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Comparison of international design criteria for geosynthetic-reinforced soil structures

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ABSTRACT: A summary of currently available criteria for the design of geosynthetic-reinforced walls, geosynthetic-reinforced slopes, and embankments founded on soft soils was compiled. The objective is to evaluate the consistency among different design criteria put forth by agencies worldwide, including evaluation of different performance criteria, backfill criteria, reduction factors for geosynthetics, and design methods. This compilation includes criteria established by Australia, Brazil, Canada, Germany, Hong Kong, Italy, Japan, the United Kingdom, and the United States.

1 INTRODUCTION

The use of inclusions to improve the mechanical properties of soils dates to ancient times. However, it is only within the last three decades or so that analytical and experimental studies have led to the contemporary soil reinforcement techniques. Soil reinforcement is highly attractive for use in embankment and retaining wall projects because of the economic benefits it offers in relation to conventional retaining structures. Moreover, its acceptance has also been facilitated by a number of technical factors, which include aesthetics, reliability, proven durability, simple construction techniques, good seismic performance, and the ability to tolerate large deformations without structural distress.

As a consequence of the significant growth in the use of geosynthetic-reinforced systems, government agencies and industry agencies (e.g. NCMA) have developed guidelines for the design of reinforced soil walls, reinforced soil slopes, and reinforced embankments over soft soils. The purpose of this work is to evaluate the consistency among different design criteria that are currently available on reinforced soil design. This effort was coordinated as a task of subcommittee 3 Design and Parameter Determination of the Technical Committee 9 (TC9) Geosynthetics and Earth Reinforcement Committee, International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE). The information was gathered from members of the Society and compiled in a tabulated format. The details of the information are briefly discussed below. It should be noted that although the design criteria could be the same for two different agencies (e.g., same factor of safety for a particular design aspect), the calculations needed to dimension the structure accordingly could be different. Therefore, the same design criteria may lead to different design outcomes. Nevertheless, the summary tables reflect the perceived level of conservatism assigned for each design aspect by each agency. Furthermore, when the computations are the same, comparison then is direct. By the time of compilation of these summary tables, several additional agencies were undertaking efforts to compile standards in geosynthetic-reinforced soil design. This includes the BNSR (Bureau National Sols-Routes, France) that is preparing the French Standard NFG 38064, as well as the Nordic Geotechnical Societies that are preparing the Nordic handbook for soil reinforcement.

2 GEOSYNTHETIC-REINFORCED SOIL WALLS

Table 1 provides a comparison of the design criteria for geosynthetic-reinforced soil walls. The compiled information from government agencies includes Australia, Brazil, Canada, Germany, Hong Kong, Italy, Japan, the United Kingdom, and the United States. In addition, the National Concrete Masonry Association (NCMA) is an industry agency that has also compiled guidelines for the design of reinforced soil walls.

Guidelines for performance criteria (rows 1.4 to 1.11 in Table 1) reveal a significant consistency among the different agencies. The minimum factor of safety against sliding is typically 1.5, with the exception of Australia (FS ≥ 2.0). Design criteria from Germany account explicitly for internal sliding (i.e. sliding between reinforcement layers). Several agencies do not consider overturning as a feasible mode of failure, as indicated in Table 1 (row 1.6). Yet, factors of safety ranging from 1.35 to 2.0 were adopted by agencies that consider this mode of failure. The guidelines provided by NCMA focus on the design of segmental retaining walls. Accordingly, they provide guidelines not only against overturning of the overall reinforced soil mass, but also against local facing overturning, crest toppling, and facing shear (bulging). Eccentricity at the base of the reinforced soil wall is limited to $L/6$ in most guidelines, where L is the width of the reinforced soil mass. The factor of safety values adopted against bearing capacity range from values as low as 1.35 (U.K.) to values as high as 3.0 (Brazil). However, most guidelines recognize the flexibility of the geosynthetic-reinforced systems by adopting a factor of safety value of 2.0 against bearing capacity, which is lower than values typically adopted for most geotechnical systems. Consistent with guidelines adopted for unreinforced systems, the factors of safety adopted for compound and deep seated stability range between 1.3 and 1.5. Most agencies have not established guidelines regarding seismic stability. US agencies (FHWA and NCMA) have typically established pseudo-static approach methods to deal with seismic design and, accordingly, have established factors of safety as design criteria (typically 75% of the corresponding static values). Finally, pullout factors of safety are reasonably consistent among different agencies. They range from values as low as 1.35 in the British guidelines to values as high as 2.0 in the Brazilian and Japanese guidelines. Not all methods specify global and compound stability although such stabilities could control the design in complex walls (e.g., tiered wall systems).

In general, all agencies are conservative regarding the selection of default soil-reinforcement interaction properties. Several agencies, though, provide default coefficient of interactions that designers can use in the absence of project-specific test results (Brazil, Canada, Germany, Japan, US). Differently than regarding the design criteria, default values are not consistent among different agencies (Table 1, rows 1.13 to 1.14). The default values range from coefficient of interactions as low as 50% (Germany) to as high as 100% (Japan). Nonetheless, their selection typically depends on the type of reinforcement (geogrid, geotextile) and in some cases on the type of backfill (granular, cohesive). Little guidance is provided regarding selection of default coefficients of interaction under seismic conditions. FHWA guidelines ask for 80% of the static values, while NCMA accepts 100% of the static value.

Regarding selection of backfill properties, most agencies preclude the use of any cohesive component of the shear strength of the backfill soil. Only Japan, FHWA, and NCMA provide default values of friction angles, while most of the agencies indicate that such parameters should be design specific. Regarding selection of peak or residual friction angle values for design, most agencies indicate explicitly the use of peak friction angles for design (Germany, Hong Kong, UK, FHWA, NCMA). Only Australian guidelines explicitly call for constant volume friction angle values in the design. Gradation requirements are typically stringent, and do not allow the use of cohesive backfill soils. British and FHWA standards require no more than 10 and 15 % fines, respectively. On the other extreme, Brazilian and NCMA standards allow up to 30 and 35% fines, respectively. NCMA has provisions under which soils with up to 50% fines can be used in reinforced soil walls. There is also considerable discrepancies among agencies regarding the maximum value of Plasticity Index to be allowed for reinforced soil wall design. FHWA standards require a Plasticity Index of no more than 6 %, while Brazilian and Hong Kong guidelines allow Plasticity Index values up to 15 and 20%, respectively. It is likely that gradation and plasticity criteria are related to local availability of backfill soils and local experience. Most of the guidelines indicate restrictions regarding soil pH. Many of them establish more stringent requirements for PET than for PP and HDPE.

In relation to geosynthetic reinforcement properties and reduction factors, most agencies have already identified standard tests to define the ultimate tensile strength. Although typical and/or default reduction factors on creep, degradation, and installation damage are typically provided by the agencies, there is no consistency regarding the magnitude of these factors. Creep reduction factors (RF_{CR}) of 2.5 for Polyester products and of 5.0 for Polypropylene products have been typically established in the absence of test results. Although

Table 1. Geosynthetic-Reinforced Soil Walls.

1.1	Country	Australia		Brazil	Canada	Germany	Hong Kong	Italy	Japan	United Kingdom	United States	
1.2	Agency	RTA, NSW DOT (RTA)	QMRD, Queensland DOT (QMRD)	GeoRio-Foundation for Slope Stability Control in the City of Rio de Janeiro	Canadian Geotechnical Society	General guidance by German Soc. S. Mech. And Geot. Eng.	Geotechnical Engineering Office (GEO)	Italian Ministry of Public Works	Public Works Research Center	British Standard Institution	AASHTO/Federal Highway Administration (FHWA)	National Concrete Masonry Association (NCMA)
1.3	Reference	RTA (1997)	QMRD (1997)	GeoRio (1989)	Canadian Geotechnical Society (1992)	EBGEO (1997)	GCO (1989)	Italian Ministry of Public Works (1988)	Public Works Research Center (2000)	British Standards Institution (1995)	Elias and Christopher (1997)	NCMA (1997, 1998)
1.4	Performance criteria											
1.5	Sliding	FS ≥ 2.0	FS ≥ 2.0	FS ≥ 1.5	FS ≥ 1.5	FS ≥ 2.0 (external) FS ≥ 1.4 (internal)	FS ≥ 1.5	FS ≥ 1.3	FS ≥ 1.5	FS ≥ 1.35	FS ≥ 1.5	FS ≥ 1.5
1.6	Overturning	N/A	N/A	FS ≥ 2.0	FS ≥ 2.0	N/A	N/A	FS ≥ 1.5	N/A	FS ≥ 1.35	N/A	FS ≥ 2.0 $FS_{face\ overt.} \geq 1.5$ $FS_{crest\ toppl.} \geq 1.5$ $FS_{face\ shear} \geq 1.5$
1.7	Eccentricity at base	Meyerhof distribution e ≤ L/6	Meyerhof distribution	e ≤ L/6	e ≤ L/6	e ≤ L/6 (static analysis) e ≤ L/3 (seismic analysis)	e ≤L/6 (trapezoidal distribution)	e ≤ L/6	e ≤ L/6	N/A	e ≤ L/6 (in soil) e ≤ L/4 (in rock)	N/A
1.8	Bearing capacity	FS ≥ 1.75	FS ≥ 1.75	FS ≥ 3.0	FS ≥ 2.0	FS ≥ 2.0	FS ≥ 2.0	FS ≥ 2.0	FS ≥ 2.0	FS ≥ 1.35	FS ≥ 2.5	FS ≥ 2.0
1.9	Compound and deep seated stability	FS ≥ 1.5	FS ≥ 1.5	Design specific	FS ≥ 1.3 to 1.5	Compound: N/A Deep seated: FS ≥ 1.2 to 1.4 based on load conditions	FS ≥ 1.4	FS ≥ 1.3	FS ≥ 1.2	FS ≥ 1.35	FS ≥ 1.3	FS ≥ 1.3 to 1.5
1.10	Seismic Stability	Use of pseudo-static acceleration coefficients	Required but methodology not stated	N/A	N/A	N/A	None stated	N/A	FS ≥ 1.0	N/A	FS ≥ 75% of static FS (all failure modes)	$FS_{base\ slide} \geq 1.1$ $FS_{overt.} \geq 1.1$ $FS_{face\ overt.} \geq 1.1$ $FS_{crest\ toppl.} \geq 1.1$ $FS_{face\ shear} \geq 1.1$ $FS_{bear.\ cap.} \geq 1.5$ $FS_{deep\ seat} \geq 1.1$ $FS_{pullout} \geq 1.1$
1.11	Pullout	FS ≥ 1.67	FS ≥ 1.45	FS ≥ 2.0	FS ≥ 1.5	FS ≥ 1.5	FS ≥ 1.8	N/A	FS ≥ 2.0	FS ≥ 1.30	FS ≥ 1.5*	FS ≥ 1.5

	resistance											
1.12	**Soil reinforcement interaction properties**											
1.13	Default soil-reinforcement interaction (static)	Based on test results for specific proprietary systems	Based on test results for specific proprietary systems	<u>Granular soils</u>: W: $C_i = 0.8$ NW: $C_i = 0.9$ GG: $C_i = 0.5$ to 0.8 <u>Cohesive soils</u>: W: $C_i = 0.7$ NW: $C_i = 0.8$ GG: $C_i = 0.5$	<u>For pullout</u>: GT: $C_i = 2/3$ GG: $C_i = 0.5$ to 0.7 <u>For direct sliding</u>: $\tan(\delta) = 2/3 \tan(\phi)$	Ci = 0.5	Based on test results for specific proprietary systems		<u>Granular soils</u>: $c_{interface} = 0$ kPa $\delta = \phi$ <u>Nongranular soils</u>: $c_{interface} = 0.5\,c$ $\delta = \phi$		<u>For pullout</u>: GT: $C_i = 2/3$ GG: $C_i = 0.8$ <u>For direct sliding</u>: $\tan(\delta) = 2/3 \tan(\phi)$	$C_i = 0.65$ for preliminary design (direct sliding) GRI GG-5 (pullout) ASTM D 5321 and GRI GS-6 (direct sliding)
1.14	Default soil-reinforcement interaction (dynamic)	90% of static value	Not given	N/A	N/A	N/A	None stated		100% of static value, but $FS_{pullout} \geq 1.2$		80% of static value	100% of static value
1.15	**Reinforced fill**											
1.16	Maximum cohesion	0 kPa	0 kPa	Design specific	0 kPa	0 kPa for permanent walls 50% of test results for temporary walls	0 kPa		10 kPa	5 kPa	0 kPa	0 kPa
1.17	Default value of ϕ	Determined by testing	Determined by testing	Design specific	N/A	No default values	Determined by testing		35° (gravel), 30° (sand), 25° (silt or cohesive soil)	N/A	34°	26° to 36°
1.18	Peak or constant volume ϕ	Constant volume	Constant volume	Design specific	Peak	Peak (conservative assumptions)	Peak at insitu vertical stress		Not stipulated	Peak	Peak	Peak
1.19	Gradation Requirements	<u>Sieve size</u> <u>%pass</u> 100 mm 100 9.5 mm 25-100 0.6 mm 10-100 0.075 mm 0-15	<u>Sieve size</u> <u>%pass</u> 100 mm 100 9.5 mm 30-100 0.6 mm 10-80 0.075 mm 0-10	Design specific, typically not more than 30% fines.	Typically not more than 12% fines	N/A	Frictional and cohesive frictional fills		Applicability of soil is based on Japanese unified soil classification system. Organic soils not allowed.	<u>Sieve size</u> <u>% pass</u> 125 mm 100 90 mm 85-100 10 mm 25-100 600 μm 10-65 63 μm 0-10 2 μm 0-10	<u>Sieve size</u> <u>% pass</u> 102 mm 100 0.425 mm 0-60 0.075 mm 0-15	<u>Sieve size</u> <u>% pass</u> 102 mm 100-75 4.75 mm 100-20 0.425 mm 0-60 0.075 mm 0-35 Note: 50% fines permitted for carefully engineered structures
1.20	Plasticity Index	PI ≤ 12	PI ≤ 12	Design specific, typically PI < 15	N/A	PI ≤ 7	PI ≤ 20				PI ≤ 6	PI ≤ 6 (PI ≤ 20 for carefully engineered structures)
1.21	Soundness	Inert, hard and durable	Inert, hard and durable	Design specific	N/A	N/A	Inert, hard and durable			Depends upon Agreement Certificate	Magnesium sulfate soundness loss < 30% after 4 cycles	N/A

1.22	PH	5-10 for steel 4-9 for PET 3-12 for HDPE		4 < pH < 10	N/A	5<pH<9	5<pH<10 for steel. Based on testing for others		Not stipulated	Depends upon Agreement Certificate	3 < pH < 9 (PET) PH > 3 (PP and HDPE)	3 < pH < 9
1.23	**Geosynthetic Reinforcements**											
1.24	Ultimate Tensile strength, T_{ult}	Based on test results of specific proprietary systems at 35°C	Based on test results of specific proprietary systems at 30°C	ABNT 12824	ASTM D4595	ISO DIN 10319	Based on test results of product, conducted at 35°C	UNI EN ISO 10319	Testing manual for geosynthetics (Public Works Research Center)	BS 6906 Part I	ASTM D4595 (GRI:GG1 for geogrids)	ASTM D4595 or GRI GG-1 for geogrids
1.25	RF_{CR}			Typical: 1.6 to 2.0 (PET) 3.0 to 5.0 (PP and PE)	Default: 2.5 (PET) 5.0 (PP, PE) Typical: 1.25 to 3.0. Minimum: 1.25	Default: 2.5 (PET) 5.0 (PP) 5.0 (HDPE) 2.5 (AR) 2.5 (PA)			1.67 (value used in example of design manual)	N/A	Typical: 2.5 to 2.0 (PET) 5.0 to 4.0 (PP) 5.0 to 2.5 (PE)	Typical: 1.5 to 5.0 For seismic design, RF_{CR} taken as 1.0.
1.26	RF_D			Typical: 1.1 to 2.0 Minimum: 1.1	N/A	2.0			1.0 (value used in example of design manual)	N/A	Typical: 1.1 to 2.0 Minimum: 1.1	Typical: 1.1 to 2.0 Minimum: 1.1
1.27	RF_{ID}			Typical: 1.1 to 1.7 Minimum: 1.1	Typical: 1.1 to 2.0 Minimum: 1.2	Default: 1.5 (rounded sands) 2.0 (rounded gravels) > 2.0 (crushed materials)			1.0 (value used in example of design manual)	N/A	Typical: 1.05 to 3.0 Minimum: 1.1	Typical: 1.05 to 3.0 Minimum: 1.1
1.28	Factor of safety, FS	1.1	1.1	Minimum: 1.1	Typical: 1.5	1.4 (static loads) 1.3 (hydraulic and seismic loads)	1.7		1.0	Depends upon Agreement Certificate	Typical: 1.5	Typical: 1.5
1.29	**Design methods /considerations**											
1.30	External stability	Gravity approach with use of Coulomb pressure coefficient in backfill	Gravity approach	Coherent gravity approach (similar to traditional gravity structure analysis)	Coherent gravity approach (Rankine theory)	Bishop, Janbu, Block analysis	Gravity approach		Coherent gravity approach (similar to traditional gravity structure analysis)	Limit Equilibrium	Coherent gravity approach (similar to traditional gravity structure analysis)	Coherent gravity approach (Coulomb theory with fully mobilized interface friction between reinforced and retained zones)
1.31	Internal stability	Active earth pressure distribution	Active earth pressure distribution	Rankine-type surface through reinforced soil	Rankine-type surface through reinforced soil	Rankine-type approach Block analysis	Rankine earth pressure, but with vertical line of		Coulomb-type analysis	Limit Equilibrium, Rankine stress state	Rankine-type surface through reinforced soil mass	Coulomb theory with interface friction between soil and facing

				mass	mass		maximum reinforcement tension at 0.15*H*					column. Typical interface friction angle $\delta = 2/3\ \phi$
1.32	Seismic stability	Mononobe-Okabe type analysis	Not given	N/A	N/A	N/A	Not given		Mononobe-Okabe type analysis	N/A	Mononobe-Okabe type analysis	Mononobe-Okabe type analysis
1.33	Preliminary L/H	≥ 0.6	≥ 0.7	≥ 0.8	0.5 to 0.7	≥ 0.7	≥ 0.5		Not stipulated	≥ 0.7	0.7	≥ 0.6
1.34	Minimum L and La.	$L \geq 2$ m $La \geq 1$ m	$L \geq 2$ m $La \geq 1$ m	N/A	Minimum L not stipulated. $La \geq 1$ m	N/A	Not given		Minimum L not stipulated. $La \geq 1.0$ m	$L \geq 3$ m $L \geq 7$ m for abutments	$L \geq 2.4$ m $La \geq 1$ m	$La \geq 1$ m
1.35	Live load	Live load considered when decreasing stability	Live load considered when decreasing stability	Design specific.	Contributes to load side of FS equations and not to resistance side.	Always considered	Live load considered when decreasing stability		Load is 10 kPa in highway applications	In accordance with BS 5400 (HA and HB)	Live load considered only when decreasing stability	Contributes to load side of FS equations and not to resistance side.
1.36	Distribution of reinforcements force with height	Meyerhof vertical stress distribution	Meyerhof vertical stress distribution	Rankine-based triangular distribution.	Rankine-based triangular distribution	Rankine-based distribution recommended	Rankine-based triangular distribution		Triangular distribution	Ka stress state	Rankine-based triangular distribution	Rankine-based triangular distribution (modified for seismic analysis)
1.37	Connection to facing (factors of safety, reduction factors)	FS = 1.6 (pullout from connectors) FS = 1.3 (pullout of connection from facing0	Dependent on proprietary system	N/A	Must perform laboratory tests.	$T_{facing} = 0.8\ T_{design}$			Analyses depend on type of wall (wrapped around wall, steel mesh wall, concrete wall)		$FS \geq 1.5$ (pullout) $FS \geq 1.5$ (breakage) RF_{CR} and RF_D are used for breakage. Both pullout and break are function of interblock confining pressure.	$FS \geq 1.5$ (connection) $FS \geq 1.5$ (interface shear) $FS \geq 1.5$ (internal sliding) $FS \geq 1.5$ (toppling) All FS equal 1.1 for seismic analyses.

German standards penalize the ultimate tensile strength with default reduction factors for degradation (RF_D) as high as 2.0, most guidelines indicate degradation reduction factors ranging from 1.0 to 1.1. Finally, regarding reduction factors to account for construction damage (RF_{CD}), minimum values of 1.1 have been generally adopted by the different agencies. The factor of safety FS, typically adopted in addition to the reduction factors to penalize the ultimate tensile strength of the geosynthetics ranges from values as low as 1.0 (Japan) to values as high as 1.7 (Hong Kong).

While most of the agencies use coherent gravity approaches for the design of geosynthetic-reinforced soil walls (similar to traditional gravity structure analyses), German and British standards adopt a more generic limit equilibrium approach for external stability analysis. Regarding internal stability, all agencies adopt some type of Rankine- or Coulomb-type surface through the reinforced soil mass. Not all agencies provide guidelines regarding seismic stability, but those that provide it adopt a pseudostatic (Mononobe-Okabe type) analysis. Most agencies adopt a minimum length of reinforcements equal to 70% of the structure height. However, Hong Kong guidelines ask for a length as low as 50% of the height, while Brazilian guidelines ask for a minimum of 80% of the height. The minimum reinforcement length is typically on the order of 2 m, while the minimum active reinforcement length is typically on the order of 1 m. The distribution of maximum reinforcement force with height adopted by all agencies is typically a Rankine-based triangular distribution. Some guidelines establish factors of safety and reduction factors to be used in the analysis of the connection of the reinforcement to facing. Detailed modes of failure have been identified by NCMA, with factors of safety of 1.5 adopted for connection to facing, interface shear (bulging), internal sliding, and toppling.

3 GEOSYNTHETIC-REINFORCED SOIL SLOPES

Table 2 provides a comparison of the design criteria for geosynthetic-reinforced soil slopes. The compiled information from government agencies includes Brazil, Hong Kong, Italy, Japan, the United Kingdom, and the United States. In a general sense, there have been fewer efforts by government agencies in compiling design criteria for geosynthetic-reinforced soil slopes than for geosynthetic-reinforced walls.

As for the case of geosynthetic-reinforced walls, performance criteria for geosynthetic-reinforced slopes (rows 2.4 to 2.12 in Table 2) are reasonably consistent among the different agencies. The factors of safety against sliding, local bearing failure, deep seated stability, and compound failure are typically 1.3. Eccentricity at the base is typically not considered in the design. Factors of safety against internal slope stability are also typically 1.3, while factors of safety against pullout range from values as low as 1.2 (Hong Kong) to values as high as 2.0 (Brazil, Japan). Most agencies have not established guidelines regarding seismic stability. Yet, FHWA and Japan establish minimum factors of safety of 1.1 and 1.0, respectively, in pseudo-static analyses. It should be noted that in lieu of the conservatism of the pseudostatic stability analysis, FHWA recommends to use a design seismic coefficient that is half the maximum ground acceleration.

Agencies are conservative regarding the selection of default soil-reinforcement interaction properties. Several agencies, though, provide default coefficient of interactions that designers can use in the absence of project-specific test results (Brazil, Japan, FHWA). The values selected by these agencies are the same as those adopted for geosynthetic-reinforced soil wall design. Little guidance is provided regarding selection of default coefficients of interaction under seismic conditions.

Regarding selection of backfill properties, there is less guidance on the selection of backfill material for geosynthetic-reinforced soil slopes than in the case of geosynthetic-reinforced soil walls. However, agencies are in general more permissive regarding the use of cohesive soils, with FHWA standards allowing up to 50% fines and Brazilian standards typically up to 30% fines. There are also more relaxed requirements regarding the Plasticity Index to be allowed for reinforced soil slope design, with most agencies allowing up to a Plasticity Index value of 20 %. Most of the guidelines indicate restrictions regarding soil pH.

Criteria adopted by different agencies regarding geosynthetic reinforcement properties and reduction factors for geosynthetic-reinforced soil slopes are consistent with those adopted for geosynthetic-reinforced soil walls. Regarding methods of design, most agencies adopt some type of limit equilibrium analysis. A triangular distribution of maximum reinforcement tension with depth is typically adopted in the design considerations. As noted in Table 2, FHWA adopts a uniform distribution of reinforcements with depth for walls lower than 6 m. Not all agencies provide guidelines regarding

Table 2. Geosynthetic-Reinforced Soil Slopes

2.1	**Country**	**Brazil**	**Hong Kong**	**Italy**	**Japan**	**United Kingdom**	**United States**
2.2	**Agency**	GeoRio-Foundation for Slope Stability Control in the City of Rio de Janeiro	Geotechnical Engineering Office (GEO)	Italian Ministry of Public Works	Public Works Research Center	British Standards Institution	Federal Highway Administration (FHWA)
2.3	**Reference**	GeoRio (1999)	GEO (1993)	Italian Ministry of Public Works (1988)	Public Works Research Center (2000)	BS 8006 (1995)	Elias and Christopher (1997)
2.4	**Performance criteria**						
2.5	Sliding	N/A	FS ≥ 1.2	FS ≥ 1.3	FS ≥ 1.2	FS ≥ 1.3	FS ≥ 1.3
2.6	Eccentricity at base	N/A	Trapezoidal distribution	N/A	N/A	N/A	N/A
2.7	Local bearing failure	N/A	FS ≥ 1.2	N/A	N/A	FS ≥ 1.3	FS ≥ 1.3
2.8	Deep seated stability	FS ≥ 1.3	FS ≥ 1.4	N/A	FS ≥ 1.2	FS ≥ 1.3	FS ≥ 1.3
2.9	Compound failure	FS ≥ 1.3	FS ≥ 1.2	FS ≥ 1.3	N/A		FS ≥ 1.3
2.10	Internal slope stability	N/A	FS ≥ 1.2	FS ≥ 1.3	FS (long term) ≥ 1.2 FS (short term) ≥ 1.1	Partial factor design	FS ≥ 1.3
2.11	Seismic Stability	N/A	None stated	N/A	FS ≥ 1.0	N/A	FS ≥ 1.1 (pseudo-static analysis)
2.12	Pullout resistance	FS ≥ 2.0	FS ≥ 1.2	N/A	FS ≥ 2.0 FS ≥ 1.2 (seismic case)	Partial factor design	FS ≥ 1.5 FS ≥ 2.0 (cohesive soils)
2.13	**Soil reinforcement interaction properties**						
2.14	Default soil-reinforcement interaction properties (static case)	Sandy Soils: W: $C_i \le 0.8$ NW: $C_i \le 0.9$ GG: $C_i \le 0.5$ to 0.8 Silty Soils: W: $C_i \le 0.7$ NW: $C_i \le 0.8$ GG: $C_i \le 0.5$	Based on test results for specific proprietary reinforcements		Granular soils: $c_{interface} = 0$ kPa $\delta = \phi$ Nongranular soils: $c_{interface} = 0.5\,c$ $\delta = \phi$		For pullout: GT: $C_i = 2/3$ GG: $C_i = 0.8$ For direct sliding: $\tan(\delta) = 2/3 \tan(\phi)$
2.15	**Reinforced fill**						
2.16	Cohesion	Design specific	0 kPa		10 kPa	≤ 5 kPa	0 kPa
2.17	Default ϕ	Design specific	Peak at insitu vertical stress		35° (gravel), 30° (sand), 25° (silty or cohesive soil).	N/A	30°
2.18	Gradation Requirements	Design specific, typically not more than 30% fines (smaller than 0.075 mm)	Cohesive frictional and frictional fills		Applicability of soil is based on Japanese unified soil classification system. Organic soil not allowed.	Based upon Highway Specification	Sieve size / % pass 20 mm 100-75 4.76 mm 100-20 0.425 mm 0-60 0.075 mm 0-50
2.19	Plasticity Index	N/A	PI ≤ 20				PI ≤ 20
2.20	Soundness	N/A	Inert, hard and durable				Magnesium sulfate soundness loss < 30% after 4 cycles
2.21	PH	4 < pH < 10	Based on test results for specific proprietary reinforcements		Not stipulated		3 < pH < 9 (PET) pH >3 (PP & HDPE)
2.22	**Geosynthetic Reinforcements**						

2.23	Ultimate Tensile strength, T_{ult}	ABNT 12824	Based on test results of specific proprietary reinforcements at 30°C	UNI EN ISO 10319	Testing manual for geosynthetics (Public Works Research Center)	BS 6906 Part I	ASTM D4595 (GRI:GG1 for geogrids)
2.24	RF_{CR} (typical ranges)	Typical ranges: 1.6 to 2.0 (for PET) 3.0 to 5.0 (for PP and PE)			Typical range: 1.7 to 2.0	Agreement Certificate and Limit State Design	2.5 to 2.0 (for PET) 5.0 to 4.0 (for PP) 5.0 to 2.5 (for PE)
2.25	RF_D	1.1 to 2.0 (typical) 1.1 (minimum)			1.0 (value used in example of design manual)	Agreement Certificate and Limit State Design	1.1 to 2.0 (typical) 1.1 (minimum)
2.26	RF_{ID}	1.1 to 1.7 (typical) 1.1 (minimum)			1.0 (value used in example of design manual)	Agreement Certificate and Limit State Design	1.05 to 3.0 (typical) 1.1 (minimum)
2.27	Factor of safety, FS, or partial factors of safety (please specify in details)	Dependent of reinforcement characteristics, 1.1 (minimum)	See approved value in certificate		1.0	Agreement Certificate and Limit State Design	FS same as in stability analysis of unreinforced slopes (target is FS > 1.3)
2.28	**Design methods /considerations**						
2.29	External stability	Analysis of reinforced block against external loads	Gravity approach		Not considered	Limit equilibrium	Analysis of reinforced block against external loads
2.30	Internal stability	Limit equilibrium analysis considering effect of reinforcements	Slip surface analysis considering effect of reinforcements		Limit equilibrium analysis considering circular failure surface and the effect of reinforcements	Limit equilibrium, considering influence of reinforcement	Limit equilibrium analysis considering effect of reinforcements
2.31	Seismic stability	N/A	Not stated		Mononobe-Okabe type analysis	N/A	Pseudo-static limit equilibrium analysis
2.32	Distribution of reinforcements with height	Design specific	Horizontal stress increases linearly with depth		Uniform with height	Ka stress state	If H ≤ 6m : Uniform reinforcement distribution. If H > 6m: Uniform zones of increasing reinforcement density with depth.
2.33	Minimum anchorage length	N/A	N/A	N/A	La ≥ 1m		La ≥ 1 m

Table 3. Embankments over Soft Soils

3.1	Country	Brazil	Japan	United Kingdom
3.2	Agency	DNER-Brazilian National Highway Department	Public Works Research Center	British Standards Institution
3.3	Reference	DNER (1998a, 1998b)	Public Works Research Center (2000)	BS 8006 (1995)
3.4	**Performance criteria**			
3.5	Sliding	N/A	FS ≥ 1.5	FS ≥ 1.3
3.6	Eccentricity at base	N/A		N/A
3.7	Local bearing failure	N/A		FS ≥ 1.3
3.8	Deep seated stability	1.2 to 1.4, depending on embankment type		FS ≥ 1.3
3.9	Compound failure	1.2 to 1.4, depending on embankment type		
3.10	Internal slope stability	N/A	FS ≥ 1.2	Partial Factor Design (Limit State Code)
3.11	Seismic Stability	N/A	N/A	N/A
3.12	Pullout resistance	Design specific	Geosynthetics should be placed over the entire width of the embankment	Partial Factor Design
3.13	**Soil reinforcement interaction properties**			
3.14	Default soil-reinforcement interaction properties (static case)	N/A	Granular soils: $c_{interface} = 0$ kPa $\delta = \phi$ Nongranular soils: $c_{interface} = 0.5$ c $\delta = \phi$	N/A
3.15	**Reinforced fill**			
3.16	Gradation Requirements	N/A	Applicability of soil is based on Japanese unified soil classification system.	Based upon Highway Specification
3.17	Plasticity Index	N/A		
3.18	Soundness	N/A		
3.19	PH	N/A	Not stipulated	
3.20	**Geosynthetic Reinforcements**			
3.21	Ultimate Tensile strength, T_{ult}	ABNT 12824	Testing manual for geosynthetics (Public Works Research Center)	BS 6906 Part I
3.22	RF_{CR} (typical ranges)	Design specific	Typical range: 1.7 to 2.0	Agreement Certificate and Limit State Design
3.23	RF_D	1.1 to 2.0 (typical) 1.1 (minimum)	1.0 (value used in example of design manual)	Agreement Certificate and Limit State Design
3.24	RF_{ID}	1.1 to 1.7 (typical) 1.1 (minimum)	1.0 (value used in example of design manual)	Agreement Certificate and Limit State Design
3.25	FS	1.2 to 1.4, depending on embankment type	1.0	Agreement Certificate and Limit State Design
3.26	**Design methods/considerations**			
3.27	External stability	Circular and non-circular slip surfaces taking into account reinforcement contribution. Soft soil expulsion has to be verified	Limit equilibrium analysis	Limit Equilibrium FEM/FDM also permitted
3.28	Internal stability	Typically wedge analyses, with part of the slip surface along the fill-reinforcement interface	Limit equilibrium analysis	Limit Equilibrium FEM/FDM also permitted
3.29	Seismic stability	N/A	Not stipulated	N/A
3.30	Minimum anchorage length	N/A	Geosynthetics should be placed over the entire width of the embankment	Partial Factor Design

seismic stability, but there is a tendency in adopting pseudostatic limit equilibrium analyses.

4 EMBANKMENTS OVER SOFT SOILS

Table 3 provides a comparison of the design criteria for embankments founded on soft soils. Only three agencies (from Brazil, Japan, and U.K.) were identified that provide guidelines regarding the use of reinforcements to support embankments over soft soils. Because of the typically site-specific nature of these projects, less guidance is provided by the different agencies on criteria for embankments over soft soils than for geosynthetic-reinforced soil walls and geosynthetic-reinforced soil slopes.

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NOTATIONS

AR: Aramid
C_i: Coefficient of interaction ($C_i = \tan\delta / \tan\phi$)
e: Eccentricity
FS: Factor of safety
GG: Geogrid
GT: Geotextile
H: Height of the slope or wall
HDPE: High density polyethylene
L: Reinforcement length at base
La: Anchorage length for pullout evaluation
NW: Nonwoven geotextiles
PA: Polyamid
PET: Polyester
PE: Polyethylene
PP: Polypropylene
RF_{CR} : Creep Reduction Factor
RF_{ID} : Installation damage reduction factor
RF_D : Durability Reduction Factor
W: Woven geotextiles
δ: Interface friction angle
ϕ: Peak soil friction angle

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