

I GEOSINTETICI NELLE OPERE DI SOSTEGNO DEL TERRENO: MURI RINFORZATI E PENDII RINFORZATI

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21 maggio 2019

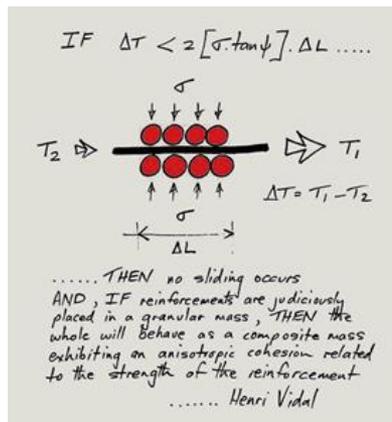


Sommario

- I materiali geosintetici
- Opere di sostegno in terra rinforzata: muri rinforzati e pendii rinforzati
- Geofiltri tubolari
- Conclusioni

Un po' di storia

- 1935 → prima pubblicazione di un materiale tessuto in cotone impiegato come un attuale geotessile, per separare e stabilizzare il terreno in sito per una strada non pavimentata – South Carolina (Beckham and Mills).
- 1963 → Henry Vidal, ingegnere-architetto francese, brevetta la «Reinforced Earth®» coprendo con il brevetto ogni tipo di rinforzo e di fronte (nel 1971, invitato dalla FHWA, presenta negli stati Uniti il suo metodo)



Pragnière (Francia)
da <http://www.terre-armee.com>

- 1964-1965 → Prima struttura rinforzata realizzata in Pragnière (Francia)
- 1966 → Barrett, in un articolo, descrive un lavoro iniziato nel 1950 nel quale geotessili sono impiegati con funzione di controllo erosione al di sotto di rip-rap
- 1971 → Primo MSEW realizzato negli Stati Uniti, State Route 39, vicino a Los Angeles
- 1974 → US Forest Service utilizzarono geotessili nontessuti per realizzare pendii rinforzati
- 1985 → J.P. Giroud conia il nome «geotextiles», attualmente utilizzato in tutto il mondo.



Opere di Ingegneria con Geosintetici

Ingegneria Geotecnica

- Indagini e prove
 - Indagini in sito
 - Indagini in laboratorio
- Metodi di calcolo
 - SLU: Equilibrio Limite, Analisi Limite, Strength Reduction Technique, Metodo delle Caratteristiche...
 - SLE: analisi delle deformazioni (Terzaghi, Shmertmann, Analisi Numerica...)
- Monitoraggio

Ingegneria dei Geosintetici

- Proprietà fisiche
 - Tipo di polimero
 - Massa areica
 - T_g ,...
- Proprietà meccaniche
 - Resistenza a trazione
 - Deformazione a carico massimo
 - Rigidezza,...
- Proprietà idrauliche
 - Permeabilità normale
 - Permeabilità nel piano
- Durabilità
 - RF_{CR}
 - RF_{ID}
 - RF_W
 - RF_{CH}
 - f_s
- Interfacce

Codici

- Eurocodici
 - Anche se non trattano le strutture con geosintetici
- Annessi nazionali agli Eurocodici e altre linee guida europee
 - British Standard BS 8006-1:2010+A1:2016
 - EBGEO, 2° German Edition, 2010
 - NF P 94-270 (muri) e XP G 38064 (pendii ripidi rinforzati)
- Codici internazionali
 - AASHTO Bridge Design Specifications
 - FHWA-NHI-10-024 e 025 Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes
 - ...



International Organization for Standardization

Great things happen when the world agrees



ISO/TC 221

Geosynthetics

About

Secretariat: BSI

Secretary: [Mr David Hyde](#)

Chairperson (until end 2017): Steve Corbet

ISO Technical Programme Manager: [Dr Anna Caterina Rossi](#)

ISO Editorial Programme Manager: [Ms Katherine Dunlop](#)

Creation date: 2000

Scope

Standardization of all geosynthetic products including geotextiles, geomembranes, geocomposite clay liners and other geosynthetic related products.

Structure

Liaisons

Reference

Title

Reference	Title
ISO/TC 221/WG 2	Terminology, Identification and sampling
ISO/TC 221/WG 3	Mechanical properties
ISO/TC 221/WG 4	Hydraulic properties
ISO/TC 221/WG 5	Durability
ISO/TC 221/WG 6	Design using geosynthetics

Chi elabora le norme sui geosintetici?

36

published ISO standards*
under the direct responsibility
of ISO/TC 221

27

ISO standards under
development*
under the direct responsibility
of ISO/TC 221

29

Participating members

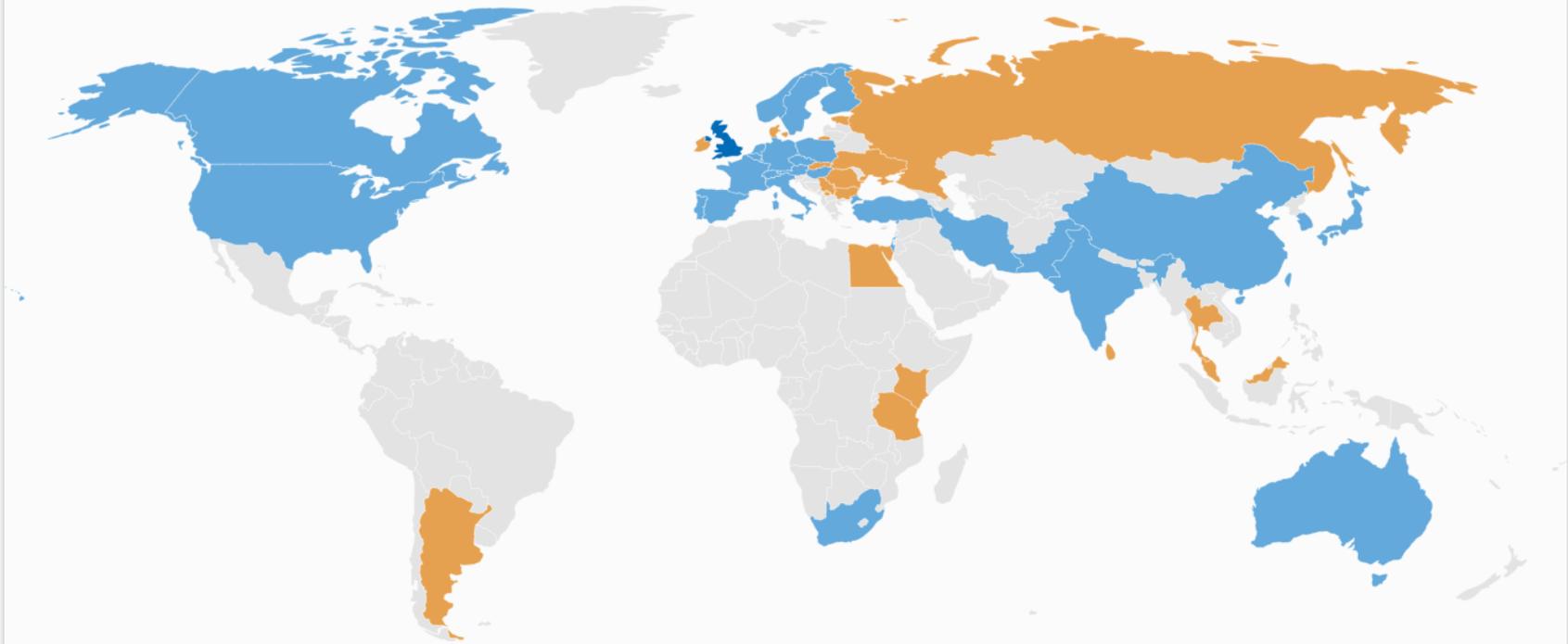
16

Observing members

Participation

ISO/TC 221

Geosynthetics



Secretariat ■

United Kingdom - British Standards Institution (BSI)

Participating Members (29) ■

Observing Members (16) ■



European Committee for Standardization



GEN COMMUNITY

TECHNICAL BODIES

STANDARDS EVOLUTION AND FORECAST

SEARCH STANDARDS

[Technical Bodies](#) > **CEN/TC 189**

CEN/TC 189 - Geosynthetics

General

Structure

Work programme

Published Standards

EN

FR

DE

CEN/TC 189 Subcommittees and Working Groups



Working group	Title
CEN/TC 189/WG 1	Geotextiles and geotextile-related products - General and specific requirements
CEN/TC 189/WG 2	Terminology, identification, sampling and classification
CEN/TC 189/WG 3	Mechanical testing
CEN/TC 189/WG 4	Hydraulic testing
CEN/TC 189/WG 5	Durability
CEN/TC 189/WG 6	Geosynthetic barriers - General and specific requirements



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All

Page 1 of 110 results

ASTM D6706-01(2013) Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil

Active Standard(Latest Version)

1.1 Scope Previous Next | Top Bottom 1.1 Resistance of a **geosynthetic** to pullout from soil is determined using a laboratory pullout box. 1.2 The test method is intended to be a performance test conducted as closely as possible to replicate design or as-built conditions. It can also be used to compare different geosynthetics, soil types, etc., and ... More

Other Searches:

Geosynthetics • Laboratory Apparatus • Performance Test • Pull-Out Boxes • More...

ASTM D5993-14 Standard Test Method for Measuring Mass Per Unit of Geosynthetic Clay Liners

Active Standard(Latest Version)

1.1 Scope Previous Next | Top Bottom 1.1 This test method covers the laboratory determination of the mas



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GSI's Mission is to develop and transfer knowledge, assess and critique geosynthetics, and provide services to the member organizations.

The Geosynthetic Institute (GSI) is a consortium of organizations interested in, and involved with, geosynthetics. All types of polymeric geosynthetic materials are involved: geotextiles, geomembranes, geogrids, geonets, geocomposites, geosynthetic clay liners, geopipe, geocells, and geofam. The organizations include federal and state governmental agencies, facility owners, designers, consultants, QC and QA organizations, testing laboratories, resin and additive suppliers, manufacturers, manufacturer's representatives and installation contractors. GSI currently consists of 71 member organizations of which ca. 45% are international.

New 6th Edition of Designing with Geosynthetics



Alcuni Standard ISO

Standard and/or project (53)	Stage	TC
ISO 9863-1:2016 Geosynthetics -- Determination of thickness at specified pressures -- Part 1: Single layers	60.60	ISO/TC 221
ISO 9863-2:1996 Geotextiles and geotextile-related products -- Determination of thickness at specified pressures -- Part 2: Procedure for determination of thickness of single layers of multilayer products	90.93	ISO/TC 221
ISO 9864:2005 Geosynthetics -- Test method for the determination of mass per unit area of geotextiles and geotextile-related products	90.93	ISO/TC 221
ISO 10318-1:2015 Geosynthetics -- Part 1: Terms and definitions	60.60	ISO/TC 221
ISO 10318-1:2015/DAmD 1 [Under development]	40.60	ISO/TC 221
ISO 10318-2:2015 Geosynthetics -- Part 2: Symbols and pictograms	60.60	ISO/TC 221
ISO 10318-2:2015/DAmD 1 [Under development]	40.60	ISO/TC 221
ISO 10319:2015 Geosynthetics -- Wide-width tensile test	60.60	ISO/TC 221
ISO 10320:1999 Geotextiles and geotextile-related products -- Identification on site	90.92	ISO/TC 221

Alcune definizioni dalla UNI EN ISO 10318-1:2015

Geosintetico; GSY: Termine generico che descrive un prodotto, del quale almeno uno dei componenti è composto di un polimero sintetico o naturale, sotto forma di foglio, striscia o struttura tridimensionale, utilizzato in contatto con il terreno e/o altri materiali in applicazioni geotecniche e di ingegneria civile.

Geotessile; GTX: Materiale tessile polimerico (sintetico o naturale), permeabile, planare, che può essere non tessuto, a maglia o tessuto, utilizzato in contatto con il terreno e/o altri materiali in applicazioni geotecniche e di ingegneria civile.

Geogriglia; GGR: Struttura planare, polimerica, che consiste di una rete regolare aperta di elementi resistenti a trazione interamente collegati, che possono essere connessi mediante estrusione, saldatura o interlooping o intrecciatura, le cui aperture sono più larghe dei costituenti.

E gli Eurocodici?

CEN/TC 250 - Structural Eurocodes

General Structure Work programme Published Standards

 European Committee for Standardization

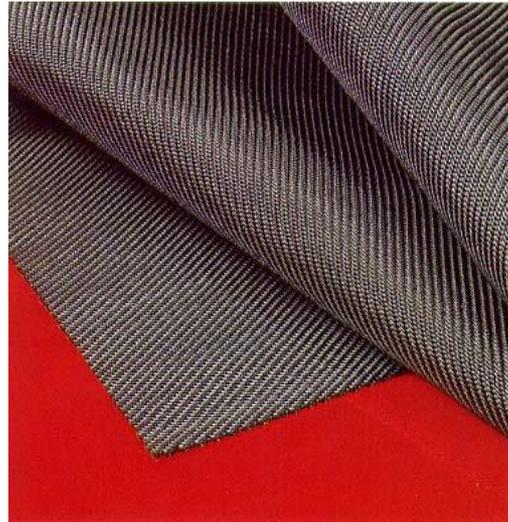
EN FR DE

CEN/TC 250 Subcommittees and Working Groups 

Subcommittee	Title
CEN/TC 250/SC 1	Eurocode 1: Actions on structures
CEN/TC 250/SC 10	EN 1990 Basis of structural design
CEN/TC 250/SC 11	Structural Glass
CEN/TC 250/SC 2	Eurocode 2: Design of concrete structures
CEN/TC 250/SC 3	Eurocode 3 - Design of steel structures
CEN/TC 250/SC 4	Eurocode 4: Design of composite steel and concrete structures
CEN/TC 250/SC 5	Eurocode 5: Design of timber structures
CEN/TC 250/SC 6	Eurocode 6: Design of masonry structures
CEN/TC 250/SC 7	Eurocode 7 - Geotechnical design
CEN/TC 250/SC 8	Eurocode 8: Earthquake resistance design of structures
CEN/TC 250/SC 9	Eurocode 9: Design of aluminium structures
Working group	Title
CEN/TC 250/WG 1	Policy, procedures and links with other standards
CEN/TC 250/WG 2	Assessment and Retrofitting of Existing Structures
CEN/TC 250/WG 4	Fibre reinforced polymer structures
CEN/TC 250/WG 5	Membrane Structures
CEN/TC 250/WG 6	Robustness

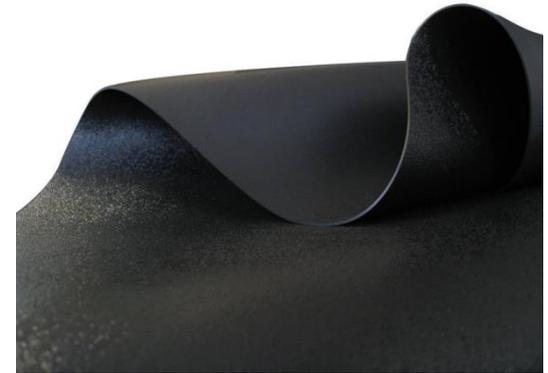


Geotessili nontessuti



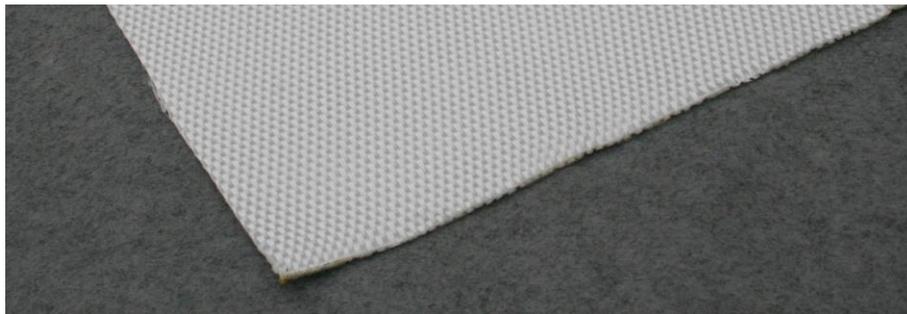
Geotessili tessuti in PP

I materiali geosintetici

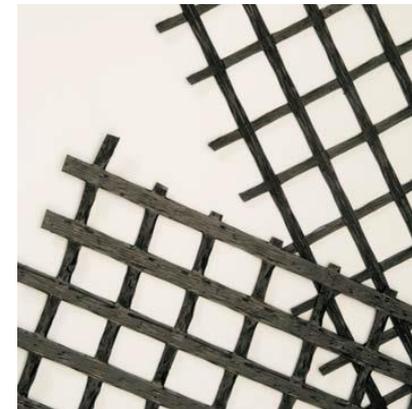


Geomembrane

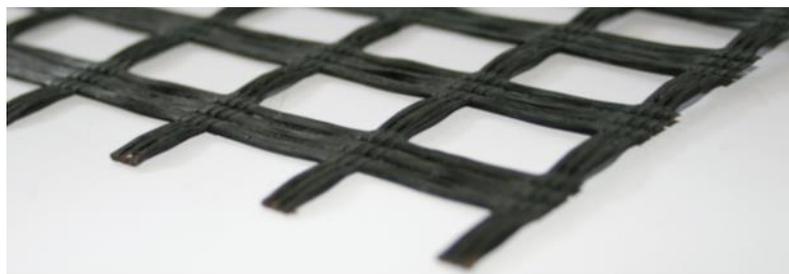
Geotessili tessuti in PET



Geogriglie



I materiali geosintetici



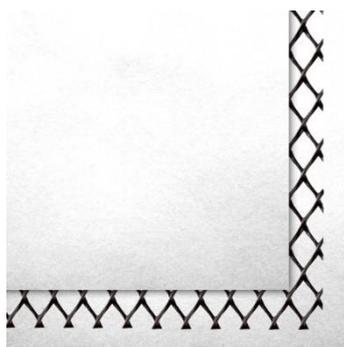
Geogriglie rinforzo asfalto



Biostuoie



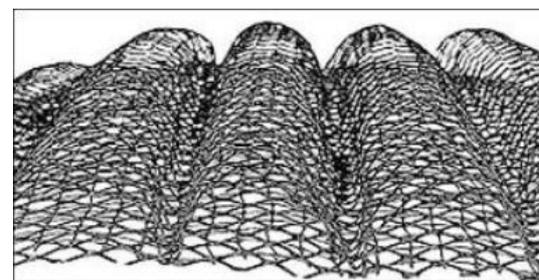
Geocompositi bentonitici



Geocompositi drenanti



Geocelle



Geostuoie



Geomaterassi

I materiali geosintetici



Filter Unit



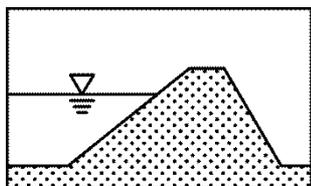
Geofiltri tubolari



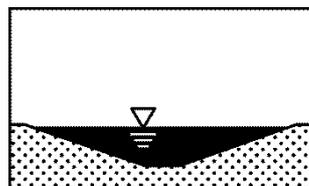
Applicazioni

UNI EN ISO 10318-2:2015

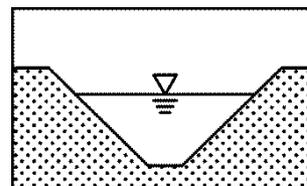
Geosintetici Parte 2 – Simboli e pittogrammi



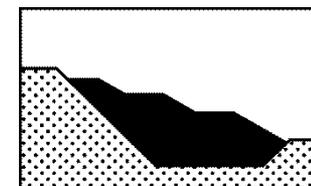
Bacini e dighe



Rifiuto liquido



Canali



Rifiuti solidi



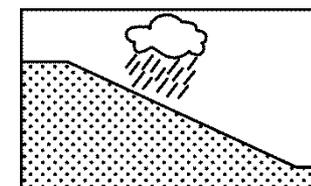
Infrastruttura di trasporto e strade



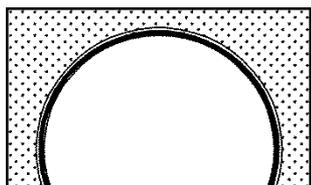
Fondazioni e pareti di sostegno



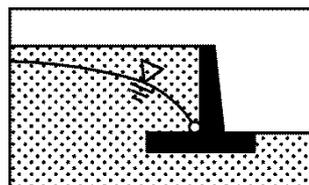
Ferrovie



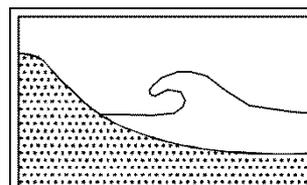
Sistemi di controllo dell'erosione di superficie



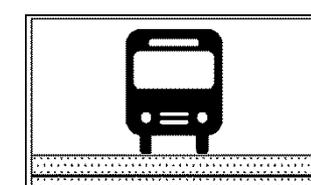
Gallerie e strutture in sotterraneo



Sistemi drenanti



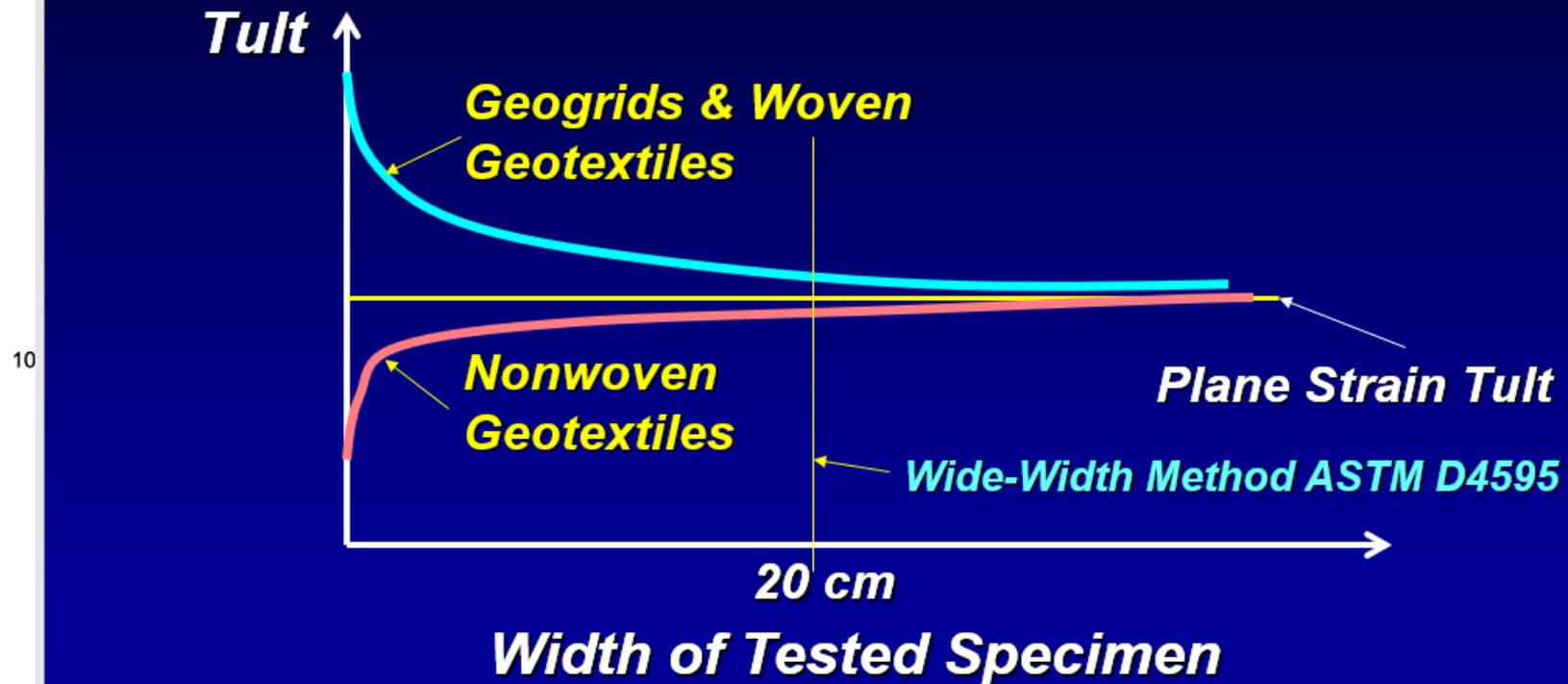
Controllo dell'erosione costiera



Armatura dell'asfalto

La funzione di rinforzo

Effect of Specimen Width on Measured Tult



La resistenza dei geosintetici con funzione di rinforzo 1/2

- Si può determinare in base a **PD ISO/TR 20432:2007** «Guidelines for the determination of the long strength of geosynthetics for soil reinforcement»
- Essa definisce:
 - **Resistenza**
 - T_{char} = Resistenza caratteristica dei geosintetico
 - **Fattori di riduzione**
 - RF_{CR} = fattore di riduzione che tiene conto del creep (per tener conto degli effetti di un carico statico applicato alla temperatura di esercizio)
 - RF_{ID} = fattore di riduzione per tener conto del danneggiamento durante la posa
 - RF_W = fattore di riduzione per tener conto degli effetti degli agenti atmosferici (UV, umidità...) durante l'esposizione prima dell'installazione o copertura o per materiali permanentemente esposti
 - RF_{CH} = fattori di riduzione per tener conto della riduzione di resistenza prodotta dagli effetti chimici e biologici alla temperatura di progetto
 - fs = fattore di sicurezza per tener conto della variazione statistica dei fattori di riduzione calcolati. Non tiene conto delle incertezze relative alla struttura in terra da realizzare e ai carichi.

La resistenza dei geosintetici con funzione di rinforzo 2/2

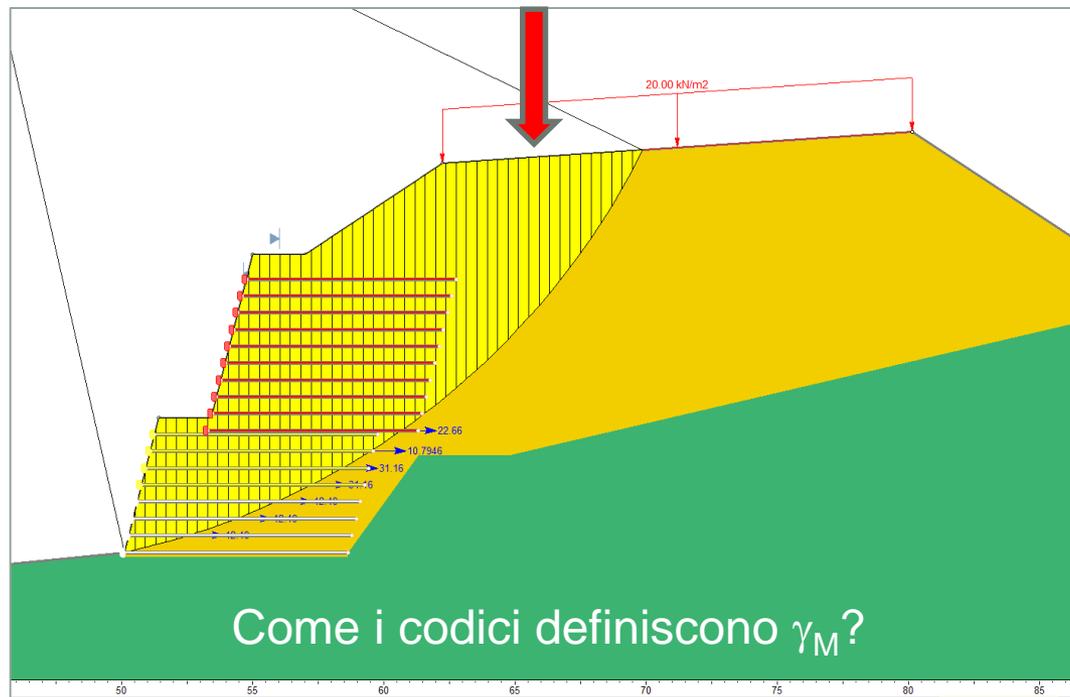
Long term strength per width:

$$T_D = T_{char} / (RF_{CR} \cdot RF_{ID} \cdot RF_W \cdot RF_{CH} \cdot f_s)$$

$$T_D = T_{LTDS} = T_{avail}$$

Si può utilizzare T_D nei dimensionamenti delle strutture?

$$T_{all} = T_D / \gamma_M$$



EBGEO 2011

$$R_{B,k} = \frac{R_{B,k0}}{A_1 \cdot A_2 \cdot A_3 \cdot A_4 \cdot A_5}$$

where:

$R_{B,k0}$ = characteristic value of the short-term of the geosynthetics (5% quantile)

$R_{B,k}$ = characteristic value of the long-term of the geosynthetics

A_1 = reduction factor for considering creep strain or creep rupture behavior,

A_2 = reduction factor for considering any damage caused during installation, transportation and compaction

A_3 = reduction factor for considering processing (seams, connections, joints)

A_4 = reduction factor for considering environmental impact

A_5 = reduction factor for considering the impact of dynamic actions

$$R_{B,d} = \frac{R_{B,k}}{\gamma_M}$$

	LC1	LC2	LC3
γ_M	1,40	1,30	1,20

BS 8006-1:2010+A1:2016

$$T_D = T_{CR} / f_m$$

T_{CR} = is the long-term tensile creep rupture strength of the reinforcement at the specified design life and design temperature

f_m = is the material safety factor to allow for the strength reducing effects of installation damage, weathering (including exposure to sunlight), chemical and other environmental effects and to allow for the extrapolation of data required to establish the above reduction factors

$$T_{CR} = T_{char} / RF_{CR}$$

T_{char} = is the characteristic short-term strength of the geogrid taken from Table 2

RF_{CR} = is the reduction factor for creep

$$f_m = RF_{ID} \times RF_W \times RF_{CH} \times f_s$$

RF_{ID} = is the reduction factor for installation damage

RF_W = is the reduction factor for weathering, including exposure to ultra violet light

RF_{CH} = is the reduction factor for chemical/environmental effects

f_s = is the factor of safety for the extrapolation of data

$$\frac{T_D}{f_n} \geq T_r$$

Table 9 Category of structure depending upon ramification of failure

Category	Partial factor f_n	Examples of structures
1 (low) ^{A)}	1.0 If analysis is undertaken	Retaining walls and slopes less than 1.5 m in retained height above finished ground level in front where failure would result in minimal damage and loss of access
2 (medium)	1.0	Embankments and structures where failure would result in moderate damage and loss of services
3 (high)	1.1	Abutments, structures directly supporting motorway, trunk and principal roads or railways or inhabited buildings, dams, sea walls and slopes, river training walls and slopes

NOTE See Figures 10 to 14 for examples of structures in categories 1, 2 and 3.

^{A)} Structures in category 1 should be restricted to small and relatively simple structures, with negligible risk, which may be designed by experience without analysis as described in BS EN 1997-1:2004.

NF P94-270 Calcul Géotechnique Ouvrages de soutènement Remblais renforcés et massifs en sol cloué

$$R_{t;d} = \rho_{end} \rho_{flu} \rho_{deg} \frac{R_{t;k}}{\gamma_{M;t}}$$

$R_{t;d}$ est la valeur de calcul de la résistance ultime de traction au point considéré ($R_{tc;d}$ ou $R_{ta;d}$) par mètre de parement

ρ_{end} , ρ_{flu} et ρ_{deg} sont des coefficients de réduction qui traduisent les diminutions de résistance possibles au point considéré (point de traction maximale ou point d'attache au parement) du fait respectivement :

ρ_{end} des agressions mécaniques (endommagement) subies par les renforcements lors de leur mise en place dans l'ouvrage

ρ_{flu} des phénomènes de fluage qui peuvent affecter la structure du matériau constitutif des renforcements

ρ_{deg} des agressions chimiques (dégradation) dues à l'environnement dans lequel les renforcements se trouvent placés

$R_{t;k}$ est la résistance de traction caractéristique du lit de renforcement ($R_{tc;k}$ ou $R_{ta;k}$) par mètre de parement

$\gamma_{M;t}$ est le facteur de sécurité partiel pour la résistance à la traction des éléments de renforcement.

NF P94-270

Tableau C.4.2.1- Facteurs partiels de matériau (γ_M) pour les renforcements géosynthétiques des ouvrages en sol renforcé

Propriétés	Symbole	Ensemble	
		M1 ^a	M2 ^b
Résistance en traction caractéristique	$\gamma_{M,t}$	1,25	1,25
a Stabilité interne.			
b Stabilité mixte.			

Tableau C.4.3 – Facteurs partiels de la résistance d'interaction sol-lit de renforcement

Résistance	Symbole	Ensemble	
		M1 ^a	M2 ^b
μ ^c tiré d'une base de données documentée	$\gamma_{M,t}$	1,35	1,1
q_s ^d déduit d'essais d'arrachement	$\gamma_{M,t}$	1,4	1.1
a Stabilité interne.			
b Stabilité mixte.			
c Interaction sol-renforcement τ_{max} pour les ouvrage en remblai renforcé.			
d Interaction terrain-clou τ_{max} pour les massifs cloués.			

Altre indicazioni in:

- FHWA-NHI-10-024 e FHWA-NHI-10-024
- GSI GRI GG4(b) - Determination of the Long-Term Design Strength of Flexible Geogrids

Determinazione degli RFs

- **PD ISO TR 20432:2007**
«Guidelines for the determination of the long-term strength of geosynthetics for soil reinforcement»
- Non confondere **index test** con il “**protocollo**” (tests e relative estrapolazioni) **per la determinazione degli RFs**:
- Ci sono tre livelli di attività di laboratorio:
 - **Index test**
 - **Quasiperformance test**
 - **Performance test**
- **Index tests**: sono i più semplici ma non simulano le condizioni in sito (Quality control purpose).
- **Performance tests**: I più complessi e simulano le condizioni in sito (per corroborare metodi di calcolo empirici).
- **Quasiperformance tests**: molto vicini a performance tests ma con alcune semplificazioni e approssimazioni (conformance testing e sviluppo di metodi di calcolo empirici).

Esempio: Installation Damage

Definizione

- RF_{ID} = is a reduction factor to allow for the effect of mechanical damage;

ISO TR 20432

- RF_{ID} = a number reflecting the expected damage to occur during construction. This value depends mainly on the type of backfill soil, the construction equipment, procedure, and field quality control. It also depends on the geosynthetic type.

Leshchinsky, D., ReSSA User Manual

Esempio: Installation Damage

- UNI EN ISO 10722:2007

Geosintetici - Procedimento di **prova indice** per la valutazione del danneggiamento meccanico sotto carico ripetuto - Danneggiamento causato da materiale granulare

1 Scope

This International Standard describes an index test procedure for simulating mechanical damage to geosynthetics, caused by granular material, under repeated loading. The damage is assessed visually and by the loss of tensile strength.

Other reference tests may be used to assess the damage caused by this test. The test method described is an index test procedure, using a standard granular material.

4 Principle

A geosynthetic specimen is placed between two layers of a synthetic aggregate and subjected to a period of dynamic loading. The geosynthetic specimen is then removed from the test apparatus, examined for any visual damage and then subjected to a mechanical or hydraulic test, to measure the change in mechanical or hydraulic properties. The result is expressed as the change (in percent) of the reference property. The visual damage is also reported.

7 Apparatus

7.1 **Compression machine**, which can be controlled to produce a sinusoidal pressure of between (500 ± 10) kPa and $(5 \pm 0,5)$ kPa on the loading plate, at a frequency of 1 Hz.

7.2 **Test container**.

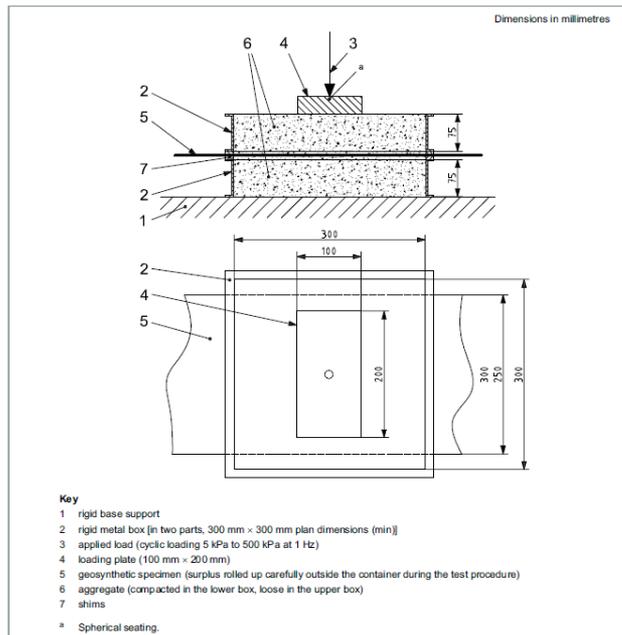
The test container shall be a rigid metal box of minimum 300 mm by 300 mm internal dimensions in the plane, and shall consist of two parts, each 75 mm deep. The two parts of the box may be bolted or clipped together during the damage procedure. Shims of sufficient thickness to allow the specimen to be retained without generating any tension shall be fitted between the two parts. The lower part of the box shall be mounted on a rigid base which deflects less than 1 mm when the test load is applied directly to the base, or shall be rigidly fixed to the hydraulic piston of the compression machine. Figure 1 shows a typical arrangement of the apparatus.

7.4 Aggregate.

The aggregate used in the damage procedure is a sintered aluminium oxide¹⁾, with the following requirement on grain size:

- when tested in accordance with EN 933-1, 100 % of the aggregate shall pass a 10 mm sieve and 0 % shall pass a 5 mm sieve.

If necessary, the aggregate shall be wet sieved on a 5 mm aperture sieve at the first use and after every three uses, and any material passing the sieve shall be discarded. The aggregate shall be totally discarded after twenty uses.



8.1 Damage procedure

Fill the lower part of the test container with the aggregate. Place the aggregate in two equal layers, each compacted with a flat plate loaded to a pressure of (200 ± 2) kPa, for 60 s, over the whole area of the test container. Strike the top layer off level with the top of the lower part of the container.

Place the specimen across the top of the lower part of the container, the centre of the specimen aligned with the centre of the container and with the free edges equally spaced from each side of the container. If necessary, roll up the free ends of the specimen outside the container in such a way that the specimen is not damaged during the test. Position the upper part of the container and bolt or clip it into position making sure the specimen is smooth, free from wrinkles but not pre-tensioned. Then, loosely fill the upper part of the container with aggregate to a depth of 75 mm above the specimen.

Position the loading plate centrally in the container and apply a load of $(5 \pm 0,5)$ kPa. Set the loading apparatus to produce a cyclic loading of between (10 ± 10) kPa minimum and (500 ± 10) kPa maximum, at a frequency of 1 Hz for 200 loading cycles. Determine the pressure using the area of the loading platen, not the area of the test container.

Remove the specimen carefully from the test container, ensuring that no additional damage occurs during removal.

Repeat the procedure for other specimens from the sample.

8.2 Measurement of damage

Measure the damage by subjecting the reference sub-specimen and the damaged sub-specimen to the same reference test. The reference test shall normally be ISO 10319, but other mechanical or hydraulic reference tests may be selected, i.e. ISO 12236, ISO 12956 or ISO 11058.

The reference sub-specimen is not a virgin product, but it consists of a specimen installed over the aggregate, subsequently subjected to a static pressure of (500 ± 10) kPa for 60 s, and finally taken out to be submitted to the reference test.

The reference test to be used shall be agreed between parties prior to the commencement of the test.

Optionally, the specimen is visually examined for damage and an assessment of the damage is made. The examination includes making a record of the number of holes in the specimen and a qualitative assessment of any damage.

9 Calculations

Calculate the change in the reference property as

$$\Delta R = 100 \times \left(\frac{R_d}{R_0} \right)$$

where

R_d is the reference test value of the damaged specimen;

R_0 is the reference test value of the reference specimen;

ΔR is the percentage retained of the reference test value (damage index).

Calculate the mean value of ΔR , the standard deviation and the coefficient of variation.

Attenzione!

- **200** carichi ciclici da 10 kPa a 500 kPa (\approx pressione gonfiaggio ruote gommate di mezzi pesanti)
- Confronto con campione sottoposto a carico statico di 500 kPa per 60 s (quindi danneggiato)
- Aggregati da 5 mm a 10 mm

Index test per geosintetici già posati in opera (campione di riferimento danneggiato) e sottoposti a carichi ciclici ($p=500$ kPa), non è un index test per il danneggiamento durante la posa in opera.

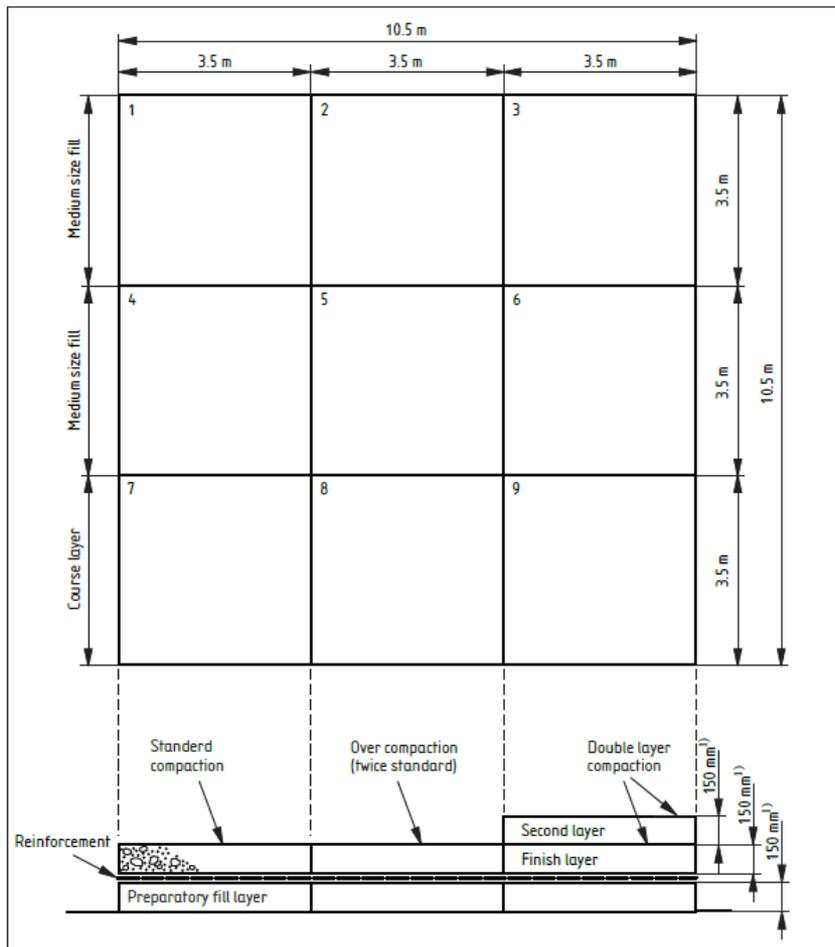
Cosa suggerisce PD ISO TR 20432?

8.4.4 Laboratory damage tests

It should be noted that ISO 10722 is intended as an index test for comparative purposes and should not be used for the derivation of reduction factors for geosynthetic soil reinforcements.

«Site damage test»

Figure D.1 Schematic layout of test bays



- Inoltre PD ISO TR 20432 suggerisce, per determinare RF_{id} , di utilizzare la procedura riportata in BS 8006, 1995 Annex D «**Site damage test**»
 - 3 tipo di terreno (coarse, middle, fine fill)
 - Compattazione secondo «Specification for Highway Works», come avverrà in sito
 - 3 livelli di compattazione: Standard Compaction; Over-Compaction; Double layer compaction.
 - Confronto con campioni vergini.

Site damage test (da Durability of Geosynthetics, SBR CUR net)



1)

Fig. 3.73 Smoothing the first layer of backfill (ERA Technology Ltd/Geoblock Ltd).



2)

Fig. 3.74 Placing the aggregate on the samples (ERA Technology Ltd/Geoblock Ltd).



3)

Fig. 3.75 Compacting the second layer of backfill (ERA Technology Ltd/Geoblock Ltd).



4)

Fig. 3.76 Final excavation using hand tools (ERA Technology Ltd/Geoblock Ltd).

ASTM D5818-11

Standard Practice for Exposure and Retrieval of Samples to Evaluate Installation Damage of Geosynthetics

1. Scope

1.1 This practice covers standardized procedures for obtaining samples of geosynthetics from a test section for use in assessment of the effects of damage immediately after installation caused only by the installation techniques. The assessment may include physical testing. This practice is applicable to any geosynthetic.

1.2 This practice is limited to full-scale test sections, and does not address laboratory modeling of field conditions. This practice does not address which test method(s) to use for quantifying installation damage.

1.3 *This standard safety concerns, if the responsibility of the user, appropriate safety and healthability of regulatory limits.*

6.4 Installation Procedures

6.4.1 The soil subgrade and geosynthetic(s) will be placed under conditions of soil type, moisture content and compaction. Construction equipment used should be the same as to be used

to construct overlying lifts, unless otherwise requested. The geosynthetic should be installed in accordance with project specific procedures. When project specific procedures and/or materials are not known, representative equipment, materials and procedures should be used and thoroughly documented.

6.4.2 The material to be placed above the geosynthetic under investigation will typically be a soil or rock fill, or another geosynthetic material(s) with soil then placed upon it. In the case of another geosynthetic, it also shall be placed in accordance with project specific procedures. When project specific procedures and/or materials are not known, representative equipment, materials and procedures should be used and thoroughly documented.

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NOTE 2—Commonly used procedures include the following: 200 mm soil lifts compacted to >90 % Modified Proctor density using a minimum 4500 kg (total) vibratory steel roll (single or tandem). Typical soils/aggregates include coarse gravel (GP, d₅₀ > 20 mm) concrete sand (SW d₅₀ > 1.0 mm) or silty sand (SM d₅₀ > 0.4 mm).

NOTE 3—Following placement and compaction for the lift, procedures could include the simulation of post construction traffic such as loaded trucks moving transversely over the test section.

6.4.3 The lift shall be constructed to specified conditions. Construction equipment used in fill spreading shall be operated in accordance with project specific procedures. When project specific equipment, procedures and/or materials are not known, representative equipment, materials and procedures should be used and thoroughly documented.

6.4.4 Spread fill into lifts above the geosynthetic modeling expected field conditions. Construction equipment used in fill spreading should be in accordance with project specific procedures. When project specific equipment, procedures and/or materials are not known, representative equipment, materials and procedures should be used and thoroughly documented.

6.4.5 Fill lift compaction above the geosynthetic shall model expected field conditions. Construction equipment used in soil compaction should be in accordance with project specifications. When project specific equipment, procedures and/or materials are not known, representative equipment, materials and procedures should be used and thoroughly documented.

Tipologie di decadimento della resistenza

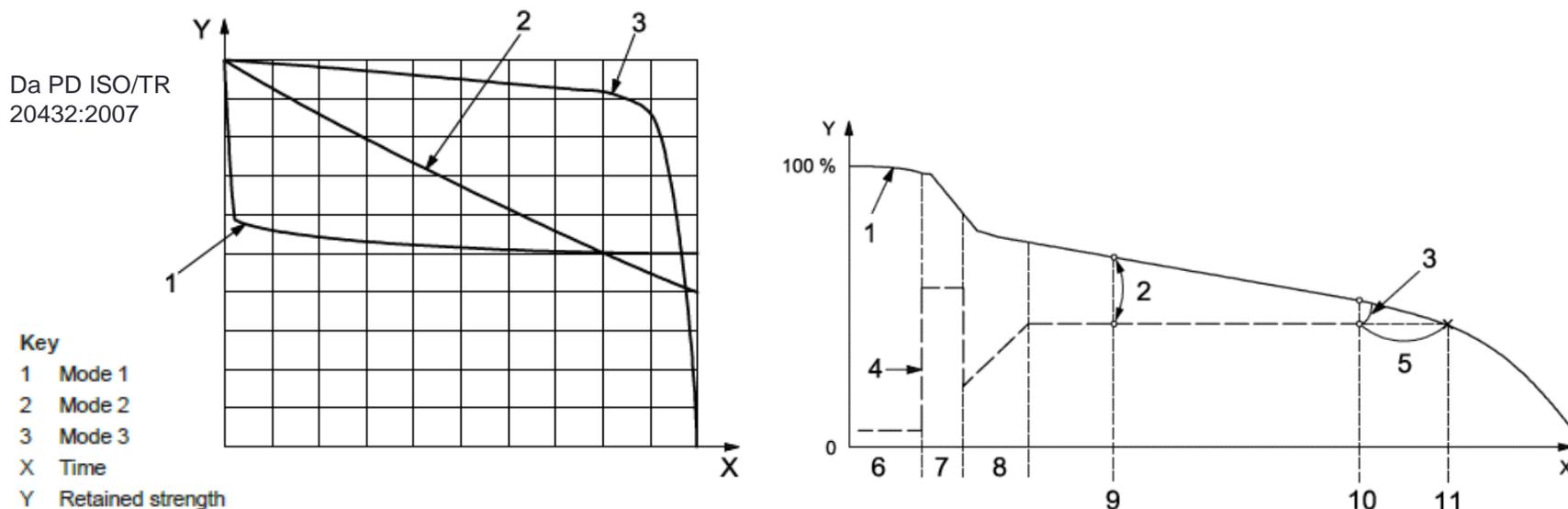


Figure 3 — Retained strength plotted against time for the three Modes of degradation

Tipo 1: Immediata riduzione della resistenza, insignificanti ulteriori riduzioni (es. danneggiamento durante l'installazione)

Tipo 2: Riduzione graduale, ma non necessariamente costante, della resistenza (es. degradazione Chimica - Idrolisi del PET)

Tipo 3: Nessuna riduzione per un certo periodo, poi inizio rapido di perdita di resistenza (es. Creep)

Quale significato ha la resistenza caratteristica T_{char} per un Geosintetico? 1/2



Ripreso da Ghersi (2004).

Quale significato ha la resistenza caratteristica T_{char} per un prodotto? 2/2

- Valore medio: $\bar{X} = \frac{X_1 + X_2 + \dots + X_N}{N}$
- Deviazione Standard corretta: $S = \sqrt{\frac{(X_1 - \bar{X})^2 + (X_2 - \bar{X})^2 + \dots + (X_N - \bar{X})^2}{N-1}}$
- Valore caratteristico: $X_k = \bar{X} - k \cdot S$
- $k=1,64$ se la grandezza ha la probabilità di essere superata pari al 95% (EN 1990:2002+A1 2005 – Par. 4.2.(3))

Per i geosintetici, soprattutto negli USA, si definisce anche:

- MARV=Minimum Average Roll Value. Rappresenta il valore di una grandezza che, in un rotolo di materiale, ha una probabilità di essere superata pari al 97,5%.

$$MARV = \bar{X} - 2 \cdot S$$

- $k=2$

Generalmente in Europa, il valore nominale del geosintetico corrisponde a T_{char} .

La terra rinforzata

- Definizione da UNI EN 14475:2006 «Esecuzione di lavori geotecnici speciali - Terra rinforzata»

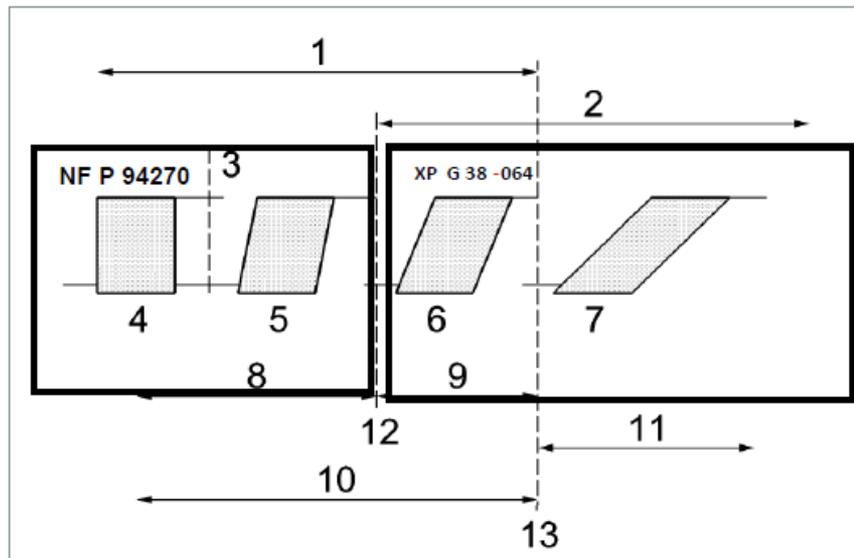
3	TERMINI E DEFINIZIONI
	Ai fini della presente norma europea, si applicano i termini e le definizioni seguenti.
3.1	terra: Materiale naturale o realizzato dall'uomo, formato da particelle solide, comprese alcune rocce, utilizzato per costruire terra ingegnerizzata.
3.2	rinforzo: Termine generico per le inclusioni di rinforzo poste all'interno della terra.
3.3	terra ingegnerizzata: Terra che è posta in opera e compattata in condizioni controllate.
3.4	terra rinforzata: Terra ingegnerizzata che incorpora strati discreti di rinforzo di terreno, posizionato in generale in orizzontale, che sono disposti tra strati successivi di terra durante la costruzione.
3.5	rinforzo della terra: Rinforzo che aumenta la stabilità della massa della terra rinforzata mediante mobilitazione della resistenza a trazione assiale del rinforzo della terra per interazione con il terreno per tutta la sua lunghezza.
	<i>Nota</i> È solitamente sotto forma di una striscia, un foglio, una barra, una griglia o una rete, ed è di solito posto in strati discreti.
3.6	geosintetici: Ai fini della presente norma europea, "geosintetici" sta per "geotessili e prodotti affini".
3.7	paramento: Copertura della facciata esposta di una struttura di terra rinforzata, che trattiene la terra tra gli strati di rinforzo e protegge la terra contro l'erosione.
3.8	fondazione: La fondazione di una struttura di terra rinforzata è l'area totale della superficie sulla quale lo strato più basso del rinforzo è installato.
3.9	unità di paramento discreta: Unità del paramento ad altezza parziale, utilizzata per costruire per incrementi successivi la struttura di terra rinforzata.
3.10	unità di paramento a tutta altezza: Unità del paramento uguale all'altezza della facciata della struttura.

Principali documenti disponibili per la progettazione

- **Germania:** **EBGEO** Recommendations for design and Analysis of Earth Structure using Geosynthetic Reinforcements-**2011**
- **Inghilterra:** **British Standard 8006-1:2010** Code of practice for strengthened/reinforced soils and other fills
- **Francia:** **NF P 94-270 e XP G 38064** Calcul géotechnique – Ouvrages de soutènement. Remblais renforcés et massifs en sol cloué.
- **Stati Uniti:** **FHWA-NHI-10-024 e FHWA-NHI-10-025** Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes **2009**
- **Giappone:** Railway Technical Research Institute (**2007**). **Design standard for railway earth structures**, Maruzen (in Giapponese)
- **Italia:** in via di definizione da parte dell'AGI - Associazione Geotecnica Italiana

Muri e pendii rinforzati

- **BS 8006-1:2010**
 - Strutture con inclinazione 0-20° dalla verticale → muri e spalle di ponte
 - Strutture con inclinazione minore → pendii ripidi
- **NF P 94270 (2009)**. Calcul géotechnique – Ouvrages de soutènement. Remblais renforcés et massifs ensol cloué.
- **XP G 38-064 (2010)**. Murs inclinés et talus raidis en sols renforcés par nappes géosynthétiques – justification du dimensionnement et éléments de conception



$i > 1H/4V \rightarrow$ NF P 94270 (2009).

$i \leq 1H/4V \rightarrow$ XP G 38-064 (2010)

$1H/4V \cong 76^\circ$

Muri e pendii rinforzati

- **FHWA**

- Per inclinazioni rispetto alla verticale $i \geq 70^\circ$ → Muri rinforzati (Mechanically Stabilized Earth Wall - MSEW)
- Per inclinazioni rispetto alla verticale $i < 70^\circ$ → Pendii rinforzati (Reinforced Soil Slope - RSS)

$i \geq 70^\circ$



$i < 70^\circ$



Cosa cambia tra muro e pendio rinforzato?

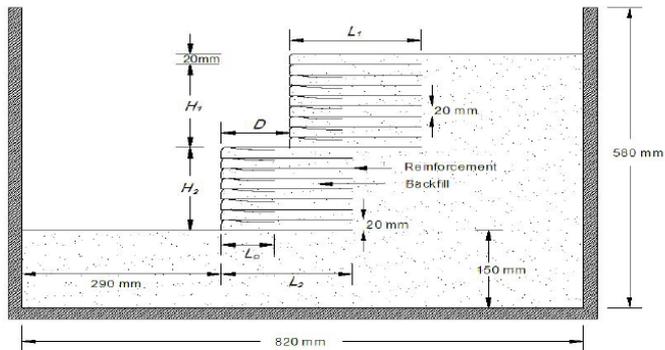


Figure 1. Schematic profile view of a centrifuge two-tiered GRS wall model (D is the off-set distance; L_1 and L_2 are the reinforcement length of the upper and lower tier; L_0 is the overlap length of reinforcement).

Hung (2008).

Breaking failure behavior and internal stability analysis of geosynthetic reinforced earth walls, Ph. D. Dissertation, National Central University, Zhongli, Taiwan.

Lateral Earth Pressure ←

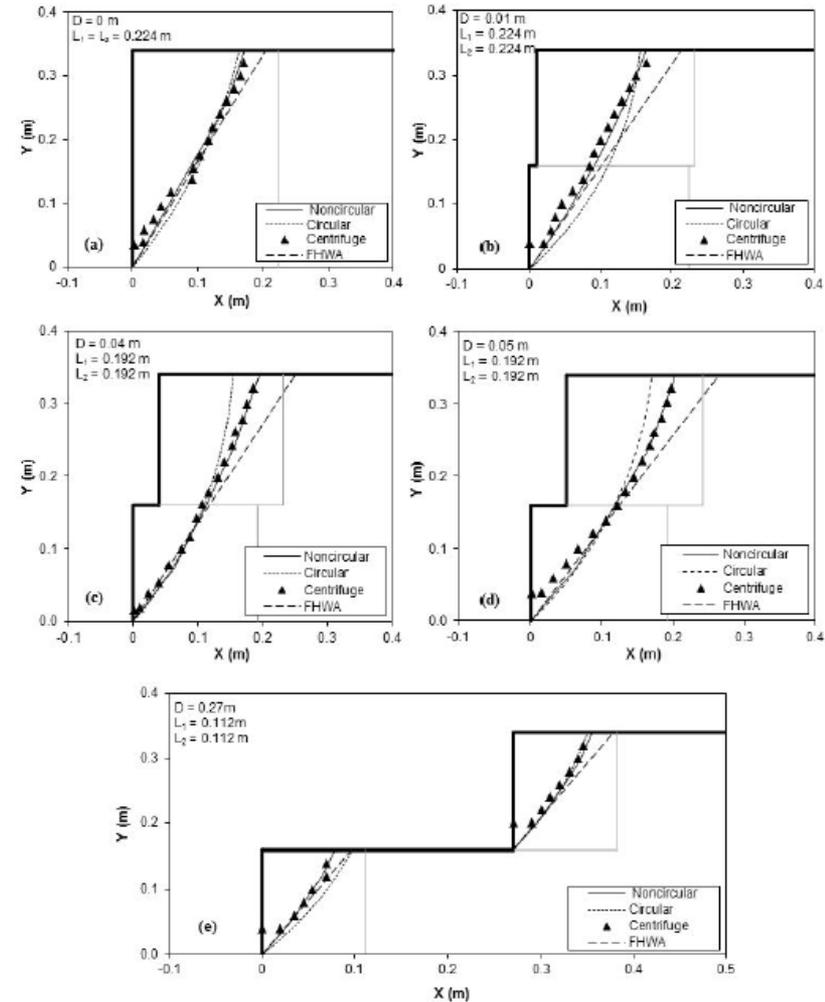


Figure 3. Predicted and measured locations of failure surfaces from centrifuge tests: (a) Test S1; (b) Test S2; (c) Test C4; (d) Test C5; (e) Test I8.

Cosa cambia tra muro e pendio rinforzato?

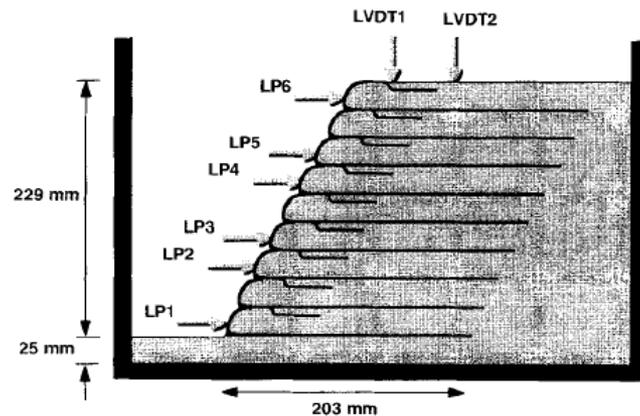
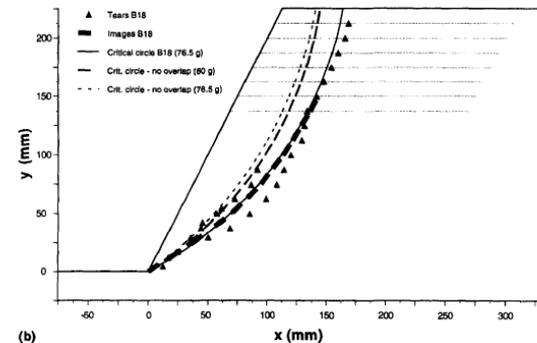


FIG. 1. Centrifuge Model with 25.4 mm Reinforcement Spacing [Linear Potentiometers (LPs) and Linear Variable Displacement Transducers (LVDTs) Are Indicated in Figure]



↓
Slope Stability

Zornberg (1998)

Limit equilibrium as basis for design
of geosynthetic reinforced slopes.

*Journal of Geotechnical and
Geoenvironmental Engineering,*
ASCE, 124(8), 684-698

Ricerche attuali propongono di utilizzare sempre LE

- Mohamed, S.B.A et al. (2013). Limit Equilibrium Analysis of Two-Tiered Geosynthetic-Reinforced Soil Wall Models in a Geotechnical Centrifuge
- FHWA-HIF-17-004. (2016) Limit Equilibrium Design Framework for MSE Structures with Extensible Reinforcement → [Dov Leshchinsky](#)

Le norme tecniche per le costruzioni NTC 2008

- Si parla di terre rinforzate?

6.5 OPERE DI SOSTEGNO

Le norme si applicano a tutte le opere geotecniche e agli interventi atti a sostenere in sicurezza un corpo di terreno o di materiale con comportamento simile:

- muri, per i quali la funzione di sostegno è affidata al peso proprio del muro e a quello del terreno direttamente agente su di esso (ad esempio muri a gravità, muri a mensola, muri a contrafforti);
- paratie, per le quali la funzione di sostegno è assicurata principalmente dalla resistenza del volume di terreno posto innanzi l'opera e da eventuali ancoraggi e puntoni;
- strutture miste, che esplicano la funzione di sostegno anche per effetto di trattamenti di miglioramento e per la presenza di particolari elementi di rinforzo e collegamento (ad esempio, ture, terra rinforzata, muri cellulari).

6.8 OPERE DI MATERIALI SCIOLTI E FRONTI DI SCAVO

6.8.2 VERIFICHE DI SICUREZZA (SLU)

Le verifiche locali devono essere estese agli elementi artificiali di rinforzo, eventualmente presenti all'interno ed alla base del manufatto, con riferimento anche ai problemi di durabilità. Nel caso di manufatti su pendii si deve esaminare l'influenza dell'opera in terra sulle condizioni generali di sicurezza del pendio, anche in relazione alle variazioni indotte nel regime idraulico del sottosuolo.

Le norme tecniche per le costruzioni NTC 2018

6.5. OPERE DI SOSTEGNO

Le norme si applicano a tutte le costruzioni e agli interventi atti a sostenere in sicurezza un corpo di terreno o di materiale con comportamento simile. In particolare :

- muri, per i quali la funzione di sostegno è affidata al peso proprio del muro e a quello del terreno direttamente agente su di esso (ad esempio muri a gravità, muri a mensola, muri a contrafforti);
- paratie, per le quali la funzione di sostegno è assicurata principalmente dalla resistenza del volume di terreno posto innanzi l'opera e da eventuali ancoraggi e puntoni;
- strutture miste, che esplicano la funzione di sostegno anche per effetto di trattamenti di miglioramento e per la presenza di particolari elementi di rinforzo e collegamento.

In presenza di azioni sismiche, oltre a quanto previsto nel presente paragrafo, le opere di sostegno devono rispettare i criteri di verifica di cui al successivo § 7.11.6.

6.8. OPERE DI MATERIALI SCIOLTI E FRONTI DI SCAVO

6.8.2. VERIFICHE DI SICUREZZA (SLU)

Le verifiche locali devono essere estese agli elementi artificiali di rinforzo eventualmente presenti all'interno ed alla base del manufatto, con riferimento anche ai problemi di durabilità. Nel caso di manufatti su pendii si deve esaminare l'influenza dell'opera in terra sulle condizioni generali di sicurezza del pendio, anche in relazione alle variazioni indotte nel regime delle pressioni interstiziali nel sottosuolo.

Circolare 2 febbraio 2009 – Istruzioni applicazione NTC 2008

C6.8.1 CRITERI GENERALI DI PROGETTO

C6.8.1.1 Rilevati e rinterri

Per i muri in terra armata o rinforzata i materiali da preferire sono costituiti da terre con passante ai 15 micron non superiore al 20%, le cui caratteristiche meccaniche e chimico fisiche devono soddisfare i requisiti richiesti comunemente per tali tipi di opere.

Per gli elementi di rinforzo dei muri in terra armata o rinforzata è necessario effettuare verifiche locali, di rottura e di sfilamento, e verifiche nei riguardi dell'azione aggressiva dell'ambiente ed in particolare delle acque.

Circolare NTC 2018

C6.8.1 CRITERI GENERALI DI PROGETTO

C6.8.1.1 RILEVATI E RINTERRI

Per i muri in terra armata o rinforzata i materiali da preferire sono costituiti da terre con passante ai 15 μm non superiore al 20%.

Per gli elementi di rinforzo dei muri in terra armata o rinforzata è necessario effettuare verifiche locali, di rottura e di sfilamento, e verifiche nei riguardi dell'azione aggressiva dell'ambiente ed in particolare delle acque.

NTC 2008 – alcuni richiami (Come NTC2018)

- Per ogni stato limite ultimo deve essere rispettata la condizione:

$$E_d \leq R_d$$

- E_d è il valore di progetto dell'azione o dell'effetto dell'azione:

$$E_d = E \left[\gamma_F F_k; \frac{X_k}{\gamma_M}; a_d \right]$$

- R_d è il valore di progetto della resistenza del sistema geotecnico:

$$R_d = \frac{1}{\gamma_R} R \left[\gamma_F F_k; \frac{X_k}{\gamma_M}; a_d \right]$$

- Dove:

- $\gamma_F F_k$ = azioni di progetto;

a_d = geometria di progetto

- $\frac{X_k}{\gamma_M}$ = parametri di progetto;

Progettazione

La scelta, non univoca, spetta al Progettista. La nostra è la seguente:

- Consideriamo le indicazioni FHWA e il loro metodo di calcolo, con la distinzione:
 - Per inclinazioni rispetto alla verticale $i \geq 70^\circ$ → Muri rinforzati (Mechanically Stabilized Earth Wall - MSEW)
 - Per inclinazioni rispetto alla verticale $i < 70^\circ$ → Pendii rinforzati (Reinforced Soil Slope - RSS)
- Utilizziamo i coefficienti parziali indicati dalle NTC2008, che si rifanno agli Eurocodici con i relativi Annessi Nazionali
- Per quello che manca seguiamo:
 - PD ISO/TR 20432:2007 «Guidelines for the determination of the long strength of geosynthetics for soil reinforcement»
 - Annessi Nazionali di altri stati: BS8006; AFNOR, EBGEO

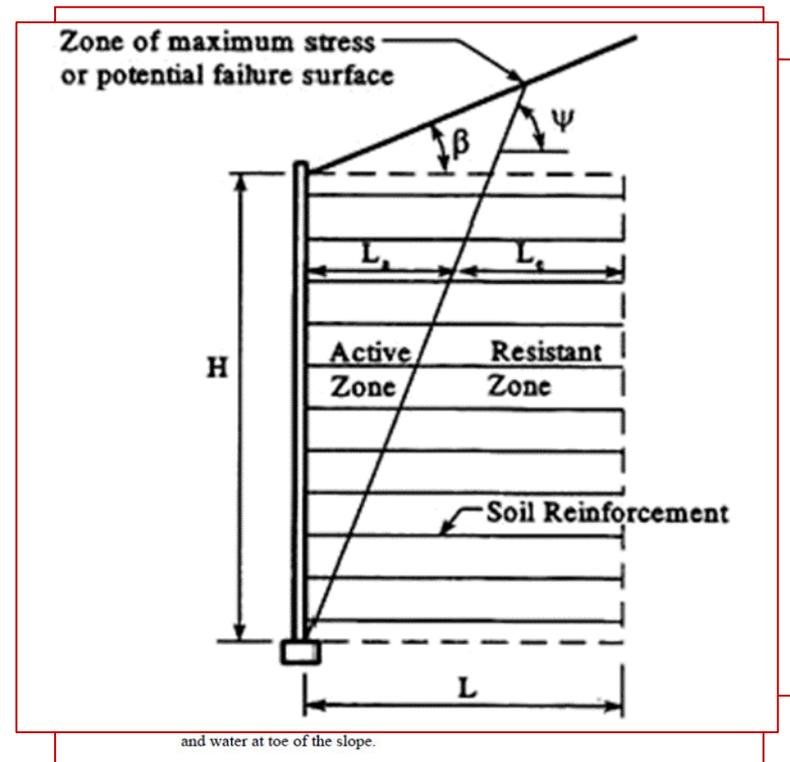
Per i codici di calcolo:

- **Stati Limite Ultimi**
 - MSEW 3.0, Adama Eng. (Indicato da FHWA, implementa la Spinta delle Terre)
 - ReSSA 3.0, Adama Eng. (Indicato da FHWA, Implementa metodi delle strisce)
 - Slide 2D 7.0, RocScience (Implementa metodi delle strisce)
 - RSPile 1.0, RocScience (p-y curves)
 - LimitState:GEO 3.4, Limit State, (Analisi limite, Upper Bound)
 - Flac 2D 8.0, Itasca (Strength Reduction Techniques)
- **Stati Limite di Esercizio**
 - Flac 2D 8.0, Itasca

Tipologie di verifiche (da EBGEO)

Table 3.1 Allocation of analyses to the limit states defined in DIN 1054

Analysis group	Analysis	Limit state	Notes/standards Remarks
Ultimate limit state	Bearing capacity failure	STR	Regarded as quasi-monolithic, cf. DIN 1054:2005-01, Para. 12.4.4 (2), 7.5.2
	Sliding	STR	Regarded as quasi-monolithic, cf. DIN 1054:2005-01, Para. 12.4.4 (2), 7.5.3
	General failure/slope failure	GEO	Regarded as quasi-monolithic, cf. DIN 1054:2005-01, Para. 12.3
	'Overturning' alternatively via position of bearing pressure resultant	EQU ¹⁾	Regarded as quasi-monolithic, cf. DIN 1054:2005-01, Para. 7.5.1 (1)
	General failure/slope failure	GEO	Reinforcement layer intersected, cf. DIN 1054:2005-01, Para. 12.3, 12.4.3, cf. EBGEO, Section 3.4
	Design strength of reinforcement	STR	cf. EBGEO, Section 3.4
	Pull-out resistance of reinforcement	GEO/ STR ²⁾	cf. DIN 1054:2005-01, Para. 12.4.3 cf. EBGEO, Section 3.4
	Analysis of connection of the outer skin	STR	cf. EBGEO, Section 3.4
	Analysis of reinforcement overlapping/joining (reinforcement junctions)	STR	cf. DIN 1054:2005-01, Para. 7.5.3 cf. EBGEO, Section 3.4
Serviceability limit state	Deformation of the structure	SLS	cf. EBGEO, Section 3.1
	Settlement in the contact area	SLS	cf. EBGEO, Section 3.1
	Analysis of bearing pressure resultant	SLS	Regarded as quasi-monolithic, cf. DIN 1054:2005-01, Para. 12.4.4 (2), 7.6.1



Per Muri Rinforzati

Approccio 2

A1+M1+R3

- Capacità portante della fondazione
- Scorrimento
- *Stabilità interna*
- *Sfilamento*

EQU+M2 ($E_d \leq R_d$)

- Ribaltamento

A2+M2+R2

- Stabilità globale

Dall'approccio Francese:

- Sfilamento
 - $M1 + \gamma_{M,f} = 1,35$
 - $M2 + \gamma_{M,f} = 1,10$
- } EBGEO (1,40)
- Fattore di sicurezza sul geosintetico, per ogni approccio $\gamma_M = 1,25$ (oppure 1,40 da EBGEO)

- Stabilità composta: come globale

NTC 2008

Tabella 6.2.I – Coefficienti parziali per le azioni o per l'effetto delle azioni.

CARICHI	EFFETTO	Coefficiente Parziale γ_F (o γ_E)	EQU	(A1) STR	(A2) GEO
Permanenti	Favorevole	γ_{G1}	0,9	1,0	1,0
	Sfavorevole		1,1	1,3	1,0
Permanenti non strutturali ⁽¹⁾	Favorevole	γ_{G2}	0,0	0,0	0,0
	Sfavorevole		1,5	1,5	1,3
Variabili	Favorevole	γ_{Qi}	0,0	0,0	0,0
	Sfavorevole		1,5	1,5	1,3

(1) Nel caso in cui i carichi permanenti non strutturali (ad es. i carichi permanenti portati) siano compiutamente definiti, si potranno adottare gli stessi coefficienti validi per le azioni permanenti.

Tabella 6.2.II – Coefficienti parziali per i parametri geotecnici del terreno

PARAMETRO	GRANDEZZA ALLA QUALE APPLICARE IL COEFFICIENTE PARZIALE	COEFFICIENTE PARZIALE γ_M	(M1)	(M2)
Tangente dell'angolo di resistenza al taglio	$\tan \phi'_k$	γ_ϕ	1,0	1,25
Coesione efficace	c'_k	γ_c	1,0	1,25
Resistenza non drenata	c_{uk}	γ_{cu}	1,0	1,4
Peso dell'unità di volume	γ	γ_γ	1,0	1,0

Tabella 6.5.I - Coefficienti parziali γ_R per le verifiche agli stati limite ultimi STR e GEO di muri di sostegno.

VERIFICA	COEFFICIENTE PARZIALE (R1)	COEFFICIENTE PARZIALE (R2)	COEFFICIENTE PARZIALE (R3)
Capacità portante della fondazione	$\gamma_R = 1,0$	$\gamma_R = 1,0$	$\gamma_R = 1,4$
Scorrimento	$\gamma_R = 1,0$	$\gamma_R = 1,0$	$\gamma_R = 1,1$
Resistenza del terreno a valle	$\gamma_R = 1,0$	$\gamma_R = 1,0$	$\gamma_R = 1,4$

Le verifiche devono essere effettuate secondo l'Approccio 1:

– Combinazione 2: (A2+M2+R2)

tenendo conto dei valori dei coefficienti parziali riportati nelle Tabelle 6.2.I, 6.2.II e 6.8.I.

Tabella 6.8.I – Coefficienti parziali per le verifiche di sicurezza di opere di materiali sciolti e di fronti di scavo.

Coefficiente	R2
γ_R	1.1

Per Muri Rinforzati

Approccio 2

A1+M1+R3

- Capacità portante della fondazione
- Scorrimento
- **Ribaltamento**
- *Stabilità interna*
- *Sfilamento*

A2+M2+R2

- Stabilità globale

Dall'approccio Francese:

- Sfilamento
 - $M1 + \gamma_{M,f} = 1,35$
 - $M2 + \gamma_{M,f} = 1,10$
- } EBGEO (1,40)
- Fattore di sicurezza sul geosintetico, per ogni approccio $\gamma_M = 1,25$ (oppure 1,40 da EBGEO)
 - Stabilità composta: come globale

NTC 2018

Tab. 6.2.I – Coefficienti parziali per le azioni o per l'effetto delle azioni

	Effetto	Coefficiente Parziale γ_R (o γ_{R1})	EQU	(A1)	(A2)
Carichi permanenti G_1	Favorevole	γ_{G1}	0,9	1,0	1,0
	Sfavorevole		1,1	1,3	1,0
Carichi permanenti $G_2^{(1)}$	Favorevole	γ_{G2}	0,8	0,8	0,8
	Sfavorevole		1,5	1,5	1,3
Azioni variabili Q	Favorevole	γ_Q	0,0	0,0	0,0
	Sfavorevole		1,5	1,5	1,3

⁽¹⁾ Per i carichi permanenti G_2 si applica quanto indicato alla Tabella 2.6.I. Per la spinta delle terre si fa riferimento ai coefficienti γ_{G1}

Tab. 6.2.II – Coefficienti parziali per i parametri geotecnici del terreno

Parametro	Grandezza alla quale applicare il coefficiente parziale	Coefficiente parziale γ_M	(M1)	(M2)
Tangente dell'angolo di resistenza al taglio	$\tan \varphi'_k$	γ_φ	1,0	1,25
Coesione efficace	c'_k	γ_c	1,0	1,25
Resistenza non drenata	c_{uk}	γ_{cu}	1,0	1,4
Peso dell'unità di volume	γ_γ	γ_γ	1,0	1,0

Tab. 6.5.I – Coefficienti parziali γ_R per le verifiche agli stati limite ultimi di muri di sostegno

Verifica	Coefficiente parziale (R3)
Capacità portante della fondazione	$\gamma_R = 1,4$
Scorrimento	$\gamma_R = 1,1$
Ribaltamento	$\gamma_R = 1,15$
Resistenza del terreno a valle	$\gamma_R = 1,4$

Terreno per muri rinforzati (da FHWA NHI-10-024)

Table 3-1. MSE Wall Select Granular Reinforced Fill Requirements.

<u>Gradation:</u> (AASHTO T-27)	<u>U.S. Sieve Size</u>	<u>Percent Passing</u> ^(a)
	4 in. (102 mm) ^(a,b)	100
	No. 40 (0.425 mm)	0-60
	No. 200 (0.075 mm)	0-15
<u>Plasticity Index, PI</u> (AASHTO T-90)	PI ≤ 6	
<u>Soundness:</u> (AASHTO T-104)	The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles (or a sodium sulfate value less than 15 percent after five cycles).	
Notes:		
(a) To apply default F* values, C _u should be greater than or equal to 4.		
(b) As a result of recent research on construction survivability of geosynthetics and epoxy coated reinforcements, it is recommended that the maximum particle size for these materials be reduced to ¾-in. (19 mm) for geosynthetics, and epoxy and PVC coated steel reinforcements unless construction damage assessment tests are or have been performed on the reinforcement combination with the specific or similarly graded large size granular fill. Prequalification tests on reinforcements using standard agency fill materials should be considered.		

Esempio

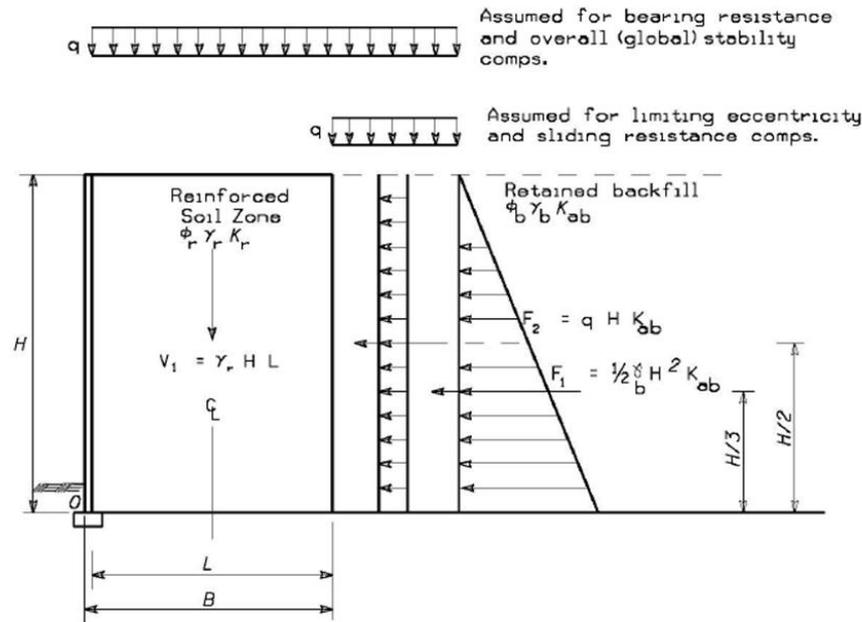


Figure 4-2. External analysis: nominal earth pressures; horizontal backslope with traffic surcharge (after AASHTO, 2007).

Traslazione A1+M1+R3

Spinta della terra $F_1 = \frac{1}{2} \cdot \gamma_{G1} \cdot \gamma_b \cdot H^2 \cdot k_{ab}$

Sovraccarico $F_2 = \gamma_Q \cdot q \cdot H \cdot k_{ab}$

Resistenza $R_r = \gamma_{G1} \cdot V_1 \cdot \mu$

$\mu = \text{minimo tra } \tan\phi_f,$

$\tan\phi_r$ e per rinforzi continui $\tan\delta$

$\gamma_R = 1,10$

Ribaltamento EQU+M2

Spinta della terra $F_1 = \frac{1}{2} \cdot \gamma_{G1} \cdot \gamma_b \cdot H^2 \cdot k_{ab}$

Sovraccarico $F_2 = \gamma_Q \cdot q \cdot H \cdot k_{ab}$

Peso proprio terreno $P = \gamma_{G1} \cdot V_1$ (Favorevole)

$E_d \leq R_d$

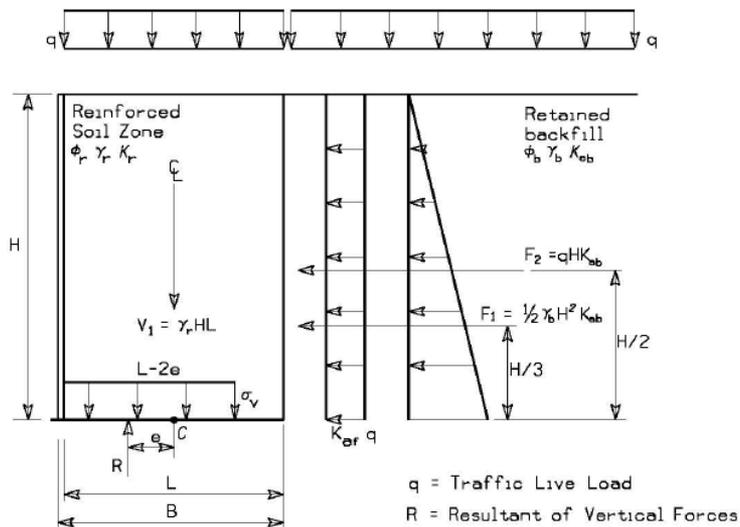


Figure 4-7. Calculation of eccentricity and vertical stress for bearing check, for horizontal backslope with traffic surcharge condition.

Capacità portante A1+M1+R3

Spinta della terra $F_1 = \frac{1}{2} \cdot \gamma_{G1} \cdot \gamma_b \cdot H^2 \cdot k_{ab}$

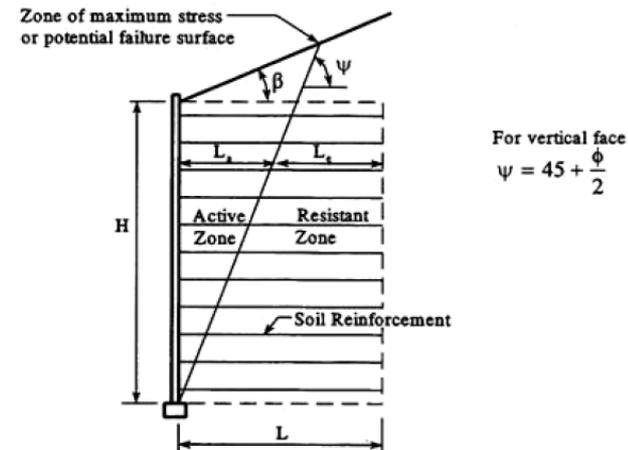
Sovraccarico $F_2 = \gamma_Q \cdot q \cdot H \cdot k_{ab}$

Peso proprio terreno $P = \gamma_{G1} V_1$

Sovraccarico $N = \gamma_{Q1} \cdot q \cdot L$

Carico limite $q_n = \frac{1}{2} \gamma B N_\gamma s_\gamma i_\gamma b_\gamma g_\gamma$
 $+ c' N_c s_c d_c i_c b_c g_c$

$\gamma_R = 1,40$



Stabilità interna A1+M1+R3

Pressione orizzontale $\sigma_H = k_a (\gamma_{G1} Z \gamma + \gamma_{Q1} q)$

Trazione agente nel rinforzo $T_{max} = \sigma_H S_v$

Resistenza del rinforzo $R_D = \frac{T_{char} / \Pi R F}{\gamma_R}$

$\gamma_R = 1,25$ (da AFNOR) (in alternativa EB GEO)

$$T_{max} \leq R_D$$

Pullout

$$L_e \geq \frac{\gamma_{Mf} T_{max}}{C_{p0} \tan \varphi \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c}$$

$\sigma_v = \gamma_{G1} \gamma_r Z + \gamma_{Q1} q$ dove $\gamma_{Q1} = 0,0$

Vertical Wall and a Surcharge Slope: The active coefficient of earth pressure is calculated for near vertical walls (defined as walls with a face batter of less than 10 degrees from vertical) and a sloping backfill from:

$$K_{ab} = \frac{\sin^2(\theta + \phi'_b)}{\Gamma \sin^2\theta \sin(\theta - \delta)} \quad (4-2)$$

where:

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_b + \delta) \sin(\phi'_b - \beta)}{\sin(\theta - \delta) \sin(\theta + \beta)}} \right]^2 \quad (4-3)$$

β = Nominal slope of backfill behind wall (deg)

δ = Angle of friction between retained backfill and reinforced soil, set equal to β (deg)

ϕ'_b = effective friction angle of retained backfill (deg)

θ = 90° for vertical, or near ($< 10^\circ$) vertical, wall (deg)

Note that the earth pressure force, (F_T) in Figure 4-3, is oriented at the same angle as the backslope, β , as it is assumed that $\delta = \beta$.

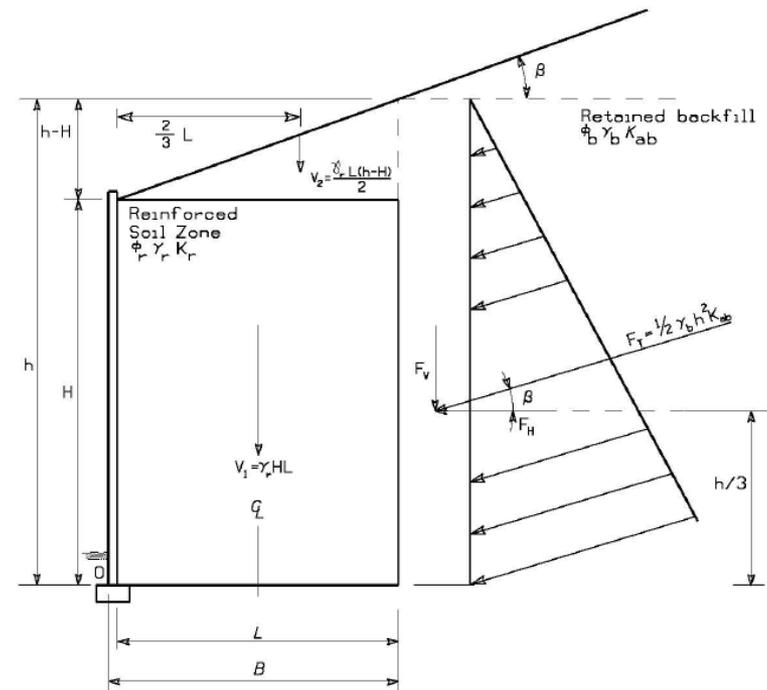


Figure 4-3. External analysis: earth pressure; sloping backfill case (after AASHTO, 2007).

Teoria di Coulomb-
Equilibrio limite

Vertical Wall with Broken Backslope: The active earth pressure coefficient (K_a) for this condition is computed using Equations 4-2 and 4-3, with the design β angle and the interface angle δ both set equal to I , as defined in Figure 4-4.

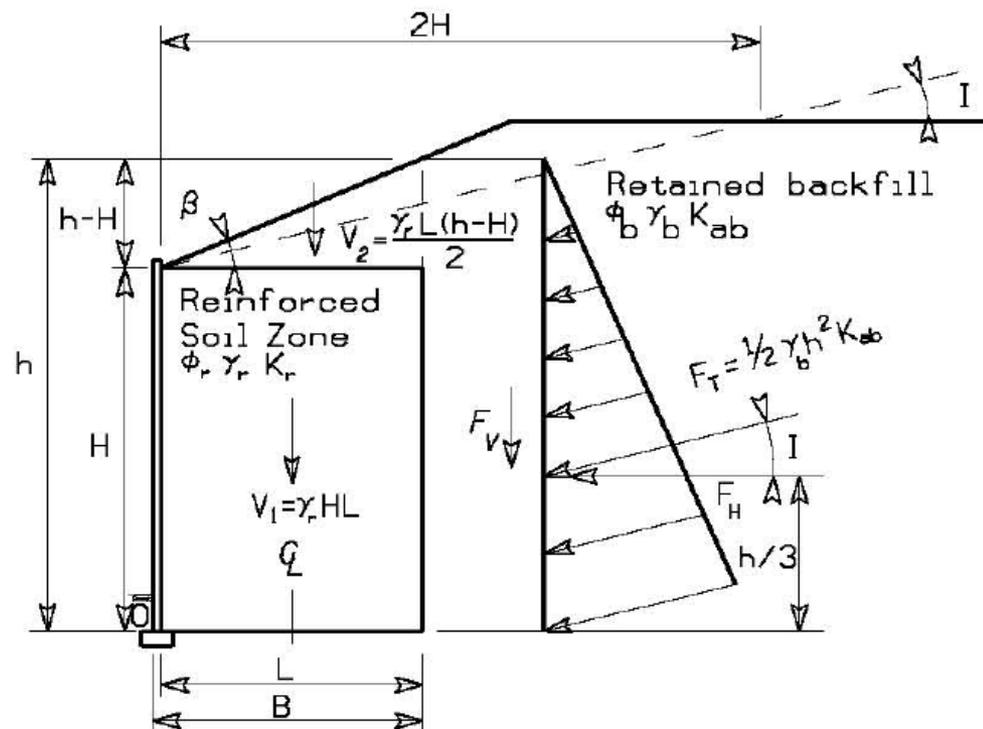
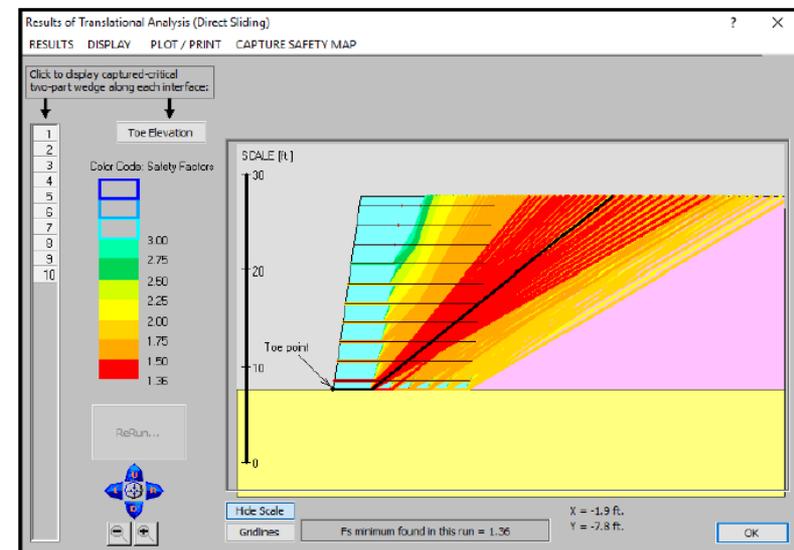
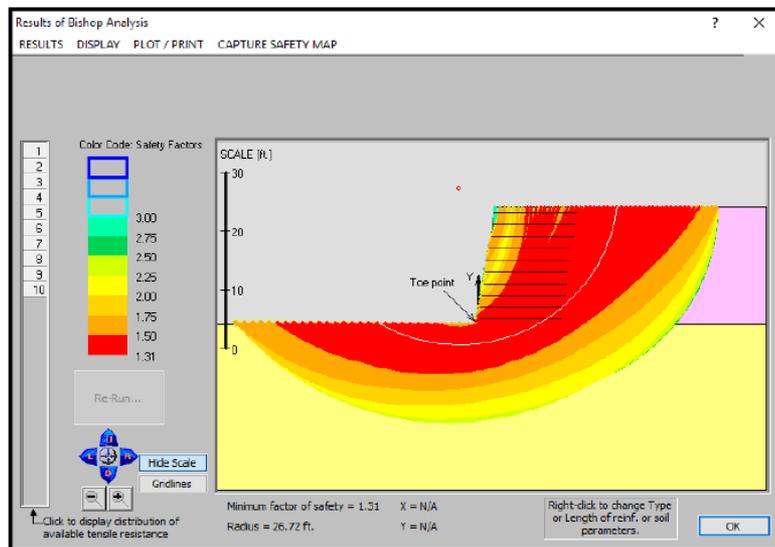


Figure 4-4. External analysis: earth pressure; broken backslope case (after AASHTO, 2007).

Stabilità globale A2+M2+R2

Alla resistenza del rinforzo applicare γ_M

Occorre tuttavia capire come funziona il software utilizzato



In condizioni sismiche

Azione e resistenza del terreno

NTC 2008

7.11.1

Le verifiche agli stati limite ultimi devono essere effettuate ponendo pari **all'unità i coefficienti parziali sulle azioni** e impiegando i parametri geotecnici e le resistenze di progetto, con i loro valori dei coefficienti parziali indicati nel Cap. 6.

NTC 2018

7.11.1.

Le verifiche degli stati limite ultimi in presenza di azioni sismiche devono essere eseguite ponendo pari a **1 i coefficienti parziali sulle azioni e sui parametri geotecnici** e impiegando le resistenze di progetto, con i coefficienti parziali γ_R indicati nel presente Capitolo 7 , oppure con i γ_R indicati nel Capitolo 6 laddove non espressamente specificato

Resistenza del geosintetico di rinforzo:

PD ISO TR 20432:2007

If the design is based on a **sustained load together with the ability to withstand a temporary seismic or other overload**, then the design lifetime should be a fraction (e.g. < 10 %) of the time over which the sustained load would lead to rupture. The residual strength at any point during this lifetime should then be taken to be:

$$T_{DR} = T_{char} / (RF_{ID} \cdot RF_W \cdot RF_{CH} \cdot f_s)$$

Ricordarsi γ_M !

Resistenza del geosintetico in condizioni sismiche

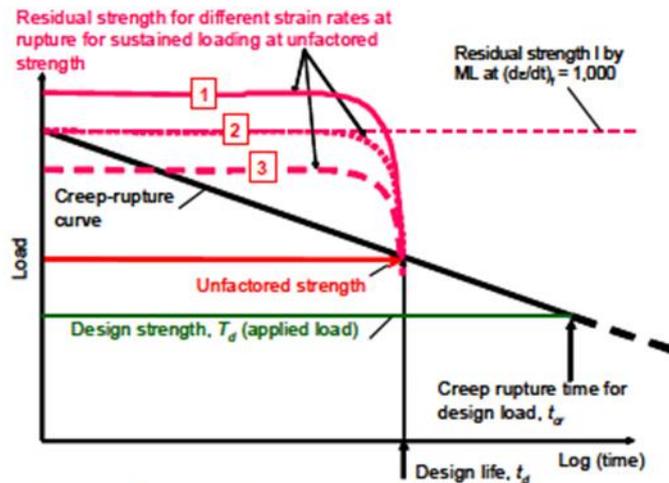


Fig.1.8 Effects of strain rate at rupture on residual strength (Tatsuoka et al., 2004)

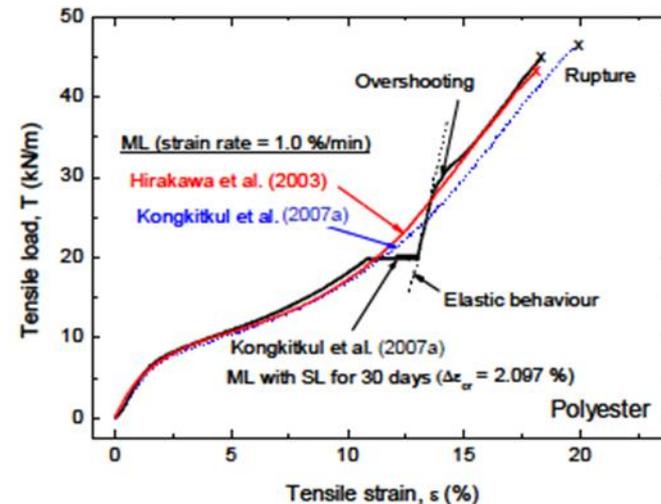
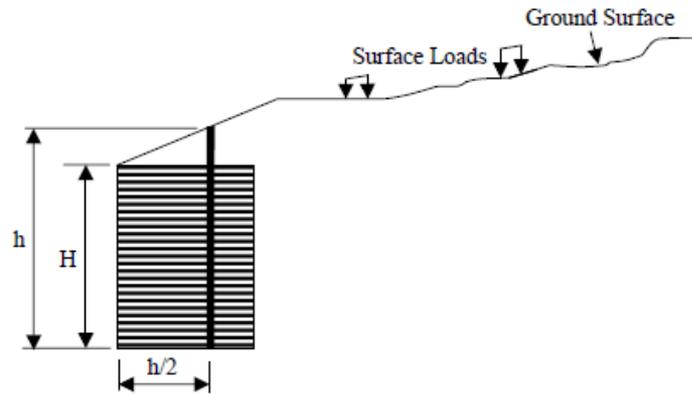


Fig. 1.9 Comparison of tensile load - strain relations from three ML tests with and without creep loading for 30 days at an intermediate load level, a PET geogrid (Kongkitkul et al., 2007a).

Tatsuoka F., Koseki J., Tateyama M. (2009). Seismic behaviour of geosynthetic-reinforced structures: lesson from recent earthquakes and design approaches, *Proc. of XXII Convegno Nazionale Geosintetici*, Bologna, 7-22

FHWA-Condizioni sismiche



$$h = H + \frac{\tan I(0.5H)}{(1 - 0.5 \tan I)}; \text{ where } I \text{ is the backfill slope angle}$$

$h/2$ is measured from back of wall facing

Figure 7-1. Definition of heights for seismic analyses.

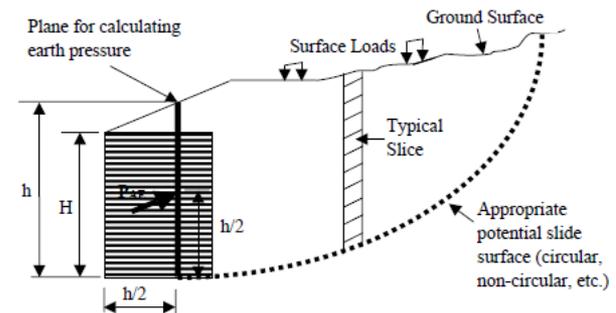


Figure 7-2. Use of a slope stability approach to compute seismic earth pressure.

Method 1: Mononobe-Okabe (M-O) formulation

$$P_{AE} = 0.5(K_{AE})\gamma_b h^2 \quad (7-5)$$

where h is the wall height along the vertical plane within the reinforced soil mass as shown in Figure 7-1, γ_b is the unit weight of the retained fill and K_{AE} is obtained as follows:

$$K_{AE} = \frac{\cos^2(\phi'_b - \xi - 90 + \theta)}{\cos \xi \cos^2(90 - \theta) \cos(\delta + 90 - \theta + \xi) \left[1 + \sqrt{\frac{\sin(\phi'_b + \delta) \sin(\phi'_b - \xi - I)}{\cos(\delta + 90 - \theta + \xi) \cos(I - 90 + \theta)}} \right]^2} \quad (7-6)$$

Method 2: Generalized Limit Equilibrium (GLE) slope stability

Stabilità esterna

FHWA-Condizioni sismiche

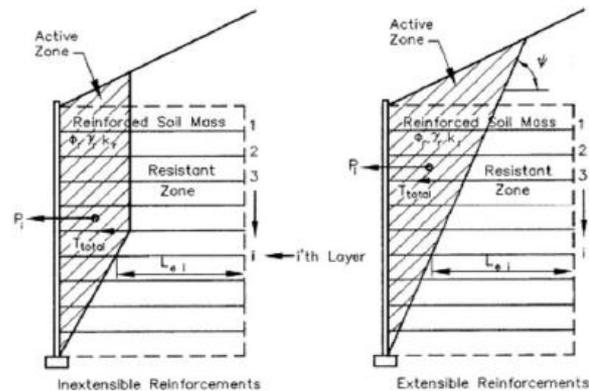


Figure 7-5. Seismic internal stability of a MSE wall.

$$P_i = k_{av} W_a \quad (7-10)$$

$$T_{md} = \frac{P_i}{n} \quad (7-11)$$

$$T_{total} = T_{max} + T_{md} \quad (7-12)$$

RF = combined strength reduction factor to account for potential long-term degradation due to installation damage, creep, and chemical aging, equal to $RF_{CR} \times RF_{ID} \times RF_D$ (see Chapter 3)

RF_{ID} = strength reduction factor to account for installation damage to reinforcement

RF_D = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation

For the static component:

$$S_{IS} \geq \frac{T_{max} RF}{\phi R_c} \quad (7-13)$$

For the dynamic component:

$$S_{rt} \geq \frac{T_{md} RF_{ID} RF_D}{\phi R_c} \quad (7-14)$$

where:

ϕ = resistance factor for combined static/earthquake loading = 1.20 from Table 11.5.6-1 of AASHTO (2007)

S_{IS} = ultimate reinforcement tensile resistance required to resist static load component

S_{rt} = ultimate reinforcement tensile resistance required to resist dynamic load component

R_c = reinforcement coverage ratio

Case History – Muro in terra rinforzata

SP 156 - Lusernetta

SEZ 130

Scala altezze 1: 200
Scala lunghezze 1: 200

Q = 462.00 m s.l.m.

SEZIONI	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	
DISTANZE PARZIALI		4.08	1.72	3.29		5.45	0.88	3.12	1.78	4.09	2.49	0.90	0.94		5.33		10.65		18
DISTANZE PROGRESSIVE	0.00	4.08	5.81	9.10		14.55	15.43	18.55	20.33	24.42	26.91	27.81	28.75	29.69	35.02		45.67		64.27
QUOTE TERRENO	476.44	476.06	474.30	473.08		470.15	468.48	468.48	468.52	468.66	465.38	466.05	464.94	464.13	464.13		464.10		463.95
QUOTE PROGETTO								478.04		478.15		478.26							



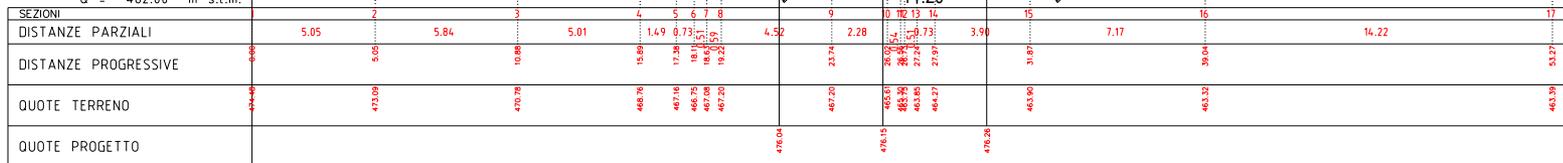


Muro in terra rinforzata

SEZ 129

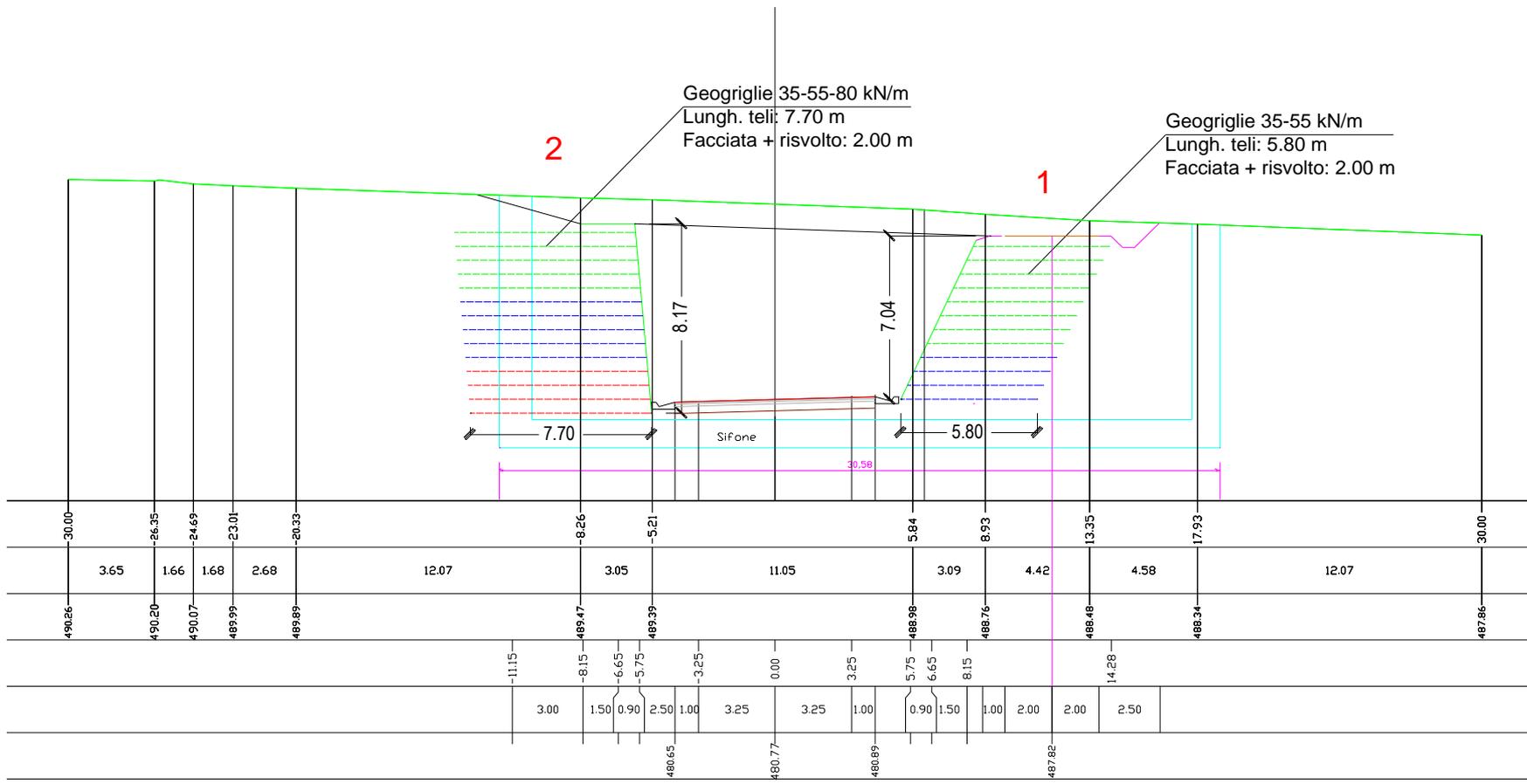
Scala altezze 1: 200
Scala lunghezze 1: 200

0 = 462.00 m s.l.m.



Geogriglie 35-55-80-110 kN/m
Facciata + risvolto: 2.00 m

Geogriglie 110 kN/m
Lungh. teli: 11.20 m
Facciata + risvolto: 2.00 m



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AASHTO LRFD Bridge Design Specifications

**SI Units
4th Edition
2007**



American Association of State Highway
and Transportation Officials

CON MS

?

ANALYSIS

-- ANALYSIS MODE --

Performance: Acceptable
reference website

Reinforcement :

Geogrid
Geotextile
Metal Mat
Metal Strip

Facia :

Wrap Arou
Full Height Pr
Segmental P
Modular Blo

DATA

Version :

Western
Asian Pacific

Results



IL MINISTRO DELLE INFRASTRUTTURE
di concerto con
IL MINISTRO DELL'INTERNO
e con
IL CAPO DEL DIPARTIMENTO DELLA PROTEZIONE CIVILE

Vista la legge 5 novembre 1971, n. 1086, recante norme per la disciplina delle opere in conglomerato cementizio armato, normale e precompresso e da struttura metallica;

Vista la legge 2 febbraio 1974, n. 64, recante provvedimenti per le costruzioni con particolari prescrizioni per le zone sismiche;

Vista la legge 21 giugno 1986, n. 317 recante "Procedura di informazione nel settore delle norme e regolamentazioni tecniche delle regole relative ai servizi della società dell'informazione in attuazione della direttiva 98/34/CE del Parlamento europeo e del Consiglio del 22 giugno 1998, modificata dalla direttiva 98/48/CE del Parlamento europeo e del Consiglio del 20 luglio 1998";

Visto il decreto del Presidente della Repubblica 21 aprile 1993, n. 246, recante «Regolamento di attuazione della direttiva 89/106/CEE relativa ai prodotti da costruzione»;

Visto il decreto legislativo 31 marzo 1998, n. 112, recante conferimento di funzioni e compiti amministrativi allo Stato, alle regioni e agli enti locali in attuazione del capo I della legge 15 marzo 1997, n. 59;

Visto il decreto del Presidente della Repubblica 6 giugno 2001, n. 380, testo unico delle disposizioni legislative e regolamentari in materia edilizia;

Vista la legge 17 luglio 2004, n. 186, di conversione del decreto-legge 28 maggio 2004, n. 136 ed in particolare l'art. 5, comma 1, che prevede la redazione, da parte del Consiglio superiore dei lavori pubblici, di concerto con il Dipartimento della protezione civile, di normative tecniche, anche per la verifica sismica ed idraulica, relative alle costruzioni, nonché per la progettazione, la costruzione e l'adeguamento, anche sismico ed idraulico, delle dighe di ritenuta, dei ponti e delle opere di fondazione e sostegno dei terreni, per assicurare uniformi livelli di sicurezza;

Visto il decreto del Ministro delle infrastrutture e dei trasporti 14 settembre 2005, con il quale sono state approvate le "Norme tecniche per le costruzioni";

Visto l'art. 14-undecies del decreto-legge 30 giugno 2005, n. 115, convertito, con modificazioni, in legge 17 agosto 2005, n. 168, che inserisce il comma 2-bis all'art. 5 del citato decreto-legge 28 maggio 2004, n. 136, convertito, con modificazioni, dalla legge 27 luglio 2004, n. 186, il quale prevede che "al fine di avviare una fase sperimentale di applicazione delle norme tecniche di cui al comma 1, è consentita, per un periodo di diciotto

Esempio: Stabilità interna di un muro in terra rinforzata

- Muro in terra rinforzata
- NTC 2018; A1+M1+R3
- Terreno ingegnerizzato (tra gli strati di GG): $\gamma=20 \text{ kN/m}^3$, $\varphi'=32^\circ$, $c'=0.0 \text{ kPa}$
- Condizioni statiche

$$K_a = \tan^2 \left(45 - \frac{\varphi'}{2} \right) = 0,307$$

$$\alpha = 45 + \frac{\varphi'}{2} = 61^\circ$$

Trazione nella geogriglia n. 2:

$$\sigma_v = \gamma_{G1} \cdot \gamma \cdot h_2 = 1,30 \cdot 20 \frac{\text{kN}}{\text{m}^3} \cdot 1,20\text{m} = 31,20 \text{ kPa}$$

$$\sigma_h = K_a \cdot \sigma_v = 0,307 \cdot 31,20 = 9,58 \text{ kPa}$$

$$T_{max} = \sigma_h \cdot \Delta h_2 = 9,58 \text{ kPa} \cdot 0,60 \text{ m} = 5,75 \text{ kN/m}$$

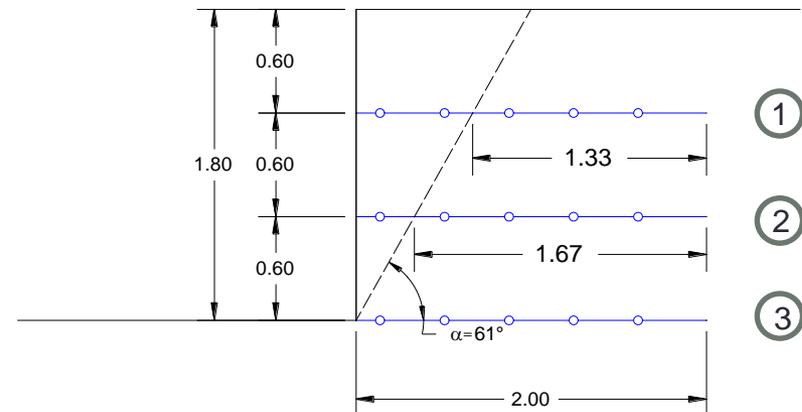
Pullout per la geogriglia n.2:

$$L_e = \frac{T_{max} \cdot F_s}{2 \cdot \sigma_v \cdot F^* \cdot \alpha \cdot R_c} \quad \text{AASHTO/DEMO 82 (ASD: Allowable Stress Design, come TA). Nota: } F_s > 1,0.$$

$$\phi L_e = \frac{T_{max}}{2 \cdot \sigma_v \cdot F^* \cdot \alpha \cdot R_c} \quad \text{(LRFD: Load and Resistance Factor Design, come SL). Nota: } \phi < 1,0.$$

$$L_e \geq \frac{T_{max}}{2 \cdot \sigma_v \cdot F^* \cdot \alpha \cdot R_c \cdot \phi} = \frac{5,75 \frac{\text{kN}}{\text{m}}}{2 \cdot 24 \text{ kPa} \cdot 0,667 \cdot \tan 32^\circ \cdot 0,8 \cdot 1,0 \cdot 0,7} = 0,513 \text{ m} \quad \phi = \frac{1}{1,4} \cong 0,7$$

$$\sigma_v = \gamma \cdot h_2 = 20 \frac{\text{kN}}{\text{m}^3} \cdot 1,20\text{m} = 24,0 \text{ kPa (non fattorizzata)}$$



Esempio: Stabilità interna di un muro in terra rinforzata

- Muro in terra rinforzata
- NTC 2018; A1+M1+R3
- Terreno ingegnerizzato (tra gli strati di GG): $\gamma=20 \text{ kN/m}^3$, $\varphi'=32^\circ$, $c'=0.0 \text{ kPa}$
- Sisma, $k_h=0.043$, procedura AASHTO 2007

$$P_i = K_{av} W_a \quad K_{av}=K_h$$

W_a =peso del terreno nella zona attiva (non fattorizzato, superficie statica)

$$T_{md} = \frac{P_i}{n} \quad \text{Metodo 1}$$

$$T_{md} = P_i \cdot \frac{L_{ei}}{\sum_{i=1}^n (L_{ei})} \quad \text{Metodo 2}$$

$$A = 0,8980 \text{ m}^2$$

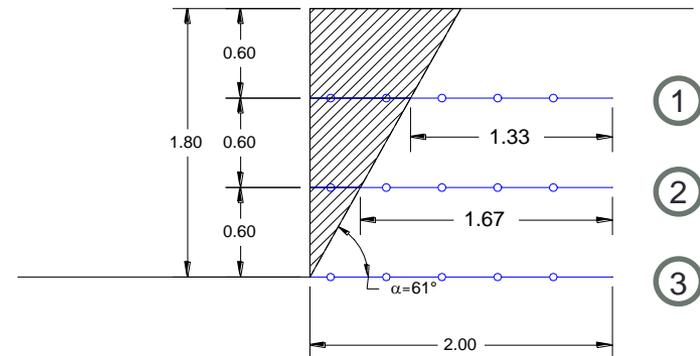
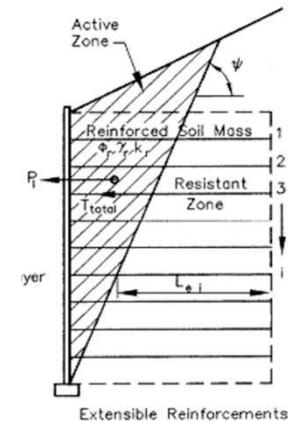
$$P_i = K_v \cdot W_a = 0,043 \cdot 20 \frac{\text{kN}}{\text{m}^3} \cdot 0,8980 \text{ m}^2 \cdot 1,0 \text{ m} = 0,7722 \text{ kN}$$

$$\sum_{i=1}^n L_{ei} = 2,00 + 1,67 + 1,33 = 5,00 \text{ m}$$

$$T_{md,1} = 0,7722 \text{ kN} \cdot \frac{1,33}{5,00} = 0,21 \text{ kN}$$

$$T_{md,2} = 0,7722 \text{ kN} \cdot \frac{1,67}{5,00} = 0,26 \text{ kN}$$

$$T_{md,3} = 0,7722 \text{ kN} \cdot \frac{2,00}{5,00} = 0,31 \text{ kN}$$



Pendii rinforzati



$i < 70^\circ$

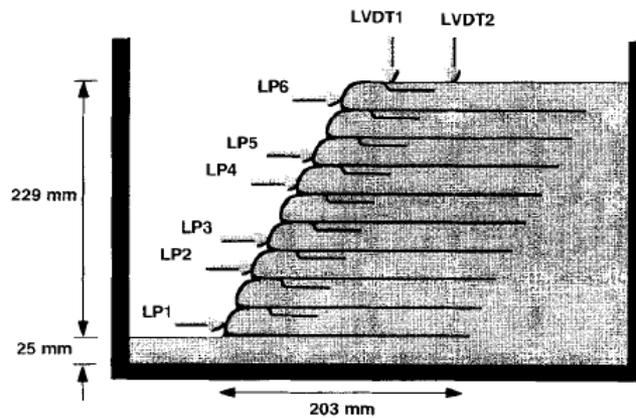
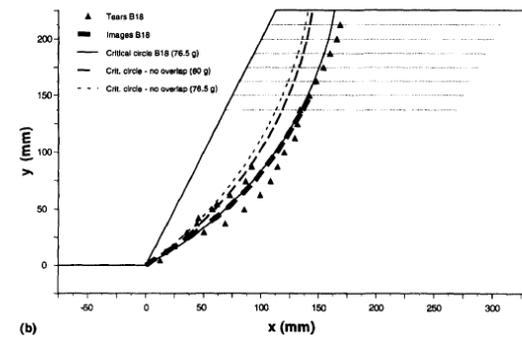


FIG. 1. Centrifuge Model with 25.4 mm Reinforcement Spacing [Linear Potentiometers (LPs) and Linear Variable Displacement Transducers (LVDTs) Are Indicated in Figure]



(b)

Il metodo all'equilibrio limite per la stabilità dei pendii

- Esistono molte soluzioni possibili, con diverse ipotesi (Bishop semplificato, Janbu semplificato, Spencer, Morgenstern-Price, Corps of Engineers, Lowe-Karafiath...)

Comprehensive Bishop (Leshchinsky D., 2001)

- si inserisce il rinforzo nel metodo di Bishop semplificato
- FS applicato alle proprietà meccaniche del terreno (c' , φ') (Alan W. Bishop, 1954)
- Contributo dei rinforzi inteso come T_{\max} a lungo termine (è una forza stabilizzante, anche se interna)
- Equazioni sviluppate **coerentemente**, in modo logico, eseguendo tutti i passaggi matematici, con ipotesi ben definite.
- Metodo implementato nel software ReSSA 3.0

$$S_i = (c_i \Delta x_i / \cos \alpha_i + N_i \tan \varphi_i) / FS$$

$$FS = \frac{\sum \{ [c_i \Delta x_i + (W_i - T_{\max} \sin \theta) \tan \varphi_i] / m_{\alpha i} \}}{\{ \sum [W_i \sin \alpha_i - T_{\max} \cos(\alpha_i - \theta)] \}}$$

$$m_{\alpha i} = \cos \alpha_i + (\sin \alpha_i \tan \varphi_i) / FS$$

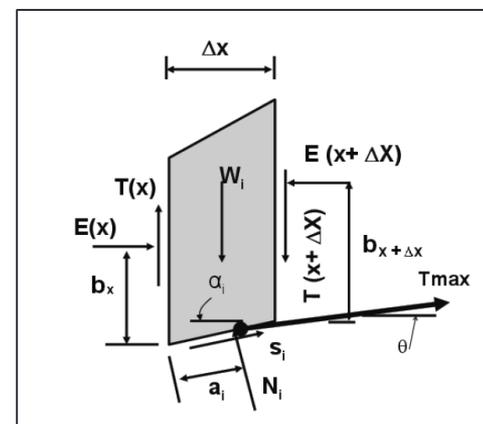


Figura 1. Il Metodo Comprehensive Bishop (Leshchinsky, D.)

Dove viene applicato il Fattore di Sicurezza?

Ducan J.M., Wright S.G., Brandon T. L., Soil Strength and Slope Stability, *Second Edition*, Wiley.

- **Method A.** The reinforcement forces used in the analysis are *allowable* forces and *are not divided* by the factor of safety calculated during the slope stability analysis. Only the soil strength is divided by the factor of safety calculated in the slope stability analysis.

$$F = \frac{\text{soil resisting moment}}{\text{overturning moment} - \text{reinforcement moment}}$$

- **Method B.** The reinforcement forces used in the analysis are *ultimate* forces and *are divided* by the factor of safety calculated in the slope stability analysis. Both the reinforcing force and the soil strength are divided by the factor of safety calculated in the slope stability analysis.

$$F = \frac{\text{soil resisting moment} + \text{reinforcement moment}}{\text{overturning moment}}$$

Come capire cosa implementa il software?

If it is not clear which method is used by a computer program, this can be determined by analyzing the reinforced slope problem shown in Figure 8.1. This slope is 20 ft high and is inclined at 2.0 vertical on 1.0 horizontal. The soil within the slope is uniform; with $\gamma = 100$ pcf, $\phi = 0$, and $c = 500$ psf. There is a firm layer beneath the toe. A reinforcing force of 10,000 lb/ft acts horizontally at mid height, 10 ft above the toe of the slope. Results for two analyses are shown in Figure 8.1. For consistency with the reported results, the analysis should be performed using **Bishop's Simplified method**, and the failure circles should be tangent to the top of the firm layer. The critical circles, located as shown in Figure 8.1, exit slightly above the toe of the slope.

The method used by any computer program can be determined based on the computed factor of safety:

- If the program computes $F = 2.19$, the program uses method A. Allowable reinforcement forces should be used with the program.
- If the program computes $F = 1.72$, the program uses method B. Ultimate reinforcement forces should be used with the program.

Slight deviations from $F = 2.19$ or $F = 1.72$ can be expected, depending on the number of slices used by the program, and the method used to locate the critical circle. The differences should be no more than 1 or 2 percent, however

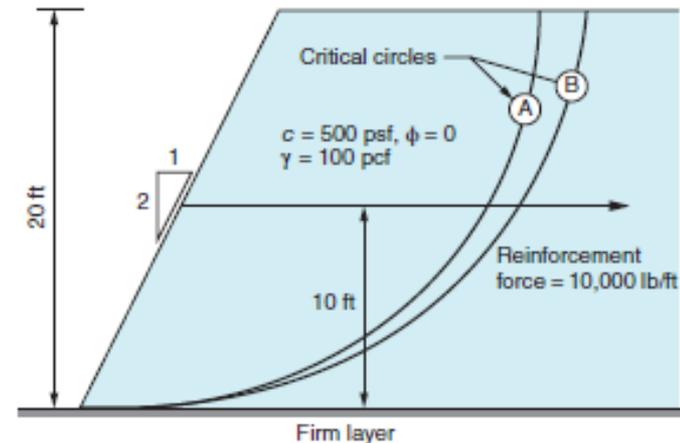
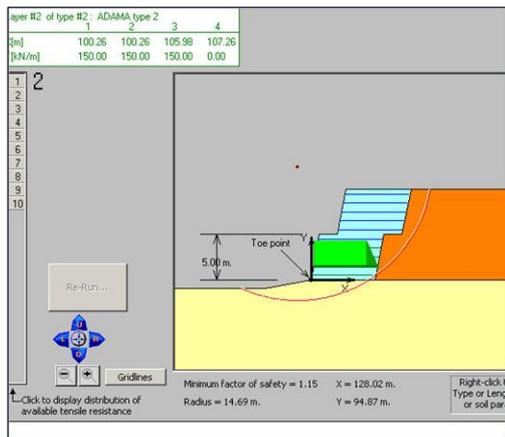
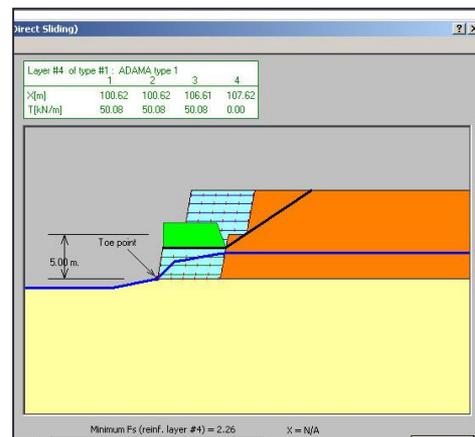


Figure 8.1 Check problem for determining whether a computer program is using method A or method B for analysis of reinforced slopes.

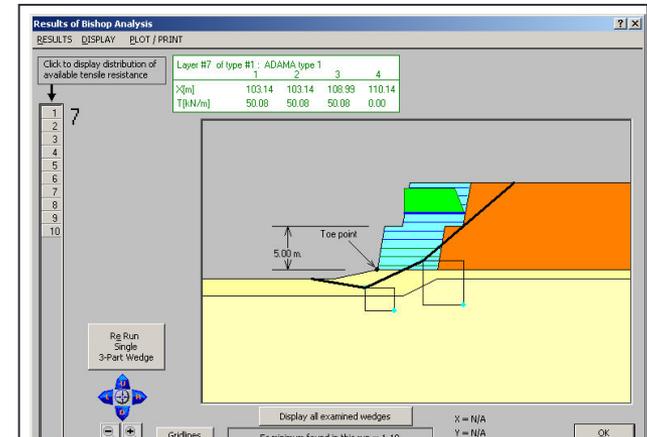
Quali superfici considerare?



Bishop Comprehensive:
superficie circolare



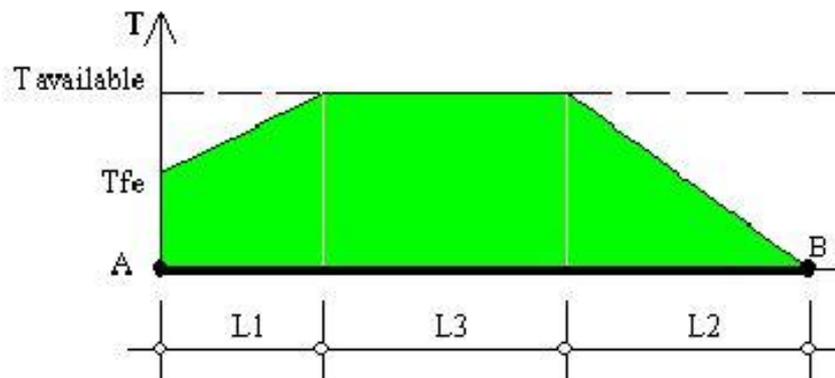
Spencer:
2 part wedge



Spencer:
3 part wedge

Andamento della resistenza lungo il rinforzo in geosintetico

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)

B = Rear-end of reinforcement

$AB = L1 + L2 + L3 =$ Embedded length of reinforcement

$T_{available}$ = Long-term strength of reinforcement

T_{fe} = Available front-end strength (e.g., connection to facing)

$L1$ = Front-end 'pullout' length

$L2$ = Rear-end pullout length

$T_{available}$ prevails along $L3$

$$T_{available} = \frac{T_{ult} \cdot R_c}{R_{F_{id}} \cdot R_{F_d} \cdot R_{F_c} \cdot R_{F_a}} \quad T_{allowable} \leq T_{available}$$

Per pendii rinforzati (NTC 2018 come NTC 2008)

A2+M2+R2 per tutte le superfici (interne, esterne e composte)

Dall'approccio Francese:

- Sfilamento
 - $M2 + \gamma_{R;f} = 1,10$ (1,40 EBGEO)
- Fattore di sicurezza sul geosintetico, per ogni approccio $\gamma_M = 1,25$ (1,40 EBGEO)

Tabella 6.2.I – Coefficienti parziali per le azioni o per l'effetto delle azioni.

CARICHI	EFFETTO	Coefficiente Parziale γ_F (o γ_E)	EQU	(A1) STR	(A2) GEO
Permanenti	Favorevole	γ_{G1}	0,9	1,0	1,0
	Sfavorevole		1,1	1,3	1,0
Permanenti non strutturali ⁽¹⁾	Favorevole	γ_{G2}	0,0	0,0	0,0
	Sfavorevole		1,5	1,5	1,3
Variabili	Favorevole	γ_{G3}	0,0	0,0	0,0
	Sfavorevole		1,5	1,5	1,3

(1) Nel caso in cui i carichi permanenti non strutturali (ad es. i carichi permanenti portati) siano compiutamente definiti, si potranno adottare gli stessi coefficienti validi per le azioni permanenti.

Tabella 6.2.II – Coefficienti parziali per i parametri geotecnici del terreno

PARAMETRO	GRANDEZZA ALLA QUALE APPLICARE IL COEFFICIENTE PARZIALE	COEFFICIENTE PARZIALE γ_M	(M1)	(M2)
Tangente dell'angolo di resistenza al taglio	$\tan \phi'_k$	$\gamma_{\phi'}$	1,0	1,25
Coesione efficace	c'_k	γ_c	1,0	1,25
Resistenza non drenata	c_{uk}	γ_{cu}	1,0	1,4
Peso dell'unità di volume	γ	γ_r	1,0	1,0

Tabella 6.5.I - Coefficienti parziali γ_k per le verifiche agli stati limite ultimi STR e GEO di muri di sostegno.

VERIFICA	COEFFICIENTE PARZIALE (R1)	COEFFICIENTE PARZIALE (R2)	COEFFICIENTE PARZIALE (R3)
Capacità portante della fondazione	$\gamma_R = 1,0$	$\gamma_R = 1,0$	$\gamma_R = 1,4$
Scorrimento	$\gamma_R = 1,0$	$\gamma_R = 1,0$	$\gamma_R = 1,1$
Resistenza del terreno a valle	$\gamma_R = 1,0$	$\gamma_R = 1,0$	$\gamma_R = 1,4$

Le verifiche devono essere effettuate secondo l'Approccio 1:

- Combinazione 2: (A2+M2+R2)

tenendo conto dei valori dei coefficienti parziali riportati nelle Tabelle 6.2.I, 6.2.II e 6.8.I.

Tabella 6.8.I – Coefficienti parziali per le verifiche di sicurezza di opere di materiali sciolti e di fronti di scavo.

Coefficiente	R2
γ_k	1.1

Terreno per pendii rinforzati

Table 3-2. RSS Granular Reinforced Fill Requirements.

<u>Gradation:</u> (AASHTO T-27)	<u>U.S. Sieve Size</u>	<u>Percent Passing</u>
	4 in. (102 mm) ^(a,b) ¾-inch (20 mm) ^(a)	100
	No. 4 (4.76 mm)	100 – 20
	No. 40 (0.425 mm)	0-60
	No. 200 (0.075 mm)	0 – 50
<u>Plasticity Index, PI</u> (AASHTO T-90)	PI ≤ 20	
<u>Soundness:</u> (AASHTO T-104)	Magnesium sulfate soundness loss less than 30% after 4 cycles, based on AASHTO T-104 or equivalent sodium sulfate soundness of less than 15 percent after 5 cycles.	
Note:		
(a) To apply default F* values, C _u should be greater than or equal to 4.		
(b) As a result of recent research on construction survivability of geosynthetics and epoxy coated reinforcements, it is recommended that the maximum particle size for these materials be reduced to ¾-in. (19 mm) for geosynthetics, and epoxy and PVC coated steel reinforcements unless construction damage assessment tests are or have been performed on the reinforcement combination with the specific or similarly graded large size granular fill. Prequalification tests on reinforcements using standard agency fill materials should be considered.		



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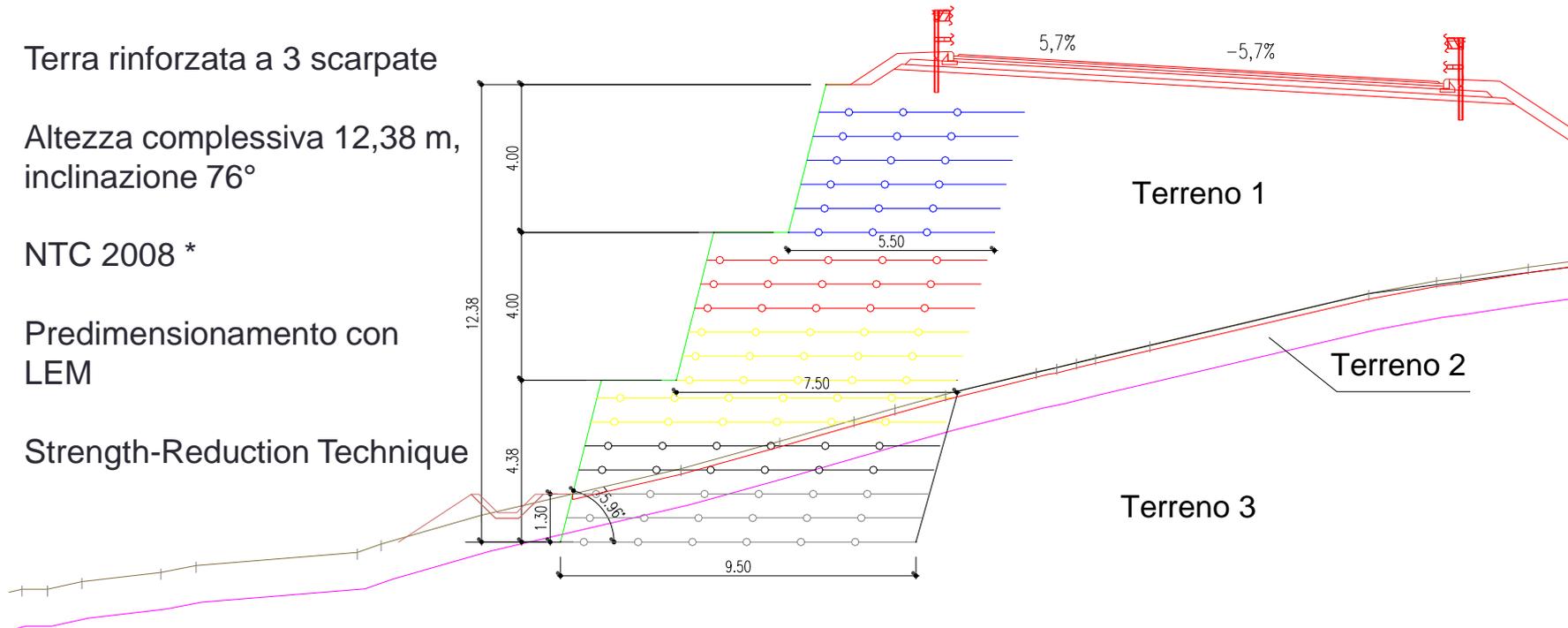
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Analisi numerica – Flac 2D v. 8.0

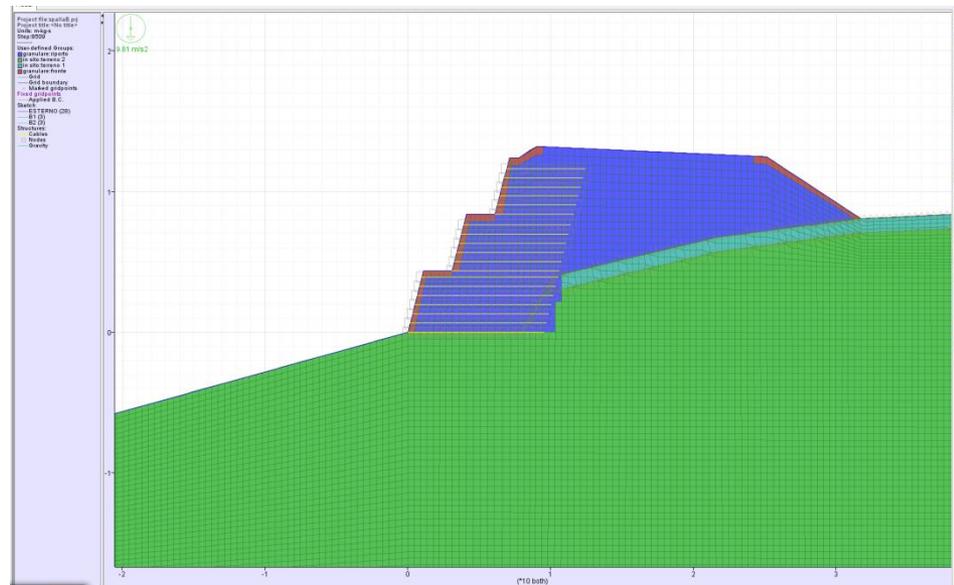
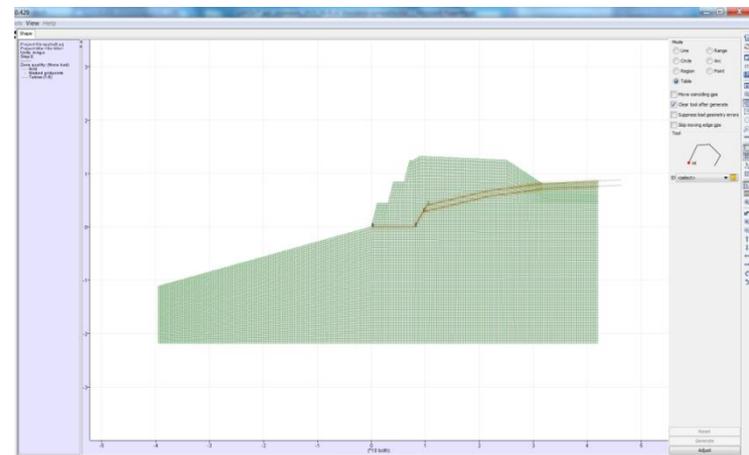
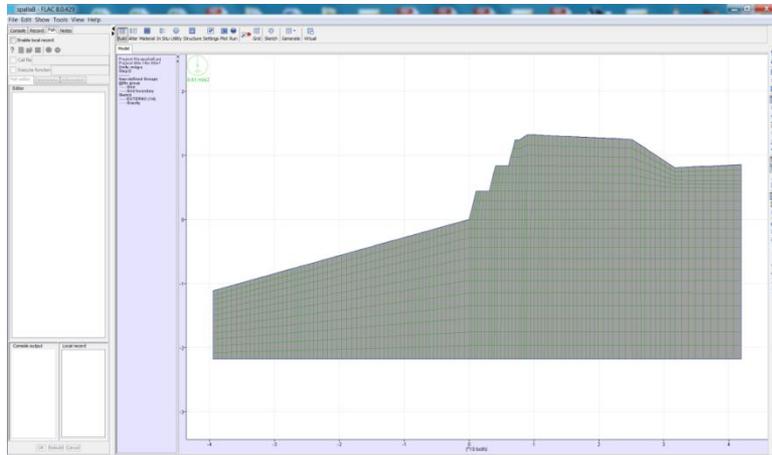
- Terra rinforzata a 3 scarpate
- Altezza complessiva 12,38 m, inclinazione 76°
- NTC 2008 *
- Predimensionamento con LEM
- Strength-Reduction Technique

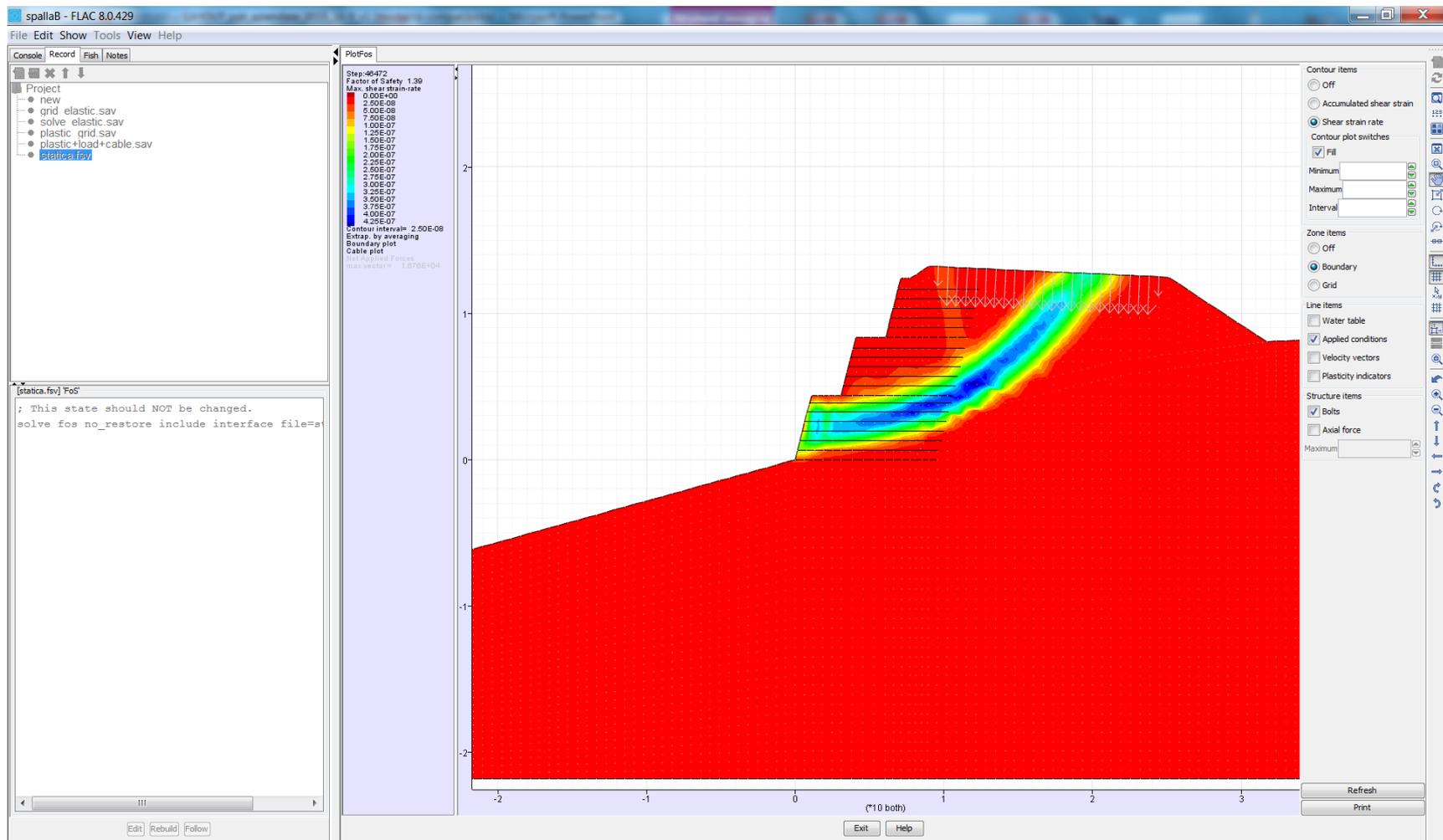


$$(*) R_d = \frac{1}{\gamma_R} R \left[\gamma_F F_K; \frac{X_K}{\gamma_M}; a_d \right] \cong R \left[\gamma_F F_K; \frac{X_K}{\gamma_R \cdot \gamma_M}; a_d \right]$$

$$\gamma_M \cdot \gamma_R = 1.25 \cdot 1.10 = 1.375$$

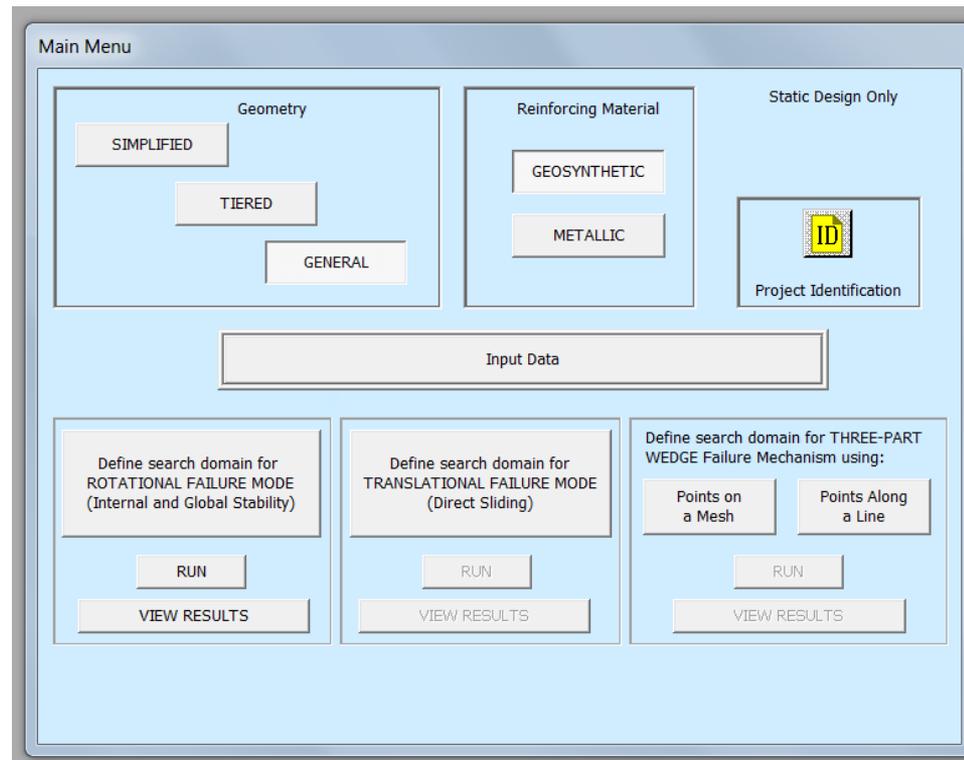
Es. 2	γ [kN/m ³]	c' [kPa]	ϕ' [deg]
Terreno 1	19	0	30
Terreno 2	20	10	32
Terreno 3	23	65	35





Pendii rinforzati

- Esempio: verifica di un pendio in terra rinforzata con il software ReSSA (Adama eng.)



E le deformazioni?

- Stati limite di esercizio

Indicazioni da BS 8006-1:2010+A1:2016

I movimenti della struttura, dopo la sua completa costruzione, possono essere classificati in:

- Cedimenti della fondazione
- Compressione del terreno di riempimento
- Deformazione per creep dei rinforzi
- Cedimenti uniformi o differenziali per chiusura di cavità al di sotto della struttura
- Deformazione per creep del terreno di riempimento con elevato contenuto di fine

Tutti questi movimenti possono essere con una attenta pratica costruttiva eccetto per il creep del rinforzo che deve essere limitato.

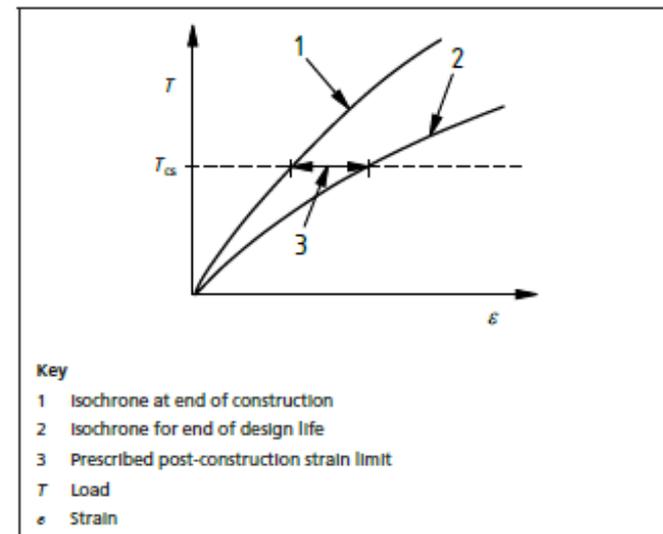
Table 19 Serviceability limits on post-construction internal strains for bridge abutments and retaining walls

Structure	Strain %
Bridge abutments and retaining walls with permanent structural loading	0.5
Retaining walls, with no applied structural loading i.e. transient live loadings only	1.0

7.4.6.3 Post-construction strain in reinforcement

Pendii In general post-construction strains in the reinforcement do not comprise a limit state and consequently strains of the order of 5% may be acceptable. However in situations where special tolerances apply (e.g. where a settlement sensitive dead load is to be supported) the values given in Table 19 may be adopted.

Figure 43 Assessment of serviceability limit state base strength



Come valutare lo stato limite di esercizio?

- Analisi **tenso-deformativa** con parametri caratteristici del terreno, per il geosintetico $\gamma_M=1,00$ (gli altri RF restano, dipendono da ε_{adm}) e limitazione della tensione del geosintetico di rinforzo

- **Metodo alternativo :**

Jewell R.A., 1996. Soil reinforcement with geotextiles. CIRIA

7.3 Serviceability limit states

Analysis to estimate the *expected deflections* in reinforced soil structures at the end of construction, and during the remaining design life of the structure is strongly recommended for design.

The simplest form of analysis for serviceability is to adopt a limiting *maximum allowable elongation* (strain) in the reinforcement, from which the *serviceability strength* of the reinforcement can be found corresponding to the design time and temperature (Section 6.4.1). The internal and overall equilibrium calculations may then be repeated, using the *design values* for *serviceability*, to demonstrate that equilibrium can be achieved without exceeding the serviceability strength.

Un dilemma: picco, critico o residuo?

(Prof. D. Leshchinsky, ReSlope User Manual)

• 4.2 Soil Shear Strength and Factor of Safety,

- Slip surface development in soil is a progressive phenomenon, especially in reinforced soil where reinforcement layers delay the formation of a surface in their vicinity. That is, a slip surface does not develop at the same instant along its full length and thus, the peak shear strength of the compacted soil is not being mobilized simultaneously as assumed in the LE analysis. Consequently, it is recommended that the design values of ϕ and c will not exceed the residual strength of the soil. This will assure that at the LE state, the shear strength utilized in each analysis is indeed attainable all along the slip surface.
- The value of the shear strength parameters reported by laboratories typically corresponds to peak shear strength. In this case, a minimum factor of safety of $F_s=1.3$ practically assures that the design strength parameters will be at or below their residual values [i.e., $\phi_d = \tan^{-1}(\tan \phi_{\text{peak}}/F_s)$ and $c_d = c_{\text{peak}}/F_s$]. It is recognized that by using the residual values, the gain in soil strength due to compaction is basically ignored in the analysis and thus, has an overall conservative impact on the reinforced slope system. However, the complex issue of progressive failure is then avoided while assuring results 'on the safe side.' Use of residual strength in analysis should not undermine the importance of compaction for structural performance.
- There are cases in which the soil will not exhibit a peak strength behavior. If the soil is lacking peak strength characteristics or the reported shear strength corresponds to a residual value, a factor of safety of $F_s=1.0$ can then be used. Note that for residual shear strength parameters, a value of $F_s=1.0$ is typically specified in design of critical structures such as geosynthetic reinforced walls. Though such a value seems to be low, recall that the stability of a steep slope is hinging on the tensile strength of the reinforcement; that is, without reinforcement a slide will occur. The soil just contributes its shear resistance to slide.

• 4.2.1 Cohesion and Factor of Safety

- If cohesive fill is used, extreme care should be used when specifying the cohesion value. Cohesion has significant effects on stability and thus the required reinforcement strength. In fact, a small value of cohesion will indicate that no reinforcement at all is needed at the upper portion of the slope. However, over the long-run cohesion of manmade embankments tends to drop and nearly diminish. Since long- term stability of reinforced steep slopes is of major concern, it is perhaps wise to ignore the cohesion altogether. It is therefore recommended to limit the design value of cohesion to 100 psf (5 kPa). It should be pointed out, however, that end-of-construction analysis must be also conducted if a soft foundation is present. In this case stability against deepseated failure must be assured.
- (BS 8006 consiglia di non considerarla)

Conclusioni

- Geosintetici possono essere ben applicati per le opere di ingegneria civile,

ma attenzione:

“Geotechnical engineering without geosynthetic engineering is cost-ineffective, while geosynthetic engineering without geotechnical engineering is a disaster.” Tatsuoka Fumio, 2009.



*Prof. Fumio Tatsuoka,
Geotechnical Eng.,
Dept. of Civil Eng.,
Tokyo Univ. of Science*

Conclusioni

- Materia in evoluzione, esempio: unico metodo di calcolo per muri e pendii rinforzati, creep, strutture con elementi rinforzati, discariche...
- Molti software disponibili, ma attenzione agli algoritmi implementati e a come viene considerato il geosintetico
- Ottima ricerca e implementazione condotta dal prof. Dov Leshchinsky e Ora Leshchinsky (Adama eng.)



Dov Leshchinsky
Emeritus Professor
University of Delaware
College of Engineering

UD's Dov Leshchinsky, professor of civil engineering, was honored with a conference in Italy devoted to his work.

Testi utili

- Koerner R.M., 2012. Designing with Geosynthetics. Vol. 1. Xlibris.
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- Davis R. O., Selvadurai A. P. S., 2002. Plasticity and Geomechanics. Cambridge.

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Never a dull moment when you're 120 feet above the ground

Ken Maschke
Civil Engineer
Years on the job: 5



What's been your favorite project so far?
"That's easy—repairing the practice facility for the Chicago Bears. We had to complete the job in time for summer practices. When we finished, we were invited to a closed practice. It was great! I got to meet some of the players. And the next year the team made it to the Super Bowl. I like to think it was because they had a better practice facility."

Do you get out of the office much?
"Last year my job sent me to Denmark for six months. I traveled to ten different countries while I was there."

How do you keep work and life in balance?
"As an engineer, I get lots of time to do the things I love outside of work, like soccer, teaching, and playing guitar."

It's your life. Your world...
How will you make a difference?

I design animal park exhibits that educate and entertain



Engineering named one of **Best Careers 2008** by *U.S. News & World Report*

How long will I be in school?
4 years for a bachelor's degree.

Will there be jobs when I graduate?
You bet! New civil engineering jobs are expected to increase 18% by 2016.

How much money will I make?
\$48,500 is the starting salary with a bachelor's degree. It goes up from there!

Source: U.S. Department of Labor, 2007

Got 5 minutes?

Find out if civil engineering is for you

Play a game

Tap into your inner engineer with online games: asce.org/kids



Watch Design Squad

Eight high school contestants tackle engineering challenges for actual clients. pbskids.org/designsquad



Find a summer program

Experience engineering first-hand and have a fantastic time. Look for programs at: engineeringedu.com/summercamps.html



I plan city bike routes that make travel safe and eco-friendly



I build dams that improve the lives of Cambodian villagers



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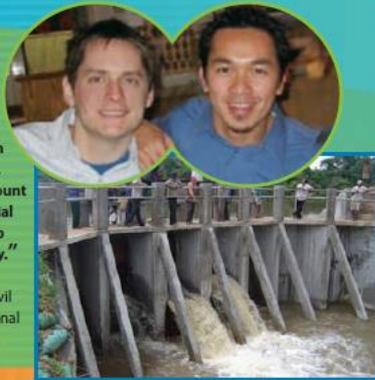
Civil Engineers. We make a difference.

Civil engineers are changing the world all the time. We dream up creative, practical solutions and work with teams of smart, inspiring people to invent, design, and build things that matter.

Design things that matter

"To help Cambodian villagers grow rice year round, we built a dam that collects water during the rainy season and releases it during the dry season. Now the villagers can double the amount of rice they grow, giving them essential food and income. It feels great to help change the lives of people so far away."

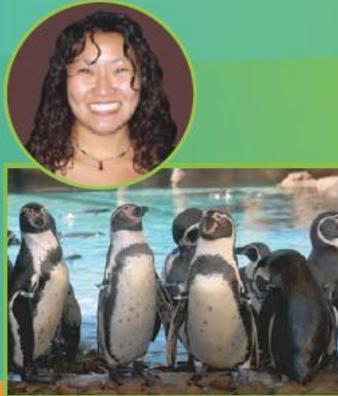
—Matt Sisul and Wil Cao, friends and civil engineers, volunteer with the international humanitarian organization, Engineers Without Borders.



Be creative

"I work with animal caretakers to design habitats that penguins will accept as their real life home. I love the creative challenges that are part of my work, like simulating rocks that the birds would naturally climb on but are also easy to clean."

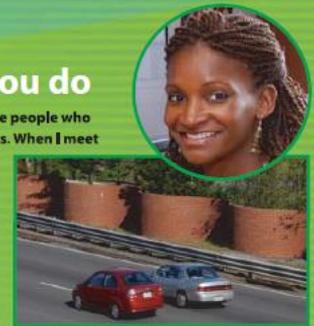
—Jennifer Nakayama, civil engineer, designs animal habitats at SeaWorld.



Love what you do

"I love spending time with the people who will benefit from my projects. When I meet people at their homes and we can't hear each other over the freeway noise, I know my work will change the quality of their lives."

—Erin Fletcher, civil engineer, develops noise barriers that make life quieter for neighborhoods near freeways.



Imagine... building a better world

Schools for quake victims

When the 2008 Sichuan earthquake hit, cities and towns were destroyed and millions were left homeless. Yan Xiao, a USC engineering professor who was visiting China at the time, saw the disaster area first-hand and got to work designing new schools that could be constructed quickly.

What makes these schools great? They're inexpensive, easy-to-assemble, earthquake resistant, and fireproof. To top it off, they're made from bamboo—a green and sustainable construction material that's widely available in China.



Better bike lanes

"Share the road." That's the call for cars and bikes to coexist peacefully, but it's not always easy. Sometimes it can be downright dangerous. To help change that, a group of civil engineering students at Northeastern University in Boston designed bike lanes to make their city safer for cycling.

Although Boston has several bike paths running through parks, they are separated from each other by busy roads. The students mapped out a plan to connect three popular routes, making it possible to ride uninterrupted from one end of the city to the other.



People helped per day

Want to help people in a BIG way? Become a civil engineer

Doctor: 20 

Teacher: 120 

Civil Engineer: 25 million 

How one engineer makes a difference



What she does

Samantha Salvia works on improving the water supply for 25 million Californians.

Why she loves it

"In one word? Relevance. Water is one of the most pressing challenges facing our society."

What she does for fun

Ultimate Frisbee national champion. New mom.

