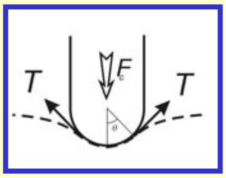
QUESTION:
Should the same tensile strength be specified for wovens and nonwovens?

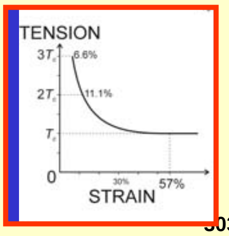
The answer is NO

NEXT QUESTION: on what basis should the different specifications be established?

302

A theoretical study of the action of concentrated stresses on a geotextile in contact with stones or other protruding objects has shown that the required tensile strength of the geotextile is given by a curve.

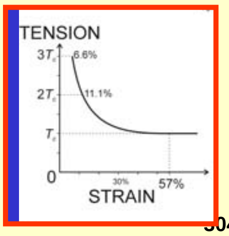




303

This curve shows that the required tensile strength of the geotextile depends on the geotextile strain.

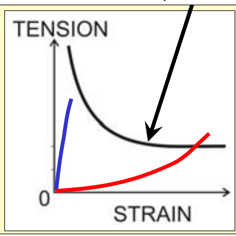
A smaller tensile strength is required from geotextiles with a larger strain.



304

Required Tensile Strength Curve for one set of conditions
(magnitude of applied load, size and shape of stones)

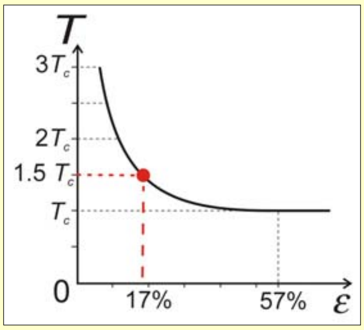
The peak of a geotextile tension-strain curve must be above the Required Tensile Strength Curve.



Giroud (1999)

The geotextile with the blue curve, is not suitable.
The geotextile with the red curve is suitable (in spite of a lower tensile strength).

305



Giroud (1999)

For example, the required tensile strength of a geotextile with a strain at failure of 17% should be 1.5 times the required tensile strength of a geotextile with a strain at failure of **57%** or greater.

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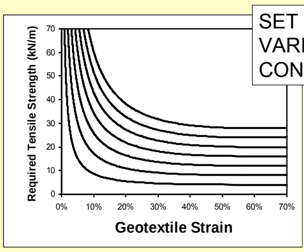


SET OF CURVES FOR VARIOUS LOADING CONDITIONS

INCREASING LOAD
INCREASING STONE SIZE

The curves show that,
from the viewpoint of specifications for separation,
there is a progressive transition
from geotextiles with low elongation
to geotextiles with high elongation.
Therefore, geotextiles should not be in two categories,
wovens and nonwovens.

Giroud (1999)

307

In conclusion:

It is right to have different tensile strength specifications for woven and nonwoven geotextiles, but there is a variety of woven geotextiles and a variety of nonwoven geotextiles.

Therefore, "progressive specifications" (i.e. based on the curve) are preferable to specifications by category (wovens and nonwovens).

308

PAVED ROADS

The geotextile performs two functions:

- **SEPARATION**
- **REINFORCEMENT**

309

From the viewpoint of the reinforcement function, a major difference between paved roads and unpaved roads is the magnitude of deformation, because acceptable rutting is much less in paved roads than in unpaved roads.

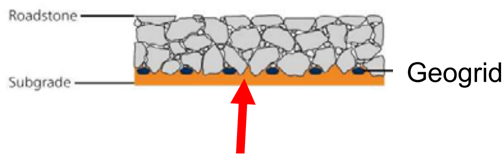
310

The only mechanism of reinforcement that is effective in paved roads is the load distribution improvement that results from lateral restraint because this mechanism works with small deformation.

Also, lateral restraint has a long-term beneficial effect by reducing aggregate base deterioration.

311

Reinforced road bases



Importance of interlocking to ensure lateral restraint

312

Geogrids provide lateral restraint.

**Therefore, it is appropriate
to use a geogrid
for road base reinforcement.**

EXAMPLE OF ROAD CONSTRUCTION
WITH GEOGRID.
(NEXT SLIDE)

313

USE OF GEOGRID FOR ROAD WIDENING

**Design methods to account
for the beneficial effects
of geosynthetics in paved roads
vary from one country to another,
and are not discussed here.**

318

APPLICATIONS OF GEOSYNTHETICS

IN ROAD STRUCTURE

- Unpaved roads
- Paved roads
- **Asphalt overlay**

319

Asphalt impregnated geotextile retards crack propagation.

This geotextile function is quite different from the functions discussed so far.

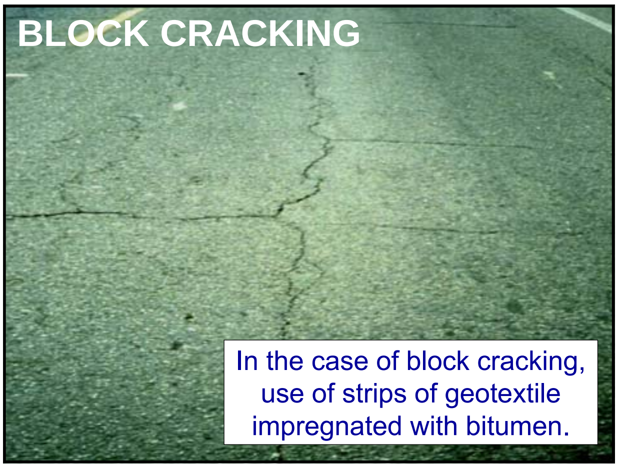
320

GEOTEXTILE FUNCTION IN ASPHALT OVERLAY

- **The geotextile impregnated with bitumen is impermeable and acts as a water barrier, preventing precipitation water from percolating into the road base and subgrade.**
- **The geotextile impregnated with bitumen has a visco-elastic behavior and acts as a crack barrier, slowing down crack propagation.**

321

BLOCK CRACKING



In the case of block cracking, use of strips of geotextile impregnated with bitumen.

Strips of geotextile impregnated with bitumen

ALLIGATOR CRACKING

In the case of alligator cracking, use of full-width geotextile.

ASPHALT OVERLAY

It is important to use a system that places the geotextile without pleats and wrinkles.

This is important because any extra thickness of the geotextile (due to a crease or a flattened wrinkle) may initiate a crack in the asphalt overlay.

326

Geotextile placed without wrinkles

APPLICATIONS OF GEOSYNTHETICS IN ROADS

THREE CATEGORIES:

- Applications in road foundation
- Applications in road structure
- Applications in controlling water

328

APPLICATIONS OF GEOSYNTHETICS IN WATER CONTROL

- Drainage
- Groundwater control
- Groundwater protection
- Moisture control

329

DRAINAGE IN ROADS

- Edge drains
- Drainage in pavement structure

330

EDGE DRAINS

It is important to understand the filtration function.

Intimate contact between the filter and soil is essential.

331

The function of a filter in geotechnical engineering is not to stop particles.

A filter that stops particles will clog, because particles will accumulate on or in the filter.

332

The function of a filter in geotechnical engineering is to prevent the soil from moving.

In order to prevent the soil from moving, the filter must be in intimate contact with the soil.

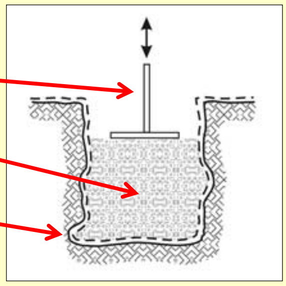
333

EXAMPLE OF INTIMATE CONTACT

Compaction

Small aggregate

Flexible geotextile



The geotextile is subjected to a **uniform pressure** and, as a result, it is in intimate contact with the soil.

334

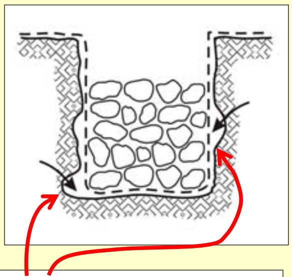
Filling a trench with small aggregate.

Compacting aggregate to ensure intimate contact.

(light compaction to avoid damaging the geotextile.)

EXAMPLE OF POOR CONTACT

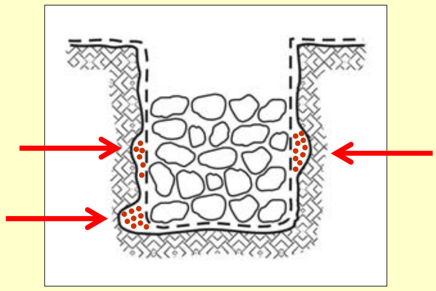
If coarse aggregate is used, the pressure applied on the geotextile is not uniform and the geotextile is not in good contact with the soil.



Soil erosion occurs at locations where water can flow between the soil and the geotextile.

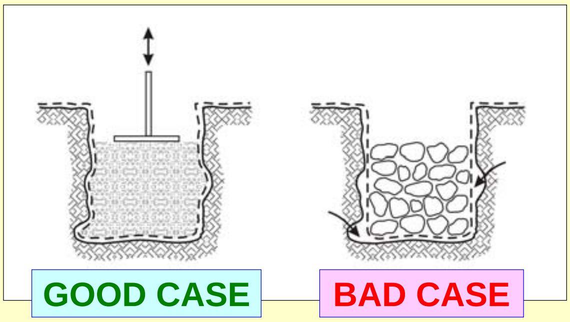
337

Soil erosion results in accumulation of fine soil particles at locations where there is a space between the geotextile filter and the soil.



338

We can now summarize the lesson learned from the good case and the bad case.



339

LESSON LEARNED FROM THE GOOD CASE AND THE BAD CASE

- In the **good case**, small aggregate is used, and this aggregate is slightly compacted. As a result, the geotextile filter is pushed against the soil and there is **intimate contact** between the geotextile filter and the soil. Consequently, the soil is retained.
- In the **bad case**, coarse stones are used, and, as a result, the geotextile filter is not in intimate contact with the soil. Therefore, when water flows from the soil toward the drain, particles **accumulate** in the space between soil and geotextile, and **accumulate** on or in the geotextile filter. This is one of the main causes of **clogging** of geotextile filters.

340

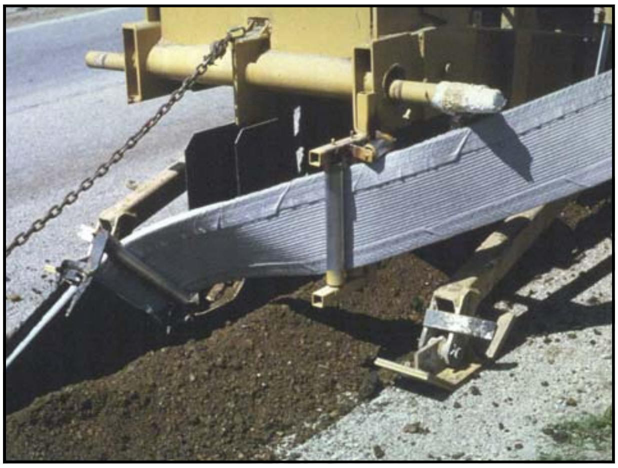
From the viewpoint of intimate contact, there is a significant difference between a geotextile filter and a granular filter.

Sand conforms to the shape of the soil. Therefore, sand is always in intimate contact with the soil.

341

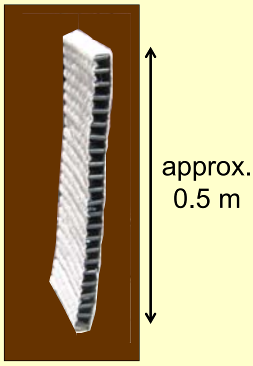
This property of sand is used in the case of geocomposites used as edge drains.

342



TYPICAL GEOCOMPOSITE USED AS EDGE DRAIN

The relatively rigid structure makes it difficult to achieve intimate contact with the soil.



Typical thickness
30 mm

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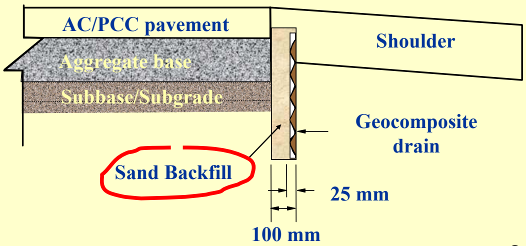
BACKFILLING WITH SAND
BETWEEN RIGID GEOCOMPOSITE EDGE DRAIN AND SOIL
TO ENSURE INTIMATE CONTACT



345

DESIGN OF EDGE DRAINS

This is a well-known subject in road engineering.

Design charts are available in road design manuals.

Here, a simple method is proposed for preliminary analyses.

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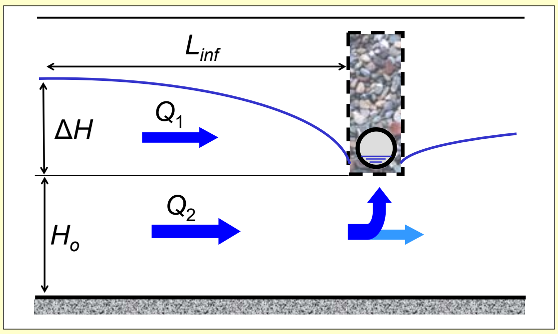
EDGE DRAIN DESIGN



$$Q = Q_1 + Q_2$$

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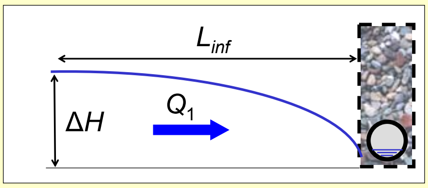
EDGE DRAIN DESIGN



Classical demonstration for flow with zero slope

$$\frac{Q_1}{B} = \frac{k\,(\Delta H)^2}{L_{inf}}$$

Usual approximation

$$L_{inf} \approx 3.8\,\Delta H$$

Hence

$$\frac{Q_1}{B} = \frac{k\ \Delta H}{3.8} = 0.26\ k\ \Delta H$$

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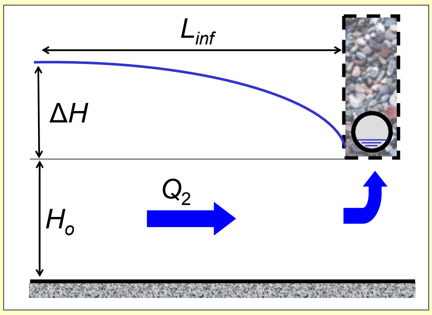
Darcy's equation:

$$Q_2 = k\, i\, A$$

$$\frac{Q_2}{B} = k\, i\, H_o$$

Gradient:

$$i = \frac{(2/3)\,\Delta H}{(1/2)\,L_{inf}}$$



Usual approximation $L_{inf} \approx 3.8\,\Delta H$

Hence: $$\frac{Q_2}{B} = \left(\frac{4}{3}\right)\frac{k\, H_o\, \Delta H}{3.8\,\Delta H}$$

$$\boxed{\frac{Q_2}{B} = 0.35\, k\, H_o}$$

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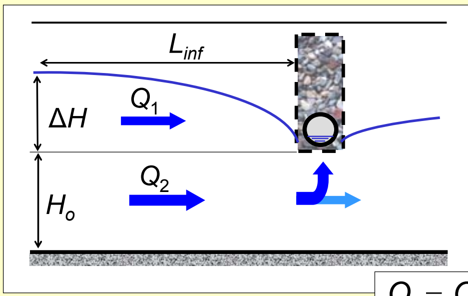
EDGE DRAIN DESIGN



$$\boxed{Q = Q_1 + Q_2}$$

$$\boxed{\frac{Q}{B} = k\,(0.26\,\Delta H + 0.35\, H_o)}$$

350

PIPE DESIGN

Classical Manning's equation:

$$\boxed{Q_{max} = 0.3353\, \frac{d^{8/3}\, i^{1/2}}{n}}$$

d = pipe diameter
i = gradient (pipe slope)
n = Manning's roughness coefficient
Typical value: $n = 0.008$ to 0.011 $sm^{-1/3}$ for smooth plastic pipe

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DRAINAGE IN ROADS

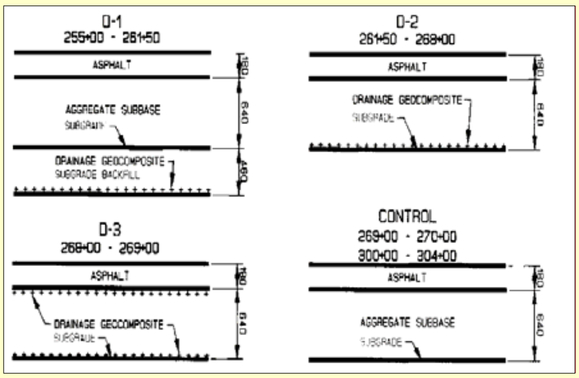
- Edge drains
- **Drainage in pavement structure**

352

Maine Department of Transportation
Frankfort to Winterport Highway

353

Drainage Test Sections



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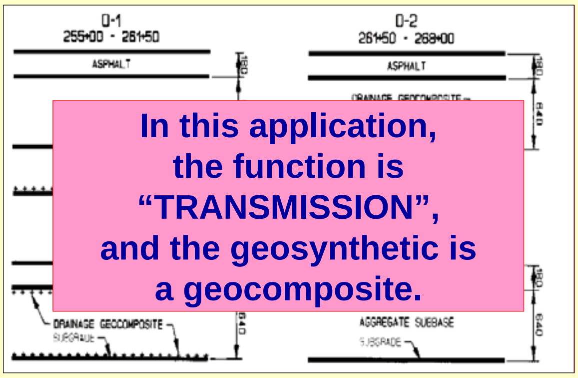
Drainage Test Sections



In this application, the function is "TRANSMISSION", and the geosynthetic is a geocomposite.

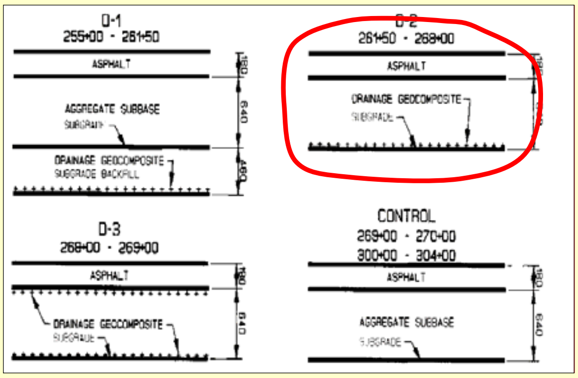
355

DEPLOYING THE GEOCOMPOSITE

Average installation time: 4 min. for 4m x 60m panel.

356

Drainage Test Sections



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Base course aggregate placed on top of geocomposite

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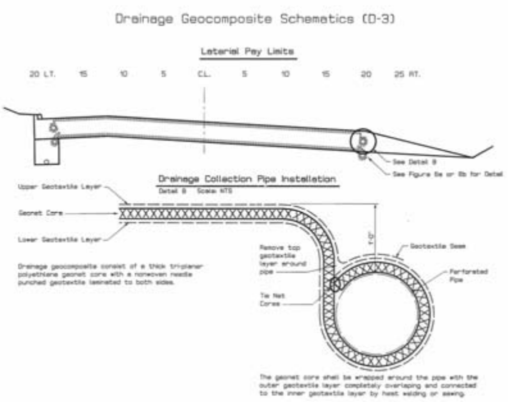


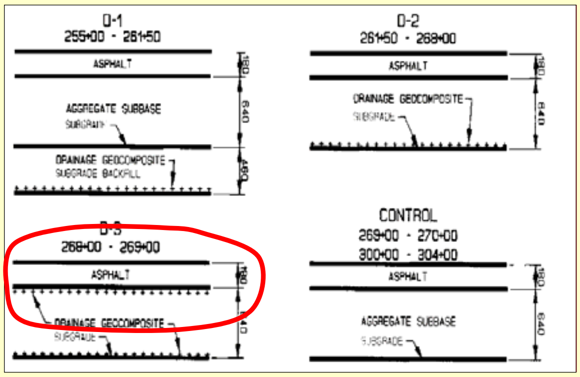
CONNECTION WITH COLLECTOR PIPE

CONNECTION WITH COLLECTOR PIPE

Texas Department of Transportation
Southwest Parkway Street Reconstruction Project

360

Drainage Test Sections



361

GEOCOMPOSITE UNDERNEATH HOT-MIX ASPHALT

GEOCOMPOSITE UNDERNEATH HOT-MIX ASPHALT

363

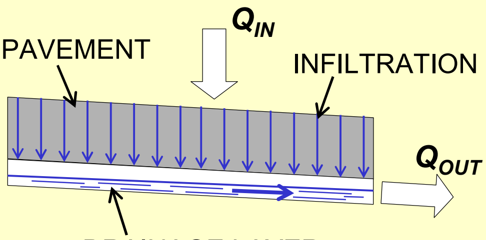
DESIGN OF GEOSYNTHETIC DRAIN UNDER PAVEMENT

This drain must collect:

- **water that infiltrates through the pavement; and**
- **water that seeps from the ground.**

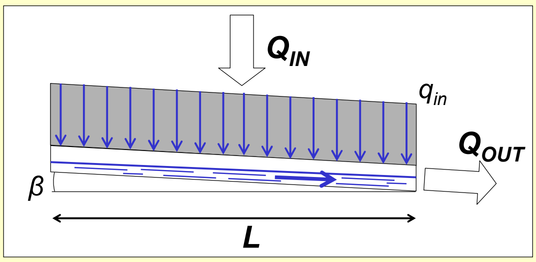
364

INFILTRATION AND DRAINAGE



Steady state : $Q_{IN} = Q_{OUT}$

365



$$Q_{IN} = q_{in} L$$
$$Q_{OUT} = \theta \, i = \theta \sin\beta$$
$$\Rightarrow \quad \theta_{req} = \frac{q_{in} L}{\sin\beta}$$

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$$\theta_{req} = \frac{q_{in}\, L}{\sin\beta}$$

This equation makes it possible to determine the required hydraulic transmissivity of the drainage layer.

To use this equation we need to know the values of the three parameters:

q_{in} = rate of infiltration through the pavement
L = length of drainage path
β = slope angle

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RATE OF INFILTRATION

q_{in} is the rate of infiltration per unit area.
It is expressed in units of
volume per unit area per unit of time,
$m^3 / m^2 / s = m / s$

TYPICAL VALUES
Asphalt pavement:

$$q_{in} = 0.10 - 0.15\,\text{m/day} = 1.15 - 1.75 \times 10^{-6}\,\text{m/s}$$

Concrete pavement:

$$q_{in} = 0.15 - 0.20\,\text{m/day} = 1.75 - 2.3 \times 10^{-6}\,\text{m/s}$$

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RATE OF INFILTRATION

The rate of infiltration is essentially due to cracks.
It can be calculated as follows:

$$q_{in} = R_{in}\left(\frac{1}{d_L} + \frac{1}{d_T}\right)$$

d_L = distance between longitudinal cracks
d_T = distance between transverse cracks
R_{in} = rate of infiltration per unit length of crack
Typical value: $R_{in} = 3 \times 10^{-6}\,\text{m}^2/\text{s}$

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LENGTH AND SLOPE OF DRAINAGE PATH

There is always a lateral slope
and there may be a longitudinal slope.
If there is a longitudinal slope,
the direction of flow forms an angle, α,
with the transverse direction.
Along this direction, the length of flow is *L*
and the slope is *S*.

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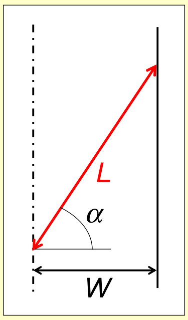
LENGTH AND SLOPE OF DRAINAGE PATH



$$L = W\sqrt{1 + \left(\frac{S_L}{S_T}\right)^2}$$

$$S = \sqrt{S_L^2 + S_T^2}$$

$$\tan\alpha = \frac{S_L}{S_T}$$

S_L = slope in longitudinal direction
S_T = slope in transverse direction
S = slope in direction of flow (typical: 0.02)
L = drainage length in direction of flow
W = half-width of pavement in case of drainage on both sides

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In addition to the steady-state flow calculation discussed in the preceding slides, it is traditional in road design to check that the drainage layer can be drained rapidly after the end of infiltration.

The purpose is to make sure the road pavement will dry rapidly after a storm.

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To do this evaluation, it is assumed that the drainage layer is saturated and, then, infiltration stops (i.e. end of rainfall).

Then, a calculation is done to evaluate the time required for 50% of the water to drain from the drainage layer after the infiltration has stopped.

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TRADITIONAL TABLE FOR DRAINAGE QUALITY EVALUATION

Drainage quality	50% water removed within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1month

Based on this table, one should try to get a drainage time less than 2 hours.

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The time required to drain 50% of the water stored in the drainage layer when it is saturated can be calculated using the following equation:

$$T_{50} = \frac{n\,L}{2\,k\,\sin\beta}$$

n = porosity of the drainage layer material
L = drainage length in direction of flow
k = hydraulic conductivity of the ddrainage layer material
β = slope angle in the direction of flow

Giroud, to be published

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In the case of geosynthetics drainage layers, if the geosynthetic meets the steady-state flow requirements, it generally provides a time T_{50} very short.

In other words, in the case of geosynthetics, the steady-state flow requirement is more stringent than the traditional requirement for rapid drainage after a rainfall.

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DESIGN OF GEOSYNTHETIC DRAIN UNDER PAVEMENT

This drain must collect:

- **water that infiltrates through the pavement; and**
- **water that seeps from the ground.**

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Two cases will be considered for water that seeps from the ground:

- **water coming from below (artesian aquifer) (rare); and**
- **water that comes laterally (frequent)**

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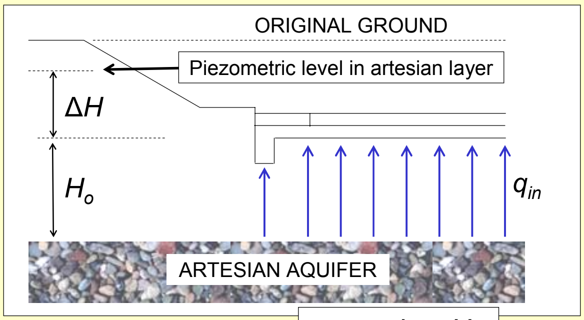
WATER FROM ARTESIAN AQUIFER



Darcy's equation: $q_{in} = \dfrac{k\ \Delta H}{H_o}$

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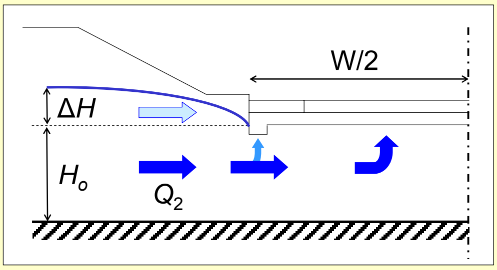
WATER FLOWING LATERALLY



$\dfrac{Q_2}{B} = 0.35\ k\,H_o$ known from earlier calculation

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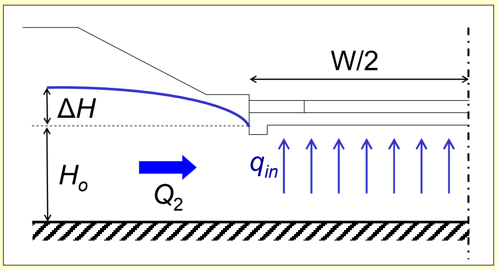
WATER FLOWING LATERALLY



$q_{in} = \dfrac{Q_2 / B}{W/2}$ hence: $q_{in} = \dfrac{0.35\ k\,H_o}{W/2}$

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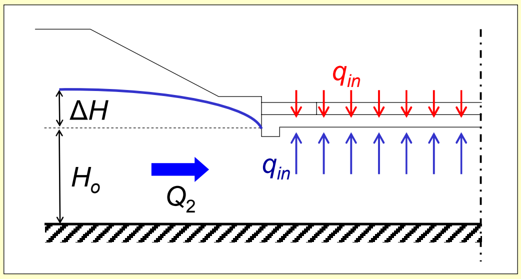

Numerical applications show that : q_{in} and q_{in} may be on the same order of magnitude, depending on the soil permeability and the pavement degradation.

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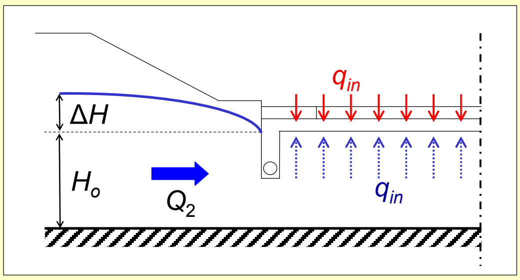

Therefore, it is recommended to have an effective edge drain that collects Q_2, thereby eliminating q_{in}.

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APPLICATIONS OF GEOSYNTHETICS

IN WATER CONTROL

- Drainage
- **Groundwater control**
- Groundwater protection
- Moisture control

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GROUNDWATER CONTROL

Highway under groundwater table

**In this application,
the function is
“WATER BARRIER”,
and the geosynthetic is
a geomembrane.**

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Low-permeability soil on top of geomembrane

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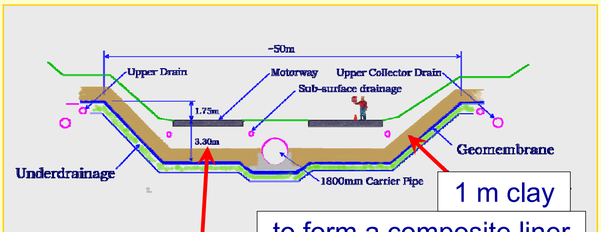
CROSS SECTION



1 m clay

to form a composite liner
with the geomembrane

weight of 3.3 m of soil (including the clay)
to counteract uplift by ground water pressure

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APPLICATIONS OF GEOSYNTHETICS

IN WATER CONTROL

- Drainage
- Groundwater control
- **Groundwater protection**
- Moisture control

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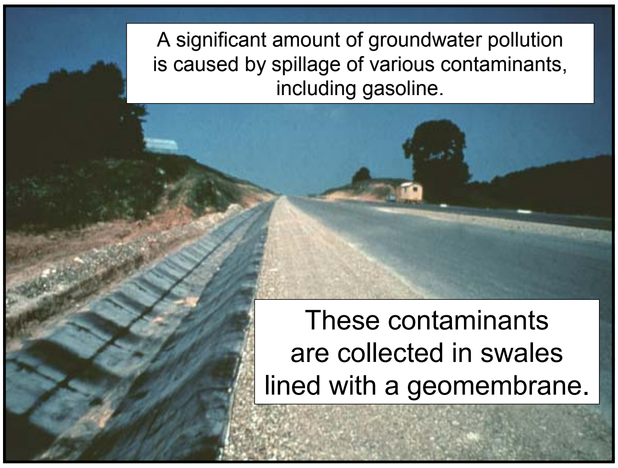

Chartres, France, 2000
Courtesy B. Breul

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or the geomembrane
can be covered with a layer of soil

Autoroute A 20, France, Courtesy of B. Breul 397

The geomembrane covering the road shoulder can be welded to concrete ditches.

Cahors, France 2002
Courtesy of B. Breul

1995 Switzerland

To protect an important aquifer, the entire highway excavation is lined with a geomembrane.

Swales lined with geomembrane can be constructed on the slope above the road.

APPLICATIONS OF GEOSYNTHETICS

IN WATER CONTROL

- Drainage
- Groundwater control
- Groundwater protection
- **Moisture control**

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MOISTURE CONTROL

- **Use of geomembranes along highways to control the moisture content of expansive soils**
- **MESLs (membrane encapsulated soil layers)**

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USE OF GEOMEMBRANES ALONG HIGHWAYS TO CONTROL THE MOISTURE CONTENT OF EXPANSIVE SOILS

- **With expansive subgrade soil, the service life of a road may be 5 years instead of 20 years.**
- **The purpose of the geomembrane is to stabilize the moisture content of the subgrade soil.**
- **Typically, moisture content fluctuates down to a depth of 1.5 to 3.0 m.**

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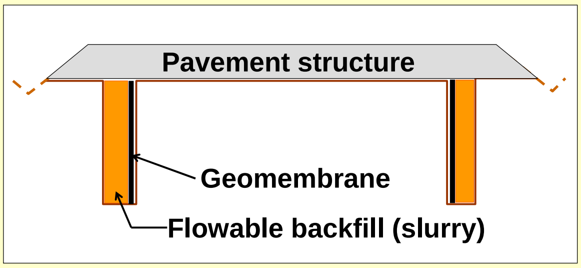
VERTICAL GEOMEMBRANE



Trench depth:
1.5 m to 3.0 m
typical: 2.5 m

Trench width:
0.5 m backhoe
0.1 m trencher

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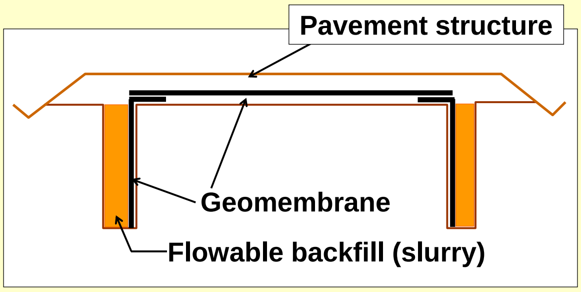
VERTICAL AND HORIZONTAL GEOMEMBRANE



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MOISTURE CONTROL

- Use of geomembranes along highways to control the moisture content of expansive clays
- **MESLs (membrane encapsulated soil layers)**

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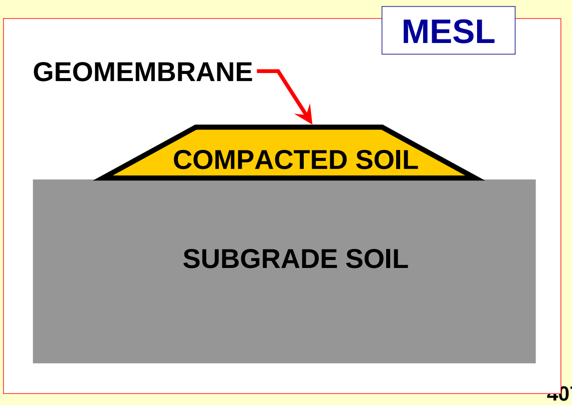
MEMBRANE ENCAPSULATED SOIL LAYER



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The geomembrane is delivered.

The bituminous geomembrane is unrolled.

The soil layer is compacted.

The geomembrane is wrapped around the compacted soil layer.

The completed MESL

CONCLUSION

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Properties & FUNCTIONS

Properties	FUNCTIONS
Thickness	
	TRANSMISSION
Permeability	
	FILTRATION
Retention	
	SEPARATION
Strength	
	REINFORCEMENT
Interface	

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COMMENTS ON THE PRECEDING SLIDE

Based on what we have learned,
this appears to be a very simplified relationship
between properties and functions of geosynthetics.

In reality, more functions are performed
and, for a given function,
several mechanisms can be considered.

This is particularly true for the reinforcement function.
Remember: load distribution, lateral restraint,
tensioned membrane, and subgrade confinement.

However, this slide summarizes the spirit of this presentation.
For each application, the relevant functions are identified,
which leads to the relevant properties, which in turn leads
to the selection of the most appropriate geosynthetic.

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Thank you

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