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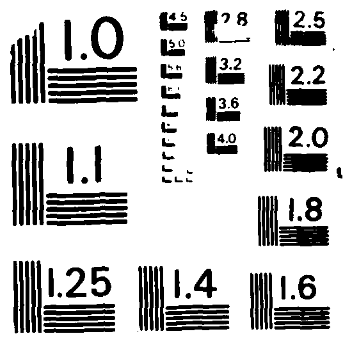

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STABILIZATION OF VERY SOFT SOILS USING GEOSYNTHETICS

by

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The introduction of high-strength geosynthetic materials has allowed for construction on heretofore impossible sites. Saturated, fine-grained soils with in-situ unconsolidated-undrained strengths as low as 1.0 kPa (20.8 lb/ft^2) have been successfully used for embankment foundations using geotextiles or geogrids for direct ground support. When rapid consolidation of the subsoil is desirable, the procedure sometimes calls for insertion of strip (wick) drains and, for lateral flow, drainage geocomposites as well.

This paper reviews and summarizes a number of soft-soil stabilization projects. Drawing from these experiences, a five-part design methodology is presented to select and size the various geosynthetic material properties involved. The paper closes with recommendations for future research and development.

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Preface

This report describes the design and construction considerations for fabric-reinforced embankments constructed on soft soils.

This report was prepared by the US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, during the period Sep 86 to Dec 86.

Concept formulation and general supervision of this research and design effort was conducted by Dr. J. Fowler, Geotechnical Laboratory (GL), WES, and Dr. R. M. Koerner, Professor, Drexel University, Philadelphia, Pennsylvania.

This report was written by Dr. J. Fowler under the general supervision of Mr. G. B. Mitchell, Chief, Engineering Group, Soils Mechanics Division (SMD), Mr. C. L. McAnear, Chief, SMD, and Dr. W. F. Marcuson III, Chief, GL. Dr. R. M. Koerner assisted Dr. Fowler in the writing and final preparation of the report.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

Contents

STABILIZATION OF VERY SOFT SOILS USING GEOSYNTHETICS

Introduction

1. A general category of in-situ foundation soils which has been extremely difficult for the geotechnical engineer to utilize is characterized by high water content and fine grained composition. High water content soils are too weak and compressible to build upon directly; because they are silts and clays, they drain too slowly for effective and economical utilization in a short time period. The usual remedies are to excavate and replace them with suitable soil or to install deep foundations through them to an adequate bearing stratum. Indeed, there have been many attempts made and techniques developed for using alternate methods of soil strengthening, e.g., electro-osmosis, deep dynamic compaction, chemical grouting, lime stabilization, stone columns, sand drains, etc., but all are site specific and relatively expensive to deploy.

2. In this paper, emphasis is on the basic use of a soil embankment or soil (surcharge) fill which is founded upon and which mobilizes consolidation of the in-situ soil. The unique part of this technique is its use on in-situ soils with shear strengths as low as 1 kPa (20.8 lb/ft^2). However, such weak in-situ soil will simply not support the dead load of the soil fill much less the live loads from the construction placement equipment. Clearly, the in-situ soil needs help, which is precisely why a geosynthetic (usually a geotextile which will be referred to hereafter, although geogrids could also be used) is necessary. The function of the geotextile is tensile reinforcement for it initially must support the soil fill which is placed directly upon it. Subsequently, the consolidation process will generate increased shear strength in the in-situ foundation soil allowing it to support part, or all, of the permanent load.

3. The soil fill which is placed on top of the geotextile, can take one of two geometric forms;

<u>a</u>. A <u>linear</u> soil fill, such as a dike or containment embankment, which is long relative to its width. This is the geometry often utilized by the U.S. Army Corps of Engineers when constructing containment dikes for dredged soil or levee embankment. In these cases the fill stays in place permanently and is upgraded as consolidation settlement occurs. Containment dike heights

are in the 2-5 m (6' - 15') range with side slopes of 1-on-5 to as low as 1-on-20. Levees are often higher.

b. An areal soil fill, such as a temporary surcharge, which is large in both its length and width. This situation is one in which the entire site is necessary for subsequent construction and the fill is only placed temporarily. After sufficient consolidation settlement has occurred (along with its strengthening of the in-situ foundation soil) all, or part, of the surcharge fill is then removed. It is then replaced by the permanent facility which is now founded using a shallow foundation on a preconsoli dated soil.

While both situations just described have great similarities, they are distinct enough to warrant certain differentiations in both design and construction.

Case Histories

4. Beginning in the late 1970's, the U.S. Army Corps of Engineers began experimenting with geotextiles placed over soft dredged soil to support containment dikes (1,2). The first major project was Pinto Pass at Mobile, Alabama, where a 160 kN/m (900 lb/in.) ultimate strength fabric was used (see Table 1). This project set the direction for a number of linear stabilization projects where high strength in the fabric's warp direction could be aligned perpendicular to the longitudinal axis of the dike. Full length rolls the entire width of the dike could be utilized without the necessity of seams. Seams were required, however, in the weft direction (i.e., along the long edges of each of the fabric rolls) which is also the direction of the minor principal stress. This is fortunate, because joining of high strength fabrics (usually by sewing) was still in an initial stage of development at the time. Required seam strengths were approximately 35 kN/m (200 lb/in.) which was considered (at the time) to be a high strength seam. A succession of linear stabilization projects followed; i.e., Minnesota DOT, three Corps of Engineers projects at Norfolk, Virginia and a Brunswick Pulp and Paper Co. project in Brunswick, Georgia (see Table 1).

5. More recently, projects involving areal stabilization have been undertaken. Washington National Airport in Washington, DC was the first in which length was approximately equal to width, but it was actually designed as two adjacent linear embankments. Thus the weft direction and its seams could

Table 1

Review of Recent USA Reinforcement Embankment Project

Using High Strength Geotextiles

Project No.	Date Constructed	Owner	Name/Location	Foundation Conditions: Soil Type	Water Content (%)	Depth (m)	Shear Strength (kN/m^2)
1	1978	USAE COE Mobile District	Pinto Pass Mobile, AL	Organic Clay	70–150	12	2–5
2	1981	Minnesota DOT	St. Paul, MN Interstate 494	Peat	200–300	6	?
3	1982	USAE COE Norfolk Dist	Test Sect #1 Craney Is	Organic Clay	60–180	11	5
4	1982	USAE COE Norfolk Dist	Test Sect #2 Craney Is Norfolk, VA	Organic Clay	60–180	10	1–4
5	1982	USAE COE Norfolk Dist	Test Sect #3 Craney Is Norfolk, VA	Organic Clay	60–180	10	1–2
6	1983	USAE COE Norfolk Dist	Craney Division Dike Norfolk, VA	Organic Clay	60–180	11	1–5
7	1983	Brunswick Pulp & Paper	Brine Storage Facility Brunswick, GA	Marine Clay	50–100	10–12	1–2
8	1983	Washington, DC	Washington Airport Extension	Clayey Silt	75–100	5–6	2–5

(Continued)

Table 1 (Continued)

Project No.	Date Constructed	Owner	Name/Location	Foundation Conditions: Soil Type	Water Content (%)	Depth (m)	Shear Strength (kN/m^2)
9	1984	USAE COE Huntington, Dist	Dike #2 Mohicancile, OH	Peat & Clay	117-636 peat 29-180 clay	6-18	8 peat 9 clay
10	1985	USAE COE Mobile, AL	South Blakney Is Mobile, AL	Organic Clay	70-150	5	2-5
11	1985	USAE COE Mobile, Al	Greenwood Is Mobile, AL	Organic Clay	70-150	5-9	2-5
12	1986	Alaska DOT Nenana, AK	Parks Hwy	Glacial Till	50-100	4-8	1-3
13	1986	Maryland Port Adm.	Seagirt Surcharge Baltimore, MD	Organic Clay	50-100	9	2-5
14	1986	USAE COE Mobile, AL	North Blakney Is Mobile, AL	Organic Clay	70-150	5	4-5
15	1986	USAE COE Norfolk, VA	Craney Is South Perimeter Dike Norfolk, VA	Organic Clay	70-180	5	5
16	1986	USAE COE Philadelphia, PA	Wilmington Harbor South, Disposal Area Wilmington, DE	Organic Clay	70-160	9-12	2-5
17	1986	USAE COE New Orleans, LA	Reach A Test Plaquemines Parish, LA	Organic Clay	70-100	9-18	1-2

(Continued)

Table 1 (Continued)

Project No.	Geotextile: Warp/Fill Ultimate Strength (kN/m)	Geotextile: Seam Ultimate Strength (kN/m)	Geotextile: Warp/Fill Secant Modulus (kN/m)	Dike Geometry: Height (m)	Dike Geometry: Width/Length (m)	Dike Geometry: Dike Slope H:V	Fill Material	Soil Response: Settlement (m)	Fabric Distributor and Type
1	160/30	45	634/187 @30%	2	52/244	10:1	Sand	1	Nicolon 66475 Polypropylene
2	175/35	18	613/114 @ 30%	3	30/152	5:1	Glacial Till	0.3-1	Nicolon 1250 X Polypropylene
3	186/39	53	606/114 @ 30%	3	72/229	11:1	Sand	1	Nicolon 1250 Polypropylene
4	186/39	53	606/114 @ 30%	2	72/152	11:1	Sand	2	Nicolon 1250 Polypropylene
5	186/39	53	606/114 @ 30%	2	66/91	12.5:1	Sand	5	Nicolon 1250 Polypropylene
6	186/105	53	606/350 @ 30%/22%	3	72/	10:1	Sand	1-2	Nicolon 1250 X Polypropylene
7	186/53		606/350 @ 30%/22%	8	15/150	2:1	Salt	?	Nicolon 1250 Polypropylene
8	184/39	39	527/? @ 10%	5	183/213	5:1	Sand	1	Burlington
9	876/86	Not Required	$21 \times 10^8 kN/m^2$ Steel	7	55/366	3:1	Clayey Sand/ Gravel	0.5	U.S. Steel

(Continued)

Table 1 (Concluded)

Project No.	Geotextile: Warp/Fill Ultimate Strength (kN/m)	Geotextile: Seam Ultimate Strength (kN/m)	Geotextile: Warp/Fill Secant Modulus (kN/m)	Dike Geometry: Height (m)	Dike Geometry: Width/Length (m)	Dike Geometry: Dike Slope H:V	Dike Geometry: Fill Material	Soil Response: Settlement (m)	Fabric Distributor and Type
10	175/114	52	665/700 @ 10%	3	24/	4:1	Sand	2	Mirafi 2100 Polyester
11	175/114	52	665/700 @ 10%	3	24/	4:1	Sand & Clay	1	Mirafi 2100 Polyester
12	361/187	88	3940/1883 @ 11%/10%	2	15/300	2:1	varied	?	Nicolon Polyester
13	193/245	116/137	1786/? @ 10%/10%	1-5	671/488	-	Sand/ Clay	1-2	Nicolon Polyester & Polypropylene
14	175/114	91	?	2	21/	5:1	Sand	1	Nicolon Polyester
15	361/187	88	3940/1883 @ 11%/10%	3	61/762	10:1	Sand	?	Nicolon Polyester
16	263/263	140	3415/2627 @ 10%/10%	5-6	122/213	12:1	Sand/ Gravel	NA	Wellman/ Polyester
17	506/105	105	59401	1.5-3	183/30	15:1	Sand/ Silt	NA	Nicolon Polyester

be weaker than the fabric strength in the warp direction. The Corps of Engineers South Blakney Island project along with the Maryland Port Authority's Seagirt stabilization project, were true areal stabilization projects. In these cases, high strength in both warp and weft directions were required, along with a major increase in seam strength. Seam strengths of up to 140 kN/m (800 lb/in.) were now required. Today, seam strengths approaching 175 kN/m (1000 lb/in.) can successfully be made even under adverse field conditions of working on floating barges and on very soft soils (see projects by the Corps of Engineers at Philadelphia, Pennsylvania and New Orleans, Louisiana, respectively, in Table 1).

6. While the above mentioned projects focused on the fabric's ultimate strength and elongation at failure (3), the Corps of Engineers, Huntington District project at Mohicanville, Ohio brought out the importance of the modulus of the reinforcement (4). Usually expressed as a secant modulus at a specific strain, this project required the use of a steel mesh to obtain the required stiffness. The use of this type of reinforcement should not be surprising, however, for there is a logical extension of the mechanical properties of plastic-to-glass-to-steel (5,6) as shown in Table 2. Note that these values are in stress units and to convert them to force per unit width as in Table 1, they would have to be multiplied by a suitable thickness. For a high strength fabric this would be approximately 2.5-5.0 mm (0.1-0.2 in.).

7. While proper design and careful construction should allow for initial stability of these dikes and surcharge fills, the time for primary consolidation to occur is often excessively long. Thus the use of strip drains has been employed on several of these projects. The Maryland Port Authority project had approximately one million linear meters (3 million ft) of strip drain placed through the surcharge, the fabric and the compressible foundation soil (7). The Corps of Engineers project in Wilmington, Delaware is somewhat similar and is currently ongoing. Both of these projects reflect the current state of the art in using geosynthetics to build upon and stabilize very soft foundation soils.

Table 2

Intrinsic Properties of Polymeric Materials Contrasted to Glass and Steel

(after Lawson, 1982)

Material Type	Tensile Strength MPa	Tensile Strength $kip/in.^2$	Elastic Modulus MPa	Elastic Modulus $kip/in.^2$	Strain at Failure %
Polypropylene					
● monofilament	400	58	3250	471	17-19
● continuous filament	700	101	6500	942	17-19
Polyethylene-Monofilament					
● low density	80-120	12-17	800	116	25-50
● high density	350-500	51-72	4000	580	20-30
Polyester-Cont. Filament					
● medium tenacity	500-650	72-94	12,500-14,500	1810-2100	15-30
● high tenacity	750-1400	109-203	14,000-18,500	2030-2680	6-15
Polyester-Staple Fibers	500-700	72-101	4000-7500	580-1090	20-50
Nylon 6,6-Cont. Filament					
● medium tenacity	450-600	65-87	< 2500	< 362	20-30
● high tenacity	700-1000	101-145	3000-12,500	435-1810	12-15
Nylon, 6,6-Staple Fibers	400-700	58-101	< 2000	< 290	20-35
Aramid-Cont. Filament					
● low tenacity	500-900	72-130	15,000-20,000	2180-2900	8-12
● high tenacity	2500-2900	362-420	55,000-130,000	8000-18,900	2-4
Glass					
● Type E	1750-3500	254-508	70,000-150,000	10,000-21,800	2-4
Steel					
● Prestressing	1450-2150	210-312	200,000	29,000	2

Design Concepts - Reinforcement

8. While a generic design for all of the projects just described is essentially impossible to provide, some similarities have evolved and can be presented. In this regard there are five major concerns, each of which leads to a specific design parameter or property of the reinforcement geotextile or embankment.

> Bearing Capacity (which leads to maximum height of embankment) - Overall bearing capacity of the site must be satisfied or a failure as shown in Figure 1(a) will occur. This is essentially the case with or without the reinforcing geotextile. Analysis follows along classical geotechnical engineering methods for infinitely long strip foundations and for undrained ($\phi = 0°$) conditions; i.e.,

$$q = c\ N_c$$

and

$$q_{ave} = \gamma H$$

so

$$\gamma H = c\ N_c$$

$$H_{ave} = c\ N_c/\gamma$$

where

- q = ultimate bearing capacity
- c = undrained shear strength
- N_c = bearing capacity factor (3.5 to 5.7)
- γ = unit weight
- H = height of embankment

For soils of low undrained shear strength (recall the values in Table 1), the height of the dike or embankment is greatly limited. It also forces wide and flat side slopes (again recall the values in Table 1). Even though the height and side slopes will be initially low, the foundation soil will consolidate and fill heights can be increased with time. If this time is too long, however, installation of strip drains will be necessary.

9. Regarding the design of strip drains, a considerable amount has been written (8). The simplified formula of Hansbo (9) appears to be adequate to obtain the proper spacing versus the time for consolidation to occur.

$$t = \frac{D^2}{8c_h} [\ell n \frac{D}{d} - 0.75] [\ell n \frac{1}{1-U}]$$

where

t = time for consolidation

c_h = horizontal coefficient of consolidation of soil

D = spacing of strip drain

d = equivalent diameter of strip drain (circumference/π)

U = average degree of consolidation

Seen in the above equation is that by making the spacing D small, the time for consolidation under a given fill height can become as short as desired. For Maryland Port Authority's Seagirt project, the strip drain spacing was 1.5 m (60") which allowed for time for a 90% consolidation of approximately six months. Once consolidation is achieved, c increases and H may then be increased if desired.

Global Stability (which leads to fabric ultimate strength) - The next consideration one must assess is the determination of what ultimate strength is necessary vis-a-vis the applied loads (embankment plus live load) and the strength of the in-situ soil. Here one usually uses a limit equilibrium method as illustrated by the circular arc method shown in Figure 1(b). Taking moments about the origin results in a factor of safety equation as follows:

$$FS = \frac{(\tau_e L_e + \tau_f L_f) R + T_a a}{Wx_s + Lx_0}$$

where

τ_e = shear strength of embankment soil (often neglected)

τ_f = shear strength of in-situ soil

L_e = arc length in the embankment soil

L_f = arc length in the foundation soil

T_a = allowable strength of geotextile

a = moment arm about center of failure arc

W = weight of soil mass

x_s = moment arm of soil mass

L = weight of live load

x_ℓ = moment arm of live load

FS = factor of safety

R = radius of the failure arc

For a given factor of safety (e.g., 1.1 to 1.3) one can solve for "T_a" as the unknown. This value of fabric allowable strength is related to the ultimate strength of the geotextile and, most directly, to its polymer type. In order to avoid tertiary creep, Lawson (5) recommends the following:

Table 3 - Allowable Strengths of Polymeric Geosynthetics to Avoid Tertiary Creep, after Lawson (5)

Polymer Type	Embankments* (% of Ultimate)	Walls and Slopes (% of Ultimate)
polypropylene	20-40	20
HDPE geogrid	30-40	30
polyester	40-60	40
aramid	45-60	45

* Problems of the type described in this paper.

Note that criteria for embankments founded on soils which will consolidate and gain in strength with time are less restrictive than permanent walls and slopes. Such calculations for embankments on very soft soils usually indicate geotextiles of 175 kN/m (1000 lb/in.) or greater ultimate strength.

10. The distinction between linear and areal fills can now be made. For linear fills the geotextile can be designed anisotropically. By knowing the direction of maximum stress, an unbalanced geotextile is possible. As note previously, its stronger direction must be in the direction of maximum stress and its weaker direction perpendicular to that, in the direction of the

minimum stress. Note that this really demands a three-dimensional slope stability analysis; a subject about which little is available and, when it is attempted, is quite complex. Lawson (5) recommends a longitudinal ultimate strength of 25% or more of the transverse ultimate strength.

11. For areal fills, however, the geosynthetic must be balanced in both the warp and weft (or machine and cross machine) directions and thus becomes essentially isotropic. This is because no preferential stress direction can be defined and all directions must be considered worst case situations. Obviously, the seams in both directions must be of a strength compatible with the fabric.

a. Elastic Deformation (which leads to fabric elastic modulus and strain at failure) - A decision as to allowable embankment deformation must be made in order to numerically define the elastic modulus and strain at failure (see Figure 1(c). Finite element methods have been used (4) and empirical relationships based on experience have been developed. It appears as though a modulus of 5 to 25 times the ultimate strength allows for tolerable deformations for the projects described herein. Using this value and on the basis of a completely elastic material:

$$E_s = 5 \text{ to } 25\ T_{ult}$$

$$\varepsilon_f = \frac{T_{ult}}{E_s}$$

thus:

$$\varepsilon_f = 0.20 \text{ to } 0.04$$

$$\varepsilon_f\ (\%) = 20\% \text{ to } 4\%$$

where

E_s = elastic (secant) modulus
T_{ult} = ultimate fabric strength
ε_f = strain at fabric at failure

Note from Table 2 that this consideration alone eliminates some of the potential polymeric materials listed. The amount of centerline deflection that such strain levels allow is quite large. Using an average value of 10% failure strain and assuming the deflected shape of the geotextile to be a parabola, i.e.,

$$S = \sqrt{4w^2 + (B/2)^2} + \frac{(B/2)^2}{2w} \ln \frac{2w + \sqrt{4w^2 + (B/2)^2}}{B/2}$$

where

S = arc length

W = centerline deflection

B = base width

an embankment 30.5 m (100') wide (and an arc length S of 1.10 B) results in a centerline deflection of 6.1 m (20')! Such large deformations strongly suggest limiting the strain at failure to a minimum and making the elastic modulus a maximum.

b. Pullout (which leads to required anchorage length) - Once the embankment height and type of geotextile reinforcement have been selected, the anchorage distance must be determined. As seen in Figure 1(d), the anchorage zone extends behind the slip zone and back into the stable soil zone. It must be sufficiently long to mobilize the full strength of the geotextile. The analysis uses the following concept.

$$T_{ult} = 2\tau\, E\, L$$

where

T_{ult} = ultimate strength (therefore it includes the FS)

τ = shear strength of foundation soil

E = efficiency factor for the particular soil/fabric combination

L = the unknown length

Regarding the efficiency factor in the above equation, considerable research is ongoing. Geogrids appear to have the highest value with E = 1.0 to 2.0; rough geotextiles next with E = 0.80 to 1.2; and smooth geotextiles the lowest, with E = 0.60 to 0.8. It must be recognized that in many situations using high strength geotextiles with soft soils, very long anchorage lengths will result from the above analysis.

c. Lateral Spreading (which leads to required fabric friction) - Utilizing techniques common to lateral earth pressure theory, one can obtain the required frictional characteristics of the geotextile. Using Figure 1(e) one can work with a factor of safety concept as follows:

$$FS = \frac{\text{Resisting Forces}}{\text{Driving Forces}}$$

$$FS = \frac{\tau L}{P_a}$$

$$FS = \frac{0.5\ \gamma H \tan \delta\ L}{0.5\ \gamma H^2\ K_a}$$

and,

$$\tan \delta = \frac{(FS)\ H\ K_a}{L}$$

where

δ = angle of shearing resistance between the fabric and the embankment soil

H = embankment height

L = embankment length

K_a = coefficient of active earth pressure = $\tan^2 (45 - \phi/2)$

ϕ = angle of shearing resistance of soil

For the typical case of low slope conditions, (recall Table 1), the required "tan δ" values can usually be met by a reasonably competent embankment soil having good frictional characteristics. The calculation should be done incrementally from maximum height of embankment to the toe of the embankment, where conditions usually become more severe. It becomes a very critical problem when soft soils are used for the embankment above the reinforcing geotextile.

Summary and Conclusions

12. Presented in this paper are a number of case histories using geotextiles on extremely soft foundation soils. Distinctions were made; between linear embankments for containment dikes or barrier purposes and areal fills for stabilization purposes. Both situations are similar and each have tremendous areas for application, they are currently seeing intense activity.

13. The design aspects for both cases were outlined where the allowable fill height and slope conditions above the geotextile were first described. It is at this point where one decides about the use of a rapid consolidation technique. If considered to be desirable, then the use of strip drains should be implemented. Two case histories were discussed using this technique. At this point the design focuses on the calculation of the following required fabric properties:

- Ultimate strength in major stress direction.
- Ultimate strength in minor stress direction.
- Seam strength in minor stress direction.
- Elastic modulus in both directions.
- Strain at failure in both directions.
- Anchorage length.
- Friction.

14. While the design methods presented are considered to be reasonable, the monitoring of the in-situ performance of the various systems has lagged behind the relatively large number of construction projects. The actual performance of the reinforcing geotextile material as determined by stress and/or strain monitoring will eventually tell of the appropriateness of these design methods. Work is also ongoing in this regard and will be reported in the near future. At a minimum, it is considered that better insight is needed in the following areas;

- Actual stress levels immediately after construction versus the models proposed.

- Long term stress levels to see if creep allowances are justified, (recall Table 3).
- Effect of punching holes in the geotextile when using strip drains.
- Mechanism of load transfer over seams, particularly field seams.
- Innovative, and possibly new, joining methods to transfer tensile stresses over 175 kN/m (1000 lb/in.).
- Verification of required values of elastic modulus and strain at failure.
- Information on anchorage mechanisms, design and mobilization.
- Information on friction behavior and mobilization along the length of the fabric.

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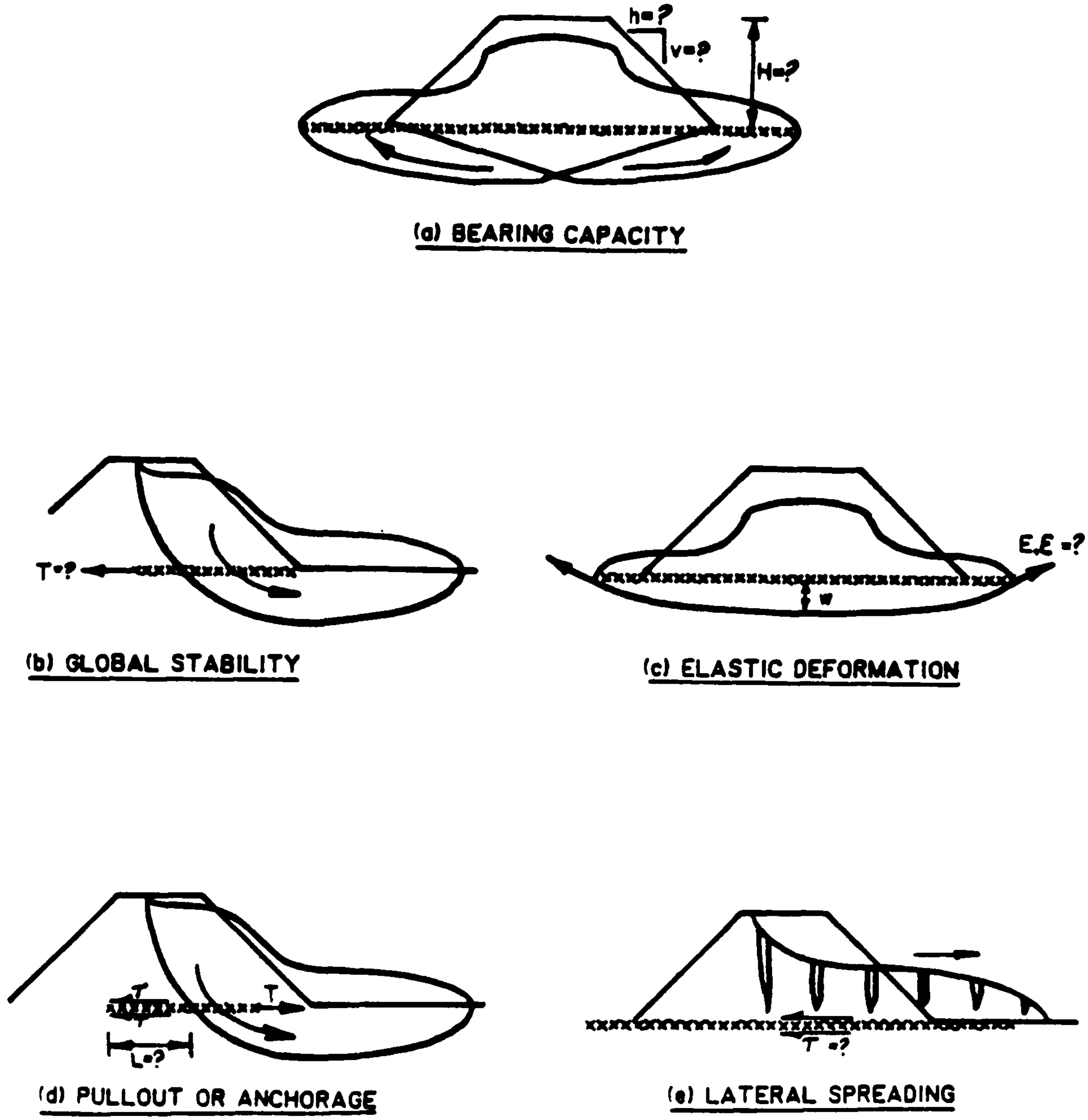

Figure 1. Geotextile design models for use in soft soil stabilization

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