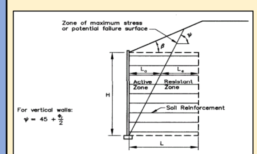
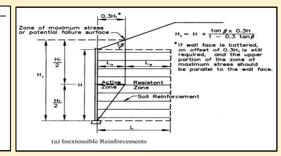
EXISTING CODES AND GUIDELINES

CODE	REFERENCE DOCUMENTS	COUNTRY OF ORIGIN	COUNTRIES WHERE USED	EXPERTS
AASHTO	AASHTO LRFD Bridge Design Specifications, 8th Edition (reference documents: FHWA-NHI-10-024 / FHWA-HIF-17-004 / NCMA)	USA	USA	Jim Collin, Giulia Lugli, Keith Brabant, Franciscus Hardianto, Michael Bernardi
BS 8006	BS 8006 -2016 Code of practice for strengthened/reinforced soils and other fills	UK	UK, IRELAND	Yuli Doulala-Rigby,Pat Naughton, Nico Brusa, Ivan Puig, Patricia Guerra, Ian Scotland, David Woods, A. Belton
EBGEO	FGSV AA 5.6 Bauwerk und Boden - Merkblatt für den Entwurf und die Bemessung von Stützkonstruktionen aus stahlbewehrten Erdkörpern	GERMANY	GERMANY	Oliver Detert, Lars Vollmerts
NF P 94-270	NF P 94-270 Calcul géotechnique - Ouvrages de soutènement - Remblais renforcés et massifs en sol cloué; EN 1997-1 and NF EN 1997-4/NA, EN 14475	FRANCE	FRANCE; Accepted in Eastern Europe, Balkans, Northern and Central Africa	Jeremy Plancq, Giulia Lugli, Juan Lima
ITALIAN AGI GUIDELINES	Linee Guida AGI sulle strutture in terra rinforzata con geosintetici (under discussion)	ITALY	ITALY	Pietro Rimoldi
NEW EUROCODE 7	EN 1997-3 (under discussion)	EUROPEAN UNION	EUROPEAN UNION	Nicolas Freitag, Castorina Silva Vieira, Pietro Rimoldi
CANADA	Canadian Standards Association (CSA). 2019. Canadian Highway Bridge Design Code. CAN/CSA-S6-14, Mississauga, Ontario, Canada (in press).	CANADA	CANADA	Richard Bathurst, Shahariar Mirmirani
JAPAN		JAPAN	JAPAN	Yoshihisa Miyata
HONG KONG	CEDD GeoGuide 6 2017. Guide to Reinforced Fill Structure and Slope Design	HONG KONG	HONG KONG	Robert Lozano, Pietro Rimoldi
SOUTH AFRICA		SOUTH AFRICA	SOUTH AFRICA	Edoardo Zannoni
AUSTRALIA	AS4678 "Earth retaining structure" & R57 "Design of reinforced soil walls"	AUSTRALIA	AUSTRALIA, NEW ZEALAND	Chris Lawson
INDIA	IRC SP:102-2014 -"Guidelines for design and construction of Reinforced Soil walls" & Ministry of Roads Transport & Highways Specification Sect 3100. IRC SP 102 - based on BS8006 & FHWA. MoRTH 3100 refers to BS8006-1, FHWA and AFNOR	INDIA	INDIA, NEPAL, BHUTAN	Ratnakar Mahajan

TOPIC	SUB-TOPIC	USA - AASHTO / FHWA
1. VERTICAL SPACING	1.1. CALCULATION APPROACH	Empirically derived (research by Allen and Bathurst)
	1.2. LIMITS	• For segmental concrete facing blocks, Sv shall be limited to twice the width of the block Wu or 2,7 ft (0,88 m), whichever is less • For welded wire, expanded metal or similar facing, Sv should be limited to 2 ft (0,65 m) max • In general, a vertical spacing greater than 2,7 ft should not be used without full scale data, except for systems with facing width Wu equal or greater than the facing units height. For these larger facing units, Sv shall not exceed 3,3 ft (1 m)
2. INTERNAL STABILITY DESIGN METHODS FOR MSE WALLS	2.1. AVAILABLE METHODS	Coherent Gravity Method (for steel reinforcements) or Simplified method (for both steel and geosynthetic reinforcements
	2.2. DIFFERENTIATIONS	• CGM for steel reinforced structures, SM for both steel and gsy reinforced structures, • For CGM, the vertical earth pressure σv at each reinforcement level shall be computed using an equivalent uniform base pressure distribution over an effective width of reinforced wall mass -->$\sigma v = \Sigma V/(L-2e)$ (Articles 11.6.3.1 and 11.6.3.2), and • For CGM, the lateral earth pressure coefficient used shall be equal to $k0$ at the point of intersection of the theoretical failure surface with the ground surface at or above the wall top, transitioning to ka at a depth of 20.0 ft (6m) below that intersection point, and constant at ka at depths greater than 20.0 ft (6m). If used for geosynthetic reinforced systems, ka shall be used throughout the wall height.
	2.3. LIMITS	• ϕ limited to 40° max, allowing a default value of 34° if no shear testing is performed • For extensible reinforcements, Rankine method to be used up to a face batter of 10° • A minimum length Le of 3ft (0.985m) shall be used • Ka shall be determined assuming no wall friction ($\delta = \beta$) • A vertical spacing Sv greater than 2,7ft (0,88m) shall not be used without full scale wall data
3. DESIGN METHODS FOR STEEP	3.1. AVAILABLE METHODS	Covered only by FHWA: modified limit equilibrium analysis, allowable stress approach for slope stability (FHWA-NHI-10-025, Volume II, Chapter 8.3.3). Analytical method e.g., Bishop & other for simple structures, and Modified Bishop, Spencer etc. for more complex structures
	3.2. DIFFERENTIATIONS	Circular or wedge-type potential failure

SLOPES	3.3. LIMITS	• For slopes up to 70° • The deformability of the reinforcements not taken into account • Tensile force direction is dependent on the extensibility and continuity of the reinforcements used, and the following inclination is suggested: - Discrete, strip reinforcements: T parallel to the reinforcements. - Continuous, sheet reinforcements: T tangent to the sliding surface.
4. EXTENSIBLE / INEXTENSIBLE REINFORCEMENT	4.1. DIFFERENTIATION LINES	• Inextensible reinforcements reach their peak strength at strains lower than the strain required for the soil to reach its peak strength. • Extensible reinforcements reach their peak strength at strains greater than the strain required for soil to reach its peak strength. Refer to AASHTO 11.10.6.3 
	4.2. REINFORCEMENT TYPE (STRIP, GRID) INFLUENCE	• Inextensible reinforcements consist of metallic strips, bar mats, or welded wire mats • Extensible reinforcements consist of geotextiles or geogrids
	4.3. SOIL STIFFNESS INFLUENCE	Soil failure limit state is considered only in the K-Stiffness Method (developped by Allen and Bathurst). The method is based on the following concept (refer to FHWA-NHI-10-025, Volume II, Appendix F): "the geosynthetic reinforcement continues to strain and gain tensile load long after the soil has reached its peak strength and begun dropping to a residual value. Therefore, if the strain in the soil is limited to prevent it from going past peak to a residual value, failure by excessive deformation or rupture is prevented and equilibrium is maintained." The WSDOT GDM should be consulted for further details.
5. MISSING REINFORCEMENTS	5.1. GEOCELLS	Not inlcuded in AASHTO, included in FHWA
	5.2. GEOSTRIPS	Not inlcuded in AASHTO, included in FHWA
	5.3. OTHER (specify)	
	6.1. LOCAL STABILITY ANALYSES	Potential for reinforcement rupture and pullout are also evaluated at the connection of the reinforcement to the wall facing. The factored tensile load applied to the soil reinforcement connection at the wall face, To, shall be equal to the maximum factored reinforcement tension, Tmax, for all wall systems regardless of facing and reinforcement type. Tmax is calculated as per AASHTO 11.10.6.4.1-2

6. CONNECTION CAPACITY	6.2. STATIC AND SEISMIC DIFFERENCES	Facing elements shall be designed to resist the seismic loads determined as specified in AASHTO Article 11.10.7.2 If the connection strength is partially or fully dependent on friction between the facing blocks and the reinforcement, the connection strength to resist seismic loads shall be reduced to 80 percent of its static value. For mechanical connections that do not rely on a frictional component, the 0.8 multiplier may be removed from Eqs. Refer to AASHTO Article 11.10.7.3
	6.3. SHORT AND LONG TERM PROPERTIES	The serviceability limit state is not specifically evaluated in current practice
	6.4. REDUCTION FACTORS	• STEEL: The capacity of the embedded connector shall be checked by tests as required in AASHTO Article 5.11.3.Connection materials shall be designed to accommodate losses due to corrosion (sacrificial thickness) in accordance with Article 11.10.6.4.2a. • GSY: The portion of the connection embedded in the concrete facing shall be designed in accordance with AAHTO Article 5.11.3. The long-term creep reduced geosynthetic strength at the connection is calcuated by knowing the the creep reduced connection strength and the durability RF (AASHTO 11.10.6.4.4b)
7. SEISMIC DESIGN	7.1. DESIGN SEISMIC ACCELERATION FOR RS STRUCTURES	• External Stability: wall mass inertial forces shall be included in analysis. • Internal Stability: reinforcements shall be designed to withstand horizontal forces generated by the internal inertia force Mononobe-Okabe method is used to calculate the seismic active earth pressure
	7.2. RFcreep FOR GSY	From product specific test results as specified in AASHTO Article 11.10.6.4.2b
8. MINIMUM CHECKS	8.1. INTERNAL STABILITY	ULS Tensile Resistance of Reinforcement Pullout Resistance of Reinforcement Structural Resistance of Face Elements Structural Resistance of Face Element Connections
	8.2. EXTERNAL STABILITY	ULS Sliding on the base Overturning Bearing resistance SLS Vertical Wall Movements Lateral Wall Movements
	8.3. LOCAL STABILITY AT FACE	The connection of the reinforcements with the facing, should be designed for TMAX for all limit states (refer to INTERNAL STABILITY)
	8.4. GLOBAL STABILITY	Overall Stability Compound Stability
	9.1. AMPLIFICATION FACTORS FOR LOADS	EV = 1.35; LL = 1.75; EH = 1.5 (Table 3.4.1-1

9. AMPLIFICATION AND REDUCTION FACTORS	9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS	3.5.2.b Installation Damage Reduction Factor, RFID 3.5.2.c Creep Reduction Factor, RFCR 3.5.2.d Durability Reduction Factor, RFD 3.5.2.e Durability Reduction Factor, RFD, at Wall Face Unit Resistance Factor Table 11.5.7-1 3.5.2.f LRFD Geosynthetic Resistance Factor, 3.5.2.g Preliminary Design Reduction Factor, RF
	9.3. MODEL FACTORS / FACTORS OF SAFETY	
10. SERVICEABILITY LIMITS	10.1. FACE DEFORMATIONS	11.5.2 & C11.5.2;11.10.4.2 - Lateral Displacement - Figure C11.10.4.2-1 . Figure 2-15; section 4.4.7j
	10.2. SETTLEMENTS AT BASE	11.10.4.1&C11.10.4.1. No limits for total settlement is provided. Differential settlement is addressed in C11.10.4.1-1. Section 2.8.3 Performance Criteria
	10.3. SETTLEMENTS AT TOP	Not addressed directly. Differential internal settlement controlled by requiring high quality select fill
11. SOILS	11.1. PERMITTED TYPES OF FILL	Included in AASHTO Construction Specifcations 7.3.6.3: Granular Fill with gradation, PI, Shear Strength, Durability and Electrochemical requirements. Section 3.2.1
	11.2. MINIMUM GEOTECHNICAL PROPERTIES	Default value of friction angle of 34 degrees if gradation and plasticity properties are met. Actual friction angle can be tested and used, but it is limited to not larger than 40 degrees. Lower friction angles can be used.
	11.3. MINIMUM INTERACTION PROPERTIES	11.10.6.3.2: F* and α shall be determined from product-specific pullout tests in the project backfill material or equivalent soil, or they can be estimated empirically/theoretically. For standard backfill materials (see AASHTO LRFD Bridge Construction Specifications, Article 7.3.6.3), with the exception of uniform sands, i.e., coefficient of uniformity C u =D 60 /D 10 < 4, in the absence of test data it is acceptable to use cons ervative default values for F* and α as shown in Figure 11.10.6.3.2-2 and Table 11.10.6.3.2-1 . For ribbed steel strips, if the specific C u for the wall backfill is unknown at the time of design, a C u of 4.0 should be assumed for design to determine F*. Laboratory tests on pullout or metal losses based on the direct project conditions may be used if accepted by the authorizing agency.
	11.4. LIMITS ON BACKFILL	Included in AASHTO Construction Specifcations 7.3.6.3: Granular Fill with gradation, PI, Shear Strength, Durability and Electrochemical requirements. Section 3.2.1
	11.5. LIMITS ON FOUNDATION SOIL	Foundation Soil Bearing Resistance 11.10.5.4; See section 10.2 above for settlements. Foundation Soil Bearing Resistance 11.10.5.4; See section 10.2 above for settlements

12. WATER	12.1. MODELS OF WATER PRESSURE	11.10.10.3 Hydrostatic Pressures: For structures along rivers and streams, a minimum differential hydrostatic pressure equal to 3.0 ft of water shall be considered for design. This load shall be applied at the high-water level. Effective unit weights shall be used in the calculations for internal and external stability beginning at levels just below the application of the differential hydrostatic pressure. Section 5.3 - Drainage
	12.2. WATER INDUCED FAILURES	
	12.3. WATER IN SEISMIC CONDITIONS	
13. MISSING ELEMENTS	13.1 ITEMS NOT INCLUDED IN CODE	Definition of "Significant" Settlement. Separate pullout resistance or metal loss criteria for dry environments versus wet or submerged environments in galvanized steel reinforcements

TOPIC	SUB-TOPIC	UK - BS 8006
1. VERTICAL SPACING	1.1. CALCULATION APPROACH	Coherent Gravity / Tie Back Wedge for walls. 2 part wedge, slip circle or others for steep slopes.
	1.2. LIMITS	Not specified but general industry guidelines are 0.15 min and usually 0.6m for geogrids. Soil reinforcement with 70degree slope could be also up to 1m; for gabion faced walls the spacing can be up to 1m. Typically 0.75m for modular panel solutions.
2. INTERNAL STABILITY DESIGN METHODS FOR MSE WALLS	2.1. AVAILABLE METHODS	Coherent Gravity / Tie Back Wedge.
	2.2. DIFFERENTIATIONS	Inextensible/Extensible reinforcement.
	2.3. LIMITS	0.7H or 3m min reinforcement length. Bridge abutments, trapezoidal walls, stepped walls and walls with low thrust differ ref. Table 14 . 70 degrees to horizontal up to vertical. Design to ULS and SLS.
3. DESIGN METHODS FOR STEEP SLOPES	3.1. AVAILABLE METHODS	Two part wedge, slip circle, log spiral or coherent gravity.
	3.2. DIFFERENTIATIONS	Slip circles generally for angles less than 45 degrees, other methods for steeper facings and coherent gravity for inextensible reinforcement.
	3.3. LIMITS	70 degree to the horizontal or less. Design to ULS and SLS.
4. EXTENSIBLE / INEXTENSIBLE REINFORCEMENT	4.1. DIFFERENTIATION LINES	Inextensible reinforcement sustains loads at less than or equal to 1% strain.
	4.2. REINFORCEMENT TYPE (STRIP, GRID) INFLUENCE	Earth pressure for design is Ka for extensible reinforcement, for inextensible reinforcement a variation from Ka to Ko in the top 6m of wall.
	4.3. SOIL STIFFNESS INFLUENCE	Not included in BS
5. MISSING REINFORCEMENTS	5.1. GEOCELLS	Not included in BS.
	5.2. GEOSTRIPS	Included
	5.3. OTHER (specify)	combined drainage & reinforcement geogrids; drainage geomposites could be specified behind wall face or at the back of the wall instead of traditional pea gravel/shingle (ref 6.10.5)

6. CONNECTION CAPACITY	6.1. LOCAL STABILITY ANALYSES	75-100% connection strength required dependant on face type. Ref Table 20.
	6.2. STATIC AND SEISMIC DIFFERENCES	N/A
	6.3. SHORT AND LONG TERM PROPERTIES	N/A
	6.4. REDUCTION FACTORS	N/A
7. SEISMIC DESIGN	7.1. DESIGN SEISMIC ACCELERATION FOR RS STRUCTURES	Adopts AASHTO pseudo-static analysis where required.
	7.2. RFcreep FOR GSY	N/A
8. MINIMUM CHECKS	8.1. INTERNAL STABILITY	Rupture, adhesion/bond, pullout, wedge failure, sliding, compound stability, strain limits for serviceability.
	8.2. EXTERNAL STABILITY	Bearing capacity, sliding, overturning, global and compound stability.
	8.3. LOCAL STABILITY AT FACE	Connection rupture, overturning and sliding.
	8.4. GLOBAL STABILITY	Slip circle and sliding.
9. AMPLIFICATION AND REDUCTION FACTORS	9.1. AMPLIFICATION FACTORS FOR LOADS	Table 11 for walls, Table 21 for slopes.
	9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS	Table 11 for walls, Table 21 for slopes.
	9.3. MODEL FACTORS / FACTORS OF SAFETY	Table 11 for walls, Table 21 for slopes.
10. SERVICEABILITY LIMITS	10.1. FACE DEFORMATIONS	Table 18 & 19 give guidelines and limits on post construction strain
	10.2. SETTLEMENTS AT BASE	Table 17 gives guidelines on tolerable settlements by face type
	10.3. SETTLEMENTS AT TOP	Table 16 gives guidelines on tolerable internal settlements by face type
11. SOILS	11.1. PERMITTED TYPES OF FILL	BS8006 allows only 6I/J and Class 7D; class 2A/2B/2C or other types of fill (e.g. 6P, IBAA, PFA, LIGTH AGGREGATES) might be used for slopes and walls
	11.2. MINIMUM GEOTECHNICAL PROPERTIES	
	11.3. MINIMUM INTERACTION PROPERTIES	
	11.4. LIMITS ON BACKFILL	

	11.5. LIMITS ON FOUNDATION SOIL	
12. WATER	12.1. MODELS OF WATER PRESSURE	
	12.2. WATER INDUCED FAILURES	
	12.3. WATER IN SEISMIC CONDITIONS	
13. MISSING ELEMENTS	13.1 ITEMS NOT INCLUDED IN CODE	Accidental impact loads to parapets. Seismic analysis. Hydrolisis of Geosynthetics subject to water contact. GRS direct bridge abutments with concrete blocks. Piled embankment and vertical MSE Wall: lateral thrust with case of vertical face embankment not specified

TOPIC	SUB-TOPIC	GERMANY - EBGEO 2nd Edition
1. VERTICAL SPACING	1.1. CALCULATION APPROACH	No specific approach to calculate the spacing
	1.2. LIMITS	Chapter 7.2.2 Geometry: "...vertical distance ...is usually 0.3 to 0.6 m"
2. INTERNAL STABILITY DESIGN METHODS FOR MSE WALLS	2.1. AVAILABLE METHODS	Chapter 7.3 Analysis Principles: ULS limit state analyses from DIN 1054: STR and GEO - Force and moment equilibrium methods (e.g. block sliding and Bishop)
	2.2. DIFFERENTIATIONS	STR: design strength of reinforcement, analysis of connections/facing, analysis of reinforcement overlapping/joints (reinforcement junctions). GEO: pull-out resistance of reinforcement, failure on slip planes penetrating retaining structure
	2.3. LIMITS	The analyses are only relevant to retaining structures allowing a planar boundary at the end of the reinforcement element, such that a geomentrically defined rear wall occurs. They can be regarded as quasi-monoliths. Such a retaining structure can be modeled as a combination of several quasi-monolithic masses. The stability of the respective individual masses and the composite mass shall be investigated. For geosynthetic-reinforced retaining structures it is common to investigate the following failure mechanisms: - failure masses with circular slip planes; - failure bodies with logarithmic spirals as slip planes; - composite failure mechanisms with at least two failure masses and planar slip planes.
3. DESIGN METHODS FOR STEEP SLOPES	3.1. AVAILABLE METHODS	see 2.1
	3.2. DIFFERENTIATIONS	see 2.2
	3.3. LIMITS	see 2.3
4. EXTENSIBLE / INEXTENSIBLE REINFORCEMENT	4.1. DIFFERENTIATION LINES	EBGEO is relevant for products made of polymers (Chapter 2.2.2 Raw Materials), which are considered as "extensible reinforcements". Other raw material are not considered.
	4.2. REINFORCEMENT TYPE (STRIP, GRID) INFLUENCE	Woven, grid and round woven material.
	4.3. SOIL STIFFNESS INFLUENCE	
5. MISSING REINFORCEMENTS	5.1. GEOCELLS	not included
	5.2. GEOSTRIPS	not specifically mentioned
	5.3. OTHER (specify)	

6. CONNECTION CAPACITY	6.1. LOCAL STABILITY ANALYSES	The facing stability analyses; are based on the facing type determined to DIN EN 14475. The demands placed on the connections depend on the facing design. It is not generally possible to precisely determine the horizontal stress on the facing. It is necessary to adopt the active earth pressure for the full height of a geosynthetic-reinforced structure when analysing the geosynthetic connection to the facing elements. An examination of whether the deformations are acceptable for both the structure and the surrounding ground shall be performed. Vertical differential deformations between the facing elements and the reinforced earth structure shall be avoided. The horizontal movement of the facing shall be guaranteed for the entire height (including the toe).
	6.2. STATIC AND SEISMIC DIFFERENCES	n.a.
	6.3. SHORT AND LONG TERM PROPERTIES	n.a.
	6.4. REDUCTION FACTORS	Calibration factors for reducing the connected forces for various systems are given in Table 7.2 (Chapter 7.6 Facing Analysis).
7. SEISMIC DESIGN	7.1. DESIGN SEISMIC ACCELERATION FOR RS STRUCTURES	Quasi-static equivalent methods are adequate for dimensioning. LC 3 is generally analysed to DIN 1054 with increased earth pressure or surcharges as a function of the seismic zone. It is not necessary to adopt the composite friction coefficient, or fatigue and continious loads. DIN 4149, 12.2 for the basis for analysis. Maximum ground acceleration to DIN 4149:2005, Table 2. In special cases (liquefaction, resonance effects, anticipated deformations and/or water pressures require consideretion) detailed investigations and an analysis in the time/frequancy domain are necessary.
	7.2. RFcreep FOR GSY	The reduction factor A_5, which takes the influence of fatigue into consideration, is adopted and is determined in a pulsating load test. The creep rupture strength following cyclic effects is determined in tensile tests similar to those for the reference values.
8. MINIMUM CHECKS	8.1. INTERNAL STABILITY	In the ULS all possible failure mechanisms and slip planes intersecting reinforcement layers (previously: analysis of internal stability) are investigated. The resistance in intersected layers is the smaller of the two following resistances: - the design resistance of each reinforcement layer (reinforcement failure: STR); - the design value of the pull-out resistance of each reinforcement layer from the surrounding fill soil on both sides of the respective slip plane (pull-out: GEO)
	8.2. EXTERNAL STABILITY	In the ULS all possible failure mechanisms and slip planes not intersecting reinforcement layers (previously: analysis of external stability) are investigated.
	8.3. LOCAL STABILITY AT FACE	see above 6.1
	8.4. GLOBAL STABILITY	Global failure in terms of EBGEO is partial or complete slipping of a terrace stabilised by a retaining structure. Adequate safety against general/slope failure shall be demonstrated. This is done by demonstrating that the DIN 4084 limit state conditions are adhered to adopting the GEO limit state partial safety factors for the failure mechanisms involved (DIN 1054, 12.3 and DIN 4084) in the construction and final states.
	9.1. AMPLIFICATION FACTORS FOR LOADS	Partial factors of safety according to DIN 1054

9. AMPLIFICATION AND REDUCTION FACTORS	9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS	The structural resistance of a geosynthetic refers to its tensile strength. The long-term strength is calculated from the short-term strength by deviding by the reduction factors A1 to A5. The reduction factors take into consideration the impact of creep (A1), damage during transportation, installation and compaction (A2), the impact of junctions, seams and connections (A3), enviromental impact such as weathering, chemicals and microorganisms (A4), and the impact from predominantely dynamic actions (A5). The required product parameters and the reduction factors shall be provided by the manufacturer. Alternatively, the reduction factors from EBGEO shall be adopted.
	9.3. MODEL FACTORS / FACTORS OF SAFETY	Partial factors of safety according to DIN 1054
10. SERVICEABILITY LIMITS	10.1. FACE DEFORMATIONS	The magnitude of the allowable deformations is determined by the structur's use and the engineering design. The deformation behaviour of the composite structure consisting of soil and reinforcement is complex and can only be described approximately. … it was derived that the facing displacements of the retaining structures can be determined in the service load range from the changes in length of the individual reinforcement layers. The following analysis steps are necessary to estimate the horizontal facing displacements: - analysis of the tensile forces and their distribution in all reinforcement layers for the SLS; - determine the associated axial stiffness of the reinforcement layers; - determine the strain distributions for all reinforcement layers; - investigate the strain in all reinforcement layers to determine the change in length of each layer. The failure mechanism gouverning the equilibrium of each reinforcement layer is determined iteratively for analysis of the tensile forces in all reinforcement layers, taking SLS partial safety factors into consideration. The strains involved are then identified on this basis. The maximum value can be conservatively adopted as a constant for the entire length of the reinforcement. The changes in length are acquired by integrating the strains along the reinforcement layers. These correspond approximately for to the frontal displacement at the level of each layer.
	10.2. SETTLEMENTS AT BASE	The displacement in the base plane is determined to DIN 1054, 7.6.2. Ground settlement if determined to DIN 1054, 7.6.3 and DIN 4019. The reinforcement retaining structure may be adopted as a flexible load area. These settlements may also be a gouverning factor on soft ground. Special attention shall be paid to the evolution of the settlement with time (consolidation).
	10.3. SETTLEMENTS AT TOP	The vertical, surface displacements of a retaining structure result from the settlement of the ground, the intrinsic settlement of the fill material and the shear deformation of the reinforced earth structure.
11. SOILS	11.1. PERMITTED TYPES OF FILL	Following soil types classified to DIN 18196 may be used: coarse-grained soil types (sand, gravel); mixed grained soil types (silty sand, silty gravel); fine-grained soil types
	11.2. MINIMUM GEOTECHNICAL PROPERTIES	n.a.
	11.3. MINIMUM INTERACTION PROPERTIES	n.a.
	11.4. LIMITS ON BACKFILL	-
	11.5. LIMITS ON FOUNDATION SOIL	-
12. WATER	12.1. MODELS OF WATER PRESSURE	Water is considered in regard to its level; Excess pore water pressure in the subsoil has to be considered in the design
	12.2. WATER INDUCED FAILURES	
	12.3. WATER IN SEISMIC CONDITIONS	

13. MISSING ELEMENTS	13.1 ITEMS NOT INCLUDED IN CODE	

TOPIC	SUB-TOPIC	FRANCE - NF P 94-270
1. VERTICAL SPACING	1.1. CALCULATION APPROACH	No calculation - Depends of company's technology
	1.2. LIMITS	see NFP94270-1-1.2-JP-2018-08-09.docx
2. INTERNAL STABILITY DESIGN METHODS FOR MSE WALLS	2.1. AVAILABLE METHODS	So-called "Coherent Gravity Method"
	2.2. DIFFERENTIATIONS	Not applicable (only one method)
	2.3. LIMITS	Not applicable for inclined walls with a tilt >14° built with geosynthetic reinforcements in the form of sheets
3. DESIGN METHODS FOR STEEP SLOPES	3.1. AVAILABLE METHODS	So-called "Coherent Gravity Method"
	3.2. DIFFERENTIATIONS	Not applicable (only one method)
	3.3. LIMITS	Not applicable for inclined walls with a tilt >14° built with geosynthetic reinforcements in the form of sheets
4. EXTENSIBLE / INEXTENSIBLE REINFORCEMENT	4.1. DIFFERENTIATION LINES	No difference is made in the document
	4.2. REINFORCEMENT TYPE (STRIP, GRID) INFLUENCE	see NFP94270-4-4.2-JP-2018-08-09.docx
	4.3. SOIL STIFFNESS INFLUENCE	Not applicable
5. MISSING REINFORCEMENTS	5.1. GEOCELLS	Not included
	5.2. GEOSTRIPS	Included
	5.3. OTHER (specify)	-
6. CONNECTION CAPACITY	6.1. LOCAL STABILITY ANALYSES	Not detailed, applicable standards depending on technology used must be applied.
	6.2. STATIC AND SEISMIC DIFFERENCES	No difference is made in the document

6. CONNECTION CAPACITY	6.3. SHORT AND LONG TERM PROPERTIES	Must be evaluated the same way as reinforcements.
	6.4. REDUCTION FACTORS	No specific reduction factors, the same factors as for the reinforcement apply.
7. SEISMIC DESIGN	7.1. DESIGN SEISMIC ACCELERATION FOR RS STRUCTURES	Acceleration from EN1998-1 ($\gamma_1 . a_{gr} . S$) is divided by 2 except for external stability if the structure is particularly sensitive to displacements.
	7.2. RFcreep FOR GSY	RFcreep=1.00 in seismic case.
8. MINIMUM CHECKS	8.1. INTERNAL STABILITY	For each reinforcing layer: the structural resistance of the reinforcements; the soil-reinforcement interaction resistance; the strength of the facing connectors; the structural resistance of the facing.
	8.2. EXTERNAL STABILITY	the load-bearing capacity of the subsoil of the structure; the slip resistance of the structure at its base.
	8.3. LOCAL STABILITY AT FACE	see point 8.1
	8.4. GLOBAL STABILITY	must be verified both during construction and once the structure is completed, for failure surfaces crossing or not crossing the reinforcements.
9. AMPLIFICATION AND REDUCTION FACTORS	9.1. AMPLIFICATION FACTORS FOR LOADS	see NFP94270-9-9.1.9.2.9.3-JP-2018-08-09.docx
	9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS	see NFP94270-9-9.1.9.2.9.3-JP-2018-08-09.docx
	9.3. MODEL FACTORS / FACTORS OF SAFETY	see NFP94270-9-9.1.9.2.9.3-JP-2018-08-09.docx
10. SERVICEABILITY LIMITS	10.1. FACE DEFORMATIONS	Must be defined in the project specifications
	10.2. SETTLEMENTS AT BASE	Must be defined in the project specifications
	10.3. SETTLEMENTS AT TOP	Must be defined in the project specifications
	11.1. PERMITTED TYPES OF FILL	see NFP94270-11-11.1,11,2,11,3-GL-2018-08-22.docx
	11.2. MINIMUM GEOTECHNICAL PROPERTIES	see NFP94270-11-11.1,11,2,11,3-GL-2018-08-22.docx

11. SOILS	11.3. MINIMUM INTERACTION PROPERTIES	see NFP94270-11-11.1,11,2,11,3-GL-2018-08-22.docx
	11.4. LIMITS ON BACKFILL	Must be defined in the project specifications
	11.5. LIMITS ON FOUNDATION SOIL	Must be defined in the project specifications
12. WATER	12.1. MODELS OF WATER PRESSURE	Water pressure must be considered as permanent action, hydrodynamic actions are considered variable or accidental
	12.2. WATER INDUCED FAILURES	Not specified
	12.3. WATER IN SEISMIC CONDITIONS	Not included, refer to NF EN 1998-5 (and appropriate Annex)
13. MISSING ELEMENTS	13.1 ITEMS NOT INCLUDED IN CODE	Values for traffic loads (Eurocode loads are for bridges, not for fill structures).

TC218 Reinforced Soil Structures
Group 2 - Codes Interpretation

COMPARISON OF THE EXISTING CODES AND GUIDELINES

REFERENCE DOCUMENT: NF P 94-270

TOPIC: 1. VERTICAL SPACING

SUB-TOPIC: 1.2 LIMITS

ANSWER:

To enable the reinforced mass to retain its nature as a composite material and comply with the standard conditions of compound stability, the vertical spacing of the reinforcing layers, s_v, should not be excessive relative to their length, especially in the lower part of the mass (Figure 1).
Notes:

- The spacing s_v is generally of the order of 0.20 m to 0.80 m.
- For guidance, Table 1 gives, as a function of the ratio $L_{inf} \leq h_m$, the maximum relative spacings s_v/h_m the adoption of which is recommended for conventional structures consisting of class 1 or 2 fill.

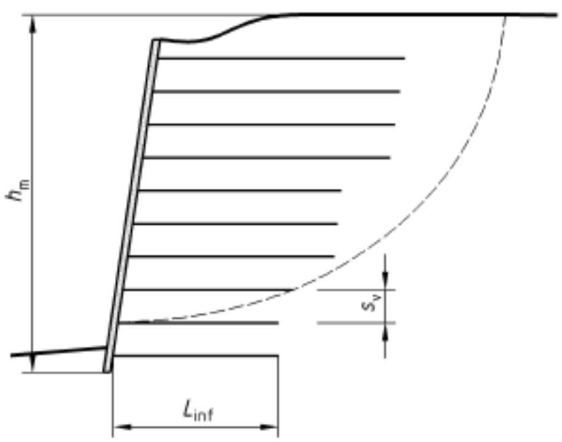


Figure 1. Length and spacing of lower layers that could be decisive for compound stability.

Table 1. "Conventional" structures consisting of class 1 or 2 fill - Recommended maximum vertical spacing of reinforcements.

Relative length of reinforcements L_{inf}/h_m	Maximum relative vertical spacing s_v/h_m
$L_{inf}/h_m \leq 0.55$	$\leq 1/8$
$0.55 < L_{inf}/h_m \leq 0.65$	$\leq 1/6$
$0.65 < L_{inf}/h_m \leq 0.75$	$\leq 1/4,5$
$0.75 < L_{inf}/h_m$	-

REFERENCES

NF P 94-270 A.1.1(4), AFNOR, France

COMPARISON OF THE EXISTING CODES AND GUIDELINES

REFERENCE DOCUMENT: NF P 94-270

TOPIC: 4. EXTENSIBLE / INEXTENSIBLE REINFORCEMENT

SUB-TOPIC: 4.2. REINFORCEMENT TYPE (STRIP, GRID) INFLUENCE

ANSWER:

The reinforcement type has obviously an influence on:
- The soil/reinforcement friction coefficient
- The calculation of the contact surface between the soil and the reinforcement
- The calculation of the reinforcement strength and durability
- The allowable grain size and chemical properties of the backfill material

In addition, the type of reinforcement has an influence on the value of the K coefficient which links the vertical stress and the horizontal stress by:

$$\sigma_h = K \cdot \sigma_v \qquad (1)$$

The coefficient K depends on the depth z of the reinforcing layer in question:
- if $z \leq z_0$:

$$K_{(z)} = \Omega_1 \cdot K_a \left[1.6\left(1 - \frac{z}{z_0}\right) + \frac{z}{z_0}\right] \qquad (2)$$

- if $z > z_0$:

$$K_{(z)} = \Omega_1 \cdot K_a \qquad (3)$$

where:
- z_0 is a depth taken as equal to 6 m
- K_a is the active earth pressure coefficient of the mass fill, given by $K_a = tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$
- Ω_1 is a coefficient (≥ 1.0) related to the type of reinforcement

The coefficient Ω_1 covers the risk of localized extra tension which may be caused by the largest fill elements entering the meshes of the reinforcements in the form of welded mesh, ladders or grids. If the fill material could contain elements larger than $s_x/2$ or $s_y/2$ (see figure 1), Ω_1=1.25 should be adopted in the calculations concerning verification of the structural resistance of reinforcements. Otherwise, especially for strip or sheet reinforcements, Ω_1=1.00.
Note: The coefficient Ω_1=1.25 does not apply to verification of interaction resistance.

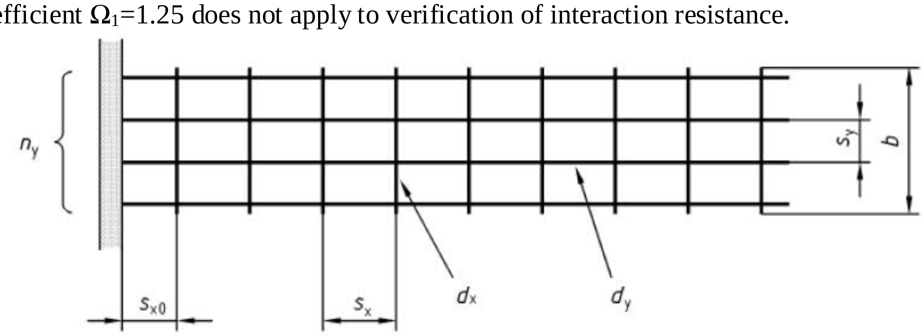


Figure 1. Top view of a welded mesh reinforcement.

REFERENCES

NF P 94-270 E.2.3.3, AFNOR, France

COMPARISON OF THE EXISTING CODES AND GUIDELINES

REFERENCE DOCUMENT: NF P 94-270

TOPIC: 9. AMPLIFICATION AND REDUCTION FACTORS

SUB-TOPIC: 9.1. AMPLIFICATION FACTORS FOR LOADS
9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS
9.3. MODEL FACTORS / FACTORS OF SAFETY

ANSWER:

The combination of sets of partial factors to be considered for verifying an ultimate limit state of a reinforced soil structure is determined by the design approach that is associated with that ultimate limit state. Only design approaches 2 and 3 are permitted by National Annex NF EN 1997-1/NA to Eurocode 7.

Design approach 2 is used for verifying the external stability (GEO) and internal stability (STR) limit states. In this approach, the partial factors are applied not only to actions and the effects of actions, but also to the resistance parameters of the ground and possibly the structure, and the combination of sets of partial factors to be applied is: A1 “+” M1 “+” R2

Design approach 3 is used for verifying the general stability (GEO) and compound stability (GEO and STR) limit states. In this approach, the partial factors are applied not only to actions and the effects of actions, but also to the resistance parameters of the ground and possibly the structure, and the combination of sets of partial factors to be applied is: A2 “+” M2 “+” R3

The values of the partial factors are detailed in the following tables.

Table 1. Partial factors for actions (γ_F) or effects of actions (γ_E).

Action		Symbol	Set	
			A1	A2
Permanent	Unfavorable	γ_{Gsup}	1.35	1.0
	Favorable	γ_{Ginf}	1.0	1.0
Variable	Unfavorable	γ_{Qsup}	1.5	1.3
	Favorable	γ_{Qinf}	0	0

Table 2. Partial factors for ground parameters (γ_M).

Ground parameters	Symbol	Set	
		M1	M2
Internal friction angle*	$\gamma_{\varphi'}$	1.0	1.25**
Effective cohesion	$\gamma_{c'}$	1.0	1.25**
Undrained cohesion	γ_{cu}	1.0	1.4**
Unit weight	γ_γ	1.0	1.0

* This factor is applied to tan φ'

** For reinforced fill structures made up of fill material with specified or known properties, in the conditions set out in 6.3.1(4) of NF P 94-270, the partial factors γ_M applicable to the shear strength of the material may be multiplied by an adjustment factor λ equal to 0.8.

Table 3. Material partial factors (γ_M) for the reinforcements.

Reinforcement type	Property	Symbol	Set	
			M1	M2
Steel	Yield strength f_y	γ_{M0}	1.0	1.0

	Tensile failure f_u	γ_{M2}	1.25	1.25
Geosynthetic	Characteristic tensile strength	$\gamma_{M;t}$	1.25	1.25

Table 4. Partial factors for soil-reinforcing layer interaction resistance (γ_M).

Resistance	Symbol	Set	
		M1	M2
Soil-reinforcement interaction τ_{max}*	$\gamma_{M;f}$	1.35	1.1

* For values derived from a documented database

For the structural verification (STR) of the facing of a reinforced soil structure, the provisions of the appropriate design standard for the constituent material of the facing apply. For example, for a reinforced concrete facing, the provisions of NF EN 1992-1-1 apply with the recommended partial factors.

Table 5. Resistance partial factors (γ_R) for the verification of the external stability.

Resistance	Symbol	Set
		R2
Bearing capacity	$\gamma_{R;v}$	1.4
Slip resistance	$\gamma_{R;h}$	1.1

Table 6. Resistance partial factor (γ_R) for the verification of the compound stability and general stability.

Resistance	Symbol	Set
		R3
Total shear strength on a failure surface	$\gamma_{R;e}$	1.0
Mobilization of soil shear strength model partial factor	$\gamma_{R;d}$	1.1*

* A value greater than 1.10 shall be applied when the intended use of the structure makes it highly sensitive to such deformations, without prejudging the serviceability limit state justifications required elsewhere. For example, $\gamma_{R;d}$ = 1.20 shall be applied when the structure is located in the immediate vicinity of a sensitive structure.

REFERENCES

NF P 94-270 Annex C, 10.2, 10.5, 12.5 and 12.6(3) , AFNOR, France

COMPARISON OF THE EXISTING CODES AND GUIDELINES

REFERENCE DOCUMENT: NF P 94-270

TOPIC: 11. SOILS

SUB-TOPIC: 11.1. PERMITTED TYPES OF FILL
11.2. MINIMUM GEOTECHNICAL PROPERTIES
11.3. MINIMUM INTERACTION PROPERTIES

ANSWER:

The properties of the material in the reinforced zone shall be specified prior to project beginning, identifying the following scenario:

- the material is sourced from specific location (material available on site or in the vicinity)
- the material source is not defined

In any case, the nature and type of reinforced material, or a range of allowable materials, to be used shall be specified considering the recommendations included in NF EN 14475, and according to NF P 11-300.

When the source of the reinforced material is imposed, it is advisable to carry out a comprehensive geotechnical investigation of the site to characterize the fill properties and their impact on project conditions.

When the source of the reinforced material is not defined, a preliminary geotechnical report is not required but it is recommended to:

- before the beginning of studies: define the fill properties according to the project conditions;
- before the beginning of earthworks: source filling material according to the specified criteria, based on contract documents.

The geotechnical properties specific to filling material shall include:

- the particle size distribution (class of material as defined in standard NF EN 14475, Appendix A; coefficient of uniformity), angularity;
- the unit weight and shear parameters of the backfill material (friction angle and cohesion);
- water content, optimum density, soil compaction requirements.

When the source of the filling material is specified, the assumptions adopted for the design concerning unit weight, friction angle, cohesion and coefficient of uniformity of the material must rely on geotechnical investigation from the borrow area.

When the source of the filling material is not defined, and the specified material is class 1 or class 2 - granular soil (according to standard NF EN 14475, Appendix A), it is possible to estimate the unit weight and friction angle, based on recognized correlations with its granulometry, for the foreseen project conditions (refer to NOTE 1 and NOTE 2).

NOTE 1 – Conservative parameters shall be used. Suggested values are reported in Table 1 and Table 2. It is recommended to use a cohesion value equal to zero.
NOTE 2 – When the nature and type of filling material is known, documented parameters could be used.

Table 1. Typical unit weights for filling materials class 1 or 2, according to NF EN 14475, properly installed.

Natural weight [kN/m^3]	Saturated weight [kN/m^3]
18 - 20	20 - 22

Table 2. Typical friction angle for filling materials class 1 or 2, according to NF EN 14475, properly installed.

Class	Class 1	Class 2
Filling material	Free-draining material	Granular material
Dry condition	36°	36°
Partially under water condition	36°	30°

When the source of the filling material is not imposed, and it is possible to use intermediate class 3 soil or a fine class 4 soil (according to standard NF EN 14475, Annex A), material properties assumptions shall be based on verifiable data. NF EN 14475, Annex A is reported in Table 3.

Table 3. Typical combinations of fills, reinforcements and facings

FILL TYPE			**Type 1**	**Type 2**		**Type 3**		**Type 4**
			Draining	Granular		Intermediate		Fine
	Geomechanical characteristics	% passing the 80micron sieve	<5%	<12%	12 to 35%	12 to 35%	>35%	Others
		% passing the 20micron sieve	n.a.	n.a.	<10%	>10%	<40%	
		Plasticity Index	n.a.	n.a.	n.a.	<25	<25	
APPLICATION								
	Parts of structure exposed to flooding and/or rapid water draw-down		A	B	B	D	D	D
	Structure supporting bridge abutments, railways, buildings		A	A	B	C (a)	D	D
	High reinforced fill walls		A	A	B	B	D	D
	High reinforced fill slopes		A	A	B	B	C (b)	C (b)
	Common walls and slopes		A	A	A	B	C (c)	C (c)
REINFORCEMENT								
	Smooth strips or rods (metallic or polymeric)		A	A		C (d)		D
	Ribbed strips or rods, ladders (metallic or polymeric)		A	A		B	C (d)	D
	Bar mats, ladders, meshes, grids, sheets (metallic or polymeric)		A	A		B	C (d)	D
	Draining geosynthetics (in-plane permeability)		B	A		A		C (b)
FACING								
	Rigid		A	A		D(a)		D
	Semi flexible		A	A		C(e)		D
	Flexible		A	A		A	B	C(e)

KEY :

A = Often Used

B = Sometimes Used

C = Subject to Specific Study

D = Not Recommended

NOTES

General The typical combinations above are given for general guidance only and are not intended to be a specification of where various fills or components may be used. The brief descriptions of the fills above are only some of the principle characteristics and do not fully describe a fill. The design documents or a project should specify the particular fills and components which should be used. Fine fill which is too wet of optimum is difficult to compact and likely to cause facings, if used, to go out of alignment during compaction. Fine fill laid and compacted in adverse weather conditions may be problematic. Frost susceptibility should be checked if applied in cold climates.

Specific

a If adequate compaction is not achieved then differential settlements between facing and reinforcements may occur which may overload the connection.

b The effect of the drainage properties on the fill characteristics should be assessed.

c Special attention should be paid to : angle of internal friction, compaction procedure with respect to moisture content and climatic conditions, need for drainage layers.

d The fill-reinforcement interaction should be assessed for long term and during construction conditions

e Special attention should be paid to the control of the alignment of the facing units (if any) during construction.

The properties of the retained material (backfill material), adjacent to the reinforced zone, shall be specified before designing.

NF P 94-270 chapter 6.3.2 (for reinforced soils) could also be applicable, following the principle of the material source.

The interaction between the fill and the reinforcement shall be considered to assess compatibility with the design assumptions. The values shall be defined prior to the beginning of the project.

Unless described in specific bibliography, the soil-reinforcement interaction coefficient shall be evaluated before designing, based upon the project specific conditions.

As per EN 14475, assessment of the fill reinforcement interaction should be based on laboratory testing such as shear box or pull-out testing, and/or previous relevant experience where available. In situ pull-out testing might also be considered.

Electrochemical, chemical and biological properties to define the aggressiveness of the backfill material with respect to reinforcement and facing must be determined, prior to the beginning of the project, according to NF P 94-270, Annex F, and EN 14475 Annex B (see Table 4 for metallic reinforcements only) for the specific reinforcing material.

Table 4. Electro-chemical properties of fills used with metallic reinforcement

STEEL REINFORCEMENTS						**Strips,**					**Welded meshes, ladders, rods**		**Woven wire meshes**	
Criteria based on corrositivity					Notes	Non-coated "black" steel	Continuously hot-dip galvanised (35 µm)	Hot dip galvanised (70 µm)	Zinc/Aluminium coated (Zn85Al15, thermal spray coated 70 µm)	Non coated black steel	Hot dip galvanised (70µm)	Zinc/Aluminium coated (Zn95Al5, hot dip coated 35µm)	Zinc/Aluminium coated (Zn85Al15, hot dip coated 35µm) + polymer coated (PVCU or PE, 0.5 mm)
Commonly used sizes						3 to 6 mm	3 mm thick	4 to 6 mm thick		bars Ø8 mm to 12 mm		Wire ϕ 2 mm to 3 mm	
Usual field of application – Class of structure (related to design life)					(1)	Class 3 or 4	Class 4	Class 4 or 5	Class 4 or 5	Class 4	Class 4 or 5	Class 1	Class 4 for steep slopes up to 70°
Electro-chemical characteristics compatible with routine design					(2)								
ENVIRONMENT	Land based, out of water	pH			(3)	5 to 10	5 to 10	5 to 10	A (9)	5 to 10	5 to 10	5 to 10	3 to 10
		Resistivity	Ω cm		(4)	> 1 000	> 1 000	> 1 000		> 1 000	> 1 000	> 1 000	B (7)
		Chlorides Cl	ppm		(5)	< 200	< 200	< 200		< 200	< 200	< 200	
		Sulfates SO_4	ppm		(6)	< 1 000	< 1 000	< 1 000		< 1 000	< 1 000	< 1 000	
	In fresh water (8)	pH			(3)	5 to 10	5 to 10	5 to 10	A (9)	5 to 10	5 to 10	5 to 10	3 to 10
		Resistivity	Ω cm		(4)	> 3 000	> 3 000	> 3 000		> 3 000	> 3 000	> 3 000	B (7)
		Chlorides Cl	ppm		(5)	< 100	< 100	< 100		< 100	< 100	< 100	
		Sulfates SO_4	ppm		(6)	< 500	< 500	< 500		< 500	< 500	< 500	
Non-routine, unusual design													
ENVIRONMENT	Marine environment, or fill of marine origin					Specific study required. Thicker strips or larger bars generally needed			pH 5 to 10 No other requirement	Specific study required. Larger bars generally needed		C	Specific study required
	Industrial waste fills, & environments of high aggressivity					Specific study required				Specific study required		Specific study required	

Key : A — Material not normally used; B — Test not relevant; C — Material not normally applicable

The reduced reinforcement capacity due to electrochemical, chemical and biological properties of the soil affects the reinforcement tensile strength.

REFERENCES

NF P 94-270 Chapter 6.1, 6.3.1, 6.3.2, 6.4, Annex F, AFNOR, France
EN 14475, Annex A

TOPIC	SUB-TOPIC	ITALY - AGI GUIDELINES
1. VERTICAL SPACING	1.1. CALCULATION APPROACH	Contributory area approach based on Coherent Gravity, Tie Back Wedge, 2-part wedge, numerical methods: the Designer has to justify the selected method
	1.2. LIMITS	No limits; the Designer has to demonstrate that with the selected spacing no ULS nor SLS is exceeded
2. INTERNAL STABILITY DESIGN METHODS FOR MSE WALLS	2.1. AVAILABLE METHODS	Coherent Gravity, Tie Back Wedge, 2-part wedge, numerical methods are possible: the Designer has to justify the selected method. Both static and seismic analyses shall be performed, according to EuroCode 7 Approach 2: A1 + M1 + R3, that is with amplification factors for loads and no reduction factors for geotechnical parameters. For static and seismic analyses R3 = 1.0. For seismic analyses all amplification factors A1 = 1.0
	2.2. DIFFERENTIATIONS	According to the selected method, which has to be justified
	2.3. LIMITS	No limits; the Designer has to demonstrate that the selected method is appropriate for the project and that with the selected method no ULS nor SLS is exceeded
3. DESIGN METHODS FOR STEEP SLOPES	3.1. AVAILABLE METHODS	Slice methods (Bishop, Morgenstern - Price, etc.), 2-part wedge, 3-part wedge, numerical methods are possible: the Designer has to justify the selected method
	3.2. DIFFERENTIATIONS	No difference between steep slopes and walls, both intended as retaining structures
	3.3. LIMITS	No limits; the Designer has to demonstrate that the selected method is appropriate for the project
4. EXTENSIBLE / INEXTENSIBLE REINFORCEMENT	4.1. DIFFERENTIATION LINES	• Inextensible reinforcements reach their peak strength at strains lower than the strain required for the soil to reach its peak strength. • Extensible reinforcements reach their peak strength at strains greater than the strain required for soil to reach its peak strength. • The Designer has to justify the selection of extensible or inextensible reinforcement
	4.2. REINFORCEMENT TYPE (STRIP, GRID) INFLUENCE	For geogrids, grids and ladders interlocking is considered; for geotextiles and geostrips interlocking is not considered. In any case the influence of friction and interlocking is assumed to be included in f_{ds} and f_{po} coefficients, obtained from direct shear and pullout tests
	4.3. SOIL STIFFNESS INFLUENCE	Considered only for defining if reinforcement is extensible or inextensible, but no set criteria

5. MISSING REINFORCEMENTS	5.1. GEOCELLS	Not included
	5.2. GEOSTRIPS	Included
	5.3. OTHER (specify)	-
6. CONNECTION CAPACITY	6.1. LOCAL STABILITY ANALYSES	Local stability at the face shall be addressed. The Designer has to justify the selected methods of analysis, considering all the components of the reinforced soil system
	6.2. STATIC AND SEISMIC DIFFERENCES	Both static and seismic analyses shall be performed, according to Approach 2: A1 + M1 + R3, that is with amplification factors for loads and no reduction factors for geotechnical parameters. For static and seismic analyses R3 = 1.0. For seismic analyses all amplification factors A1 = 1.0. Specific tests can be added for the connection strength in seismic conditions.
	6.3. SHORT AND LONG TERM PROPERTIES	Must be evaluated the same way as reinforcements. Specific tests can be added
	6.4. REDUCTION FACTORS	No specific reduction factors, the same factors as for the reinforcement apply.
7. SEISMIC DESIGN	7.1. DESIGN SEISMIC ACCELERATION FOR RS STRUCTURES	Both horizontal and vertical seismic acceleration shall be considered; for pseudo-static analyses the seismic coefficients are: $K_h = (\beta_m * a_{max}/g)$; $K_v = \pm 0.5 * K_h$ $a_{max} = S_S * S_T * a_g$ a_g is the maximum acceleration at bedrock, which is defined by national seismic maps for each location; S_S is the startigraphic factor; S_T is the topographic factor; β_m is the dumping factor.
	7.2. RFcreep FOR GSY	RF_{creep} = 1.00 in seismic conditions.
8. MINIMUM CHECKS	8.1. INTERNAL STABILITY	ULS Tensile Resistance of reinforcement Pullout Resistance of reinforcement Direct sliding along reinforcement Structural resistance of face element connections
	8.2. EXTERNAL STABILITY	ULS Sliding on the base Overturning Bearing capacity SLS Vertical wall movements at top, berms, and bottom Horizontal wall movements including bulging

	8.3. LOCAL STABILITY AT FACE	The connection of each reinforcement with the facing has to be designed for the design strength T_D of the connected reinforcement
	8.4. GLOBAL STABILITY	No difference between global and compound stability. Both static and seismic analyses shall be performed, according to combination: A2 + M2 + R2, that is no amplification factors for permanent loads and with reduction factors for geotechnical parameters. For both static and seismic analyses R2 = 1.2.
9. AMPLIFICATION AND REDUCTION FACTORS	9.1. AMPLIFICATION FACTORS FOR LOADS	see AGI-9-9.1.9.2.9.3-PR-2019-02-12.docx
	9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS	see AGI-9-9.1.9.2.9.3-PR-2019-02-12.docx
	9.3. MODEL FACTORS / FACTORS OF SAFETY	see AGI-9-9.1.9.2.9.3-PR-2019-02-12.docx. Note: the Model Factor / Factor of Safety for pullout analyses is under definition
10. SERVICEABILITY LIMITS	10.1. FACE DEFORMATIONS	Deformations, displacements and settlements shall be evaluated in serviceability conditions and the compatibility with the safety and functionality of the structure and of all surrounding constructions shall be checked
	10.2. SETTLEMENTS AT BASE	See 10.1
	10.3. SETTLEMENTS AT TOP	See 10.1
11. SOILS	11.1. PERMITTED TYPES OF FILL	All types of fill are allowed. The assumption of c' and c_u for cohesive fills shall be justified. If the fill is not self draining, drainage systems shall be included in design
	11.2. MINIMUM GEOTECHNICAL PROPERTIES	No limits. The Designer has to demonstrate that with the selected geotechnical properties no ULS or SLS is exceeded.
	11.3. MINIMUM INTERACTION PROPERTIES	No limits. The Designer has to justify the selected interaction properties and has to demonstrate that with the selected interaction properties no ULS or SLS is exceeded.
	11.4. LIMITS ON BACKFILL	No limits
	11.5. LIMITS ON FOUNDATION SOIL	No limits. The Designer has to demonstrate that with the selected geotechnical properties no ULS or SLS is exceeded.
12. WATER	12.1. MODELS OF WATER PRESSURE	Water pressure shall be considered as permanent action, hydrodynamic actions are considered as variable or accidental
	12.2. WATER INDUCED FAILURES	All potential water induced failures shall be addressed

	12.3. WATER IN SEISMIC CONDITIONS	Water in seismic conditions shall be considered. The Designer has to justify if inertia ia applied to standing water or not
13. MISSING ELEMENTS	13.1 ITEMS NOT INCLUDED IN CODE	Compund stability is included in global stability and is not considered seaparately.

FACTORS ACCORDING TO THE ITALIAN CODE FOR CONSTRUCTIONS NTC 2018

9.1. AMPLIFICATION FACTORS FOR LOADS

- Actions can be permanent (persistent), permanent non structural, or variable (transient.
- Persistent actions are denoted by F_G. Transient actions are denoted by F_Q.
- Actions can be either "favourable" or "unfavourable".
- Load Coefficients (A) shall be applied to actions in each Ultimate Limit State (ULS) analysis.
- Partial coefficients (A) are denoted by γ_{G1}, γ_{G2}, γ_{Qi}, for permanent, permanent non structural, and variable loads, respectively.
- Water and soil are permanent loads (structural) when, in the used model, contribute to the behavior of the structure with their characteristics of weight, strength, and stiffness.

The combinations of factors to be applied to the actions (A) in the ultimate limit state analyses, according to Italian Norm NTC 2018, are reported in Table 1.

Table 1. Combinations of factors to be applied to the actions (A) in the ultimate limit state analyses, according to Italian Norm NTC 2018

TYPE OF LOAD	EFFECT	PARTIAL COEFFICIENT	A1	A2
PERMANENT	FAVOURABLE	γ_{G1f}	1,00	1,00
	UNFAVOURABLE	γ_{G1u}	1,30	1,00
PERMANENT NON STRUCTURAL	FAVOURABLE	γ_{G2f}	0,80	0,80
	UNFAVOURABLE	γ_{G2u}	1,50	1,30
VARIABLE	FAVOURABLE	γ_{Qif}	0,00	0,00
	UNFAVOURABLE	γ_{Qiu}	1,50	1,30

9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS

- Material properties are given the general symbol, X.
- Characteristic values of material properties are given the general symbol, X_k.
- The selection of characteristic values for geotechnical parameters shall be based on results and derived values from laboratory and field tests.
- Geotecnical coefficients (M) shall be applied to actions in each Ultimate Limit State (ULS) analysis.
- Partial coefficients (M) are denoted by $\gamma_{\phi'}$, $\gamma_{c'}$, γ_{cu}, γ_{γ}, for tangent of friction angle, drained cohesion, undrained cohesion, and unit weight, respectively.

The combinations of factors to be applied to the soil parameters (M) in the ultimate limit state analyses, according to Italian Norm, are reported in Table 2.

Table 2. Combinations of factors to be applied to the soil parameters (M) in the ultimate limit state analyses, according to Italian Norm NTC 2018

PARAMETER	APPLIED TO	PARTIAL COEFFICIENT γ_M	M1	M2
TANGENT OF FRICTION ANGLE	$\tan \varphi'_k$	$\gamma_{\varphi'}$	1,00	1,25
DRAINED COEHESION	c'_k	$\gamma_{c'}$	1,00	1,25
UNDRAINED COHESION	c_{uk}	γ_{cu}	1,00	1,40
UNIT WEIGHT	γ	γ_γ	1,00	1,00

9.3. MODEL FACTORS / FACTORS OF SAFETY

The ratio between the design resistance (R_d) and the design effect of the actions (E_d) shall be larger than the coefficient γ_R :
$R_d \; / \; E_d \; \geq \; \gamma_R$
External and internal stability analyses shall be carried out using Approach 2, with the combination A1 + M1 + R3.
The combinations of coefficients to be used for ultimate limit states verification purposes (R3) of retaining structures, according to Italian Norm, are reported in Table 3.
Moreover, global stability analyses of works made up of loose material (including veneer covers) shall be carried out with Approach 1, Combination 2 (A2 + M2 + R2), and the factor (R2) reported in Table 4 shall be applied.

Table 3. Combinations of coefficients to be used for ultimate limit states verification purposes (R3) of retaining structures, according to Italian Norm NTC 2018

ANALYSIS	PARTIAL COEFFICIENT γ_R (R3)
LIMIT BEARING CAPACITY	1.40
DIRECT SLIDING	1.10
OVERTURNING	1.15
RESISTANCE OF DOWNSLOPE SOIL	1.40

Table 4. Factor R2 for global stability analyses of works made up of loose material, according to Italian Norm NTC 2018

COEFFICIENT	R2
γ_R	1.10

TOPIC	SUB-TOPIC	CANADA - CAN/CSA-S6-14
1. VERTICAL SPACING	1.1. CALCULATION APPROACH	AASHTO Simplifed Method, Coherent Gravity method
	1.2. LIMITS	The maximum vertical spacing between reinforcement layers shall be limited to twice the depth of the modular block facing unit,or 800 mm, whichever is less. If the height of the modular block facing unit exceeds 800 mm, the maximum vertical spacing between reinforcement layers shall be 1 m.
2. INTERNAL STABILITY DESIGN METHODS FOR MSE WALLS	2.1. AVAILABLE METHODS	LRFD framework with AASHTO Simplifed Method (SM), Coherent Gravity method CGM)
	2.2. DIFFERENTIATIONS	SM for polymeric and steel reinfrocment. CGM for steel strips and steel grids
	2.3. LIMITS	Do not apply to geometrically complex systems such as tiered walls (walls stacked on top of one another), back-to-back walls, shored walls, or walls which have trapezoidal sections and MSE walls constructed with polyester strips.The maximum friction angle used for the computation of the horizontal force within the reinforced soil mass shall be assumed to be 35°, unless the project specific backfill is tested for frictional strength by accepted triaxial or direct shear testing methods. A design friction angle of greater than 40° shall not be used with the simplified method even if the measured friction angle is greater than 40°.
3. DESIGN METHODS FOR STEEP SLOPES	3.1. AVAILABLE METHODS	Use design guidelines for reinforced slopes provided in FHWA-NHI-10-024
	3.2. DIFFERENTIATIONS	MSE walls onstructed at an angle 70° or less from the horizontal are considered to be reinforced slopes
	3.3. LIMITS	See above
4. EXTENSIBLE / INEXTENSIBLE REINFORCEMENT	4.1. DIFFERENTIATION LINES	Steel (inextensible) and geosynthetics (extensible)
	4.2. REINFORCEMENT TYPE (STRIP, GRID) INFLUENCE	Metal strips, metal bar mats, metal wire mesh and polymeric geogrids.
	4.3. SOIL STIFFNESS INFLUENCE	Not considered
5. MISSING REINFORCEMENTS	5.1. GEOCELLS	Mentioned only as a potential facing treatment
	5.2. GEOSTRIPS	Excluded (prefer term "polymeric straps")
	5.3. OTHER (specify)	NA
6. CONNECTION CAPACITY	6.1. LOCAL STABILITY ANALYSES	Yes. The tensile load applied to the soil reinforcement connection at the wall face, shall be equal to the maximum reinforcement tensile load, for all wall systems regardless of facing and reinforcement type.
	6.2. STATIC AND SEISMIC DIFFERENCES	Yes

	6.3. SHORT AND LONG TERM PROPERTIES	Yes
	6.4. REDUCTION FACTORS	See above
7. SEISMIC DESIGN	7.1. DESIGN SEISMIC ACCELERATION FOR RS STRUCTURES	Yes
	7.2. RFcreep FOR GSY	Yes
8. MINIMUM CHECKS	8.1. INTERNAL STABILITY	Yes
	8.2. EXTERNAL STABILITY	Yes. Traditional requirements for reinforcement length not less than 70% of the wall height
	8.3. LOCAL STABILITY AT FACE	Yes
	8.4. GLOBAL STABILITY	Yes
9. AMPLIFICATION AND REDUCTION FACTORS	9.1. AMPLIFICATION FACTORS FOR LOADS	LRFD load factors
	9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS	LRFD resistance factors with different values for different limit states and for different project "levels of understanding"
	9.3. MODEL FACTORS / FACTORS OF SAFETY	Not used
10. SERVICEABILITY LIMITS	10.1. FACE DEFORMATIONS	Must be considered but no quantitative guidance
	10.2. SETTLEMENTS AT BASE	Must be considered but no quantitative guidance
	10.3. SETTLEMENTS AT TOP	Must be considered but no quantitative guidance
11. SOILS	11.1. PERMITTED TYPES OF FILL	
	11.2. MINIMUM GEOTECHNICAL PROPERTIES	
	11.3. MINIMUM INTERACTION PROPERTIES	
	11.4. LIMITS ON BACKFILL	
	11.5. LIMITS ON FOUNDATION SOIL	
12. WATER	12.1. MODELS OF WATER PRESSURE	
	12.2. WATER INDUCED FAILURES	
	12.3. WATER IN SEISMIC CONDITIONS	
13. MISSING ELEMENTS	13.1 ITEMS NOT INCLUDED IN CODE	

TOPIC	SUB-TOPIC	HONG KONG - GeoGuide 6 2017
1. VERTICAL SPACING	1.1. CALCULATION APPROACH	7.8. For metallic reinforcement consisting of strips, grids and anchors, the vertical spacing (Sv) is maintained constant and the reinforcement density is increased with depth by reducing the horizontal spacing (Sh) or changing the reinfor cement size, strength or grade. For polymeric grid or geosynthetic sheet reinforcement the reinforcing density can be increased by reducing the vertical spacing (Sv) with depth. Alternatively the reinforcement density can be varied by changing the desi gn strength of the reinforcement. This is particularly useful with a wrap around form of construction where a constant wrap height is desirable. For tall structures reinforced with polymeric grids or geo synthetic sheets, double layers of reinforcement c an be provided. For segmental block walls the low height of the blocks can make it impractical/uneconomic to place reinforcement at each level. The maximum spacing should not exceed the maximum stable unreinforced height and the normal rule of thumb is the spacing of reinforcement should not exceed two times the block depth (i.e. front face to back face). For the common elemental facing systems comprising 1.5 m high units, at least two rows of einforcement should be attached to each facing element.
	1.2. LIMITS	3.3.4. The vertical spacing (Sv) of reinforcement is usually controlled by construction practice with reinforcement being located to coincide with fill lifts. Spacings of 250 to 800 mm are commo n with vertical faced structures, the closer spacing being at the base of structures. At these spacings the reinforcement is fully effective. The density of reinforcement in reinforced fill slopes may not be high and wider spacing is possible. Field studies have shown that the vertical spacing (Sv) of principal reinforcement should not exceed 1 to 1.5 m. Intermediate reinforcement is sometimes used to prevent local ravelling and deterioration of the slope face. With fine grained fill, vertical reinforcement spacings in the order of 300 to 500 mm are commonly used.
2. INTERNAL STABILITY DESIGN METHODS FOR MSE WALLS	2.1. AVAILABLE METHODS	7.5.4. Local Stability Check. The resistance of the i-th level reinforcement should be checked against rupture and pullout failure whilst carrying the design tension. 7.5.5. Wedge Stability Check. After checking rupture and pullout of individual layers of reinforcement, limit equilibrium analysis should be undertaken to check the potential wedge failures within the reinforced block. A selection of potential wedge failures should be investigated. For each of the typical points the maximum value of the total tensile force, T to be resisted by the reinforcement should be established by analysing the forces acting on a number of different wedges. Stability of any wedge inside the reinforced block is maintained when shear resistance acting on the potential failure planes in conjunction with the tensile/pullout resistance of the group of reinforcing elements embedded in the fill beyond the plane is able to resist the destabilising loads. Wedges are assumed to behave as rigid bodies and may be of any size and shape. The resistance provided by an individual layer of reinforcement should be taken as the lesser of either: the pullout resistance of that layer of reinforcement embedded in the fill beyond the potential failure plane; the design tensile resistance of that layer of reinforcement.
	2.2. DIFFERENTIATIONS	7.5.2. Design methods. Where the short term (i.e. immediately after construction) axial tensile strain of the reinforcement exceeds 1 % under the design loads, the analytical method recommended is the Tieback Method. Where the short term axial tensile strain of the reinforcement is less than or equal to 1 % under the design load, the analytical method recommended is an empirical method of analysis termed the Coherent Gravity Method.
	2.3. LIMITS	In normal situation the short-term axial tensile strain of polymeric reinforcement will most likely exceed 1 % and this is sufficient to generate the active Ka stress-state. However, if the design employs a large quantity of relatively stiff polymeric reinforcement to limit deformation, a relatively stiff structure could be developed and the Coherent Gravity Method may be the more appropriate design method.

3. DESIGN METHODS FOR STEEP SLOPES	3.1. AVAILABLE METHODS	8.2, 8.3, 8.4. The assessment of external stability for reinforced fill slopes is based on the procedures adopted for reinforced fill structures given in Section 7. 4. In checking the overall stability of the slope, it is necessary to evaluate potential deep seated failures in the ground mass containing the reinforced block and to provide adequate margin of safety against this mode of failure. The ultimate limit states that involve the following modes of external instability should be considered in the design: Loss of overall stability; Sliding failure; Bearing failure. Internal stability is concerned with the integrity of the reinforced block. A reinforced fill slope has the potential to fail due to rupture or pullout of the reinforcement or failure at the connection or facing. In checking internal stability, c onsideration should be given to the following: local stability of individual reinforcing elements, and stability of the yielding reinforced fill mass. The ultimate limit states that involve the following modes of internal instability should be considered in the design: Rupture of reinforcement; Pullout of reinforcement from the resisting fill mass; Pullout of reinforcement from the yielding fill mass; Rupture of structural facing elements and connection. The ultimate limit states that involve the following modes of compound instability should be considered in the design: Rupture/pullout of reinforcement; Sliding on reinforcement; Sliding on planes between reinforcement. For a simple upslope ground profile or where only simple uniform surcharge is present, a two part wedge method of analysis (Schertmann et al 1987,; Jewell 1990) can be used to give a quick, preliminary estimate of the design tension in the reinforcement. However, for less steep (say, slope angle less than 60° slopes), the two part wedge failure mechanism may not be able to model precisely the potential failure surfaces, and it may underestimate the design tension of the reinforcement.
	3.2. DIFFERENTIATIONS	8.1 Reinforced fill features with a face inclination of more than 20° from the vertical should be designed as steep slopes. Features with a facing inclined within 20° from the vertical should be designed as reinforced fill structures.
	3.3. LIMITS	8.7. For ease of construction, the minimum practical vertical spacing for the reinforcement could be a multiple of the appropriate fill lift, which is normally controlled by compaction considerations. Fill lif t s between 150 mm and 300 mm are typical. Maximum vertical reinforcement spacing should be limited to 1.0 m.
4. EXTENSIBLE / INEXTENSIBLE REINFORCEMENT	4.1. DIFFERENTIATION LINES	3.3.2. Load in the relatively in extensible reinforcement builds up rapidly and equilibrium occurs at a low er strain than that required to mobilise the peak shear strength of the fill. By contrast, the relatively extensible reinforcement strains more but mobilises a lower force that contributes to the equilibrium of the fill feature. Ductile reinforcement wil l allow larger strains to occur at ultimate limit state without the reinforcement suddenly rupturing, irrespective of the initial stiffness of the reinforcement. The benefit of the tensile strength available in a ductile reinforcement at large strain exis ts even after the peak strength of the fill has been reached (McGown et al, 1978).
	4.2. REINFORCEMENT TYPE (STRIP, GRID) INFLUENCE	Inextensible reinforments: metallic sheets, bars, strips (smooth or ribbed), anchors. Extensible reinforcements: Geogrids, Woven geotextiles, Geocomposites, Wire meshes.
	4.3. SOIL STIFFNESS INFLUENCE	3.3.6. Fill State, section 3, state of stress. The state of stress within a reinforced structure will be different with increasing depth of fill and with different quantities and types of reinforcement.
5. MISSING REINFORCEMENTS	5.1. GEOCELLS	NOT INCLUDED
	5.2. GEOSTRIPS	No limitation
	5.3. OTHER (specify)	NO
	6.1. LOCAL STABILITY ANALYSES	7.9.1. Failure of the connections between individual reinforcing elements and the connections on the facing elements should be checked using the design tensile force, Ti, developed in the individual layers of reinforcement. In addition any shear and bending stresses resulting from settlement of the fill relative to the facing should be considered in the design of the connections.
	6.2. STATIC AND SEISMIC DIFFERENCES	N.A.

<table>
<tr><td rowspan="2">6. CONNECTION CAPACITY</td><td>6.3. SHORT AND LONG TERM PROPERTIES</td><td>Section 4.2.1. Depending on the material to be used, the strength of the facing and connection materials should be obtained from the relevant standards, e.g. BS 5400: Part 4 (BSI, 1990), BS 8110: Part 1 (BSI, 1997) or Buildings and Lands Department (1987a) for reinforced concrete, and BS 449 : Part 2 (BSI, 1969), BS 5950 : Part 1 (BSI, 2000) or Buildings and Lands Department (1987b) for steel. Where the structure is part of a private development, the requirements of the Buildings Ordinance (Laws of Hong Kong, CAP 123) must be complied with.</td></tr>
<tr><td>6.4. REDUCTION FACTORS</td><td>
<table>
<tr><td rowspan="2">Material Parameter</td><td colspan="2">Partial Material Factor, γ_m</td></tr>
<tr><td>Ultimate Limit State</td><td>Serviceability Limit State</td></tr>
<tr><td>Facing units interaction:</td><td></td><td></td></tr>
<tr><td>unit-to-unit resistance</td><td>1.2</td><td>-</td></tr>
<tr><td>unit-to-reinforcement resistance</td><td>1.2</td><td>-</td></tr>
</table>
</td></tr>
<tr><td rowspan="2">7. SEISMIC DESIGN</td><td>7.1. DESIGN SEISMIC ACCELERATION FOR RS STRUCTURES</td><td>Hong Kong is situated in a region of low to moderate seismicity and seismic load is generally not critical for retaining wall design.
Guidance on seismic loading appropriate for the Hong Kong conditions is given in Section 7.4 of Geoguide 1 (GEO, 1993).</td></tr>
<tr><td>7.2. RFcreep FOR GSY</td><td>N.A.</td></tr>
<tr><td rowspan="4">8. MINIMUM CHECKS</td><td>8.1. INTERNAL STABILITY</td><td>7.2. The ultimate limit states that involve failure planes located entirely within the reinforced block are categorised as internal instabilities, and these modes of instability should be considered in the design:
- Rupture of reinforcement.
- Pullout of reinforcement.
- Sliding along reinforcement.
- Sliding on planes between reinforcement.
- Failure of connections.
- Rupture of facing panels.
- Toppling of facing blocks.
- Sliding of facing blocks.</td></tr>
<tr><td>8.2. EXTERNAL STABILITY</td><td>7.2. The ultimate limit states that involve failure planes entirely outside or at the boundary of the reinforced block (i. e. reinforced portion of ‘compacted fill’) are categorised as external instabilities, and these modes of instability should be considered in the design:
- Loss of overall stability.
- Sliding failure.
- Overturning failure.
- Bearing failure.</td></tr>
<tr><td>8.3. LOCAL STABILITY AT FACE</td><td>7.9.1. Failure of the connections between individual reinforcing elements and the connections on the facing elements should be checked using the design tensile force, Ti, developed in the individual layers of reinforcement. In addition any shear and bending stresses resulting from settlement of the fill relative to the facing should be considered in the design of the connections.</td></tr>
<tr><td>8.4. GLOBAL STABILITY</td><td>6.5.2. When designing against overall slope instability the global safety factor approach recommended in the Geotechnical Manual for Slopes (GEO, 1984) should be followed.</td></tr>
<tr><td></td><td>9.1. AMPLIFICATION FACTORS FOR LOADS</td><td>
Table 7 - Recommended Partial Load Factors for the Design of Reinforced Fill Structures and Slopes
<table>
<tr><td rowspan="2">Loading</td><td colspan="2">Partial Load Factor, γ_f</td></tr>
<tr><td>Ultimate Limit State</td><td>Serviceability Limit State</td></tr>
<tr><td>Dead load due to weight of the reinforced fill</td><td>1.0</td><td>1.0</td></tr>
<tr><td>Dead load due to weight of the facing</td><td>1.0</td><td>1.0</td></tr>
<tr><td>External dead load (e.g. line or point loads)</td><td>1.5</td><td>1.0</td></tr>
<tr><td>External live load (e.g. traffic loading)</td><td>1.5</td><td>1.0</td></tr>
<tr><td>Seismic load</td><td>1.0</td><td>1.0</td></tr>
<tr><td>Water pressure</td><td>1.0</td><td>1.0</td></tr>
</table>
</td></tr>
</table>

9. AMPLIFICATION AND REDUCTION FACTORS

9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS

Table 6 - Recommended Partial Material Factors for the Design of Reinforced Fill Structures and Slopes

Material Parameter	Partial Material Factor, γ_m Ultimate Limit State	Partial Material Factor, γ_m Serviceability Limit State
Fill: unit weight, γ	1.0	1.0
effective shear strength, tan ϕ'	1.2	1.0
Ground: effective shear strength[(2)]	1.2	1.0
base friction, tan δ_b	1.2	1.0
Granular fill and drainage materials:		
Permeability, k	10.0	-
Structural elements:		
Reinforcement tensile strength	1.5[(3)]	-
facing strength	as per relevant structural code	-
Fill-to-reinforcement interaction:		
sliding resistance	1.2[(4)]	-
pullout resistance	1.2[(4)]	-
Facing units interaction:		
unit-to-unit resistance	1.2	-
unit-to-reinforcement resistance	1.2	-

9.3. MODEL FACTORS / FACTORS OF SAFETY

Table 3 - Recommended Partial Consequence Factors for the Design of Reinforced Fill Structures and Slopes

Economic Consequence \ Consequence-to-life	Category 1	Category 2	Category 3
Category A	1.1	1.1	1.1
Category B	1.1	1.0	1.0
Category C	1.1	1.0	1.0

10. SERVICEABILITY LIMITS

10.1. FACE DEFORMATIONS

Table 9 - Tolerance of Reinforced Fill Facing Systems to Differential Settlement

Maximum Differential Settlement	Comment
1 in 1000	Not normally significantly
1 in 200	Full-height panels may be affected by joints closing or opening
1 in 100	Normal safe limit for discrete concrete panel facings without special precautions
1 in 50	Normal safe limit for semi-elliptical steel facings or geosynthetic facings
1 in < 50	Distortion may affect retaining ability of soft facings

Table 10 - Minimum Vertical Movement Capacities Required for Facing Systems to Cope with Vertical Internal Settlement of Reinforced Fill

Structural Form	Minimum Vertical Movement Capacity of System
Discrete panels	Joint closure of 1 in 150 relative to panel height
Full-height panels	Vertical movement capacity of connections 1 in 150 relative to panel height
Semi-elliptical steel facings	Vertical distortion of 1 in 150 relative to panel height
Geosynthetic wraparound facings	No specific limit except for appearance or serviceability consideration

10.2. SETTLEMENTS AT BASE

7.7.2. Differential Settlement. The possibility of differential settlement along the length of the reinforced fill structure should be considered when the foundation material is likely to be variable or when compressible spots exist. It is often the facing of the structure that determines the limits to differential settlement. Where large differential settlements are anticipated, special slip joints should be incorporated into the facing and detailed on the construction drawings. Reinforced fill bridge abutments are able to accommodate differential settlements significantly in excess of the established tolerable movements criteria for bridge decks (Moulton et al, 1982). In these conditions special structural precautions should be used with regard to the bridge superstructure (Worrall, 1989; Sims and Bridle, 1966; Jones and Spencer,1978).

10.3. SETTLEMENTS AT TOP

6.7.3. Settlement. (3) Internal Compression. Reinforced fill structures compress internally during placement and compaction of the fill layers, so the construction system and the construction tolerances must be able to accommodate these movements (see Section 2.3.1). Table 10 provides typical movement capacities of facing systems to cope with internal compression ofthe reinforced block.

<table>
<tr><td rowspan="5">11. SOILS</td><td>11.1. PERMITTED TYPES OF FILL</td><td>3.3.5. Fill Properties. (1) Particle size and grading. The ideal particle size for reinforced fill is a well-graded granular material, providing every opportunity for long-term durability of the reinforcing elements, stability during construction, and good physiochemical properties. In the normal stress range associated with reinforced fill structures and slopes, well-graded granular fill materials behave elastically, and post-construction movements associated with internal yielding will not normally occur.Fine-grained fill materials are poorly drained and difficult to compact when moisturecontent becomes high following heavy rainfall. Therefore, construction using fine-grained fill normally results in a slower construction rate. Fine-grained fill materials often exhibit elasto-plastic or plastic behaviour, thereby increasing the chance of post-construction movements. Problems of stability and serviceability can also result from the use of crushable fill materials.</td></tr>
<tr><td>11.2. MINIMUM GEOTECHNICAL PROPERTIES</td><td>Table A.1 – Properties of Selected Fill Material
<table>
<tr><th>Requirement \ Material Type</th><th>Type I</th><th>Type II</th></tr>
<tr><td>Maximum Size (mm)</td><td>150</td><td>150</td></tr>
<tr><td>% Passing 10 mm BS Sieve Size</td><td>25 – 100</td><td>-</td></tr>
<tr><td>% Passing 600 microns BS Sieve Size</td><td>10 – 100</td><td>10 – 100</td></tr>
<tr><td>% Passing 63 microns BS Sieve Size</td><td>0 – 10</td><td>0 – 45</td></tr>
<tr><td>% Smaller than 2 microns</td><td>-</td><td>0 – 10</td></tr>
<tr><td>Coefficient of Uniformity</td><td>≥ 5</td><td>≥ 5</td></tr>
<tr><td>Liquid Limit (%)</td><td>Not applicable</td><td>≤ 45</td></tr>
<tr><td>Plasticity Index (%)</td><td>Not applicable</td><td>≤ 20</td></tr>
</table>
Table A.2 – Allowable Electrical and Chemical Limits of Selected Fill and Granular Filter
<table>
<tr><th rowspan="2">Fill Property</th><th colspan="2">Allowable Limits</th></tr>
<tr><th>Submerged</th><th>Non-Submerged</th></tr>
<tr><td>pH</td><td>5 – 10</td><td>5 – 10</td></tr>
<tr><td>Resistivity (ohm m)</td><td>≥ 30</td><td>≥ 10</td></tr>
<tr><td>Organic Content</td><td>≤ 0.2</td><td>≤ 0.2</td></tr>
<tr><td>Redox Potential (volts)[3]</td><td>≥ 0.40 (Type I)
≥ 0.43 (Type II)</td><td>≥ 0.40 (Type I)
≥ 0.43 (Type II)</td></tr>
<tr><td>Microbial Activity Index[3]</td><td>≤ 5</td><td>≤ 5</td></tr>
<tr><td>Chloride Ion Content (% by weight)</td><td>≤ 0.01</td><td>≤ 0.02</td></tr>
<tr><td>Total Sulphate Content (% by weight)</td><td>≤ 0.10</td><td>≤ 0.20</td></tr>
<tr><td>Sulphate Ion Content (% by weight)</td><td>≤ 0.05</td><td>≤ 0.10</td></tr>
<tr><td>Total Sulphide Content (% by weight)</td><td>≤ 0.01</td><td>≤ 0.03</td></tr>
</table></td></tr>
<tr><td>11.3. MINIMUM INTERACTION PROPERTIES</td><td>N.A.</td></tr>
<tr><td>11.4. LIMITS ON BACKFILL</td><td>4.3. FILL MATERIALS. Reinforced fill retaining walls and bridge abutments are usually designed to use fill material of such a quality that will allow for easy and rapid construction. Similar considerations apply to the use of fill in reinforced slopes. Fill may be naturally occurring or processed materials.</td></tr>
<tr><td>11.5. LIMITS ON FOUNDATION SOIL</td><td>3.3.8. Foundation. Reinforced fill structures and slopes can be constructed on relatively weak foundations. However the nature and mode of settlement will be dictated by the foundation condition and the geometry of the structure (Jones, 1996). Reinforced fill walls often settle backwards when constructed on weak foundations. The rotational behaviour of the reinforcedfill mass in these circumstances is similar to that experienced by bridge abutments built on soft ground (Nicu et al, 1970). Reinforced fill walls constructed on sloping rock foundations are frequently designed as stepped walls. Consideration needs to be given to the possibility of fill arching at the base of the structure associated with the geometry of the steps in the foundation profile. Guidance on the dimensions of base width and height of step is provided in Section 7.14.</td></tr>
<tr><td rowspan="3">12. WATER</td><td>12.1. MODELS OF WATER PRESSURE</td><td>The influence of pore water pressures on pullout resistance should be taken into account. Zero pore water pressure within the reinforced block may be assumed if adequate drainage measures are incorporated in the design.</td></tr>
<tr><td>12.2. WATER INDUCED FAILURES</td><td>7.11.2. Design Detailing. (1) Surface drainage. Structures located along the downhill side of highways should be provided with robust surface drainage details. Channels with removable, perforated covers may be provided along the crest of downhill retaining structures as they are easy to maintain and do not become blocked easily as buried drainage pipe systems with gullies at widespacings. If deep drains are required to be constructed along the crest of reinforced fill structures, it is common practice to construct a self standing upstand on top of the reinforced fill wall to avoid clashing with the top layer of reinforcement. For part height walls, a drainage channel should be provided immediately behind the wall crest along the slope toe to remove water running off the side slope (Figure 43(a)). Guidance on the design of surface drainage for highway structures and slopes is provided in the Highway Slope Manual (GEO, 2000).</td></tr>
<tr><td>12.3. WATER IN SEISMIC CONDITIONS</td><td>N.A.</td></tr>
<tr><td>13. MISSING ELEMENTS</td><td>13.1 ITEMS NOT INCLUDED IN CODE</td><td>6.2.2. The design life of a permanent reinforced fill feature should be taken as 120 years unless otherwise specified by the owner.</td></tr>
</table>

TOPIC	SUB-TOPIC	AUSTRALIA - R57
1. VERTICAL SPACING	1.1. CALCULATION APPROACH	Limit State
	1.2. LIMITS	Not addressed or specified.
2. INTERNAL STABILITY DESIGN METHODS FOR MSE WALLS	2.1. AVAILABLE METHODS	LRFD (Load Resistance Factor Design) and Circular and Non circular stability analyses to satisfy minimum FoS requirements
	2.2. DIFFERENTIATIONS	Nil
	2.3. LIMITS	No limitations
3. DESIGN METHODS FOR STEEP SLOPES	3.1. AVAILABLE METHODS	Not covered in R57 as it is exclusively for vertical and near vertical walls
	3.2. DIFFERENTIATIONS	n/a
	3.3. LIMITS	n/a
4. EXTENSIBLE / INEXTENSIBLE REINFORCEMENT	4.1. DIFFERENTIATION LINES	Inextensible and Extensible are defined in each soil reinforcement system approval. R57 only allows soil reinforcement systems that have gained system approval. The approval process requires technical submission for which R57 will specify allowable design tensile and connection strengths and any other limitations thare are uniique or specifc to that system. That includes the type of facing that can be used with the approved soil reinforcement system and importantly nominates the system owner.
	4.2. REINFORCEMENT TYPE (STRIP, GRID) INFLUENCE	Inextensible reinforcement include strips and ladders. Extensible includes geostrip, mesh and grid.Current approved systems comprise 4 Inextensible systems and 23 x Extensible systems
	4.3. SOIL STIFFNESS INFLUENCE	Not addressed or specified.
5. MISSING REINFORCEMENTS	5.1. GEOCELLS	Currently not an approved system.
	5.2. GEOSTRIPS	Currently two different Geostrip systems and owners are approved as extensible reinforcement.
	5.3. OTHER (specify)	
6. CONNECTION CAPACITY	6.1. LOCAL STABILITY ANALYSES	Local stabilty analyses are covered under each system of reinforcement and facing in combination. The approval submission and is not publicly available however our experince with ouyr systems would suggest fairly rigorous testing of connections are required with each system.
	6.2. STATIC AND SEISMIC DIFFERENCES	No static and seismic dfferences probbaly because of relatively low seismic activity in Australia.

	6.3. SHORT AND LONG TERM PROPERTIES	All adressed in soil reinforcement system approval where design tensile and connections strengths and temperatures are specified.
	6.4. REDUCTION FACTORS	Specied as 0.85 on characteristic design strength for all systems
7. SEISMIC DESIGN	7.1. DESIGN SEISMIC ACCELERATION FOR RS STRUCTURES	Maximum aog =0.15. 50% reduction for horizontal and 75% for vertical Inertia forces.
	7.2. RFcreep FOR GSY	Covered in each reinforcement system approval. Our experince suggests Rfcreep will tyically be in the range 0.50 to 0.60 for all extensible reinforcement systems
8. MINIMUM CHECKS	8.1. INTERNAL STABILITY	For Ultimate Limit State : - 1) stability of individual soil reinforcement (i.e rupture and soil pull out failure), 2) stability of wedges, 3) forward sliding of the wall, 4) structural failure of facing connections and pull out failure from facing elements, 5) structural failure of facing elements, 6) durability. For Serviceability Limit State : -1) stability of individual soil reinforcement (post construction creep and soil pull out failure), 2) stability of wedges, 3) structural failure of facing connections and pull out failure of connections from facing elements.
	8.2. EXTERNAL STABILITY	R57 spcifies minimum strip lengths to be L=0.6H + 1m. For Ultimate Limit State : - 1) bearing failure, 2) sliding, 3) slip failures. For Serviceability Limit State : 1) settlement, tilting and rotaional movement, 2) slip failures.
	8.3. LOCAL STABILITY AT FACE	Covered by each system of soil reinforcement and facing element. Each approval system conditions are unique to that specifc combination of reinforcement and facing element.
	8.4. GLOBAL STABILITY	Specified to be carried out by a Geotechnical Engineer and is a designated hold point prior to commencement of construction.
9. AMPLIFICATION AND REDUCTION FACTORS	9.1. AMPLIFICATION FACTORS FOR LOADS	Dead Load = 1.25, Live Load =1.5, Earth Pressure =1.25.
	9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS	Select and general backfill strength = 1.0, foundation = 0.8. Reinforcemnt pull out from backfill = 0.75, sliding between soil reinforcement and foundation = 0.85, pull out of soil reinforcement from facing elements = 0.70, structural strength of connection = 0.85.
	9.3. MODEL FACTORS / FACTORS OF SAFETY	FoS required to be 1.35 for RSW's not supporting a bridge and 1.60 for RSW's supporting bridge loads.
10. SERVICEABILITY LIMITS	10.1. FACE DEFORMATIONS	Constructed position to be a maximum +/- 50mm from design position. Flatness +/-20mm pver 4.5m length. Inclination as +0, -5mm per metre height for panel facing and +10mm, -10mm per metre height for segmental blocks.
	10.2. SETTLEMENTS AT BASE	+/- 30mm from design position
	10.3. SETTLEMENTS AT TOP	+/- 15mm from design position

11. SOILS	11.1. PERMITTED TYPES OF FILL	Generally tight requirments that preclude use of any backfill other than a granular fill. Minimum requirements are CU>5, PI<12 and LL<30. Phicv (contant volume conditions) must be at least 34° and value adopted for design is not to exceed 36° irrespective of actal result.
	11.2. MINIMUM GEOTECHNICAL PROPERTIES	Maximum 10%<0.075mm and as above
	11.3. MINIMUM INTERACTION PROPERTIES	All addressed in each system approval. Friction coefficients for strips and ladders > 1.0 x tan (phicv) and grids and mesh between 0.6 and 0.9 x (tanphicv).
	11.4. LIMITS ON BACKFILL	All slect backfill must be placed in layer thickness not exceeding 150mm
	11.5. LIMITS ON FOUNDATION SOIL	No limitations provided is covered by Geotechnical Engineer certification of foundation.
12. WATER	12.1. MODELS OF WATER PRESSURE	Minimum default height of water level to be considered is at finished ground level. For floodwater or tidal imnundation, minimum drawdown levels to be 1m beteen internal and external water levels.
	12.2. WATER INDUCED FAILURES	Vertical cutoff rear drainage layer at back fo RSW block is mandatory required up to 1m below finished surface level.
	12.3. WATER IN SEISMIC CONDITIONS	Water and seismic to be not simultaneous.
13. MISSING ELEMENTS	13.1 ITEMS NOT INCLUDED IN CODE	Possibly only crash barrier or impact loading is not covered. R57 is considered by all users and designers in Australia to be a very good design specification primarily because of the soil reinforcement system approval process.

TOPIC	SUB-TOPIC	INDIA - IRC SP:102-2014
1. VERTICAL SPACING	1.1. CALCULATION APPROACH	Vertical Spacing is fixed to maximum 800mm and based on this spacing loads/stresses coming over the reinforcemnt at particular layer is calculated. Local stability of facia need to be ensured.
	1.2. LIMITS	(i) To provide a coherent reinforced soil mass, the vertical spacing of primary reinforcement shall not exceed 800mm, in all types of reinforcement. (ii) For walls constructed with modular blocks and deriving their connection capacity by friction, and also for any other facia configurations, where connection capacity is by friction, the maximum vertical spacing of reinforcement shall be two times the block width (measured from front fact to back face of the block). Further, the maximum spacing of reinforcing elements shall not exceed 800 mm in all cases. The maximum height of facing left unreinforced a) above the uppermost reinforcing layer and b) below the lowest reinforcing layer, shall not exceed the width of the block (measured from the front face to back face of the block.) (iii) In case modular blocks are used for facia, no more than one intervening block shall be left without having primary reinforcement. (iv) In case of wraparound facings for walls, the maximum spacing of reinforcing elements shall not exceed 500mm, to protect against bulging. (v) Where panels are used, the maximum spacing of reinforcement shall not exceed 800mm. The spacing of nearest reinforcing element shall be such that maximum height of facing above uppermost reinforcement layer and below the lower most reinforcement layer does not exceed 400mm. (vi) Reinforcement spacings worked out from the design procedures shall be configured to fit the above parameters.
2. INTERNAL STABILITY DESIGN METHODS FOR MSE WALLS	2.1. AVAILABLE METHODS	Tie Back Wedge Method and Coherent Graviy Method.
	2.2. DIFFERENTIATIONS	Differentiation is based on extension of reinforcement (less than 1% is coherent and more than 1% tieback) a)Tie Back Wedge Method is used for Extensible Reinforcement ,Coefficient of earth Pressure is taken as constant i.e., Ka from top to bottom and Failue plane is linear i.e, (45-ϕ/2)with vertical . b) Coherent Gravity Method is used for Inextensible Reinforcements,Coefficient of earth Pressure varies linearly from ko to ka from top to 6m and will be taken as ka till bottom from 6m and Failure plane is bilinear (Same as BS 8006-1).
	2.3. LIMITS	Cannot use coherent gravity method for Geosynthetics unless stress strain curves are produced to prove that geosynthtic reinforcement is inextensible.
3. DESIGN METHODS FOR STEEP SLOPES	3.1. AVAILABLE METHODS	Rotation stability. As per BS8006-1, FHWA & AFNOR
	3.2. DIFFERENTIATIONS	No
	3.3. LIMITS	No limit
4. EXTENSIBLE / INEXTENSIBLE REINFORCEMENT	4.1. DIFFERENTIATION LINES	When the design load is sustained at a total axial strain 1 percent or less the reinforcement is classified as inextensible .Where the design load is sustained at total axial tensile strain exceeding 1 percent the reinforcement is classified as extensible.
	4.2. REINFORCEMENT TYPE (STRIP, GRID) INFLUENCE	No Differentiation based on type of strip or geogrid except FHWA which refer different in "k"
	4.3. SOIL STIFFNESS INFLUENCE	Once the design loads (serviceability load, or working load) are carried by the metallic reinforcement such as bars, plates etc. at an axial strain less than the strain in the soil, the reinforcement is classified as "inextensible" reinforcement. Polymeric reinforcements which are characterised by temperature and time dependent strains (creep) are normally classified as extensible reinforcements. However, Polymeric and other reinforcements which show less strain as compared to soil strain may be also classified as inextensible reinforcement. No clear influence of soil stiffness
5. MISSING REINFORCEMENTS	5.1. GEOCELLS	Not Specified.
	5.2. GEOSTRIPS	Geostrips are included under heading of geogrid.

	5.3. OTHER (specify)	Geocomposite Reinforcement is to be specified .
6. CONNECTION CAPACITY	6.1. LOCAL STABILITY ANALYSES	Strength of Connection should be more than the Load at the connection (Tconnection).
	6.2. STATIC AND SEISMIC DIFFERENCES	Under Seismic loading the long term connection strength shall be reduced to 80% of its static
	6.3. SHORT AND LONG TERM PROPERTIES	Same in both long term and short term.
	6.4. REDUCTION FACTORS	a) For Tie back Wedge Method : i)All facings with movement capacity or movement capacity at connections Tconnection = 75% Tj at top linearly varies to 100 %Tj at toe .ii) Stiff face, e.g. segmental block walls and full height panels with no movement capacity at connections,Tconnection = 100% Tj from top to toe .b) For Coherent Gravity method : i) Flexible face, e.g. metal U-shape elements,Tconnection = 75% Tj at top till 0.6H and after 0.6H till toe it varies linealy to 100% Tj .ii)Articulated face, e.g. discrete concrete panels ,Tconnection = 85% Tj at top till 0.6H (Constant)and after 0.6H till toe it varies linealy to 100% .iii) Stiff face, e.g. segmental block walls and full height panels with no movement capacity at connections,Tconnection = 100% Tj from top to toe
7. SEISMIC DESIGN	7.1. DESIGN SEISMIC ACCELERATION FOR RS STRUCTURES	Seismic Ground acceleration(A) shall be calculated based on the Seimic zone and maximum Wall acceleration(Am) for RS Wall is calculated as Am=A(1.45-A) . Horizontal inertia force on block of soil mass and incremental thrust due to retained fill are the additional loads considered in the design.Both these forces are horizontal and added to the horizontal static forces. Full inertia force on the part of reinforced soil and only 50 per cent of dynamic thrust due to earth pressure by retained fill are considered for stability analysis.No dynamic increment on earth pressure due to live load surcharge is considered .
	7.2. RFcreep FOR GSY	For dynamic component, where the load is applied for a short time, creep reduction is not required.
8. MINIMUM CHECKS	8.1. INTERNAL STABILITY	Adherence,Rupture,wedge Stability & Internal Sliding.
	8.2. EXTERNAL STABILITY	Sliding,Overturning,Bearing and tilt.
	8.3. LOCAL STABILITY AT FACE	Connection Strength Check
	8.4. GLOBAL STABILITY	Slip circle failure check .It should also include Deep Seated Failure
9. AMPLIFICATION AND REDUCTION FACTORS	9.1. AMPLIFICATION FACTORS FOR LOADS	
	9.2. REDUCTION FACTORS FOR RESISTANCES/MATERIALS	For Geosynthetic Reinforcements,Following Reduction factors is to be applied and these has to be provided by the manufacturer from an accrediated laboratory/ cerfication from an accrediated agency is required For Soil,Following factors to be applied: RF_{CR} - Reduction factor for creep. RF_{ID} - Reduction factor for installation damage RF_{W} - Reduction factor for weathering RF_{CH} - Reduction factor for chemical/ environmental effects. fs - Factor for the extrapolation of data
	9.3. MODEL FACTORS / FACTORS OF SAFETY	
	10.1. FACE DEFORMATIONS	a)Location of plane of structure ± 50 mm – metallic reinforcement ,± 75 mm – synthetic reinforcement b)Bulging (Vertical) and Bowing (Horizontal) ± 20 mm in 4.5 m template (Metallic) ± 30 mm in 4.5 m template (Synthetic) c)Steps at joints ± 10 mm

10. SERVICEABILITY LIMITS	10.2. SETTLEMENTS AT BASE	Post construction settlement of the founding soil should not exceed 100 mm for discrete panels/and blocks which result in flexible structures.Typically differential settlement of 1 in 100 are considered as safe for discrete concrete panels facings (1 in 500 for full height panels).
	10.3. SETTLEMENTS AT TOP	Not Specified
11. SOILS	11.1. PERMITTED TYPES OF FILL	i.)Clean, free draining and non-plastic granular fill(GC, GM or GC-GM) ii.)Fly ash and pond ash iii)Residual/soil murrum iv)Any other mechanically stabilised soil, blended by mechanical equipment
	11.2. MINIMUM GEOTECHNICAL PROPERTIES	a)Reinforced Fill: Angle of internal friction >30 degrees b)Retained Fill :Angle of internal friction >25 degrees
	11.3. MINIMUM INTERACTION PROPERTIES	Not Specified.It has to be derived based on the tests conducted at site/laboratory on the same fill and reinforcement Used as per ASTM D 5321 Specification.
	11.4. LIMITS ON BACKFILL	a)Reinforced Fill: percentage Fines < 15%,Plasticity Index (PI)<6,Coefficient of uniformity >2 . b)Retained Fill : Plasticity Index (PI)< 20
	11.5. LIMITS ON FOUNDATION SOIL	Nothing as Such.It should satisfy Bearing Capacity Check ,else Suitable Ground Improvement is to be provided.
12. WATER	12.1. MODELS OF WATER PRESSURE	Not Included.
	12.2. WATER INDUCED FAILURES	Not Included.
	12.3. WATER IN SEISMIC CONDITIONS	Not Included.
13. MISSING ELEMENTS	13.1 ITEMS NOT INCLUDED IN CODE	