

BS 8006-1:2010+A1:2016



BSI Standards Publication

Code of practice for strengthened/reinforced soils and other fills

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Published by BSI Standards Limited 2016

ISBN 978 0 580 92713 3

ICS 93.020

The following BSI references relate to the work on this standard:

Committee reference B/526/4

Drafts for comment 09/30093258 DC, 16/30334776 DC

Publication history

First published October 2010

Amendments issued since publication

Date	Text affected
30 June 2012	C1
31 July 2016	A1: See foreword

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Summary of pages

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Foreword

Publishing information

This part of BS 8006 is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 31 October 2010. It was prepared by Subcommittee B/526/4, *Strengthened/reinforced soils and other fills*, under the authority of B/526, *Geotechnics*. A list of organizations represented on this committee can be obtained on request to its secretary.

Supersession

BS 8006-1:2010+A1:2016 supersedes BS 8006-1:2010, which is withdrawn.

Relationship with other publications

The use of reinforcement in soils has become an accepted technique for the construction of retaining walls, steep slopes and basal strengthening. This code of practice has been revised and updated to include information about new methods of soil reinforcement and to bring the document in line with BS EN 1997-1:2004, NA to EN 1997-1:2004 and BS EN 14475:2006. Reinforced soil techniques are now used extensively for a range of design lives and service requirements and are still in an active stage of development, particularly as far as the use of polymeric materials is concerned.

Information about this document

The start and finish of text introduced or altered by Amendment No. 1 is indicated in the text by tags **A1** **A1**.

Use of this document

This code of practice embodies the experience of engineers successfully engaged on the design and construction of the particular class of works. It has been assumed in the drafting of this British Standard that the execution of its provisions is entrusted to appropriately qualified and experienced people.

A code of practice represents good practice at the time it is written and, inevitably, technical developments will render parts of it obsolescent in time. It is the responsibility of engineers concerned with the design and construction of works to remain conversant with developments in good practice, which have taken place since publication of the code.

As a code of practice, this part of BS 8006 takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

Any user claiming compliance with this part of BS 8006 is expected to be able to justify any course of action that deviates from its recommendations.

Presentational conventions

The provisions in this standard are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is "should".

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard cannot confer immunity from legal obligations.

Section 1: General

1.1 Scope

This British Standard contains recommendations and guidance for the application of reinforcement techniques to soils, as fill or in situ, and to other fills. The standard is written in a limit state format and guidelines are provided in terms of partial material factors and load factors for various applications and design lives.

This code is to be read in conjunction with BS EN 1997-1:2004, NA to BS EN 1997-1:2004 and BS EN 14475:2006.

BS EN 1997-1:2004 does not cover the design and execution of reinforced soil structures. The values of partial factors and load factors given in BS EN 1997-1:2004 have not been calibrated for reinforced soil structures. BS EN 1997-1:2004 is not for use in the design and execution of reinforced soil. The partial factors set out in BS 8006-1 cannot be replaced by similar factors in BS EN 1997-1:2004.

The code is divided into eight sections. Section 1 identifies the scope, definitions and notation of the code. Section 2 describes the concepts and fundamental principles of reinforced soil. Section 3 provides recommendations for the use of materials where existing standards are available. Where materials are used that are not covered by existing standards or where known materials are to be used in ways not covered by existing standards Section 4 gives recommendations for the testing and approval of such materials.

Sections 5 to 8 relate to design, construction and maintenance of walls and abutments, slopes and foundations. They include specific recommendations for characterization of the soils to be used and other factors affecting the design and performance of the structures. Emphasis is placed on quality control both with regard to the consistency of the properties of the fill and reinforcing materials and to the handling of the materials on site.

In line with current practice the design methods described are based on limit state principles. The partial factors included are based on previous experience and have been calibrated to maintain consistency with current practice.

The clauses are supplemented by a substantial list of references to enable the user to consider in greater depth the applications of the technique.

1.2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 1377-3, *Methods of test for soils for civil engineering purposes – Chemical and electro-chemical tests*

BS 1377-7, *Methods of test for soils for civil engineering purposes – Shear strength tests (total stress)*

BS 1377-8, *Methods of test for soils for civil engineering purposes – Shear strength tests (effective stress)*

- BS 1377-9, *Methods of test for soils for civil engineering purposes – In-situ tests*
- BS 1449-1, *Steel plate, sheet and strip – Specification for carbon and carbon-manganese plate, sheet and strip*
- BS 2569, *Control cable fittings and turnbarrels – Technical specification*
- BS 3416, *Specification for bitumen-based coatings for cold application, suitable for use in contact with potable water*
- BS 3692:2001, *ISO metric precision hexagon bolts, screws and nuts – Specification*
- BS 4147, *Specification for bitumen-based hot-applied coating materials for protecting iron and steel, including suitable primers where required*
- BS 4164, *Specification for coal-tar-based hot-applied coating materials for protecting iron and steel, including a suitable primer*
- BS 4449:2005, *Steel for the reinforcement of concrete – Weldable reinforcing steel – Bar, coil and decoiled product – Specification*
- BS 5930:1999+A1:2007, *Code of practice for site investigations*
- BS 6349, *Maritime structures*
- BS 7263-1:2001, *Precast concrete flags, kerbs, channels, edgings and quadrants – Part 1: Precast, unreinforced concrete paving flags and complementary fittings – Requirements and test methods*
- BS EN 771-3:2003+A1:2005, *Specification for masonry units – Part 3: Aggregate concrete masonry units (dense and light-weight aggregates)*
- BS EN 1990:2002+A1:2005, *Eurocode – Basis of structural design*
- BS EN 1997-1:2004, *Eurocode 7: Geotechnical design – Part 1: General rules*
- BS EN 1997-2, *Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing*
- BS EN 10025-2:2004, *Hot rolled products of structural steels – Part 2: Technical delivery conditions for non-alloy structural steels*
- BS EN 10080:2005, *Steel for the reinforcement of concrete – Weldable reinforcing steel – General*
- BS EN 12225, *Geotextiles and geotextile-related products – Method for determining the microbiological resistance by a soil burial test*
- BS EN 12878, *Pigments for the colouring of building materials based on cement and/or lime – Specifications and methods of test*
- BS EN 14475:2006, *Execution of special geotechnical works – Reinforced fill*
- BS EN ISO 1461, *Hot dip galvanized coatings on fabricated iron and steel articles – Specifications and test methods*
- BS EN ISO 10319, *Geotextiles – Wide-width tensile test*
- BS EN ISO 10320, *Geotextiles and geotextile-related products – Identification on site*
- BS EN ISO 10321, *Geosynthetics – Tensile test for joints/seams by wide-width strip method*

BS EN ISO 14688-2, *Geotechnical investigation and testing – Identification and classification of soil – Part 2: Principles for a classification*

BS EN ISO 22475, *Geotechnical investigation and testing – Sampling methods and groundwater measurements – Part 1: Technical principles for execution*

NA to BS EN 1997-1:2004, *UK National Annex to Eurocode 7 – Geotechnical design – General rules*

PD 6694-1, *Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004*

PD ISO/TR 20432, *Guidelines for the determination of the long-term strength of geosynthetics for soil reinforcement*

Other publications

- [1] DEPARTMENT OF TRANSPORT, *Manual of Contract Documents for Highway Works, Specification for Highway Works*, Department of Transport. London: HMSO, 1993.

1.3 Terms and definitions

For the purposes of this British Standard the terms and definitions in BS EN 14475:2006 and the following apply.

1.3.1 anchored earth

form of reinforced soil which uses anchors embedded within the soil mass to provide stability; resistance to pull-out is provided by passive action of the anchor and friction along the anchor shaft or loop

1.3.2 axially flexible reinforcement

reinforcement that can absorb tensile loads only

1.3.3 axially stiff reinforcement

reinforcement that can absorb tensile, shear and bending loads

1.3.4 cohesive frictional fill

fill containing at least 15% material passing a Φ_{63} 63 μm or 63 micron Φ_{63} sieve

NOTE It is described in the Specification for Highway Works [1] under fill material classes 7C and 7D.

1.3.5 extensible reinforcement

reinforcement that sustains the design loads at strains greater than 1%

1.3.6 frictional fill

fill containing less than 15% material passing a Φ_{63} 63 μm or 63 micron Φ_{63} test sieve

NOTE It is described in the Specification for Highway Works [1] under selected fill material classes 6I and 6J.

1.3.7 geogrid

polymeric, planar structure consisting of an open network of connected tensile elements used in geotechnical and civil engineering applications

1.3.8 geotextile

permeable, polymeric material, which can be woven, non-woven or knitted, used in geotechnical and civil engineering applications

- 1.3.9 inextensible reinforcement**
reinforcement that sustains the design loads at strains less than or equal to 1%
- 1.3.10 partial factors**
specific design parameters to account for uncertainty
- 1.3.11 polymeric reinforcement**
generic term that encompasses geosynthetic materials used in geotechnical engineering such as geotextiles, geogrids and geostrips
- 1.3.12 reinforced soil**
general term which refers to the use of placed or in situ soil or other material in which tensile reinforcements act through interface friction, bearing or other means to improve stability
- 1.3.13 reinforced soil segmental block structure**
reinforced soil structure that has a facing comprising dry stacked blocks without compressible joints between blocks and without joint fillers/covers as described in BS EN 14475:2006
- 1.3.14 reinforcement base strength**
unfactored strength of the reinforcement at the end of its selected design life
- 1.3.15 reinforcement design strength**
factored strength of the reinforcement at the end of its selected design life; it is the reinforcement base strength divided by the appropriate partial material factor
- 1.3.16 retained backfill**
fill material located between the reinforced mass and the natural soil

1.4 Symbols

- a Size or diameter of pile caps
- a_{bc} Adhesion factor
- a_c Cross-sectional area of connections or connection components
- a_{eb} Cross-sectional area resisting bearing stresses in connection components
- a_q Cross-sectional area of the shear plane of a connection component resisting shear
- b Width of the loading strip contact area at right angles to the structure or Width of footing or foundation
- b' Variable, defined in Figure 36
- b_i Width of slice i in stability analysis
- c' Cohesion of the soil under effective stress conditions
- c_u Undrained shear strength
- d Maximum vertical deflection of unsupported reinforcement or Distance of strip load from wall face
- d' Variable, defined in Figure 36
- d_c Diameter of a fastener in a connection
- d_s Vertical displacement of surface of pavement or embankment due to subsidence below

e	Eccentricity of an applied force or Eccentricity of vertical strip load with respect to the centre line of the contact area of the load on top of a structure
e_j	Eccentricity of resultant vertical load at j th level about centreline of reinforcement
f	Partial load factor
f_f	Partial load factor applied to external dead loads
f_{fs}	Partial load factor applied to soil unit weight loads
f_m	Partial material factor applied to reinforcement
f_{m1}	Partial material factor related to material properties of the reinforcement
f_{m2}	Partial material factor related to construction effects and tolerances of the reinforcement
f_{ms}	Partial material factor applied to soil parameters
f_n	Partial factor associated with ramification of failure
f_q	Partial load factor applied to external live loads
f_p	Partial factor against pull-out failure of the reinforcement
f_s	Partial factor against sliding failure across the reinforcement or soil
h	Average height of fill above reinforcement
h_j	Height of fill above j th layer of reinforcements
h_p	Height of the prop above toe of a structure
h_t	Height of fill above toe of a structure
k_b	Variable in 6.8.4.3.5
k_r	Variable in 6.8.4.3.6
l	Variable, defined in Figure 36
n	Number of shear planes resisting applied shear, in 6.8.4.3.3 ; or Number of slices in a moment stability analysis; or Sideslope of embankment
p'_c	Vertical stress on pile cap
pH	Value of acidity of an aqueous solution
q_r	Bearing pressure
q_{ult}	Ultimate bearing capacity of foundation soil
r_u	Pore pressure ratio
s	Pile spacing
t_a	Long term height of an anchor head
t_d	Design life
t_t	Test duration
u	Pore water pressure
u_i	Average pore pressure acting on the base of slice i in stability analysis
w_s	Uniformly distributed surcharge on top of a structure
z	Depth measured from top of wall or slope

z_a	Depth of design line 1; coherent gravity method
z_c	Critical depth of foundation sliding block
z_o	6 m critical depth
z_s	Depth of dispersal of horizontal shear
B	Width of an element of reinforcement or Length of Meyerhof pressure distribution
B_a	Long term width of an anchor head
B_s	Long term horizontal projection area of shaft or loop of an anchor
C_c	Arching coefficient
D	Cavity diameter formed at the underside of a pavement or embankment
D_m	Embedment depth of reinforced walls and abutments
D_s	Deformation diameter at surface of pavement or embankment due to subsidence below
E	Elastic modulus
F_B	Function in 6.6.4.2.1
F_d	Factored design load
F_k	Unfactored characteristic load
F_L	Horizontal shear applied to the strip contact area of width b on top of the structure per metre "run"
F_p	Resistance of pile
H	Overall height of reinforced structure or fill, mechanical height of structure
H_t	Total height
K	Coefficient of earth pressure
K_a	Coefficient of active earth pressure
K_o	Coefficient of earth pressure at rest
K_p	Coefficient of passive earth pressure
L	Length of element of reinforcement or anchor; or Length at right angles in plan to the face of the structure of the bottom layer of reinforcing elements; or Distance between centres of end fasteners of a connection
L_{aj}	Length of that part of the j th layer of reinforcements within the potential failure plane
L_b	Reinforcement bond length needed beyond outer row of piles in piled embankment
L_e	Effective length of reinforcement
L_{ej}	Length of that part of the j th layer of reinforcing elements beyond the potential failure plane
L_j	Bond length of basal reinforcement within slip circle; or Length of nail j in a slope
L_p	Horizontal distance between outer edge of pile cap and toe of embankment
L_s	Length of side slope of an embankment

M	Maximum bending moment in a connection component
M_B	Reinforcement base bending moment resistance
M_d	Bending resistance in a connection component
M_D	Disturbing moment in slip circle or log-spiral analysis
M_j	Maximum bending moment in nail j in the slope
M_o	Out of balance moment, $M_o = M_D - M_R$
M_R	Restoring moment in slip circle analysis
M_{RP}	Restoring moment due to piles under an embankment
M_{RR}	Restoring moment due to reinforcement
M_{RS}	Restoring moment due to soil strength
M_{RT}	Restoring moment due to tension in nails
M_{RV}	Restoring moment due to shear in nails
M_x	Moment about X-X axis
M_y	Moment about Y-Y axis
N	Normal force acting on potential failure plane
P_{aj}	Bearing resistance of anchor at level j in wall
P_j	Total horizontal width of top and bottom faces of the j th layer of reinforcements per metre "run" of structure.
<i>NOTE In case of grid reinforcement the width of the jth layer of grid per metre "run" of structure.</i>	
P_h	Horizontal component of backfill thrust on reinforced soil block
P_L	Horizontal propping force
P_{sj}	Pull-out resistance generated on anchor shaft in cohesionless fill at level j in wall
P_{uj}	Ultimate pull-out resistance of an anchor at level j in wall
P_v	Vertical component of backfill thrust on reinforced soil block
Q	Pressure beneath strip loading; or Force acting at discontinuity between two regions of two-part wedge analysis
Q_m	Average pressure
Q_p	Bearing capacity of pile
R	Reaction
R_d	Radius of critical slip surface in rotational failure
R_{di}	Radius of slip surface at slice i in log-spiral analysis
R_{dj}	Radius of slip surface coinciding with nail j in log-spiral analysis
R_h	Horizontal factored disturbing force
R_i	Reaction at slice i in log-spiral analysis
R_v	Vertical factored resultant force
S_L	Vertical loading, applied to a strip contact area of width b on top of a structure, per metre "run"
S_{Li}	Length of strip load S_L
S_v	Vertical spacing of reinforcement
S_{vj}	Vertical spacing of j th layer of reinforcement

T	Total tensile force to be resisted by the layers of reinforcement which anchor a wedge of reinforced soil, per metre “run” (wedge analysis)
T_{avj}	Average tensile load along the length of the reinforcement at the j th level in the wall or slope
T_B	Reinforcement base strength
T_c	Applied maximum tensile load on connection
T_{cj}	Tensile force developed due to cohesion in reinforced fill at level j in wall
T_{CR}	Extrapolated creep rupture strength at the end of the design life
T_{CS}	Extrapolated tensile load based on creep strain at the end of the design life
T_{ds}	Tensile force in the reinforcement developed from the lateral thrust of the embankment
T_D	Design tensile strength of the reinforcement
T_{fj}	Tensile force developed from the horizontal shear applied to the top of the structure to be resisted by the j th layer of reinforcing elements
T_j	Total maximum tensile force resisted by the j th layer of reinforcement
T_r	Maximum tensile force in basal reinforcement
T_R	General tensile force in reinforcement
T_{rf}	Tensile force in basal reinforcement due to shear stresses at surface of foundation
T_{ro}	Tensile force in basal reinforcement needed to resist rotational failure
T_{rp}	Tensile force generated in basal reinforcement in piled embankments due to transfer of vertical loading
T_{rs}	Tensile force generated in basal reinforcement over unsupported void
T_{sj}	Tensile force developed from the external loading (S_L) on top of the structure
T_{pj}	Tensile force developed from the external loading
T_u	Ultimate tensile strength of metallic reinforcements
V_c	Applied maximum load on a connection component
W	Total weight of soil structure per metre “run”
W_i	Weight of slice i in stability analysis
W_l	Weight of soil contained within Coulomb failure wedge
W_T	Distributed vertical load acting on basal reinforcement between adjacent pile caps
X	Horizontal moment arm in rotational failure
X_d	Design soil parameter value
X_i	Horizontal moment arm to slice i in rotational failure
X_k	Characteristic soil parameter value

Y	Vertical moment arm in rotational failure
Y_j	Vertical moment arm to reinforcement j in slope or wall
Z	Section modulus
Z_x	Section modulus about X-X axis
Z_y	Section modulus about Y-Y axis
α	Inclination of slope supported by reinforced soil structure
α'	Friction coefficient relating soil friction angle to soil/reinforcement bond
α'_{bc}	Adhesion coefficient relating soil cohesion to soil/reinforcement bond
α_i	Angle of inclination of base of soil slice i .
β	Angle of inclination of backfill thrust on reinforced soil block
β'	Inclination of a potential failure plane to the vertical plane
β_s	Inclination of slope
γ	Unit weight density of soil
δ'	Angle of shearing resistance between soil and reinforcement measured under effective stress conditions
δ_h	Lateral strain of soil under an applied load
δ_v	Axial compression under an applied load
ε	Reinforcement strain
ε_{max}	Maximum allowable strain in reinforcement
θ	Angle of draw in embankment fill
θ_d	Angle of slice i in a log-spiral analysis
θ_j	Angle of nail j location in the slope in log-spiral analysis
θ_p	Angle (to the vertical) between the outer edge of the outside pile cap and the shoulder of an embankment
λ	Load shedding coefficient
μ	Coefficient of friction between the fill and reinforcing element derived from the identity ($\mu = \tan \delta'$ where δ' is the angle of bond stress between the fill and the reinforcing elements – statistical mean of results obtained)
μ^*	Apparent coefficient of friction
ρ	Rate of increase in undrained shear strength with depth of foundation
σ	Normal stress; or Statistical standard deviation of results obtained
σ_{aj}	Vertical stress acting at level j in a wall in the resistant zone
σ_b	Maximum bearing pressure between connected parts
σ_{bc}	Ultimate bearing strength of connection components
σ_c	Tensile stress in connections
σ_{ec}	Extreme fibre bending stress for compression
σ_{et}	Extreme fibre bending stress for tension
σ_h	Horizontal stress on an element of soil

σ_q	Ultimate shear strength of connection components
σ'_{ij}	Normal effective confining stress acting on nail j in the slope
σ_t	Ultimate tensile strength of connection components
σ_v	Vertical stress on an element of soil
σ_{vj}	Vertical stress acting at level j in a wall
σ'_v	Applied vertical effective stress
σ_1	Major principal effective stress
σ_3	Minor principal effective stress
τ	Shear stress
τ_c	Average shear stress
ϕ	Mohr-Coulomb angle of friction
ϕ_u	Angle of internal friction of soil under total stress conditions
ϕ'	Angle of internal friction of soil under effective stress conditions
ϕ'_{cv}	Angle of constant volume shearing resistance under effective stress conditions
ϕ'_p	Peak angle of shearing resistance under effective stress conditions
ω_j	Inclination to the horizontal of nail j in the slope

Section 2: Concepts and fundamental principles

2.1 General

Reinforcement may be incorporated in engineering fill, or inserted into natural ground either to provide steeper slopes than would otherwise be possible or to improve load carrying capacity. Reinforcement may also be used to improve the performance of weak soils to support embankments or other resilient structures. These applications, which are illustrated in Figure 1, may involve the use of a range of reinforcement types and techniques including:

- metallic strips, grids or meshes;
- geosynthetics as polymeric strips, geotextiles geogrids or meshes;
- anchors or multi-anchors (but not ground anchors).

2.2 Limit state principles

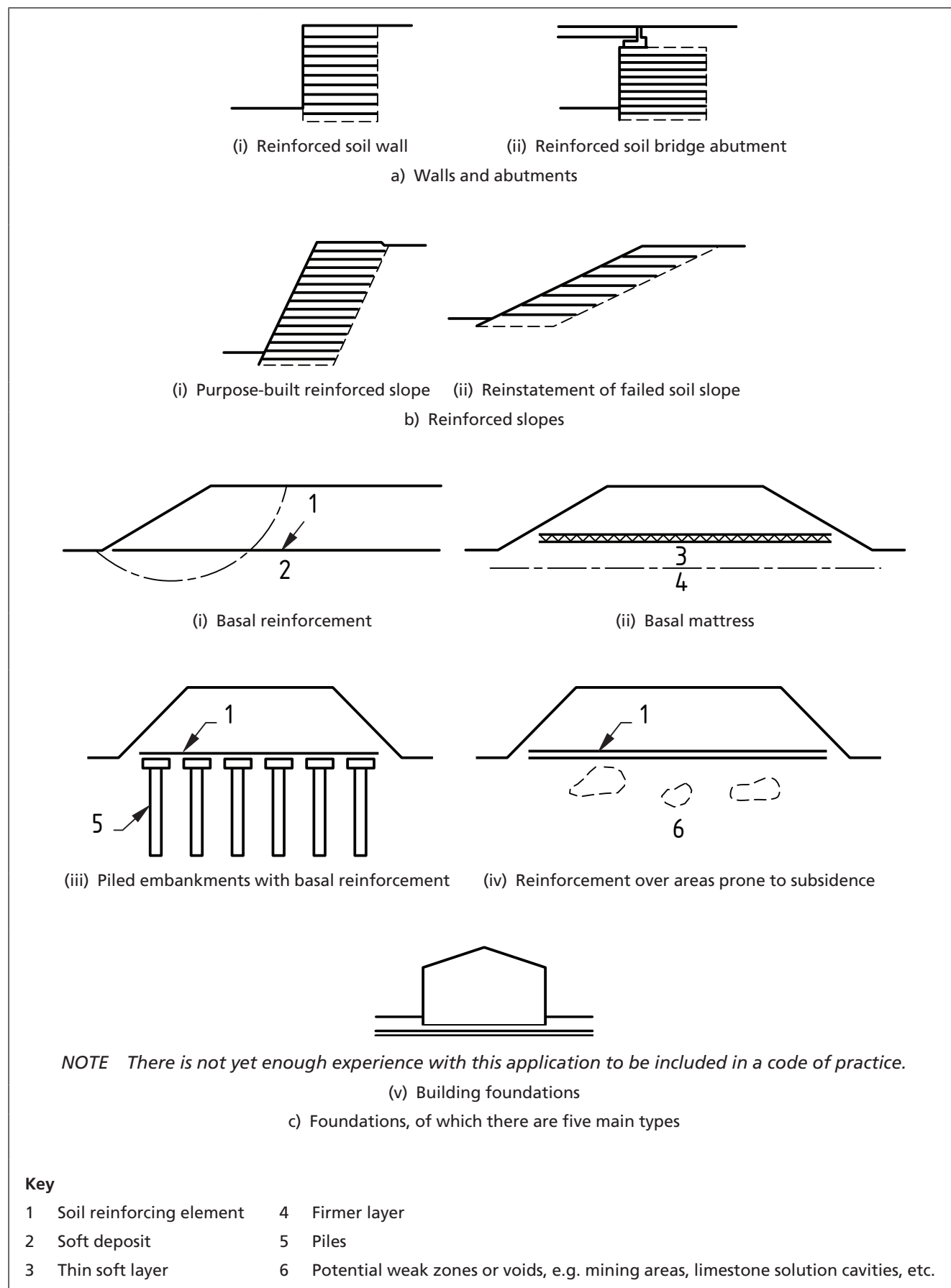
Limit state principles are applied to the design of reinforced soil walls, slopes and foundations to embankments or similarly resilient structures. The two limit states considered in design are the ultimate limit state and the serviceability limit state.

Ultimate limit states are associated with collapse or other similar forms of structural failure. These states are attained, for a specific mode of failure, when disturbing design effects equal or exceed restoring design effects. Margins of safety, against attaining the limit state considered are provided by the use of partial material factors, partial load factors and partial resistance factors. These partial factors assume prescribed numerical values of unity or greater.

Nominal loads are increased by multiplying by prescribed load factors, greater than unity for loads with a disturbing effect, to produce design loads. Material properties such as reinforcement capacity or soil properties are reduced by dividing by prescribed material factors (greater than unity) to produce design material properties. Resistances such as fill/reinforcement interaction or bearing capacity are modified by prescribed resistance factors (greater than unity).

Serviceability limit states are attained if the magnitudes of deformation occurring within the design life exceed prescribed limits or if the serviceability of the structure is otherwise impaired. Construction tolerances are subject to separate limits and are considered separately from any serviceability limit state. In assessing deformations or strains to determine compliance with the appropriate limit state, the prescribed numerical values of load factors are different to those used in assessing the ultimate limit state of collapse and usually assume a value of unity. In assessing magnitudes of total or differential settlements all partial factors are set to a value of unity, except for those pertaining to the reinforcements.

Figure 1 Range of applications of reinforced soil



2.3 Partial factors

Limit state design for reinforced soil employs four principal partial factors all of which assume prescribed numerical values of unity or greater. Two of these are load factors f_f (and f_{fs}) applied to dead loads and f_q applied to live loads. The principal materials factor is f_m (and f_{ms}). The fourth factor f_n is used to take account of the economic ramifications of failure. This factor is employed, in addition to the materials factor, to produce the reduced design resistance for the reinforcement material.

It is not feasible to uniquely define values for load or material factors. Prescribed ranges of these values are given to take account of the type of structure, the mode of loading and the selected design life and are considered in Section 6, Section 7 and Section 8 in relation to walls, slopes or embankment foundations respectively. In a particular application the load factors applied to dead loads and live loads can vary depending upon the load combination under consideration; in some circumstances the partial load factors for live loads may be set to zero to produce a worst case combination for the design load.

The material factors applied to shear strength are similarly prescribed. For a given application and design life, the material factor applied to soil reinforcement will assume a prescribed minimum value that will reflect the selected design life and type of reinforcement used. In contrast to load factors, reinforcement material factors are principal factors that can be broken down into various components and subcomponents that individually address each aspect of reinforcement strength. By definition each component or subcomponent has a value of unity or greater but the values may vary for different reinforcements.

Guidance is provided on how these material factors are determined. Partial factors are applied in a consistent manner to minimize the risk of attaining a limit state. In the case of the ultimate limit state of collapse, potential failure mechanisms will vary from one application to another and those pertaining to one application, such as a reinforced soil wall, might be different to another application such as a reinforced soil slope or reinforced soil foundation. Potential failure mechanisms to be considered are described in the particular sections of this code which address the analysis and design of walls, slopes and foundations.

Although potential failure mechanisms can vary from one application to another the ultimate limit state of collapse and the serviceability limit state of all reinforced soil applications should be considered in terms of both external and internal stability. The assessment of external stability involves consideration of the stability of the reinforced soil mass. In the case of a reinforced soil wall, for example, this would include assessment of potential failure modes such as forward sliding along the base of the wall. For each failure mode considered, prescribed load factors and material factors are appropriately applied to external disturbing forces and external restoring forces to ensure that the factored restoring force equals or exceeds the factored disturbing force. The overall, rotational or global stability of the reinforced soil mass has to be checked using slope stability procedures as described in BS EN 1997-1:2004, 11.5.1 [see Figure 22c)].

The internal stability of a reinforced soil mass is governed by the interaction between soil and reinforcement. This interaction occurs by friction, adhesion or anchoring. Where internal stability depends upon the shedding of load from the reinforcement to the soil, an appropriate

margin of safety is achieved by enhancing this load, by a load factor of prescribed value, and reducing the frictional, or adhesive and anchoring, parameters controlling the soil/reinforcement interaction by a material factor of prescribed value. Soil/reinforcement interaction also involves the transmission of load from soil to reinforcement. In addition to this load being a function of dead and live loads it will also be a function of the characteristics of the reinforcement and in particular the axial tensile stiffness and bending stiffness of the reinforcement. A margin of safety is achieved by enhancing this load by a load factor of prescribed value, and reducing the strength of the reinforcement using a material factor of prescribed value. Reinforcement design strength may be governed by an ultimate limit state of collapse or a serviceability limit state.

2.4 Design loads

Loads can be dead loads or live loads and these are assessed in an unfactored form. Consequently, if load is developed solely by the self weight of soil the load would be calculated using an unfactored characteristic value for the unit weight of the soil, the characteristic value being the cautious estimate of the value affecting the limit state.

The magnitude of disturbing loads, such as those which can be developed by lateral earth pressures, are controlled by many factors including pore water pressures and soil shear strength.

The numerical value of the calculated raw disturbing load, defined in terms of effective stress or total stress, is increased by multiplying by a prescribed load factor with a value of unity or greater. The end product of this factoring is the design load.

The magnitudes of design loads transmitted to a reinforcing element will be a function of prevailing dead and live loads. However, the magnitude, and nature, of the loads absorbed by the reinforcing element will also be affected by the physical properties of the reinforcing element.

2.5 Design strengths

A fundamental principle of limit state design is that the design strength should be equal to, or greater than, the design load.

In the case of external stability the design load is resisted by forces generated in the soil. Resisting forces will be a function of several variables including pore water pressure and soil shear strength; their characteristic values are determined as a cautious estimate of the value affecting the occurrence of the limit state. These are reduced by a material factor, of prescribed value, to produce the design strength. As with any other geotechnical problem, due account should be taken of any variation of soil shear strength with time over the selected design life.

The majority of reinforcing elements, such as strips, sheets and grids, have thicknesses that are small compared to their other dimensions. Such elements are flexible and, due to their low bending stiffness, can only resist axial tensile loads. The magnitude of the loads absorbed by reinforcing elements incorporated in compacted fill will be affected by the axial tensile stiffness of the reinforcing element. Where the design load can be sustained at a total axial tensile strain less than or equal to 1% the reinforcement is classified as inextensible and

the design load includes the effects of higher forces on a wall, or slope, as set out in Section 6 and Section 7. Where the design load is sustained at total axial tensile strains exceeding 1% the reinforcement is classified as extensible.

In considering the ultimate limit states of collapse of soil reinforced with flexible reinforcement, the design strength of the reinforcement may be determined by dividing the unfactored reinforcement base strength by a prescribed value of the partial material factor f_m .

The design strength employed may be dictated by considerations of a serviceability limit state rather than the ultimate limit state of collapse. If the economic consequences of failure are high, the derived design strength may be further reduced by dividing by a partial factor f_n to take account of these consequences. It follows that where the reinforcement design strength is governed by the ultimate limit state of collapse it is derived from the unfactored reinforcement base strength divided by the chain product $f_m \times f_n$.

2.6 Fundamental mechanisms

Soil has an inherently low tensile strength but a high compressive strength which is only limited by the ability of the soil to resist applied shear stresses. An objective of incorporating soil reinforcement is to absorb tensile loads, or shear stresses, thereby reducing the loads that might otherwise cause the soil to fail in shear or by excessive deformation. There is some similarity to the principle of reinforced concrete as the reinforced mass may be considered a composite material with improved properties, particularly in tension and shear, over the soil or concrete alone.

Although soil can be loaded under a compressive stress regime, tensile strains can develop within the soil mass. This is illustrated by the simple model in Figure 2 which shows a sample of dry sand confined by an externally applied compressive stress σ_3 and loaded by a compressive stress σ_1 where $\sigma_1 > \sigma_3$. Under this loading regime an unreinforced sample will suffer an axial compression δ_v and a lateral expansion, $\frac{1}{2}\delta_h$ [see Figure 2a)]. Clearly, this lateral expansion will be associated with the development of lateral tensile strains within the soil mass.

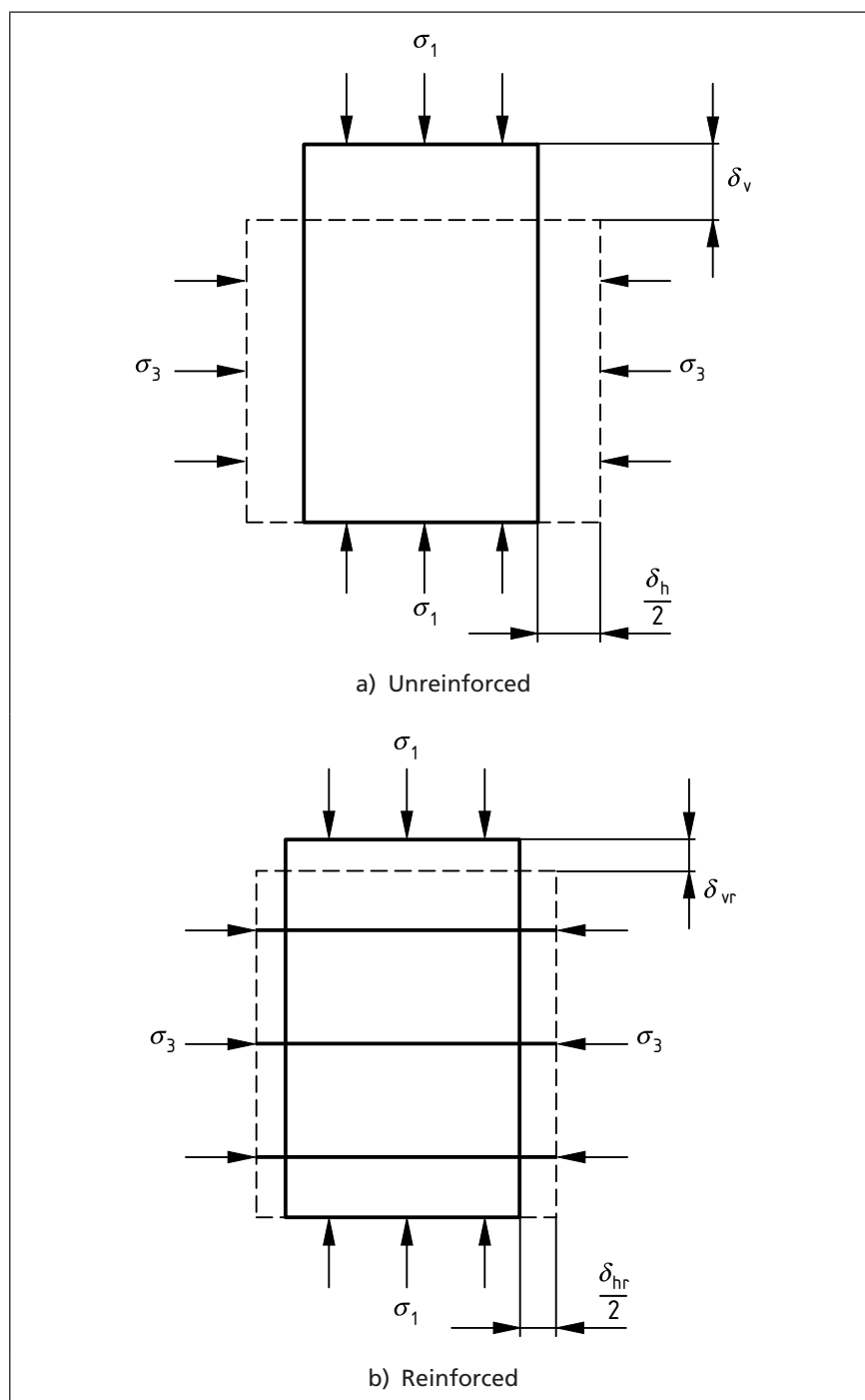
If several horizontal layers of reinforcement are inserted into the soil [see Figure 2b)] and the same external loads applied, then the resulting deformations are δ_{vr} and $\frac{1}{2}\delta_{hr}$ where $\delta_{vr} < \delta_v$ and $\frac{1}{2}\delta_{hr} < \frac{1}{2}\delta_h$. This reduction in the magnitudes of deformations is a direct result of an additional confining stress $\Delta\sigma_3$ generated by an internal interaction between the soil and the reinforcement. The factors involved in this interaction define the basic principles of reinforced soil.

When an axial load is applied to the reinforced soil [see Figure 2b)] this generates an axial compressive strain and a resulting lateral tensile strain. If the reinforcement has an axial tensile stiffness greater than that of the soil, then lateral movement of the soil will only occur if the soil can move relative to the reinforcement. Provided the surface of the reinforcement is sufficiently rough, movement of the soil, relative to the reinforcement, will generate shear stresses at the soil/reinforcement interface.

These shear stresses induce tensile loads in the reinforcement which are redistributed back into the soil in the form of an internal confining stress $\Delta\sigma_3$, which is additive to any externally applied confining stress, σ_3 .

The net external effect of this internal interaction is a reduction of both axial and lateral deformations compared to the unreinforced soil.

Figure 2 Effect of reinforcement on a soil element



In the above illustration, both reinforced and unreinforced samples are subjected to the same magnitudes of externally applied load and the addition of reinforcement decreases deformations compared to the unreinforced soil. [A1] Text deleted. [A1]

If the unreinforced soil is confined by a constant stress σ_3 and the magnitude of σ_1 is progressively increased, then the soil will be subjected to a progressively increasing shear stress, $\frac{1}{2}(\sigma_1 - \sigma_3)$. General shear failure of the unreinforced soil ensues as this applied shear stress approaches the shear strength of the soil.

When the soil is reinforced, a larger value of σ_1 is needed to cause failure. This is because increments of σ_1 induce increments of $\Delta\sigma_3$ which lead to relatively small increments in the applied shear stress, $\frac{1}{2}(\sigma_1 - [\sigma_3 + \Delta\sigma_3])$. A practical limit is imposed on the strength of the reinforced soil either by tensile rupture of the reinforcement or a bond failure caused by slippage at the soil/reinforcement interface.

2.7 Soil reinforcing mechanisms in walls and slopes

Figure 3 shows a steep slope in a dry cohesionless soil with a face inclined at β_s to the horizontal, where β_s is greater than the internal angle of shearing resistance. Without the benefit of soil reinforcement the slope would collapse, however, by the incorporation of suitable soil reinforcement the slope may be rendered stable. Investigations of the basic reinforcing mechanisms reveal that the soil in the slope comprises two distinct zones. These are shown in Figure 3 as the active zone and the resistant zone. Without reinforcement the active zone is unstable and tends to move outwards and downwards with respect to the resistant zone.

If soil reinforcement is installed across the active and resistant zones it can serve to stabilize the active zone. Figure 3 shows a single layer of reinforcement with a length L_{aj} embedded in the active zone and length L_{ej} embedded in the resistant zone. A practical reinforcement layout would contain multiple layers of reinforcement; however, the single layer shown in Figure 3 is adequate to illustrate the basic mechanisms involved.

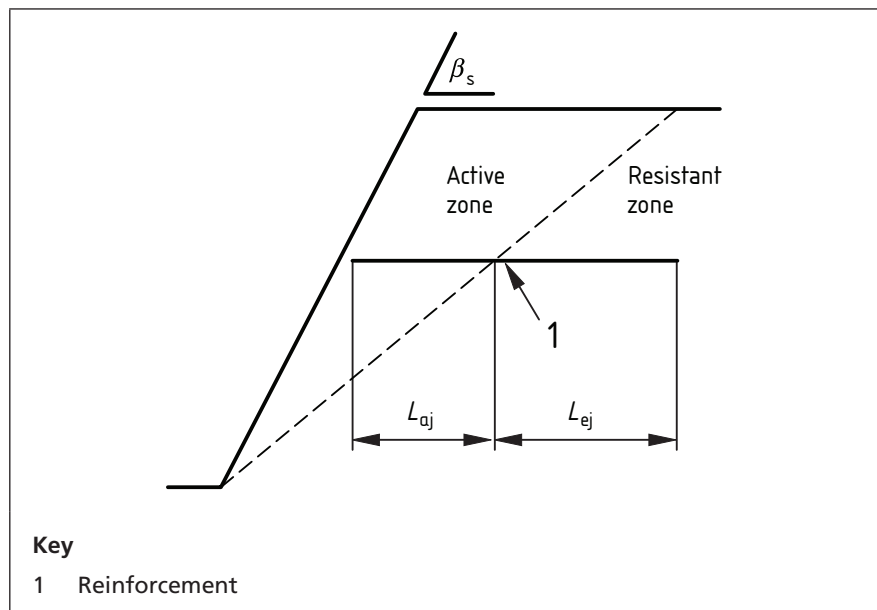
The precise reinforcing mechanism will be affected by the properties of the reinforcement. Flexible reinforcements provide stability to a reinforced mass of soil by transferring destabilizing forces from the active zone to the resistant zone where they are safely absorbed. In this process purely axial tensile loads are resisted by flexible reinforcement.

Provided the reinforcement develops an adequate bond, and has adequate tensile stiffness, it will absorb tensile strains developed in the soil in the active zone. These tensile strains are transferred from the soil, to the reinforcement, through the mechanism of soil/reinforcement bond. The tensile strains developed in the reinforcement in the active zone give rise to a corresponding tensile force in the reinforcement in the active zone.

If the total length of the reinforcement is limited to L_{aj} (see Figure 3) then the transfer of load from soil to reinforcement in the active zone would not prevent collapse of the active zone. To achieve this, the reinforcement extends a length L_{ej} into the resistant zone. Provided the reinforcement has sufficient tensile strength to sustain tensile loads absorbed from the active zone it will shed these into the soil in the resistant zone. As in the active zone, load is transferred from reinforcement to soil by the mechanism of soil/reinforcement bond. The tensile load in the reinforcement over the length L_{ej} is not constant but decreases towards the free end of the length L_{ej} remote from the slope face, as load is shed into the soil. At the free end of the reinforcement in the resistant zone the tensile load in the reinforcement is zero.

Flexible reinforcement is incorporated in fill during construction. Consequently, the layers of reinforcement are generally horizontal.

Figure 3 Reinforcing mechanisms in walls and slopes



2.8 Soil reinforcing mechanisms in embankment foundations

As well as applications involving the reinforcing of walls and slopes, soil reinforcement can also be used to improve the performance of foundation soils supporting flexible constructions such as embankments. Where soil reinforcement is used to carry vertical loads on elements spanning between pile caps [Figure 1c)iii)], or over weak zones and voids [Figure 1c)iv)], the reinforcement deforms in a manner similar to the main cables of a suspension bridge, to serve as a tensile structural element which supports the imposed vertical load.

In other foundation applications, soil reinforcement may be employed as an expedient to improve short- and intermediate-term margins of safety. The most common example of this is embankments constructed over weak cohesive foundation soils, see Figure 1c)i), and Figure 1c)ii). Although the chosen geometry of an embankment is consistent with long term stability once the foundation soil has consolidated under the imposed embankment load, the same geometry can give rise to instability, in the short and intermediate term, prior to consolidation of the foundation soil. In the foundation soil beneath an embankment, the principal tensile strain direction is horizontal. In the unreinforced condition this indicates a potential for the embankment, and the foundation soil, to fail by lateral spreading.

This mode of failure can be prevented by introducing a horizontal reinforcing layer, or mattress at the base of the embankment. Consequently, this style of soil reinforcement is often referred to as basal reinforcement.

Lateral extrusion and spreading failures can be prevented by the inclusion of horizontally inclined reinforcement at the interface of the embankment fill and the foundation soil. Provided that the reinforcement exhibits adequate surface roughness, tensile strength

and axial tensile stiffness, then horizontal shear stresses developed at the base of the embankment fill will be transferred to the reinforcement, by soil/reinforcement friction, so inducing tensile load in the reinforcement. This transfer has a twofold effect. Firstly, it limits the development of tensile strains at the base of the embankment fill, thereby inducing a lateral confinement. Secondly, shear stresses which would otherwise be transmitted directly to the weak foundation soil are intercepted by the reinforcement. This reduces the potential for lateral extrusion of the weak foundation soil and maintains the bearing capacity of the foundation soil which would otherwise be reduced by transmission of shear stresses.

Where the weak foundation soil is of limited depth, and underlain by a competent stratum, lateral movement of fill or foundation soil is the critical potential failure mode. However, where the foundation soil is deep, with a depth typically greater than one third of the final embankment height, then, additionally, consideration should be given to deep seated rotational failure involving the embankment fill and foundation soil. Basal mattress reinforcement has been shown to influence the shape of the failure surface by causing it to rotate and be driven deeper thus lengthening its path (see Williams and Sanders [2]).

The tensile load induced in the basal reinforcement gives rise to a restoring moment about the centre of the slip circle under consideration. The effect of this is to partially absorb the disturbing moment which might otherwise cause failure of the embankment.

Once this degree of consolidation is achieved the basal reinforcement becomes redundant. Thus, the design life of the reinforcement is only the time needed for the foundation soil to attain a necessary degree of consolidation. The design strength of the reinforcement will be governed by either the tension needed to provide lateral stability of both the fill and foundation soil, or the tension necessary to provide rotational stability of both the fill and foundation soil.

2.9 Soil reinforcement interaction

For soil reinforcement to be effective it should interact with the soil to absorb the stresses and strains which would otherwise cause the unreinforced soil to fail. The precise mechanisms by which this interaction occurs will be affected by the characteristics of the soil, be it fill or natural ground, the characteristics of the reinforcement, and the relationships between these two sets of characteristics.

In the context of this code, two criteria are applied to define failure. The first is the ultimate limit state of collapse, see Section 6. In terms of the interaction between soil and reinforcement this state can be brought about by either rupture of the reinforcement or a failure of the bond between the soil and the reinforcement. The second is a serviceability limit state, which occurs when the in-service deformations of the reinforced mass, or strains within the reinforcement, exceed prescribed limits.

Although characteristics of the reinforcement will vary slightly according to the manner in which the reinforcement interacts with the soil, certain characteristics are fundamental. Since loads are transferred from soil to reinforcement by relative movement between these two components it is essential that the reinforcement is axially stiff in comparison to the

soil. The load transfer mechanism from soil to reinforcement, or from reinforcement to soil, is by soil/reinforcement bond. Where the soil is cohesionless this bond resistance will be frictional and will be a function of the soil, reinforcement and its surface roughness.

In cohesive soils the bond stress is adhesive. Interlocking can develop between soil particles and the apertures of grid reinforcement. In this case, bond may be controlled by soil-to-soil shear strength at some small distance away from the soil/reinforcement interface. The magnitude of bond stress will be governed by the relative properties of the soil and reinforcement, i.e. the shear strength of the soil and the roughness of the reinforcement (see Ingold [3], and Ingold [4]).

If the reinforcement is sufficiently stiff and rough it will absorb load from the soil without invoking a serviceability failure. Having absorbed load it is necessary for the reinforcement to sustain this load, over the selected design life, without rupture, which would constitute an ultimate limit state of collapse, or without suffering time dependent deformations that might give rise to a serviceability failure. In certain applications, such as walls and steep slopes, load will be transferred to the reinforcement from soil in the active zone near the face of the wall or slope. To effect internal stability of the reinforced soil mass, all, or part, of this load will be transferred, by the reinforcement, into the stable, resistant zone of soil behind the active zone (see Figure 3). For this transfer to be effective there is again a need for adequate reinforcement axial stiffness and roughness.

Flexible reinforcing elements are not deemed to reinforce the soil by any interaction involving bending or shear across their cross-sectional area. Flexible reinforcements interact with the soil by absorbing axial tension only. As a construction expedient, and to maximize their tensile load carrying capacity, flexible reinforcements are installed horizontally, in walls, slopes and beneath embankments, to coincide with the principal tensile strain axis within the unreinforced soil. The axial forces absorbed by a flexible reinforcement are statically determinate. Consequently the design problem with respect to internal stability, for wall, slope, and basal embankment reinforcement, reduces to one of determining axial tensile forces, absorbed by the reinforcement in the active zone, and their distribution into the resistant zone, within the limits imposed by the ultimate limit state of collapse and prescribed serviceability limits. With respect to internal stability, the overall design objective for axially stiff reinforcement is again to arrange a secure distribution of stresses and strains from the active zone to the resistant zone. This will involve axial tensile force but may additionally include the effects of bending and shear developed in the reinforcement. In this case forces may be analysed in terms of moments rather than axial forces.

2.10 Soil properties to be considered

The performance of a reinforced soil mass will be dependent on the characteristics of the soil and how these are affected by internal environments such as pore water pressure regimes and external environments such as imposed loads. These, and other factors, are summarized in Table 1.

Soil is characterized by a number of factors which are considered in Section 3, and Sections 6, Section 7 and Section 8 which deal with the design of walls, slopes and embankment foundations respectively.

Two basic sets of characteristics to be considered are those which affect the loads imposed on the soil reinforcement and those which affect the durability of the reinforcement. The loads transferred from the soil to the reinforcement will be directly affected by the shear strength of the soil. In general soil shear strength is defined by the effective shear strength parameters c' and ϕ' or the undrained shear strength parameter c_u . Effective cohesion c' is only applied to cut slopes formed in overconsolidated clays. The effective cohesion of overconsolidated clays decreases with time, and the value used in design is to be based upon that value prevailing at the end of the selected design life. Small, constant, values of effective cohesion may be applied to certain fills derived from industrial wastes and the values to be employed are established in Section 3.

In particular, but with the exceptions cited above, the shear strength of fill or soil incorporating multiple layers of reinforcement [see Figure 1 a) and Figure 1 b)] is considered to be purely frictional. Since the strains induced in multiple layer reinforcement such as walls and slopes are small, the frictional strength is represented by ϕ_p , the peak effective angle of internal shearing resistance. The shear strength of fill of embankments resting on foundation soil reinforced by horizontal basal reinforcement is also considered to be purely frictional. For some types of structures, however, larger strains may sometimes need to be allowed for in design. An example would be an embankment subject to differential settlement. In these cases the frictional strength is represented by large strain values. For cohesionless soils this is ϕ'_{cv} , the value when the soil shears at constant volume. Large strains can also be mobilized in the basal reinforcement, the frictional strength of the fill being represented by ϕ'_{cv} . This value is used for short, intermediate and long term analysis with due regard for pore water pressures.

Table 1 Factors affecting performance

Reinforcement	Soil	Construction
Composition	Particle size and shape	Construction system
Durability	Grading	Compaction
Form	Index properties	Handling
Surface properties	Mineral content	
Dimensions	Durability	
Strength		
Stiffness		
Creep under load		
Reinforcement distribution	Soil states	Structure
Location	Density	Geometry
Spacing	Confinement	End use
Orientation	State of stress	Foundation conditions
	Degree of saturation	
	Drainage	

Where embankments are constructed over weak deposits of normally consolidated clays, of low to medium compressibility, short term analysis is based on the undrained shear strength of the foundation soil, c_u . If the foundation soil is not fully saturated, or is fissured by desiccation,

then it may be found that $\phi_u \neq 0$. In this case the values of undrained shear strength c_u ought to be consistent with the magnitudes of total stress operating beneath the embankment. Alternatively short term analyses may be based upon ϕ'_p with due regard for the short term pore water pressure regime. Intermediate and long term analyses may again be based upon ϕ'_p but with regard for the prevailing pore water pressure regime. In the long term, when excess pore water pressures have dissipated, the basal reinforcement becomes redundant.

In walls and slopes, the loads imposed on the soil reinforcement will be increased if positive pore water pressures are allowed to develop. The development of adverse pore water pressures in reinforced fill walls and slopes can be prevented by the installation of appropriate drainage. In natural soil slopes, or waterfront constructions, the development of positive pore water pressures can be unavoidable. In this case it is essential that due account be taken of increased reinforcement loadings and the range of any fluctuations in these loads occasioned by seasonal or tidal variations in pore water pressures.

In addition to the physical interaction between soil and reinforcement, through the agency of soil/reinforcement bond, there can also be chemical and electrochemical or thermal interactions. Over the design life of the reinforcement its durability, and therefore mechanical performance, can be adversely affected by the fill environment. The effects of different fill environments will vary for different reinforcing materials. For a given soil reinforcement the design process includes the identification of potentially aggressive fills which should be specified as unsuitable. Aggressive ground water or surface water entering the structure may also have to be considered.

2.11 Reinforcement geometry

The performance of a reinforced soil mass will be affected by the characteristics of the reinforcement and how these are affected by internal environments, such as chemistry and temperature, and external environments such as imposed loads and climatic conditions. These, and other factors, are summarized in Table 1.

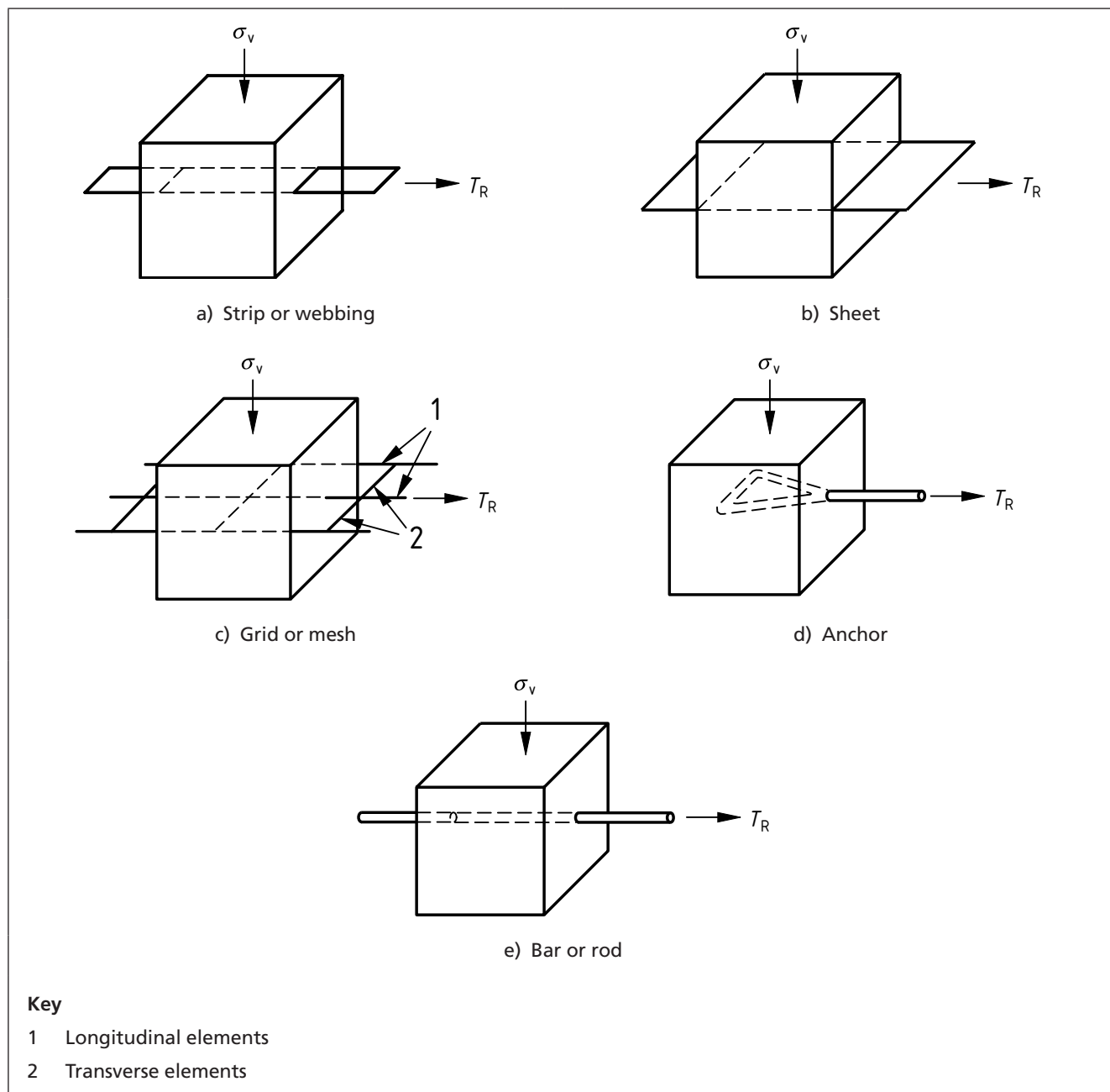
Soil reinforcement can take a variety of forms, some of which are shown in Figure 4. Grids, meshes and strips can be metallic or polymeric whilst sheet reinforcement takes the form of polymeric geotextiles. Anchored earth fill employs multiple layers of flexible steel bars or polymeric materials, which are shaped, at the end remote from the face of the wall, to form an anchor.

Sheet reinforcement and polymeric grids are generally installed full width, such that each metre length of face is associated with a one metre width of reinforcement, and so, in a multilayer system, the total stabilizing force developed by the reinforcement is a function of the number of layers of reinforcement and their vertical spacings. Strip reinforcement, including wide strips of metallic or polymeric grid, are not placed full width. Consequently, the total stabilizing force developed by such reinforcement will be a function of the number of reinforcing elements and both their horizontal and vertical spacings.

The total length of each reinforcing element will influence the overall geometry of the reinforced mass and this in turn will influence external stability. For example, in the case of a reinforced fill wall, the length of the reinforcing elements at the base of the wall determine the width of the base of the wall and therefore affect the performance of the

reinforced mass with respect to forward sliding on the base, bearing, tilting, settlement and overall stability.

Figure 4 Forms of reinforcement



2.12 Reinforcement bond

Within the reinforced mass, differentiation is made between that part of the length of the reinforcement which falls in the active zone and the remaining length which falls in the resistant zone. The length of reinforcement in both the active and resistant zones, and its bond characteristics, govern the loads that can be carried.

The length of reinforcement falling in the resistant zone is the bond length, or embedment length, which enables transmission of forces from the active zone to the resistant zone. In the case of a flexible reinforcement, where the force in the reinforcement is axial tension,

the bond length ought to be sufficient to prevent the reinforcement being pulled out of the resistant zone.

In addition to being a function of the embedment length, bond performance will be affected by the shear strength of the soil, any pore water pressures within the soil, and the characteristics of the reinforcement. If the bond resistance of reinforcement is greater than its tensile strength then the ultimate limit state is controlled by tensile rupture of the reinforcement. Conversely, if the tensile load in the reinforcement is greater than the bond resistance the ultimate limit state will be controlled by bond failure, see 2.9.

Shear stresses are not uniformly mobilized along the embedment length but are, among other things, a function of the axial tensile extensibility of the reinforcement.

In assessing bond performance for the purpose of design, shear stresses developed between the soil and the reinforcement are assumed to act uniformly over the embedment length. In frictional fills, the magnitude of these shear stresses is taken to be the product of the vertical effective stress acting on the reinforcement and the tangent of the angle of bond stress δ' . The angle of bond stress is determined by direct shear testing referred to in Section 4. For grid reinforcement the boundary conditions in the direct shear test may give rise to a measured value of δ' greater than ϕ'_p (see Ingold [3]). Such a value cannot be realized in the field and the maximum value of δ' used in design can not exceed ϕ'_p .

Pull-out resistance is calculated to be the product of the surface area of the reinforcement, along the bond length, $\tan \delta'$ and the vertical effective stress operating on the bond length. The value $\tan \delta'$ is represented by the coefficient μ . For full width reinforcement, the maximum possible value of vertical effective stress, disregarding any surcharge, is taken to be the overburden pressure above the reinforcement. For narrow, rough, strip reinforcement, embedded in dense cohesionless fill, the shear stresses developed during pull-out lead to dilatancy of the fill which causes the vertical effective stress to locally rise above the overburden pressure. This gives rise to an enhanced pull-out resistance which is modelled by μ^* , where $\mu^* > \mu$ and μ^* is determined by field pull-out tests (see Schlosser and Elias [5], Schlosser and Guilloux [6]). For cohesive foundation soils, subject to undrained loading, the bond stress developed between the soil and the reinforcement is directly related to the undrained shear strength of the soil by an adhesion factor, a_{bc} , which cannot assume a value greater than unity (see Schlosser and Elias [5]).

2.13 Effects of flexible reinforcement axial stiffness on loads

Flexible reinforcement operates in axial tension only and an ultimate limit state of collapse may be attained by a bond failure or by tensile rupture of the reinforcement. The tensile rupture mode will be controlled by the magnitudes of the loads imposed on the reinforcement and the ability of the reinforcement to sustain these loads without rupture occurring within the selected design life.

Methods used for the calculation of reinforcement loads vary from application to application and are set out in the sections dealing with design. For flexible reinforcement, installed in filled walls and

slopes, the imposed load will be related to active earth pressure. In the case of an un-surcharged vertical wall active earth pressure is the product of vertical effective stress and the coefficient of active lateral earth pressure K_a . Faced walls are assumed to be smooth and consequently K_a takes the familiar value $(1 - \sin\phi'_p)/(1 + \sin\phi'_p)$. Field observations of walls reinforced with flexible, inextensible strips have established that reinforcement loads in the upper sections of the walls are substantially higher than those consistent with K_a earth pressures (see Schlosser [7], Jones and Sims [8]).

Analyses of such walls have shown that although walls move during construction, consistent with the development of K_a , earth pressures higher than active are induced by the effects of compaction (see Ingold [9]). In the upper reaches of a wall these can be modelled by the coefficient of lateral earth pressure at rest K_o (see *Les Ouvrages en Terre Armée* [10]). There is no accumulation of field observations to suggest that reinforcements with lower axial tensile stiffness attract these additional forces. Consequently for the purposes of calculating lateral earth pressures and thus reinforcement loads a differentiation is made between extensible and inextensible reinforcement. Where the design load can be sustained at an axial tensile strain less than or equal to 1%, the reinforcement is defined as inextensible and the design load will include the effects of the higher lateral earth pressures developed in the upper sections of the wall, or slope, as set out in Section 6 and Section 7. All other reinforcements are defined as being extensible and in the upper reaches of the wall, design loads are based on the assumption of an active earth pressure distribution. However, where field observations of any given reinforcement defined as extensible record the generation of lateral earth pressures higher than active, then this reinforcement should be designed to sustain higher loads based on a K_o pressure distribution in the upper reaches of the wall or slope.

2.14 Factors affecting tensile behaviour of flexible reinforcement

A margin of safety against attaining the ultimate limit state of collapse due to tensile rupture of the reinforcement is obtained by increasing unfactored loads by a partial load factor to produce the design load and decreasing the reinforcement base strength by a partial material factor and, where appropriate, a partial factor to allow for the ramifications of failure, to produce a design strength. Provided the design strength is equal to or greater than the design load then an adequate margin of safety is deemed to operate. If the design strength based on tensile rupture is assessed to give rise to strains or deformations that exceed prescribed limits for the serviceability limit state then a lower design strength consistent with the serviceability limit state should be used.

The design strength to be calculated is that strength prevailing at the end of the design life of the reinforcement. In the case of basal reinforcement to embankments constructed over weak foundation soils the design life of the reinforcement might be quite short and ceases when the foundation soil has consolidated sufficiently to support the embankment without the need for reinforcement. In the case of walls and slopes, the design life of the reinforcement is identical to the design life of the wall or slope. Design lives can vary

from a few months or years, up to 120 years. Over the design life of a wall or slope the tensile rupture strength of the reinforcement will decrease with time through various agencies of degeneration. Different reinforcing materials are degraded by different agencies but all will be affected by the passage of time. Although rates of degradation vary with time, the general rule is that tensile rupture strength decreases with increasing time.

In the case of metallic reinforcement, which in practice is predominantly galvanized steel or ungalvanized steel, the sole agency of degradation is electrochemical corrosion. Irrespective of the properties of the soil in which it is installed, all metallic reinforcement is subject to electrochemical corrosion. Some soils and fills are more corrosive than others and therefore rates of corrosion are higher in the more aggressive soils.

Loss of reinforcement cross-sectional area due to corrosion causes a time dependent reduction in tensile rupture load and consequently the rupture load used as the basis for the design strength will be that determined to prevail at the end of the selected design life. The grades of metal used for soil reinforcement are assumed to exhibit tensile rupture stresses which are independent of time. Consequently the effects of corrosion are taken into account by allowing prescribed sacrificial thicknesses which vary according to the selected design life, the corrosivity of the fill or soil and the particular metal used for the reinforcement.

Due to their geometry and irregular cross-sectional area, the loads developed in polymeric reinforcements can not be conveniently defined in terms of stress. Consequently for sheet reinforcement, such as grids, meshes and geotextiles, load is defined per unit width, e.g. kN/m. For narrow strips, load is defined per strip. All polymeric materials are visco-elastic and, as a direct consequence of this, the load which causes tensile rupture will be a function of several variables. These are considered in detail in following sections; however, the two variables which affect all polymeric materials are time and temperature. When a constant, sustained tensile load is applied to a polymeric material of regular, solid cross section it will immediately induce a tensile stress and a tensile strain in the material. This strain will continue to increase with time through the agency of creep. As a corollary to this, the load bearing cross-sectional area decreases with time and so the tensile stress increases with time. At some time, after initial loading, the time dependent stress can increase to a critical value that results in tensile creep rupture of the material. When a lower load is applied to a sample of the same material, at the same temperature, then tensile creep rupture occurs at a longer time after initial loading. Conversely when a higher load is applied tensile creep rupture occurs at a shorter time after initial loading.

When a polymeric material is subjected to a constant tensile load, at a constant temperature, it will fail by tensile creep rupture after a certain time. If the same material is subjected to the same load, but at a higher temperature, it will fail in a shorter time. Conversely, loaded at a lower temperature, tensile creep rupture will occur after a longer time. The relationships between time to failure and temperature are nonlinear and non-reversible. For the purposes of design it is necessary to know the tensile creep rupture strength of the reinforcement prevailing at the end of the selected design life under the maximum operating temperature. The design strength, based on

tensile creep rupture strength, is obtained using the partial material factors given in Annex A.

The satisfactory performance of a reinforced soil mass might be governed by deformation (a serviceability limit state) rather than an ultimate limit state. Consequently if a load equal to a design strength based on tensile creep rupture gives rise to strain which exceeds a prescribed serviceability limit then the design strength is reduced to that which conforms to the serviceability limit. Limits are prescribed for construction tolerances for walls and similarly serviceability limits are applied to the axial tensile strains which may be allowed to develop in basal reinforcement to embankments constructed over weak ground. Development of post-construction strains in wall reinforcement can lead to wall deformations which are unsightly or, as in the case of a reinforced soil bridge abutment, render the structure unserviceable. To prevent this serviceability limits are prescribed for walls and abutments in terms of axial tensile strain increments which are not to be exceeded between completion of construction and the end of the selected design life.

Post-construction strain can be caused by increased loading such as a bridge deck applied to an abutment and in the case of polymeric materials post-construction strain will result from creep strain under constant load. For polymeric materials design strength may be limited by considerations of a serviceability limit or the ultimate limit state of collapse. Consequently both design strengths should be evaluated and used in the appropriate limit state calculations to determine which is the more critical.

Section 3: Materials

COMMENTARY ON SECTION 3

The main materials within reinforced soil structures are fill, reinforcing elements, facings and connections. This chapter addresses each of these items in respect of reinforced soil structures (walls/abutments), reinforced slopes and reinforced foundations.

Table 2 provides the summary references to each relevant component of the main materials. The following clauses provide additional details for the main reinforced soil materials. Reference has been made to the European execution standard on reinforced soil – BS EN 14475:2006.

Table 2 Summary references to the relevant component of the main materials within reinforced soil walls, abutments and slopes

	Walls and abutments	Steep slopes ($\leq 70^\circ$, $> 45^\circ$)	Shallow slopes ($\leq 45^\circ$)	Foundations	Comments
Fill materials					
Class or type	Selected granular fills (Classes 6I/J), Selected cohesive fills (Classes 7B/C/D) and Chalk (see Note 3)	Selected granular fills (Classes 6I/J), Selected cohesive fills (Classes 7B/C/D), General granular fill (Class 1), General cohesive fill (Class 2) (see Note 4) and Chalk (see Note 3)	Selected granular fills (Classes 6I/J), Selected cohesive fills (Classes 7B/C/D), General granular fill (Class 1), General cohesive fill (Class 2), and Chalk (see Note 3)	A1 Selected granular fills as specified in SHW A1	Fill classes as SHW Table 6/1. Class 7B not permitted for steel reinforcing elements. Specific drainage measures may be required with cohesive fills. See also 3.1.3.2 and 3.1.3.3.
Permitted constituents	SHW Table 6/1	SHW Table 6/1	SHW Table 6/1	SHW Table 6/1	
Grading	SHW Table 6/1	SHW Table 6/1	SHW Table 6/1	SHW Table 6/1	Grading not applicable to chalk
Shear strength (ϕ) test	BS 8006-1 4.2.1	BS 8006-1 4.2.1	BS 8006-1 4.2.1	BS 8006-1 4.2.1	SHW, cl 636
Friction (μ) test	BS 8006-1 4.3.3	BS 8006-1 4.3.3	BS 8006-1 4.3.3	BS 8006-1 4.3.3	SHW, cl 639
Compaction	SHW Table 6/1 and Table 6/4	SHW Table 6/1 and Table 6/4	SHW Table 6/1 and Table 6/4	SHW Table 6/1 and Table 6/4	
Chemical	BS EN 14475:2006 Table B.1 for steel elements	BS EN 14475:2006 Table B.1 for steel elements	BS EN 14475:2006 Table B.1 for steel elements	BS EN 14475:2006 Table B.1 for steel elements	A1 See Note 1. A1 For polymeric, see appropriate proprietary product certification.
Soil reinforcements					
Steel types and specifications	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	See Table 3 for design properties of some materials listed in BS EN 14475:2006. See Table 4 for corrosion allowances
Polymeric specifications	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	See appropriate proprietary product certification for design properties

Table 2 Summary references to the relevant component of the main materials within reinforced soil walls, abutments and slopes (*continued*)

	<i>Walls and abutments</i>	<i>Steep slopes ($\leq 70^\circ$, $> 45^\circ$)</i>	<i>Shallow slopes ($\leq 45^\circ$)</i>	<i>Foundations</i>	<i>Comments</i>
Facing materials					
Concrete	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	
Steel	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	
Polymeric	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	

NOTE 1 Polymeric materials are not normally influenced by electrochemical action but can be affected by certain chemicals. The influence of chemicals on proprietary polymeric materials ought to be considered in the product certification procedures, see Section 4.

NOTE 2 The use of secondary and recycled aggregates might be appropriate for some reinforced soil applications. Consideration ought to be given to the use of such materials and detailed guidance can be found in the Highways Agency document HD35/04 [11]. The requirements for reinforced soil fill materials as set out in the above table also apply to secondary and recycled materials.

NOTE 3 Information on chalk fill is given in 6.10.2.5.

NOTE 4 See BS EN 14475, Annex A and Annex B.

SHW = Specification for Highway Works [1]

3.1 Soils and fills

3.1.1 General

In reinforced soil walls and abutments acting as earth retaining structures, consideration should be given to both the properties of the retained fill and of the selected fill forming the reinforced soil mass. Similar considerations should be given to the use of fill in slopes. Less stringent conditions may be applied to the fill in the reinforced zones of reinstated or flatter slopes, and reinforced foundations.

NOTE Further guidance on the use of various combinations of fill types/reinforcing elements and systems can be found in BS EN 14475:2006 Annex A.

3.1.2 Electrochemical criteria

The limits of the electrochemical properties of soil fill with respect to steel metallic reinforcements are detailed in BS EN 14475:2006, Table B.1.

3.1.3 Non-standard fills

3.1.3.1 General

Fills meeting the mechanical and chemical limitations given in BS EN 14475:2006, Table B.1 should be used. However greater frequency of testing should be carried out to maintain adequate quality control when non-standard fills are used.

3.1.3.2 General cohesive fill

General cohesive fill, as defined in *Specification for Highway Works* [1] should not be used in the construction of reinforced soil walls or abutments for permanent works, either faced or unfaced. General cohesive fill, as defined in *Specification for Highway Works* [1] may be used with caution in the construction of steep slopes (see BS EN 14475).

3.1.3.3 Pulverized-fuel ash (PFA)

Only conditioned hopper ash should be used as PFA fill class 7B conforming with *Specification for Highway Works* [1]. Special anti-corrosion measures should be followed with PFA fill as detailed in **6.10.2.6.3**, and special drainage measures as in **6.10.5.3** (see Boot [13]).

NOTE PFA is generally self-hardening and its shear strength parameters, as determined by BS 1377-7 and BS 1377-8, are time dependent.

Fresh samples of PFA should be prepared and tested to assess the strength parameters relevant to the construction state. The value of c' for PFA used in design should be limited to $\leq 5 \text{ kN/m}^2$. The unit weight of PFA used in design should be obtained from the intended source. A check should be made at the construction stage to confirm that the unit weight does not exceed the design value.

3.1.3.4 Colliery spoil

COMMENTARY ON 3.1.3.4

Colliery spoil is the waste from the mining of coal normally deposited in large tips. The most common rock types found in colliery spoil are mudstones, siltstones, shales, sandstones and, in some areas, limestones.

The properties of colliery spoil vary considerably both within a tip and from tip to tip and its suitability in reinforced soil should be the subject of specific testing and assessment. Argillaceous colliery waste arising from coal production should be well burnt in order to be permitted in permanent reinforced structures.

Some materials may be labelled as colliery spoil even though they meet all the chemical and mechanical characteristics set out in BS EN 14475:2006, Table B.1. Such materials may include overburden materials won from the vicinity of mine workings. In such cases these materials may be suitable for use in reinforced soil structures and should not be excluded simply as a result of being labelled unburnt colliery spoil.

3.1.3.5 Argillaceous materials

The chemical characteristics of argillaceous materials such as clay shales used as fill for reinforced soil should be assessed to ensure consistency and compatibility with the reinforcing elements. Argillaceous materials used as fill may contain carbonaceous material and pyrite, often in localized concentrations.

COMMENTARY ON 3.1.3.5

Pyrite can oxidize and the process can generate heat which results in a rise in soil temperature. Oxidation of Pyrite produces soluble sulfates. These have the potential to raise pH, which may be detrimental to concrete, metallic reinforcements and certain polymeric compounds. Sulfates can also chemically attack concrete with a resulting loss of strength.

Polymeric reinforcements can be influenced by the increase in operational temperature caused by any exothermic reaction.

Some argillaceous materials can also be friable and breakdown under the effects of long term pressure from the weight of fill above and moisture within the fill causing significant compression of the fill. This can affect superimposed structures and it is also particularly detrimental in structures with facings that settle less and cause increased and unexpected loads at the connections of reinforcing elements and facing.

The use of argillaceous materials in permanent structures is not accepted by some authorities including the Specification for Highway Works [1] because of concern for the durability of the reinforcing elements and because the needs for testing and assessment are greatly increased over other materials (see West and Aerial [14], Rainbow [15]).

3.1.3.6 Chalk

Chalk used in reinforced soil structures should conform to the requirements in 6.10.2.5.

3.1.3.7 Friable materials

Friable soils, for example those which are susceptible to degradation by water and pressure over time, should not be used in reinforced soil structures.

3.2 Reinforcing materials

COMMENTARY ON 3.2

Reinforcing elements are made from materials that have a resistance to degradation when buried. The reinforcement may take the form of sheets, grids, meshes, strips, bars, rods, etc. that are capable of sustaining tensile loads and the effects of deformation developed in the fill. BS EN 14475:2006 provides examples of various common reinforcing materials.

3.2.1 Metallic soil reinforcements

3.2.1.1 General

Metallic soil reinforcements are made from materials having a certain resistance to corrosion when buried and may take the form of sheets, grids, meshes, strips, bars, rods, etc. that are capable of resisting tensile loads and the effects of deformation developed in the fill.

Steel reinforcement material should conform to the criteria described in BS EN 14475:2006. Table 3 gives design properties of some steels given in BS EN 14475:2006.

Reinforcing elements should not be less than 1.5 mm in thickness. Anchor elements should not be less than 10 mm in diameter, or, in the case of non-circular bar, equivalent diameter. The rupture strength used in the design of anchored earth should be based on either the strength of any welded connection between an anchor and anchor shaft, or of the anchor shaft, whichever is the lesser. When threaded end connections are used, the cross-sectional area of the anchor shaft should be based upon the tensile stress area.

Steel strips, rods, bars, ladders or welded wire meshes may be provided with a galvanized coating. The galvanizing shall conform to BS EN ISO 1461 with a local coating thickness of 70 µm.

3.2.1.2 Corrosion allowance

The non-structural sacrificial thickness on each surface of steel elements exposed to corrosion of class 6I, 6J, 7C, and 7D fills should be as listed in Table 4. For other fills a separate evaluation should be made.

It is recommended that all metallic components buried in soil, i.e. reinforcing elements, connections, facing lugs and where applicable the facing units, should be of electrolytically compatible material. Where this is not possible, electrical insulation of durability equal to the service life of the structure should be provided between different metallic components.

Table 3 Minimum properties of some different types of steel reinforcement

Type of steel reinforcement	Maximum thickness to which stresses apply mm	Tensile strength σ_t N/mm ²	Shear strength σ_q N/mm ²	Bearing strength σ_{bc} N/mm ²
Carbon steel to BS EN 10025-2:2004 S 235 JR	16	360	215	360
Carbon steel to BS EN 10025-2:2004 S 275 JR	16	410	245	410
Carbon steel to BS EN 10025-2:2004 S 355 JR	16	470	280	470
Carbon steel rod to BS 4449:2005 and BS EN 10080:2005 grade B500	40 diameter	525	315	525

Table 4 Sacrificial thickness to be allowed on each surface exposed to corrosion

Design working life A_1 years	Reinforcement material	Sacrificial thickness mm	
		Land based structure (out of water)	Fresh water structure
5	B	0.25	0.25
	G	0	0
10	B	0.35	0.4
	G	0	0
50	B	1.15	1.55
	G	0.3	0.55
60	B	1.35	1.68
	G	0.38	0.63
70	G	0.45	0.7
120	G	0.75	1.0

Key

B black steel (ungalvanized)
G galvanized steel

NOTE 1 Linear interpolation may be used for intermediate service lives.

NOTE 2 These values apply to steels embedded in fills conforming to Table 2 of this standard and Table B.1 of BS EN 14475:2006.

NOTE 3 Sites of special aggressiveness are to be assessed by specific study.

3.2.1.3 Handling

Metallic reinforcement should be loaded, unloaded and handled in such a way as to prevent bending which causes a permanent set or damage to any protective coating.

The reinforcement should not be dragged across abrasive surfaces such as reinforced concrete or coarse angular soils or through deleterious materials.

To avoid punctures or fractures in any covering that might allow ingress of corrosive media only fibre rope, webbing slings or protected chains should be used.

Connecting lugs attached to facing elements should be handled with similar care to prevent damage to the protective covering.

3.2.1.4 Storage

Metallic reinforcement should be stored in neat stacks clear of the ground at all times and supported on non-absorbent materials to avoid contamination. Ideally storage should be located close to the construction site. Items having different lengths and cross-sectional dimensions should be stacked separately and clearly marked.

Storage arrangements should preferably maintain a separation between batches delivered. Ideally one batch should be fully utilized before the next one is used to facilitate quality control.

3.2.1.5 Placing

Metallic reinforcement should be placed on compacted fill at the necessary level. The elements should be sensibly horizontal and should not span over irregularities in the fill surface.

If inspection of the elements prior to or during placement reveals bends or kinks with a radius less than two times the reinforcement thickness, these should be rejected for use in the works.

Small areas of galvanized coating damaged during handling should be repaired in accordance with BS EN ISO 1461.

3.2.2 Polymeric reinforcements

3.2.2.1 General

Design and testing polymeric reinforcements should be as given in Section 4.

NOTE Polymeric reinforcements generally take the form of woven, knitted or stitch-bonded geotextiles, geogrids or geocomposite strips. Details of packaging and hence handling and storage will vary from product to product.

Where the recommendations given below do not relate to a particular product further advice should be sought from an approving authority, the supplier or the manufacturer.

3.2.2.2 Handling

Polymeric reinforcement materials are generally supplied in rolls, which should bear a conformity identification mark, e.g. according to BS EN ISO 10320. Site handling of the reinforcements should ensure that damage to the product such as surface abrasion, slitting, notching or tearing is prevented.

Where a central tube or mandrel to facilitate lifting is supplied, the recommendations of the manufacturer should be followed. In any case, geotextiles supplied in rolls should be supported at a minimum of two points to prevent excessive bending unless a central steel tube is used for support.

3.2.2.3 Storage

The storage condition of polymeric reinforcements should take account of their characteristics and placement needs.

Generally, short term storage on site may be carried out without particular precautions as long as the products are kept within their packaging. Prolonged exposure to light should be prevented either by storage under cover or by ensuring that the product is covered with an opaque packaging.

Geotextiles should be stored in dry conditions particularly for materials that can absorb water and where low temperatures might cause freezing and make placing difficult.

Where bars or other fixings are needed in the use of a polymeric reinforcement, these should be stored in clean dry conditions.

3.2.2.4 Placing

Prior to placing of the reinforcing elements a method statement should be prepared which provides details on the sequence of operations. The method adopted should ensure that the reinforcements do not suffer deterioration during placing and that any joints or connections are formed effectively.

The placing of the reinforcement should be consistent with the direction of major stresses, considering that most sheet products have a preferential strength direction.

Geotextiles and geogrids are generally supplied in rolls of specific width; it is recommended that rolls are not cut to reduce their width, rather a greater overlap should be provided on site.

Geotextiles may be cut to a length using a sharp blade, scissors or shears. Materials which can ravel should be heat treated or bonded with adhesive tape at the cut. The cutting to size of the polymeric sheet geogrids should ideally be performed before placing.

Where geotextiles or geogrids are to be used for single or multilayer basal reinforcement to embankments, separate elements forming the basal reinforcement should be joined in accordance with 3.2.3.

NOTE During construction, the predominant stresses are likely to be in the direction of the centreline of the embankment as filling progresses. After construction, the predominant stresses will be in a transverse direction.

Ideally strips should be laid in the major load bearing direction, that is, transverse to the embankment centreline. However, construction is

facilitated and sewing time minimized if the geotextile can be laid in the direction of the centreline; this is not recommended but could be allowed if the integrity of the joints is assured and the design takes account of the likely reduced strength in the transverse direction.

3.2.3 Polymeric reinforcement joints

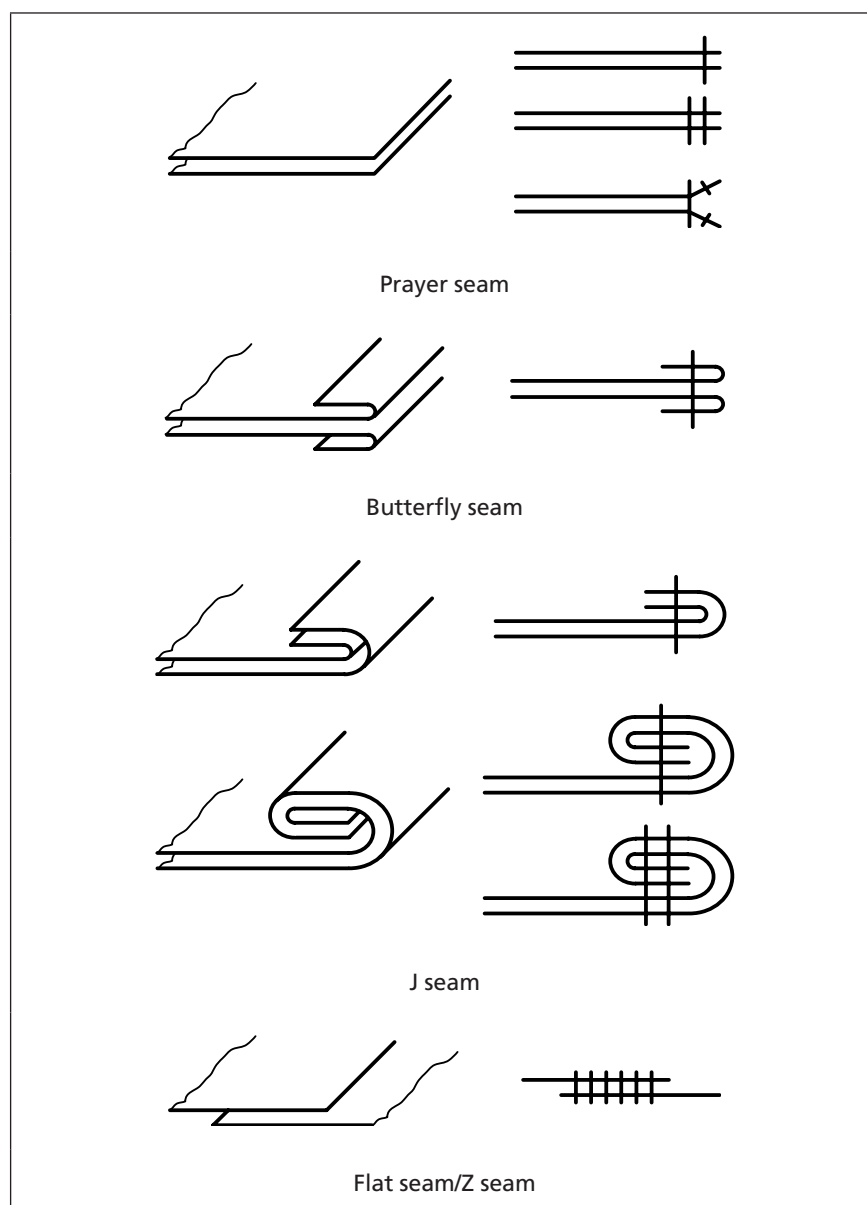
3.2.3.1 General

COMMENTARY ON 3.2.3.1

Joints are subdivided into prefabricated joints and joints made during execution of the works. A number of different jointing systems are in use.

Joints in geotextiles should normally be sewn where load transference is needed (see Figure 5 and Figure 6). For polymeric meshes or grids a bodkin may be employed whereby two overlapping sections are coupled together using a bar passed through the aperture of the grid.

Figure 5 Types of seams



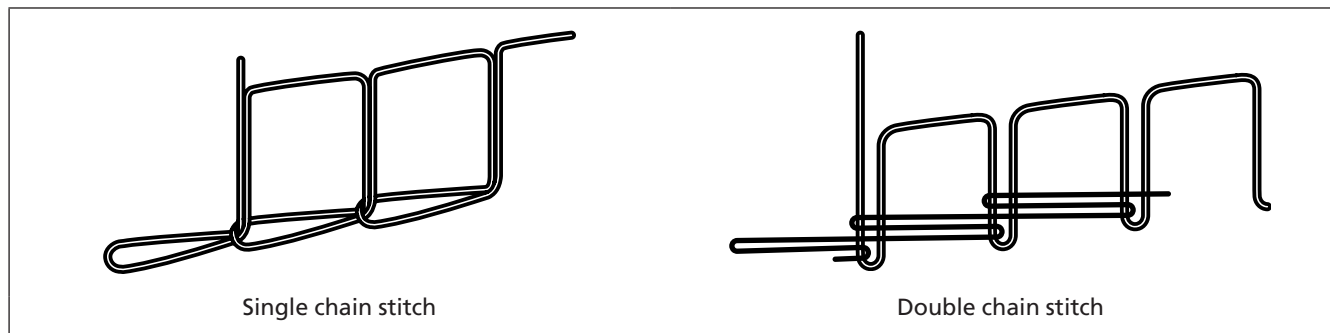
Joints should be formed to have the highest mechanical and durability efficiency possible, compared to the performance characteristics of the parent materials. Test methods used to assess joints should correspond closely to those procedures employed when determining the properties of the parent materials. All joints used in permanent structures designed to carry loads should be tested in accordance with BS EN ISO 10321.

3.2.3.2 Overlaps

In situations where relatively small tensions are developed, overlapping may be employed.

Such joints are sometimes used in the secondary tensile direction but should not be employed in the primary tensile direction in reinforced soil structures. Overlapping may also be used for jointing under water where the amount of overlap depends on design considerations and the construction conditions.

Figure 6 Stitch configuration



3.2.3.3 Bodkin joints

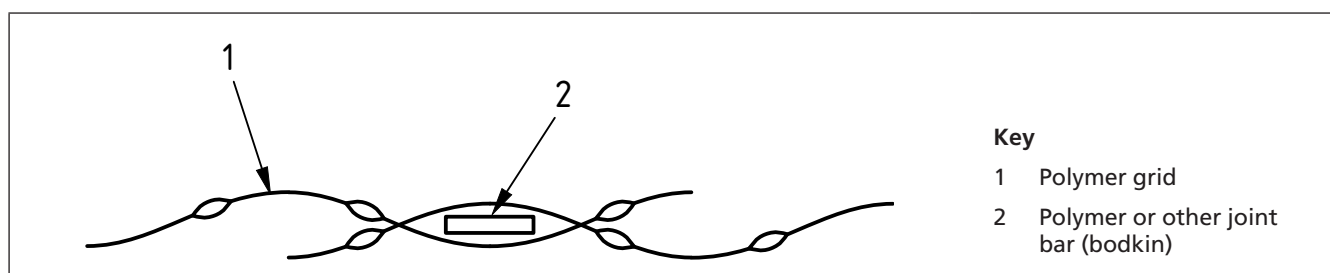
COMMENTARY ON 3.2.3.3

A bodkin joint, see Figure 7, is an effective method of joining some polymeric grid reinforcement.

Care should be taken to ensure that:

- bodkins have sufficient cross-sectional area and strength to avoid excessive deformation;
- bodkins are not so large as to distort the parent material causing stress concentrations;
- joints are pre-tensioned prior to loading, to reduce joint displacement as the components lock together.

Figure 7 Bodkin joint



3.2.3.4 Stapling

This method may be used with geotextiles to make temporary joints. Stapling should never be used for structural jointing.

3.2.3.5 Other jointing methods

Other jointing methods may be adopted, however, the general recommendations as set out in 3.2.3.1 will still apply.

3.3 Facings

3.3.1 General

COMMENTARY ON 3.3.1

There are many types of facing for reinforced soil structures and details of the various types and systems can be found in BS EN 14475:2006.

All facing units and their applications should conform with the requirements of BS EN 14475:2006.

Concrete facings backfilled with fill material that has been checked for chemical compliance with BS EN 14475:2006 Table B.1, should not require further protection measures such as bitumen painting on the buried face or equivalent concrete additives.

3.3.2 Panel facing units – Joint filler materials

3.3.2.1 General

Fillers should be durable, flexible, resistant to the effects of air pollution, insolation and water that might be contaminated with de-icing salt.

3.3.2.2 Bedding material

The selection of the bedding material should depend upon the structural behaviour of the facing assumed in the design of the wall. Cement mortar or a durable gasket material such as resin bonded cork, bitumen bonded cork, rubber or ethylene propylene diene monomer (EPDM) may be used.

3.3.2.3 Sealing material

The filling of joints other than bedding joints may consist of either closed cell polyethylene foam or closed cell polyurethane foam strip in the joint, or a geotextile strip over the rear face of the joint.

3.3.3 Concrete block facing of segmental block walls

3.3.3.1 General

The facing of segmental block walls should usually be made of un-reinforced dense concrete blocks of high durability, appropriate for the more aggressive environments commonly required for highway retaining structures. The blocks should not be those commonly used for internal walls in buildings.

Ⓐ Blocks should conform to BS EN 771-3:2011 and the following:

- Minimum concrete cube strength 30 N/mm² at 28 days;
- Minimum gross dry density: 2100 kg/m³;
- Reaction to fire: Euroclass A1;
- Exposure classes related to environmental conditions should be assessed in accordance with BS 8500-1.

Pigments used for colouration of the concrete blocks should conform to BS EN 12878. Ⓐ

3.3.3.2 Tolerances for blocks

Blocks should comply with category D2 of BS EN 771-3:2003+A1, except that the width may be outside that specified as the exposed face is often a textured face or else is a rough face created by splitting a double-width block. The location of any nibs or voids critical to strength or alignment/connection of blocks should be ± 2 mm. Width of the block is the distance from front face to the soil face and should be as defined in BS EN 771-3:2003+A1.

3.3.3.3 Shear strength between blocks and connection strength between block and reinforcing element

COMMENTARY ON 3.3.3.3

Test methods in ASTM D6916, ASTM D6638 and NCMA [16] for measuring block-to-block shear strength and block-to-soil reinforcement connection strength for blocks with plane faced bedding surfaces are available.

3.4 Fasteners and connections between the facing and reinforcing elements

3.4.1 General

Fasteners are used to make a connection between the reinforcement and the facing and take the form of dowels, rods, hexagon headed screws and nuts and bolts, and should consist of one of the following materials:

- plain steel;
- coated steel;
- galvanized steel;
- polymers.

The choice of material used to form the fastener should be compatible with the design life of the structure. Materials for fasteners and connections should conform to the criteria described in BS EN 14475:2006.

The provisions of 3.2 and Table 4 are also applicable to fasteners.

Recommended steel coatings may be divided into two groups:

- a) metallic coatings, including galvanizing in accordance with BS EN ISO 1461, aluminium and aluminium-zinc coatings in accordance with BS 2569; and
- b) organic coatings, including bitumens in accordance with BS 3416 and BS 4147, coal tar products in accordance with BS 4164, polyvinyl chloride (PVC), liquid and powder epoxies and liquid polyurethanes. The standards shown above give details of the application of the coating to the base metal of the fastener, but do not give details of the durability or service life of the coating in buried conditions. Table 4 sets out service lives for galvanized coatings in reinforced soil. Fasteners with all other coatings for structures other than temporary ones should be considered in accordance with 3.5.

3.4.2 Stresses

The ultimate stresses that should be assumed for bolts and screws up to 40 mm stock size are shown in Table 5. The ultimate stresses for dowels and rods are shown in Table 6.

3.5 Testing materials and components not covered by relevant specifications

NOTE Reinforced soil often uses materials that are either covered by existing standards or well known materials used in ways not covered by existing standards.

The acceptability of the latter materials should be determined by the designer using engineering principles and tests on components. Proprietary systems or proprietary components such as reinforcements, facings and connections may include traditional materials. These materials and systems may be certified by independent accredited approval organizations, see Figure 8.

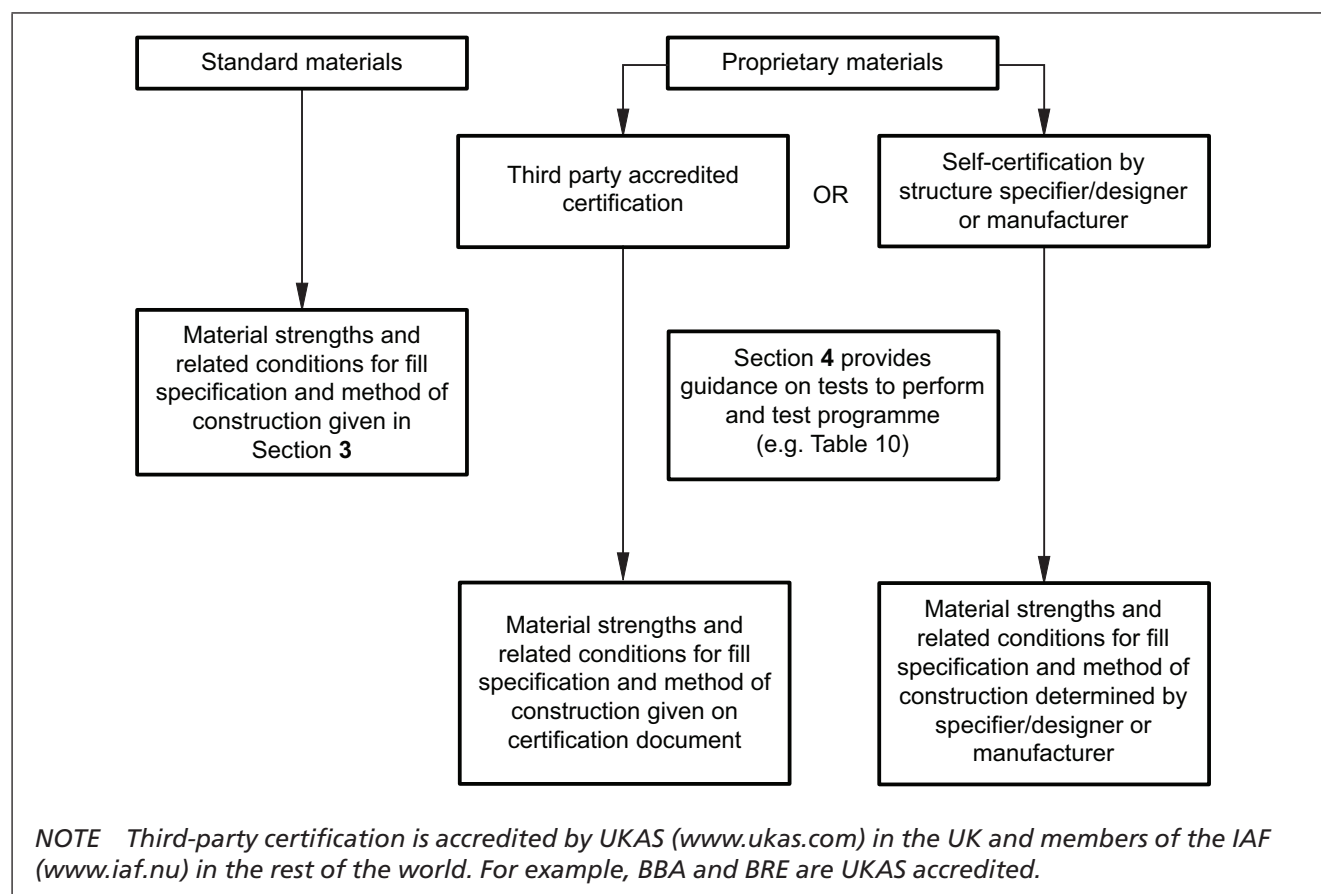
Table 5 Properties of bolts and screws up to 40 mm stock size

Material	Tensile strength σ_t N/mm ²	Shear strength σ_q N/mm ²	Bearing strength σ_{bc} N/mm ²
Alloy steel to BS 3692:2001			
grade 8.8	800	480	800
grade 10.9	1 000	600	1 000

Table 6 Properties of dowels and rods up to 40 mm stock size

Material	Tensile strength σ_t N/mm ²	Shear strength σ_q N/mm ²	Bearing strength σ_{bc} N/mm ²
Carbon steel to grade B500 of BS 4449:2005 and BS EN 10080:2005	525	315	525

Figure 8 Selection of materials for reinforcement, connections and facings for reinforced soil structures



Section 4: Testing for design purposes

4.1 General

An objective of this code is to permit a wide range of choice of fill and soil reinforcement to the design engineer, and since the properties of the soil reinforcement, and in some cases the fill, will be time dependent it is essential that the design engineer has knowledge of how these properties change with time so that these can be matched with any selected design life. Examples of service lives are given in Table 7.

It should be borne in mind that reinforced soil comprises three component parts:

- a) fill;
- b) soil reinforcement; and
- c) facing, except for reinforced soil foundations and some slopes.

Design parameters should be defined for each of these components. This section should be followed to define the available test methods to establish basic design strengths for fills/soil and reinforcements. It should be noted that no consideration is given to the materials used in facing units, as adequate specifications for both materials and test methods exist for concrete, reinforced concrete, metal and timber facing units. Soft faces may usually be formed of geotextile or geogrid sheets; specifications and test methods are as for reinforcements.

Table 7 Examples of Δ_1 design working Δ_1 life

Design working life category (BS EN 1990:2002+A1)	Category	Typical Δ_1 design working Δ_1 life years	Example
1	Temporary works	1 to 5	Contractors site structures Piling platforms
1	Short term	5 to 10	Contractors site structures Basal reinforced platforms for settlement control or in association with ground improvement
3/4	Industrial	10 to 50	Structures at mines
4/5	Long term	60	Marine structures in accordance with BS 6349 Highway embankments Railway embankments (NR/SP/CIV071 [17]) Basal reinforcement in association with Piles or other supports under embankments
5	Long term	120	Highway and railway retaining walls and highway structures and bridge abutments Basal reinforcement in association with piles or other supports under retaining walls and structures

Void spanning depending upon application and reason for use, may fall in any one of these categories, see Section 8.

The specification of metallic reinforcement has been developed based on many years of practical use with steel; however some of the essential properties that may be needed for design cannot be obtained from tests, the most significant being the rate of corrosion. The sacrificial thicknesses prescribed in 3.2.1.2 for steel reinforcements are based on experience, current practice and on measurements of corrosion in reinforced soil.

When using polymeric reinforcements the durability and reductions factors associated with these materials should be taken from Annex A. It should be noted that guidance on the means of evaluating the polymeric reinforcement properties can be found in PD ISO/TR 20432.

All polymeric materials delivered to a site should be identified in accordance with BS EN ISO 10320.

4.2 Fill and ground

4.2.1 General

Soil, either fill or natural ground, can interact with soil reinforcement with each affecting the performance of the other; broadly there are two possible effects that should be considered:

- a) firstly, the axial tensile strain of the reinforcement in service can affect the shear strength mobilized in the soil; and
- b) secondly, the chemistry of the soil, and soil fluid, can affect the durability and therefore time-dependent performance of the reinforcement.

Other effects of fill on the reinforced soil performance, which should be considered, arise from the durability of the fill, hydraulic properties of the soil and swelling characteristics of the soil; problems in this latter category are common to conventional earthworks, earth retaining structures and foundations. These are not considered further here but clearly should be taken into account in the design of reinforced soil.

Data on swelling characteristics may be found in technical journals or from site specific testing; however it is unlikely that these effects will arise with the limited range of fills such as Classes 6I, 6J, 7C and 7D.

4.2.2 Mobilized soil shear strength

It should be noted that in soil subject to a compressive loading regime, mobilized shear strength increases with increasing axial and lateral strain until the peak shear strength is mobilized. Also, in elasto-plastic soils shear strength continues to be mobilized at the peak value when strains exceed those pertaining to initial mobilization of peak strength. However, with strain softening soils, it should be borne in mind that the mobilized shear strength decreases as strains exceed those necessary to mobilize peak strength.

It should be noted that at large strains the mobilized soil shear strength achieves a constant minimum value independent of strain magnitude, referred to as ϕ'_{cv} ; and the peak value is defined by ϕ'_p and for wholly frictional fills and under plane strain conditions typical to retaining walls the strains to mobilize ϕ'_p are small; Bishop [18] quotes axial strains of 1.3% for dense sands and Cornforth [19] illustrates an axial strain of similar magnitude, see Figure 9. Consequently ϕ'_p may be assumed to be