

Code of practice for

Strengthened/ reinforced soils and other fills

ICS 93.020

NO COPYING WITHOUT BSI PERMISSION EXCEPT AS PERMITTED BY COPYRIGHT LAW



Document Sharing Hub

Committees responsible for this British Standard

The preparation of this British Standard was entrusted by Technical Committee B/526, Geotechnics, to Subcommittee B/526/4, Strengthened/reinforced soils and other fills, upon which the following bodies were represented:

- British Coal Corporation
- British Railways Board
- British Textile Confederation
- County Surveyors' Society
- Department of Transport (Highways Agency)
- Department of Transport (Transport Research Laboratory)
- Federation of Civil Engineering Contractors
- Institution of Civil Engineers
- Institution of Highways and Transportation
- Institution of Structural Engineers
- Zinc Development Association

This British Standard, having been prepared under the direction of the Sector Board for Building and Civil Engineering, was published under the authority of the Standards Board and comes into effect on 15 November 1995

© BSI 06-1999

The following BSI references relate to the work on this standard:
Committee reference B/526/4
Draft for comment 91/14831 DC

ISBN 0 580 24216 1

Amendments issued since publication

Amd. No.	Date	Comments
10252	March 1999	Indicated by a sideline in the margin

Contents

	Page
Committees responsible	Inside front cover
Foreword	vi
Section 1. General	
1.1 Scope	1
1.2 References	1
1.3 Definitions	1
1.4 Symbols	3
Section 2. Concepts and fundamental principles	
2.1 General	9
2.2 Limit state principles	9
2.3 Partial factors	9
2.4 Design loads	10
2.5 Design strengths	10
2.6 Fundamental mechanisms	11
2.7 Soil reinforcing mechanisms in walls and slopes	12
2.8 Soil reinforcing mechanisms in embankment foundations	12
2.9 Soil reinforcement interaction	13
2.10 Soil properties to be considered	14
2.11 Reinforcement geometry	15
2.12 Reinforcement bond	16
2.13 Effects of flexible reinforcement axial stiffness on loads	16
2.14 Factors affecting tensile behaviour of flexible reinforcement	17
Section 3. Materials	
3.1 Soils and fills	23
3.2 Reinforcing materials	26
3.3 Facings	28
3.4 Fasteners between the facing and reinforcing elements	30
3.5 Testing materials and components not covered by relevant specifications	30
Section 4. Testing for design purposes	
4.1 General	34
4.2 Fill and ground	35
4.3 Soil reinforcement	36
4.4 Facing units	40
4.5 Trial constructions to evaluate constructability	40
Section 5. Principles of design	
5.1 Design philosophy	41
5.2 Service life	41
5.3 Factors of safety	41
5.4 Fasteners and connections	44
5.5 Serviceability	45
5.6 Design information	45
Section 6. Design of walls and abutments	
6.1 General	53
6.2 Partial factors used in this section	53
6.3 Basis for design	55
6.4 Dimensions of the structure	56
6.5 External stability	57
6.6 Internal stability	61

	Page
6.7 Facings	72
6.8 Connections	72
6.9 Soil nailing for walls	77
Section 7. Design of reinforced slopes	
7.1 General	100
7.2 Partial factors used in the design of reinforced slopes	100
7.3 Areas of application	101
7.4 Reinforcement of fill materials	101
7.5 Reinforcement of existing ground	107
7.6 Facings	110
Section 8. Design of embankments with reinforced soil foundations on poor ground	
8.1 General	122
8.2 Partial factors used in the design of embankments with reinforced soil foundations on poor ground	122
8.3 Reinforced embankments over soft and very soft foundation soils	123
8.4 Reinforced embankments over areas prone to subsidence	134
Section 9. Construction and maintenance	
9.1 General	151
9.2 Walls and abutments	151
9.3 Slopes	156
9.4 Foundations	160
9.5 Handling, storage and placing	163
Annex A (normative) Assessment of partial material factors for reinforcements	177
Annex B (normative) Microbial activity index test	184
Annex C (normative) Determination of effective angle of internal friction (ϕ') and effective cohesion (c') of earthworks materials	185
Annex D (normative) Site damage test	186
Annex E (normative) Determination of coefficient of friction and adhesion between fill and reinforcing elements or anchor elements for reinforced soil and anchored earth structures	189
Annex F (normative) Trial constructions	190
Annex G (informative) Propping forces	190
Figure 1 — Range of applications of reinforced soil	19
Figure 2 — Effect of reinforcement on a soil element	20
Figure 3 — Reinforcing mechanisms in walls and slopes	21
Figure 4 — Forms of reinforcement	22
Figure 5 — Types of seams	31
Figure 6 — Stitch configuration	32
Figure 7 — Bodkin joint	32
Figure 8 — Selection of materials for reinforcement, connections and facings for reinforced soil structures	33
Figure 9 — Stress/strain relationship for sand under plane strain loading	40
Figure 10 — Examples of structures in category 1 — applicable to walls and slopes	48
Figure 11 — Examples of structures in category 2 — applicable to walls and slopes	49
Figure 12 — Examples of foundations in category 2	50

	Page
Figure 13 — Examples of structures in category 3 — applicable to walls and slopes	51
Figure 14 — Examples of foundations in category 3	52
Figure 15 — Definitions and types of walls and abutments	79
Figure 16 — Common facings used with structures	80
Figure 17 — Load combinations showing load factors	81
Figure 18 — Design procedure for reinforced soil walls	82
Figure 19 — Initial sizing of structures	83
Figure 20 — Sizing of walls with various geometries	84
Figure 21 — Definition of embedment, D_m	85
Figure 22 — Ultimate limit states — external stability	85
Figure 23 — Serviceability limit states — external and internal stability	86
Figure 24 — Definition of soil properties and principal loads	86
Figure 25 — Pressure distribution along base of wall	87
Figure 26 — Types of slip surface failure	88
Figure 27 — Stability — effects to be considered	88
Figure 28 — Stresses imposed due to self weight, surcharge and retained backfill	89
Figure 29 — Dispersal of vertical strip load through reinforced fill — tie back wedge method	89
Figure 30 — Dispersal of horizontal shear through reinforced fill — tie back wedge method	90
Figure 31 — Determination of adherence capacity of the reinforcement — tie back wedge method	90
Figure 32 — Types of reinforced soil anchors	91
Figure 33 — Internal wedge stability	92
Figure 34 — Internal wedge stability analysis of simple problem	93
Figure 35 — Assessment of post-construction strain under applied load T_{avj}	94
Figure 36 — Variation of coefficient of earth pressure with depth — coherent gravity method	94
Figure 37 — Dispersal of vertical strip load through reinforced fill — coherent gravity method	95
Figure 38 — Dispersal of horizontal shear through reinforced fill — coherent gravity method	95
Figure 39 — Line of maximum tension for retaining wall — coherent gravity method	96
Figure 40 — Definition of maximum tension line 2 (retaining wall without superimposed strip loads) — coherent gravity method	96
Figure 41 — Lines of maximum tension for structures with strip loads — coherent gravity method	97
Figure 42 — Definition of line 1 — coherent gravity method	97
Figure 43 — Examples of structures requiring global stability analysis — coherent gravity method	98
Figure 44 — Soil nailed walls	99
Figure 45 — Examples of slope reinforcement	111
Figure 46 — Design basis for reinforced slopes	112
Figure 47 — Ultimate limit states — external stability	112
Figure 48 — Ultimate limit states — internal stability	113
Figure 49 — Ultimate limit states — compound stability	113
Figure 50 — Serviceability limit states	114

	Page
Figure 51 — Definition of soil properties and principal loads for reinforced steep fill slopes	115
Figure 52 — Two-part wedge analysis for internal stability of reinforced fill slopes	116
Figure 53 — Other methods of internal stability analysis of reinforced fill slopes	117
Figure 54 — Force components in two-part wedge analysis of compound stability	118
Figure 55 — Applications of soil nailing	119
Figure 56 — Use of two-part wedge analysis for soil nailing	120
Figure 57 — Use of log-spiral analysis for soil nailing	121
Figure 58 — Reinforcement used to control initial stability only of embankment	138
Figure 59 — Reinforcement used to control both initial stability and settlement of embankment	138
Figure 60 — Ultimate limit states for basal reinforced embankments	139
Figure 61 — Serviceability limit states for basal reinforced embankments	140
Figure 62 — Procedure for assessing rotational stability by slip circle analysis	141
Figure 63 — Lateral sliding stability at fill/reinforcement interface	142
Figure 64 — Analysis of foundation extrusion stability	143
Figure 65 — Ultimate limit state stability analysis for basal mattress reinforcement	144
Figure 66 — Piled embankment configurations	145
Figure 67 — Ultimate limit states for basal reinforced piled embankments	146
Figure 68 — Serviceability limit states for basal reinforced piled embankments	147
Figure 69 — Outer limit of pile caps	147
Figure 70 — Variables used in determination of T_{rp}	148
Figure 71 — Lateral sliding stability at fill/reinforcement interface	148
Figure 72 — Variables used in analysis of overall stability of basal reinforced piled embankments	149
Figure 73 — Conceptual role of reinforcement in limiting surface deformations due to subsidence	150
Figure 74 — Parameters used to determine reinforcement	150
Figure 75 — Wrap-around construction techniques	166
Figure 76 — Reinforced soil retaining walls	167
Figure 77 — Typical drainage detail for abutment bankseat	168
Figure 78 — Reinforced soil mass acting as drain	168
Figure 79 — Porous pipe at wall face	169
Figure 80 — Drainage details for walls supporting cuttings	170
Figure 81 — Wrap-around facing	170
Figure 82 — Reinforced gabions	171
Figure 83 — Reinstatement of failed slope	172
Figure 84 — Soil nailing technique	173
Figure 85 — Laying and jointing sequence for basal reinforcement	174
Figure 86 — Advancing mud wave	175
Figure 87 — Inverted “U” construction	175
Figure 88 — Construction of a “U” shaped leading edge	175

	Page
Figure 89 — Basal mattress fabrication technique	176
Figure A.1 — Assessment of $f_{m 11}$	180
Figure A.2 — Assessment of $f_{m 122}$	181
Figure A.3 — Assessment of $f_{m 12}$	182
Figure A.4 — Assessment of $f_{m 21}$	183
Figure D.1 — Schematic layout of test bays	188
Table 1 — Factors affecting performance	15
Table 2 — Selection of fill for walls and abutments	23
Table 3 — Category of structure depending upon ramification of failure	23
Table 4 — Electrochemical properties of fill used with plain steel, galvanized steel and stainless steel materials	24
Table 5 — Grading of fill material for basal mattresses	26
Table 6 — Minimum properties of some different types of steel reinforcement	27
Table 7 — Sacrificial thickness to be allowed on each surface exposed to corrosion	27
Table 8 — Jointing methods and approximate strengths of polymeric materials	29
Table 9 — Material standards for different types of facing	29
Table 10 — Properties of bolts and screws up to 40 mm stock size	30
Table 11 — Properties of dowels and rods up to 40 mm stock size	32
Table 12 — Examples of service life	34
Table 13 — Checklist for investigations of reinforcement products	38
Table 14 — Factors affecting durability and performance of buried soil reinforcement materials	38
Table 15 — Partial materials factors for reinforcements	42
Table 16 — Summary of partial factors to be used in section 6	53
Table 17 — Partial load factors for load combinations associated with walls	54
Table 18 — Partial load factors for load combinations associated with abutments	55
Table 19 — Dimensions of walls and abutments	57
Table 20 — Determination of the minimum embedment as a function of the mechanical height H in metres and the factored bearing pressure q_r in kN/m^2	57
Table 21 — Minimum vertical movement capacities required for facing systems to cope with vertical internal settlement of reinforced fill	60
Table 22 — Guide to the effects of settlement	60
Table 23 — Usually accepted tolerances for faces of retaining walls and abutments	62
Table 24 — Serviceability limits on post-construction internal strains for bridge abutments and retaining walls	63
Table 25 — Connection loads for ultimate and serviceability limit states	73
Table 26 — Summary of partial factors to be used in section 7	101
Table 27 — Summary of partial factors to be used in section 8	122
Table 28 — Arching coefficient C_c for basal reinforced piled embankments	131
Table B.1 — Classification for microbial activity	185
Table B.2 — Determination of sulfate reducing bacteria classification	185
List of references	192

Foreword

This British Standard has been prepared by Subcommittee B/526/4, Strengthened/reinforced soils and other fills. It supersedes PD 6517:1988 which is withdrawn.

There has been an increasing use of soil reinforcement techniques in Great Britain over the past decade for a variety of applications from vertical walls and abutments through reinforced slopes to reinforced foundations. The modern impetus for the use of such techniques stems from the development of reinforced soil techniques for vertical or near vertical slopes some 25 years ago, generally using metallic reinforcement, usually with modular concrete facing units. The development of the use of polymeric materials in the form of geotextiles in the civil engineering industry has been accompanied by their introduction into reinforced soil applications including the reinforcing of slopes and foundations. 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, and it was felt that the absence of a code of practice covering the techniques was hindering their wider development.

This code of practice contains material which is both for the information and guidance of engineers and material which forms recommendations on good practice. Engineering judgement should be applied to determine when the recommendations of the code should be followed and when they should not.

This code of practice embodies the experience of engineers successfully engaged on the design and construction of the particular class of works. It is intended for the use of engineers with some knowledge of the subject as a basis for the design of similar works.

A code of practice represents good practice at the time it is written and, inevitably, technical developments can 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.

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.

As a code of practice, this British Standard 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.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 196, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

Section 1. General

1.1 Scope

This British Standard contains guidelines and recommendations 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 of safety margins in terms of partial material factors and load factors for various applications and design lives.

The code is divided into nine 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 which 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 9 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.

Much of the existing practice of reinforced soil is based on the use of limit equilibrium design methods which incorporate a global factor of safety. In keeping with the principles of limit state design, consistent structural dimensions and materials quantities have been maintained with existing practice by calibration of the partial factors in the limit state design relationships in this code. This approach is different to that adopted by some concurrent codes of practice, e.g. BS 8002:1994, which rely on the use of “worst credible” parameters to develop an adequate margin of safety.

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 References

1.2.1 Normative references

This British Standard incorporates, by dated or undated reference, provisions from other publications. These normative references are made at the appropriate places in the text and the cited publications are listed on page 192. For dated references, only the edition cited applies; any subsequent amendments to or revisions of the cited publication apply to this British Standard only when incorporated in the reference by amendment or revision. For undated references, the latest edition of the cited publication applies, together with any amendments.

1.2.2 Informative references

This British Standard refers to other publications that provide information or guidance. Editions of these publications current at the time of issue of this standard are listed on page 193, but reference should be made to the latest editions.

1.3 Definitions

For the purposes of this British Standard the following definitions 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

cohesive frictional fill

fill containing at least 15 % material passing a 63 μm sieve in accordance with BS 410

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

1.3.3

fill material

material in the reinforced soil structure in contact with the reinforcing elements, connections and facings, including both selected fill and any filter material

1.3.4

frictional fill

fill containing less than 15 % material passing a 63 μm test sieve in accordance with BS 410

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

1.3.5

geogrid

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

1.3.6

geotextile

permeable, polymeric material, which may be woven, nonwoven or knitted, used in geotechnical and civil engineering applications

1.3.7

partial factors

specific design parameters to account for uncertainty

1.3.8

polymeric reinforcement

generic term that encompasses geosynthetic materials used in geotechnical engineering such as geotextiles and geogrids

1.3.9

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

1.3.10

reinforcement base strength

unfactored strength of the reinforcement at the end of its selected design life

1.3.11

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

1.3.12 reinforcement

1.3.12.1

axially flexible reinforcement

reinforcement that can absorb tensile loads only

1.3.12.2

axially stiff reinforcement

reinforcement that can absorb tensile, shear and bending loads

1.3.12.3

extensible reinforcement

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

1.3.12.4

inextensible reinforcement

reinforcement that sustains the design loads at strains less than or equal to 1 %

1.3.13

retained backfill

fill material located between the reinforced mass and the natural soil

1.3.14**soil nailing**

type of reinforced soil formed by installing reinforcing elements in situ into an otherwise undisturbed soil mass

1.4 Symbols

a	Size or diameter of pile caps
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 Width of footing or foundation
b'	Variable, defined in Figure 37
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 Distance of strip load from wall face
d'	Variable, defined in Figure 37
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 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
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 37
n	Number of shear planes resisting applied shear, 6.8.4.3.3 Number of slices in a moment stability analysis 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 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 6.6.5.2.1
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 Length at right angles in plan to the face of the structure of the bottom layer of reinforcing elements 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 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. In case of grid reinforcement the width of the j th layer of grid per metre “run” of structure. For nails, it is the perimeter of individual nails
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 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_{Ll}	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
V_j	Maximum shear load in nail j in the slope
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

a_{bc}'	Adhesion coefficient relating soil cohesion to soil/reinforcement bond
a_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 of the 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
$\varepsilon_{\max}$	Maximum allowable strain in reinforcement
θ_d	Angle of draw in embankment fill
θ_i	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
	Statistical standard deviation of results obtained
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'_{rj}	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 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
χ	Moment correction factor
ω_j	Inclination to the horizontal of nail j in the slope
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 of safety against pull-out failure of the reinforcement
f_s	Partial factor of safety against sliding failure across the reinforcement or soil

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;
- polymeric strips, sheets, grids or meshes;
- anchors or multi-anchors (but not ground anchors);
- soil nails and in situ reinforcement.

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 forces equal or exceed restoring forces. Margins of safety, against attaining the limit state of collapse, are provided by the use of partial material factors and partial load factors. These partial factors assume prescribed numerical values of unity or greater. Disturbing forces are increased by multiplying by prescribed load factors to produce design loads. Restoring forces are decreased by dividing by prescribed material factors to produce design strengths. Provided the design strength equals or exceeds the design load then there is deemed to be an adequate margin of safety against attaining the ultimate limit state of collapse.

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.

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_t (and f_{ts}) 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 a reduced design strength.

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 sections 6, 7 and 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, may 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 may 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 internal stability of a reinforced soil mass is governed by the interaction between soil and reinforcement. This interaction occurs by friction or adhesion. 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, 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 may be dead loads or live loads and these are calculated 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 worst credible value.

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. In calculating disturbing loads and forces the shear strength parameters of the soil and pore fluid are used unfactored. 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 may be 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 a 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.

In the case of internal stability the design load may be resisted by forces generated in the soil and reinforcement. Resisting forces generated by reinforcement will be determined on a statistical basis (Annex A), and reduced by a material factor to produce a design strength.

The majority of reinforcing elements, such as strips, sheets and grids, have thicknesses which are small compared to their other dimensions. Such elements are flexible and, due to their low bending stiffness, can only absorb 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 sections 6 and 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, a design strength 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 which 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 2 a). 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 2 b), 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 2 b), 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.

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. The addition of reinforcement will also improve soil strength. 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. Investigation 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 absorbed, or dissipated, by flexible reinforcement.

Provided a 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 horizontal. Flexible reinforcement is also inserted into cut sections during construction (in the form of soil nails) at inclinations close to the horizontal. This inclination is convenient since it coincides with the general inclination of the tensile strains developed in the soil in the active zone.

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 span over weak zones in the foundation soil, see Figure 1 c), (iii), and (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 1 c), (i), and (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 [2].

In the deep seated rotational failure mode, tensile forces are generated in the basal reinforcement by mechanisms similar to those operating in a wall or slope. Although the tensile load induced in the basal reinforcement is both axial and horizontal this does give rise to a restoring moment about the centre of the slip circle under consideration. The effect of this is to reduce the disturbing moment and therefore the high shear stresses which would otherwise cause shear failure in the embankment fill or foundation soil. As the foundation soil consolidates, and so gains shear strength, it becomes capable of resisting the high applied shear stresses without the benefit of any reduction made available by the reinforcement.

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 2.2. 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.

Conversely, if the soil is cohesive, this bond stress will be 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, [3], [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 which 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, 7 and 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 total shear strength parameter c_u . Effective cohesion, c' is only applied to cut slopes, formed in overconsolidated clays, which are to be reinforced by soil nailing. 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.

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 $\varphi_u \neq 0$. In this case the values of undrained shear strength, c_u , should 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.

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		
Reinforcement Distribution	Soil states	Structure
Location	Density	Geometry
Spacing	Confinement	End use
Orientation	State of stress	Foundation conditions
	Degree of saturation	
	Drainage	

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, may 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 will 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. When used as soil nails, steel bars have a simple circular cross section.

Sheet reinforcement, and polymeric grids, are generally installed full width, such that each metre length of face is associated with a 1 m 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.

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 influences 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 should 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 a 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 3. For grid reinforcement the boundary conditions in the direct shear test may give rise to a measured value of δ' greater than φ'_p [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 in 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, [5], [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, α_{bc} , which can not assume a value greater than unity, [4].

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 $(1 - \sin \varphi'_p)/(1 + \sin \varphi'_p)$. 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, [7], [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, [9]. In the upper reaches of a wall these can be modelled by the coefficient of lateral earth pressure at rest K_0 , [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 sections 6 and 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_0 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 raw loads by a partial load factor to produce the design load and decreasing the reinforcement base strength by a 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 which 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 may 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 may 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.

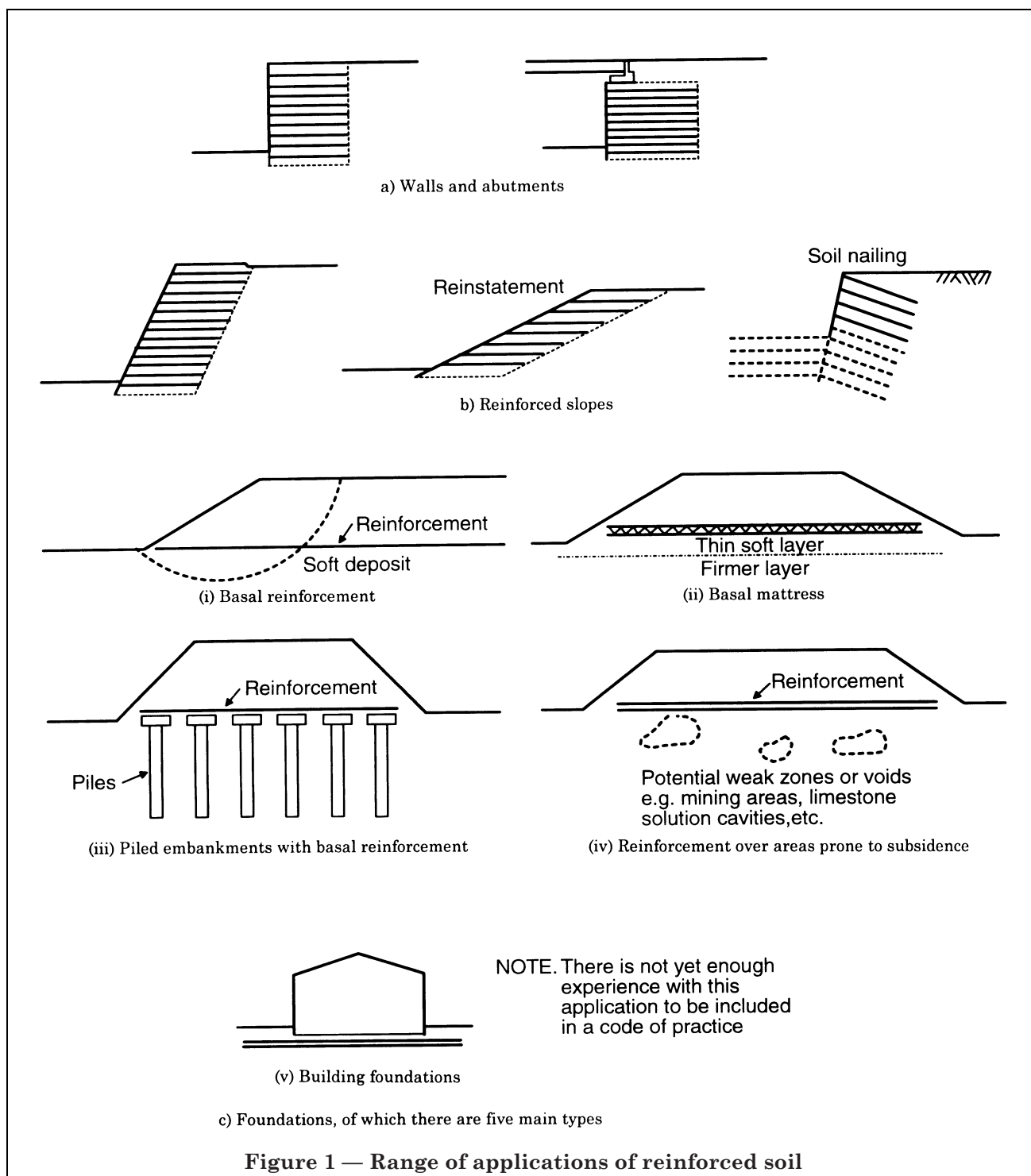
In the case of metallic reinforcement, which in practice is dominantly galvanized steel or ungalvanized steel, the sole agency of degradation is electrochemical corrosion. Irrespective of the properties of the soil in which it is installed, all metallic reinforcement is subject to electrochemical corrosion. 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 invoke 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 sometime, after initial loading, the time dependent stress can increase to a critical value which invokes 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 in 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. A 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 may be governed by deformation 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 numerated and used in the appropriate limit state calculations to determine which is the more critical.



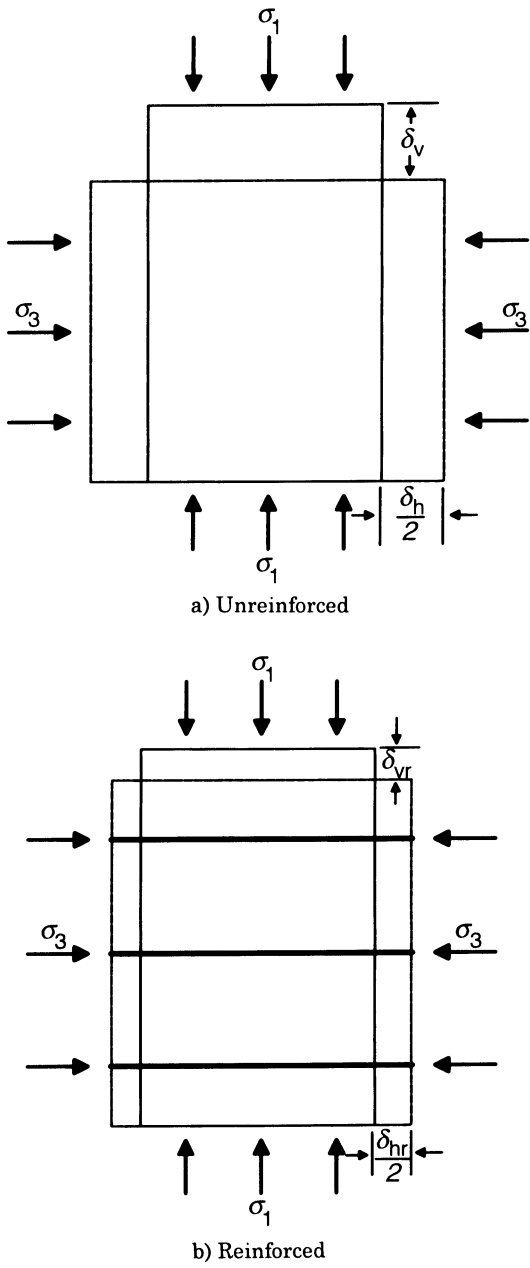
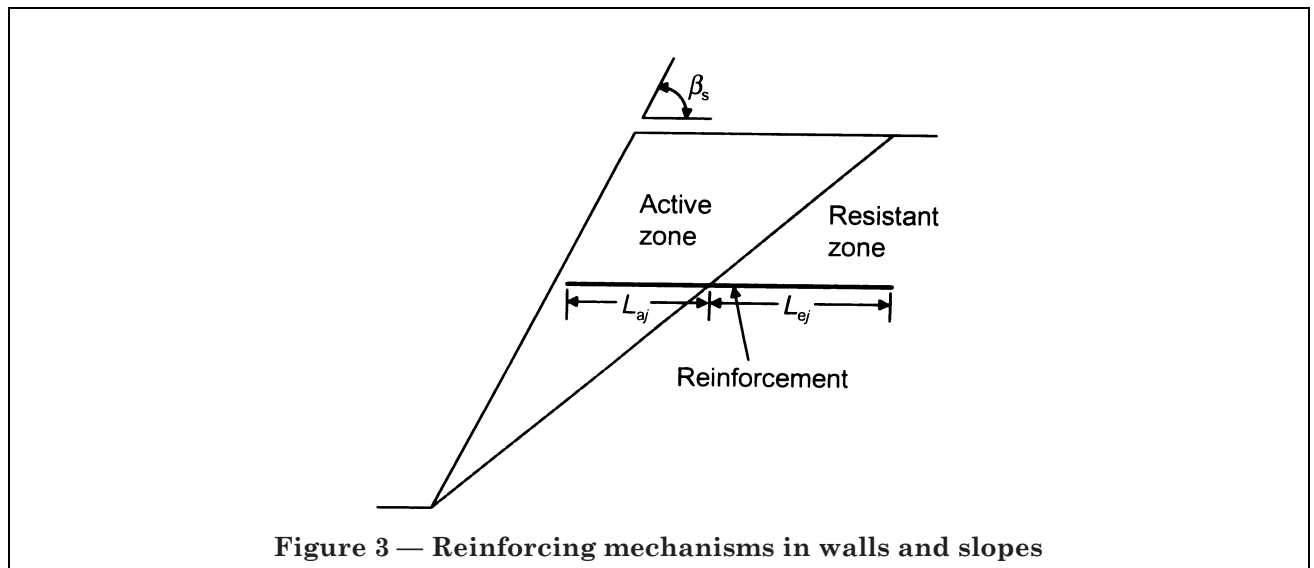
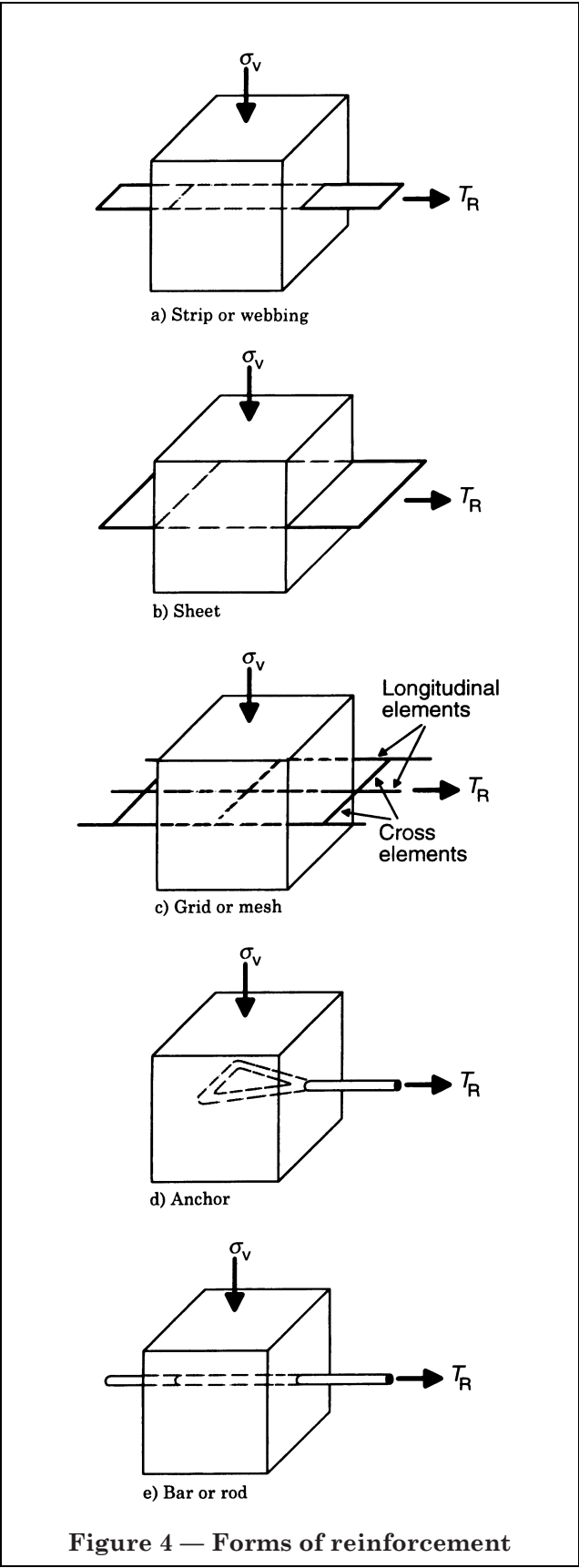


Figure 2 — Effect of reinforcement on a soil element





Section 3. Materials

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 apply to the use of fill in slopes. Less stringent conditions might be applied to the fill in the reinforced zones of reinstated or flatter slopes.

3.1.2 Fill material in walls and abutments

3.1.2.1 Mechanical criteria

For permanent works the fill used should be frictional or cohesive frictional material classes 6I, 6J, 7C, and 7D as described in the *Specification for Highway Works* [1]. Soft chalk, (SMC > 29 %), and cohesive fill should not be used in walls and abutments of structures in categories 2 and 3, Table 2 and Table 3.

Table 2 — Selection of fill for walls and abutments

Soil type ^a		Structure category (see 5.3.2)		
		1	2	3
Frictional fill (see 3.1.2.1)		✓	✓	✓
Cohesive frictional fill (see 3.1.2.1)		✓	✓	✓
Pulverized-fuel ash (see 3.1.2.4.2)		✓	✓	✓
Chalk	SMC < 29 %	✓	✓	✓
	SMC > 29 %	✓	—	—
Argillaceous material		✓ ^b	✓ ^b	—
General fill (clay fills)		✓	—	—
^a This table shows the types of soil to be used in various classes of structures based upon structural performance and constructability. Consideration should also be given to the durability of the reinforcement in connection with the service life, see 3.1.2.2. ^b Due to the variable nature of argillaceous materials the proposed source should be subject to a specific study and client approval prior to construction and a frequent quality control testing during construction, see 3.1.2.4.4 and 9.2.3.6.				

Table 3 — Category of structure depending upon ramification of failure

Category	Partial factor f_n	Examples of structures
1 (low)	Not applicable ^a	Retaining walls and slopes less than 1.5 m in retained height where failure would result in minimal damage and loss of access
2 (medium)	1.0	Embankments and structures where failure would result in moderate damage and loss of services
3 (high)	1.1	Abutments, structures directly supporting motorway, trunk and principal roads or railways or inhabited buildings, dams, sea walls and slopes, river training walls and slopes
NOTE 1 See Figure 10, Figure 11, Figure 12, Figure 13 and Figure 14 for examples of structures in categories 1, 2 and 3.		
^a Structures in category 1 should be restricted to simple structures designed by experience without analysis as described in DD ENV 1997-1:1995.		

3.1.2.2 Chemical and electrochemical criteria

The influence of the chemical properties of the fill with respect to the durability of concrete elements should be considered in accordance with BS 5400-4.

Table 4 — Electrochemical properties of fill used with plain steel, galvanized steel and stainless steel materials

Reinforcing element material	Location	Properties of fill									
		Tests required for all fills					Additional test required in some cases				
		pH ^b		Maximum chloride content ^c %	Maximum soluble sulfate content ^d %	Minimum resistivity (saturated) ^e ohm cm	Maximum organic content ^f %	Minimum redox potential ^g volts	Maximum microbial activity index ^g	Minimum resistivity (in situ) ^h ohm cm	Maximum sulfide content ⁱ %
	OUT or IN water (not sea) ^a	Min.	Max.								
Galvanized or ungalvanized steel	OUT	5	10	0.02	0.10	1 000	0.2	0.4	5	5 000	0.03
Stainless steel		5	10	0.025	0.10	1 000	0.2	0.35	5	3 000	0.03
Galvanized or ungalvanized steel	IN	5	10	0.01	0.05	3 000	0.2	0.4	5	5 000	0.01
Stainless steel		5	10	0.01	0.05	3 000	0.2	0.35	5	3 000	0.01

^a In water refers to permanently or regularly submerged.

^b The measurement of pH (test 9 in BS 1377-3:1990) should be carried out in all cases.

^c The measurement of chloride content (test 7.2 in BS 1377-3:1990) should be carried out in all cases.

^d The measurement of water soluble sulfate SO₃ (test 5 in BS 1377-3:1990) should be carried out in all cases.

^e The measurement of resistivity of a saturated sample (test 10.4 in BS 1377-3:1990) should be carried out in all cases.

^f The measurement of organic content (test 3 in BS 1377-3:1990) should be carried out for clayey soils where more than 15 % passes a 63 micron sieve.

^g For clayey soils with an organic content in excess of the specified level then the measurement of redox potential (test 10 in BS 1377-9:1990) or microbial activity index (see Annex B) should be carried out.

^h The measurement of resistivity of a saturated sample of fill provides an indication of the soil corrosion characteristics and is carried out in all cases. However for comparative purposes the designer may require, in some cases, the in situ measurement of apparent resistivity (test 9 in BS 1377-9:1990).

ⁱ The measurement of sulfide content should be carried out if the origin of the fill raises the possibility of its presence. The measurement should be carried out by a competent laboratory using a method such as that given in the *Encyclopedia of Industrial Chemical Analysis* by Snell and Hilton (Wiley Interscience Publishing, London 1974).

The limits of the electrochemical properties of soil fill with respect to steel metallic reinforcements are detailed in Table 4. There is little guidance on the soil or fill quality to be used with other metallic reinforcements. Polymeric reinforcements are not influenced by electrochemical action but can be affected by certain chemicals. The influence of chemicals on proprietary polymeric materials should be considered in the product certification procedures, see section 4.

3.1.2.3 Cohesive fill

Cohesive fill, as defined in *Specification for Highway Works* [1] should not be used in the construction of reinforced soil walls for permanent works, either faced or unfaced.

3.1.2.4 Other fills and industrial wastes

3.1.2.4.1 General

Materials meeting the mechanical and chemical limitations in 3.1.2.1 and 3.1.2.2 are permitted. A greater frequency of testing should be carried out to maintain adequate quality control.

3.1.2.4.2 Pulverized-fuel ash (pfa)

Only conditioned hopper ash should be used as pfa fill class 7B conforming with *Specification for Highway Works* [1].

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.

3.1.2.4.3 Colliery spoil

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.

3.1.2.4.4 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 can contain carbonaceous material and pyrite, often in localized concentrations. Pyrite can oxidize and the process can generate heat which results in a rise in soil temperature. Oxidation can result in an increase in acidity which can effect concrete facings, metallic reinforcements and certain polymeric reinforcements. Polymeric reinforcements can be influenced by the increase in operational temperature caused by any exothermic reaction.

The use of argillaceous materials in permanent structures is not accepted by some authorities because of concern for the durability of the reinforcing elements and because the needs for testing and assessment are greatly increased over other materials, [11], [12].

3.1.2.4.5 Chalk

Chalk with a saturation moisture content of up to 29 % and a natural moisture content of up to 23 % may be used as fill, see also 9.2.3.4.

3.1.2.4.6 Friable materials

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

3.1.3 Fill material in slopes

The use of frictional or cohesive frictional fills meeting the recommendations of 3.1.2.1 and 3.1.2.2 or other fills meeting the recommendations of 3.1.2.4 may be used for steep slopes. Cohesive fill may be used in conjunction with an appropriate reinforcement for reinstated or new slopes [13]. Embankments with reinforced foundations may also use cohesive fill and in these circumstances a basal drainage layer may be necessary.

3.1.4 Fill material in foundations

Materials used in foundations should be selected granular fill in accordance with the *Specification for Highway Works* [1]. Material used as fill in basal mattresses should conform to Table 5.

Table 5 — Grading of fill material for basal mattresses^a

Aperture size of sieve in accordance with BS 410 mm	Percentage by mass passing the sieve
125	100
90	80 to 100
75	65 to 100
37.5	45 to 100
10	15 to 60
5	10 to 45
µm	
600	0 to 25
63	0 to 12
^a Using selected granular fill (coarse grading) with the following constituents subject to the requirements of clause 601 of the <i>Specification for Highway Works</i> [1]: natural gravel, natural sand, crushed gravel, crushed rock other than argillaceous rock, crushed concrete, chalk, well burnt colliery spoil.	

3.1.5 Natural ground (for soil nailing)

Soil nailing may be used in any natural ground but some ground conditions are more suitable for application of the technique than others. The factors affecting the performance of soil nailing are identified in 2.10. The effects of non-homogeneous ground on the stability and durability of soil nails should be evaluated for all soils outside the limits of Table 4.

3.2 Reinforcing materials

3.2.1 General

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., (see Figure 4) which are capable of sustaining tensile loads and the effects of deformation developed in the fill.

Steel has been used for many years in soil environments for piles and pipelines and the degree of corrosion is reasonably well established. The long term behaviour of polymers in a soil environment is not quite so well established although they have now been employed for over a decade. Where necessary guidance should be sought from specialists on the behaviour of particular products under simultaneously applied conditions, including stress, strain and construction to ensure that the conditions model satisfactorily the intended use of the reinforcement.

NOTE Users of this British Standard are advised to consider the desirability of quality system assessment and/or approval certification procedures when using proprietary metallic and polymeric reinforcements for permanent structures and/or where the consequences of failure would be serious.

Permissible non-proprietary materials should conform to the general material clauses detailed in 3.2.2, 3.3.1 and 3.4.

3.2.2 Metallic soil reinforcements

3.2.2.1 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., which 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 3.2 and Table 6 gives examples of recommended types of steel.

Steel for galvanizing should have a silicon content which allows for a galvanized coating conforming to BS 729 except that the average zinc coating weight for an individual test specimen should not be less than 1 000 g/m².

3.2.2.2 Corrosion allowance

The non-structural sacrificial thickness to be allowed on each surface of steel elements exposed to corrosion of class 6I, 6J, 7C, and 7D fills is given in Table 7. 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 materials. Nails fully protected in accordance with the recommendations for corrosion protection in BS 8081 require no allowance for sacrificial thickness.

Table 6 — 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 10025:1993 S 235 JR	16	340	205	340
Carbon steel to BS EN 10025:1993 S 355 JR	16	490	295	490
Stainless steel to BS 1449-2:1983 316S31, 316S33	10	510	305	510
Stainless steel to BS 1449-2:1983 316S31 CR temper rolled to a minimum 0.2 %; proof stress of 310 N/mm ²	6	650	390	650
Carbon steel rod to BS 4449:1998	40 diameter	485	290	485

Table 7 — Sacrificial thickness to be allowed on each surface exposed to corrosion

Design service life years	Reinforcement material	Sacrificial thickness mm	
		Land based structure (out of water)	Fresh water structure
5	B	0.25	0.25
	G	0	0
	S	0	0
10	B	0.35	0.4
	G	0	0
	S	0	0
50	B	1.15	1.55
	G	0.3	0.55
	S	0.05	0.07
60	B	1.35	1.68
	G	0.38	0.63
	S	0.05	0.09
70	G	0.45	0.7
	S	0.05	0.1
120	G	0.75	1.0
	S	0.1	0.2

NOTE 1 B black steel (ungalvanized); G galvanized steel; S stainless steel. Black steel should not be used as a reinforcement material for a design service life greater than 60 years.
 NOTE 2 Linear interpolation may be used for intermediate service lives.
 NOTE 3 These values apply to steels embedded in fills of class 6I, 6J, 7C and 7D in the Specification for Highway Works [1].
 NOTE 4 Sites of special aggressiveness are to be assessed by specific study.

3.2.3 Polymeric reinforcements

Design and testing recommendations for polymeric reinforcements are given in section 4.

3.2.4 Polymeric reinforcement joints

3.2.4.1 *General*

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. 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.

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 6906-1 or ISO 10321.

3.2.4.2 *Overlaps*

In situations where relatively small tensions are developed, the method of 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. The method of overlapping is also used for jointing under water where the amount of overlap depends on design considerations and the construction conditions.

3.2.4.3 *Sewing*

Various seam types are used with sewn connections, frequently used methods are shown in Figure 5; stitch configurations are shown in Figure 6. Typical performance characteristics are shown in Table 8. Details concerning sewn joints including type of seam, stitching, sewing thread and sewing machines, are detailed in 9.5.4.5.

3.2.4.4 *Bodkin joints*

A bodkin joint, see Figure 7, is an effective method of joining some polymeric grid reinforcement, Table 8. 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.

3.2.4.5 *Stapling*

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

3.2.4.6 *Other jointing methods*

Other jointing methods may be adopted, however, the general recommendations as set out in 3.2.4.1 will still apply.

3.3 Facings

3.3.1 *Hard facings*

Facings may consist of concrete, steel sheet, steel grids or meshes, timber, proprietary materials or combinations of these.

The facing should conform to the appropriate material standard given above and should be sized by normal design procedures using the appropriate standards see Table 9 or else be subject to a test programme, see section 4.

Table 8 — Jointing methods and approximate strengths of polymeric materials

Material	Joint type	Load carrying efficiency ^b %	Displacement ^c mm
Woven geotextile	Prayer seam ^a (see Figure 5)	30 to 50	< 25
	Butterfly seam ^a (see Figure 5)	40 to 70	< 25
	“J” seam ^a (see Figure 5)	30 to 60	< 25
	Double “J” seam ^a (see Figure 5)	50 to 70	< 25
	Interlocking seam ^a (see Figure 5)	≤ 80	< 25
	“Z” seam ^a (see Figure 5)	70 to 100	< 10
	Bonded	Dependent upon method and area conditions	
Polymeric geogrid	Bodkin (see Figure 7)	> 95	3 to 15 ^d
	Lacing	Dependent upon method	
NOTE 1 This table gives a guide to typical joint characteristics. A suitable test method, e.g. BS 6906-1 or ISO 10321 should be used to determine actual joint characteristics. ^a These are sewn seams. ^b Measured in terms of the tensile strength of the joint divided by the tensile strength of the adjoining geotextile. ^c The measurement of the displacement of joints according to BS 6906-1 or ISO 10321. ^d Displacement after part-tensioning to remove slack in joint.			

Table 9 — Material standards for different types of facing

Facing	Standard
Concrete	BS 8110; BS 5400-4
Steel sheet	BS 1449-1
Steel grids and meshes	BS 4482; BS 4483; BS 4449
Timber	BS 5268; BS 5975 (temporary works)
Galvanizing	BS 729

3.3.2 Soft facings

These are considered in 9.2.5.3.

3.3.3 Facing unit joint filler materials**3.3.3.1 General**

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

3.3.3.2 Bedding material

The selection of the bedding material will 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 or ethylene propylene diene monomer (EPDM) has been used.

3.3.3.3 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.

3.4 Fasteners 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 may consist of one of the following materials:

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

The choice of material used to form the fastener should be compatible with the design life of the structure. The provisions of 3.2 and Table 7 are also applicable to fasteners.

Steel coatings may be divided into two groups:

- a) metallic coatings, including galvanizing in accordance with BS 729 aluminium and aluminium-zinc coatings in accordance with BS 2569;
- 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 and Table 7 set 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 for bolts and screws up to 40 mm stock size are shown in Table 10. The ultimate stresses for dowels and rods are shown in Table 11.

3.5 Testing materials and components not covered by relevant specifications

Reinforced soil often uses materials which 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 10 — 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
Carbon steel to grade 4.6 of BS 4190:1967	395	235	395
Alloy steel to BS 3692:1967			
grade 8.8	785	470	785
grade 10.9	980	590	980
Stainless steel to grade A4-70 of BS 6105:1981	700	420	700

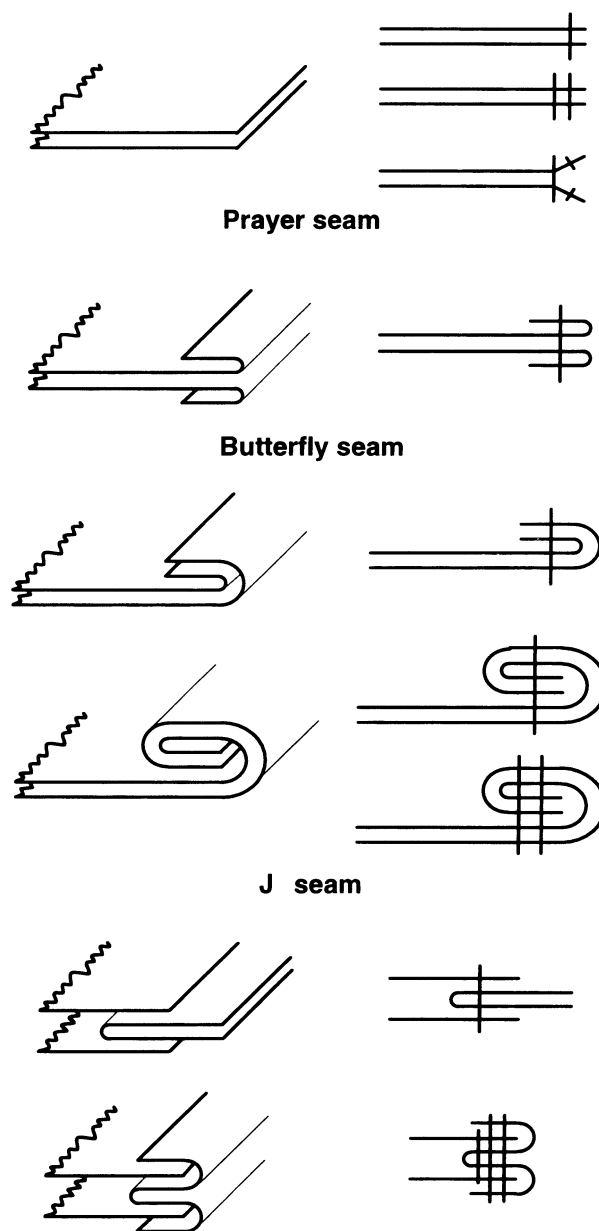
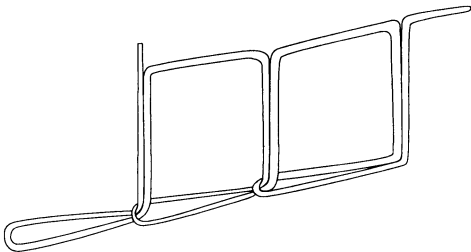


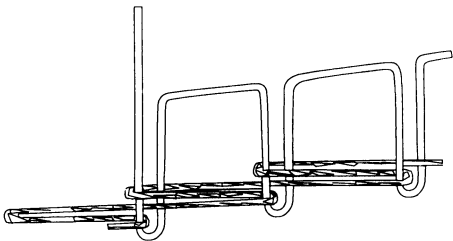
Figure 5 — Types of seams

Table 11 — 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 250 of BS 4449:1988	290	170	290
Carbon steel to grade 50B of BS 4360:1990	490	295	490
Stainless steel to 316S31 of BS 970-1:1991, cold drawn to a minimum 0.2 % proof stress of 450 N/mm ²	725	435	725



Single chain stitch



Double chain stitch

Figure 6 — Stitch configuration

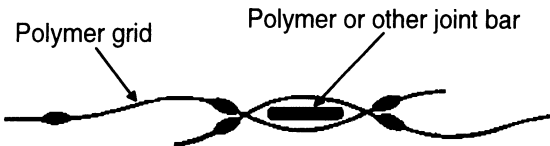
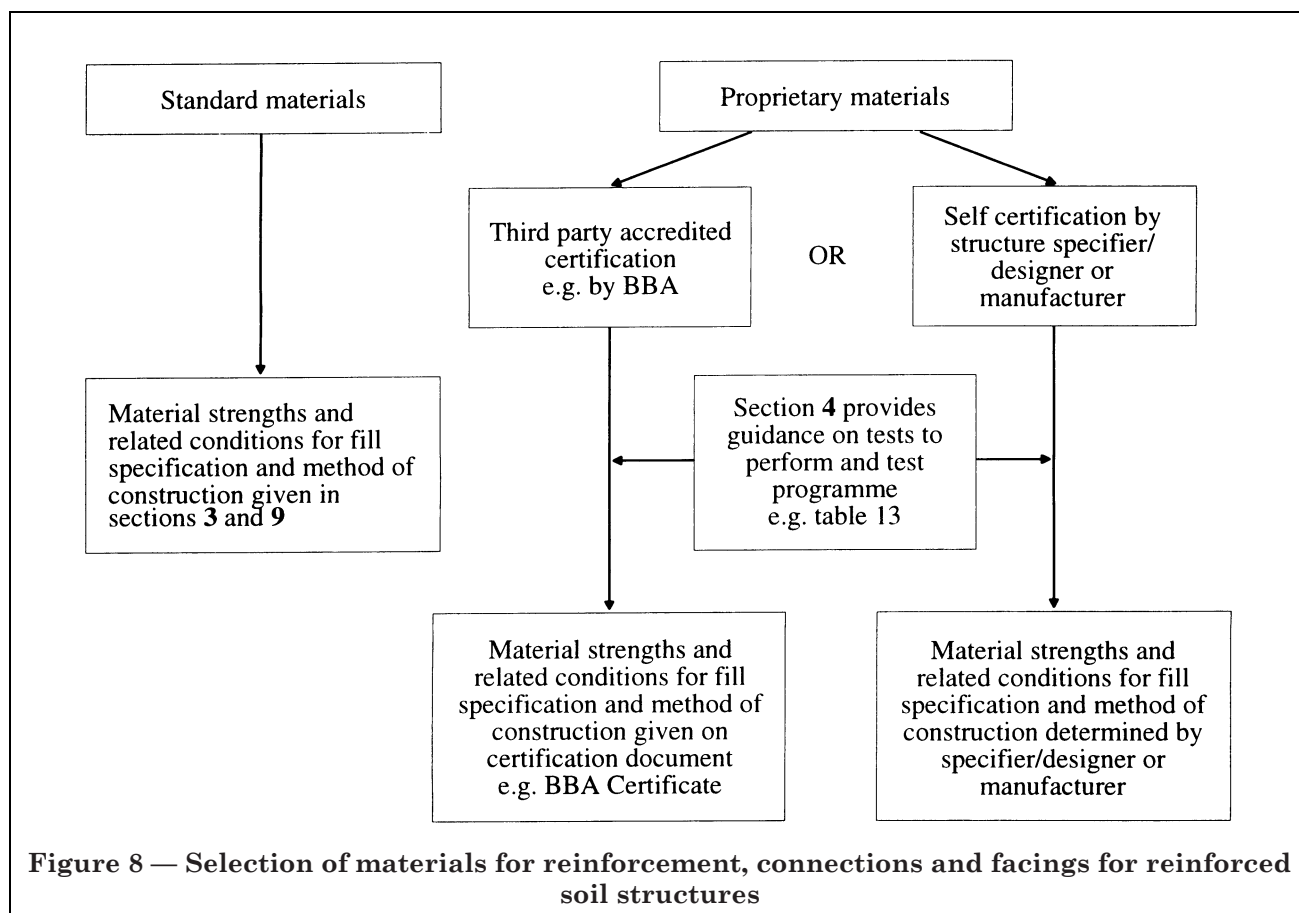


Figure 7 — Bodkin joint



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. Since the properties of the soil reinforcement, and in some cases the fill, will be time dependent it is important 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 12.

Reinforced soft comprises three component parts:

- a) fill, or soil in the case of reinforced ground;
- b) soil reinforcement;
- c) facing, except for reinforced soil foundations and slopes of less than 45°.

Design parameters should be defined for each of these components. In many cases standard test methods are not available to quantify design parameters. Consequently the following section defines test methods and basic design strengths for fills/soil and reinforcement. No consideration is given to the materials used in facing units, as adequate specifications for both materials and test methods exist for reinforced concrete, metal and timber facing units. Soft faces are usually formed of geotextile or geogrid sheets. Specifications and test methods are as for reinforcements. Soft faces may be given a further treatment such as shotcrete, bitumen or asphalt to improve their durability.

The specification of metallic reinforcement has been developed based on many years of practical use with steel. However some of the essential properties needed for design cannot be obtained from tests, the most significant being the rate of corrosion. The sacrificial thicknesses prescribed in 3.2.2.2 for steel reinforcements are based on experience, current practice and on measurements of corrosion in reinforced soil.

Polymeric reinforcements should be considered as specific products. British Standards exist for the naming and testing of polymeric materials for example BS 2782; BS 3502; BS 4618 and BS 6906 and industry has formulated test regimes for plastic pipes and other applications. An increasing range of BS, ISO and EN test procedures are becoming available for polymeric reinforcements. Materials delivered to site should be identified in accordance with BS EN 30320.

It is recommended that, particularly for structures with long service lives in a medium to high risk category, specifications should allow for access to buried specimens from the structure after a number of years to provide ongoing information on the evolution of metal corrosion or polymer degradation. This could be carried out by providing access through panels or facing units near the base of the wall.

Table 12 — Examples of service life

Category	Typical service life years	Example
Temporary works	1 to 2	Contractors site structures
Short term	5 to 10	Contractors site structures Basal reinforcement
Industrial	10 to 50	Structures at mines
Long term	60	Marine structures in accordance with BS 6349 and highway embankments
Long term	70	Retaining walls
Long term	120	Highway retaining walls and highway structures and bridge abutments to DoT requirements

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. Firstly, the axial tensile strain of the reinforcement in service may affect the shear strength mobilized in the soil. Secondly, the chemistry of the soil, and soil fluid, may affect the durability and therefore time dependent performance of the reinforcement. The fill can have other effects on the reinforced soil performance arising 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 considered no further here but clearly should be taken into account in the design of reinforced soil.

NOTE Data on swelling characteristics can be found in technical journals or from site specific testing.

4.2.2 Mobilized soil shear strength

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. 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, the mobilized shear strength decreases as strains exceed those necessary to mobilize peak strength. At large strains the mobilized soil shear strength achieves a constant minimum value independent of strain magnitude, referred to as φ'_{cv} ; the peak value is defined by φ'_p .

For wholly frictional fills and under plane strain conditions typical to retaining walls the strains to mobilize φ'_p are small. Bishop [14] quotes axial strains of 1.3 % for dense sands and Cornforth [15] illustrates an axial strain of similar magnitude, see Figure 9. Consequently φ'_p might be mobilized at axial strains of 1 % or less. For greater strains the mobilized friction angle will tend towards φ'_{cv} . For this code of practice use is made of φ'_p for walls, abutments and steep slopes constructed with frictional fill, and φ_{cv} for fill to shallow slopes and embankments founded on weak foundations.

4.2.3 Fill deformation and strength

The mobilized shear strength, and the coefficient of earth pressure K will depend on the strain properties of the fill/soil and reinforcement. Wall movements needed to mobilize the active condition (K_a) are small in frictional fill, typically a rotation of 0.001 rad. Experience has shown that with inextensible reinforcements lateral stresses in the upper fill behind a wall will be largely determined by compaction stresses, and can approximate to a K_0 condition. Guidance is given in sections 6 to 8 on the appropriate selection.

4.2.4 Fill strength related to reinforcement strain

The axial tensile stiffness of reinforcement materials is classified as extensible or inextensible, see 2.5. Steel below the yield point and some polymeric materials mobilize their design strength at a total axial strain of 1 % or less, whereas the majority of polymeric reinforcements, such as geotextiles and geogrids mobilize their strength at higher strains.

4.2.5 Determination of fill strength

The effective angle of internal friction φ' and effective cohesion c' may be determined by shear box or triaxial tests in accordance with Annex C of this standard, BS 1377-7 and BS 1377-8 or Appendix A of BS 6906-8:1991.

Shearing should not commence until each soil sample is fully consolidated under each normal stress level applied. The rate of shear should be consistent with drained conditions prevailing in the sample.

4.2.6 Determination of ground strength

The shear strength of ground subject to soil nailing may be determined using conventional techniques for site investigation and testing described in BS 5930 and BS 1377-7 and BS 1377-8 or shear box testing described above with due regard for sample disturbance.

4.2.7 Effects of soil on reinforcement durability

The chemistry of the soil and soil water can have a significant effect on the durability and therefore load carrying capacity of the soil reinforcement.

In particular the electrochemical characteristics of the soil can make it corrosive to metallic reinforcement. In recognition of this certain limits, defined in terms of chemical and electrical characteristics, have been placed on the fill used with galvanized, ungalvanized or stainless steel (see Table 4). Although the use of such low corrosivity fills do not eliminate corrosion, the rate and nature of the corrosion is less severe, and limits the corrosion to less than the limits given in Table 7.

At present there are no formalized limits applied to the aggression of fill, or soil, with respect to polymeric reinforcement. Any such limits that are subsequently defined will be a function of the specific polymeric reinforcement and in particular the specific polymer, and additives, used in the reinforcement. The performance of polymeric reinforcements, particularly in the long term, can be impaired by organic or inorganic chemicals or extreme pH values of the soil.

4.2.8 Site damage

The durability of both metallic and polymeric reinforcement can also be affected by the particle size, shape and hardness of the fill. This stems from the ability of the fill to damage the reinforcement during placement and compaction of the fill. The severity and nature of the damage caused by a particular fill and placement method will vary from one reinforcement product to another. In general surface scratches will have negligible effect on metallic reinforcement consisting of galvanized steel provided such damage does not expose the bare steel. The effects of construction damage on polymeric reinforcement need to be fully investigated for each proprietary material. However, these can be more severe than for metallic reinforcement. Although the general nature of construction induced damage to polymeric reinforcements, e.g. cuts, tears, splits and perforations can be assessed by site trials no test methods have yet been derived by which the same nature and degree of damage can be reproduced consistently in the laboratory.

Criteria exist to assess the ability of polymeric reinforcements to survive the construction process without the design function being impaired, [16]. These involve reductions in the values of tensile strength and strain to be taken into account when assessing the design tensile capacity of the reinforcements. Cuts, tears, splits and perforations can lead to some loss of performance, and this may be assessed either from site trials or empirically from laboratory tests which simulate the effect of such damage, for example see Annex D.

4.3 Soil reinforcement

4.3.1 General

Reinforced soil should be designed with an adequate margin of safety against reaching the ultimate limit state. In addition, checks for serviceability limits should be made. These are stipulated in terms of strain levels in the reinforcement which should not be exceeded.

Design against collapse requires consideration of both internal and external stability. The main influence of the soil reinforcement on external stability is in fixing the geometry of the reinforced soil mass, for example the effective width of a wall and therefore its resistance to sliding or overturning. These aspects of design are considered in sections 6, 7 and 8. Similarly these sections consider various aspects of design with respect to internal stability. However in general internal stability will be controlled either by failure of the reinforcement through tensile rupture or by loss of bond. Consequently the objective should be to devise meaningful and reproducible test methods which assess rupture and bond behaviour under full-scale conditions. The same or similar tests may be employed, where necessary, to determine load-strain characteristics with a view to designing for a defined serviceability requirement.

4.3.2 Serviceability limits

Serviceability limits are expressed in terms of limit axial tensile strains in the reinforcement which should not be exceeded. These limits will vary with the type of structure and will come into operation at different stages according to the type of structure.

4.3.3 Bond strength

The pull-out or sliding resistance of reinforcement in contact with fill is assessed on the basis of direct shear testing such as that described in Annex E or BS 6906-8:1991.

For walls and slopes bond failure is assessed using peak bond strength which is reduced by a partial factor to give a margin of safety to the design. For reinforced foundations bond failure is assessed using the large strain bond strength with the fill with a partial factor applied to give a margin of safety to the design.

The use of the shear box to assess bond strength reflects current design practice in accordance with BS 6906-8. Whilst the shear box test should prove adequate for quality control purposes the use of laboratory and in situ pull-out tests may be appropriate to assess load-displacement characteristics with respect to serviceability.

For certain reinforcing elements such as rods, used in soil nailing, or anchored fill systems shear box testing is inappropriate. It is also current practice to base the frictional bond strength of rough linear reinforcements, such as ribbed strips, on the result of pull-out tests. Unless otherwise established it is recommended that field pull-out tests be carried out to verify the bond strength of these materials assumed in design.

4.3.4 Durability and performance with time

4.3.4.1 General

The definition of durability is the ability to maintain requisite properties over the selected design life. In recognition of the fact that no material is immutable, the design engineer should be able to quantify how pertinent properties change with time and what factors will affect mechanisms or rates of change. In the context of reinforcement for reinforced soil a check list of factors to be considered is given in Table 13.

Although each of these factors should be considered, a combination of two or more factors influencing the reinforcement simultaneously can lead to a more critical situation. Testing of the reinforcement under combined conditions should be carried out as appropriate to the reinforcement.

The prime factors likely to affect the durability of reinforcing materials are listed in Table 14 which cites general factors and specific factors. General factors reflect environments and in-service conditions which will apply to all soil reinforcements. For metallic and polymeric reinforcing elements these include design life, loading, water and installation induced damage. For polymeric reinforcements there are two additional general factors of pre-installation UV exposure and operational temperature.

In addition to general factors there will be factors that affect performance which will be specific to the reinforcement product.

For metallic reinforcements in general performance and the variation of performance with time will be affected by the corrosivity of the soil. In recognition of this certain limits have been set for the electrochemical parameters of fill, see Table 4. In addition to corrosive environments associated with the soil there may be corrosive fluids introduced into the reinforced soil mass, e.g. aqueous solutions of salt (common salt used in de-icing) or spillage of corrosive fluids transported by road. The effects of loading and construction damage to metallic reinforcement may not be time dependent. Corrosion, which is related to the electrochemical nature of the soil and the air/water requirement, is time dependent and is considered as a reduction in net reinforcement cross-sectional area with time. For the conditions pertaining to reinforced soil, the effects of individual factors are additive and not interdependent. Thus rates of corrosion, for example, do not vary with load intensity.

The performance of polymeric reinforcements is affected by a number of inter-related variables. Some variables will relate directly to durability of the polymer and additives used in the reinforcement while others will relate to the visco-elastic nature of polymeric materials. In respect of tensile creep rupture, all of these variables will interact. A consequence of this is that the sum of the effects of discrete environments may be less than the total effects of a compound environment. Any such synergism should be considered in design. As with metallic reinforcement there are general and specific factors which will affect performance. The specific factors which are listed in Table 14 will vary according to the product under consideration. Much of this variation will depend upon the specific formulation of the polymer and additives used in the manufacture of the product. For example, one polymer can be degraded in a highly alkaline environment whilst another can not. In addition the additives incorporated in the polymer, the manufacturing process and the physical form of the reinforcement will affect performance.

Consequently testing is product specific. At present there are no formalized limitations on the environments in which polymeric reinforcements are expected to operate.

Table 13 — Checklist for investigations of reinforcement products

1	Physical and mechanical properties: a) full description of the material and relevant technical literature; b) short and long term data: ultimate and design values of tensile strength; stress strain curves; value of Young's modulus; creep; flexibility, extensibility; fatigue (dynamic and static) however induced e.g. structural, thermal or tidal loading; coefficient of friction with fill.
2	Durability: The effects of the following agencies on physical and mechanical properties to be investigated: extreme pH values; chlorides; sulfates; ozone; hydrocarbons and other chemicals commonly transported on highways; water; ultra-violet and infra-red rays (including short term exposure prior to installation); bacteria and other animal and vegetable life forms; temperature; fire; vandalism
3	Performance during installation and use: handling, storage ^a damage ^a extension and movement ^a effects on facing ^a
4	The effect of the combination of load, chemical environment and construction damage, i.e. combination of 1, 2 and 3
5	Construction details: fasteners: effect on 1 b) and 2
6	Quality control procedures with production processes
7	Preconstruction exposure to an aggressive environment, e.g. long term effect of short term UV exposure on polymeric reinforcements or ozone and marine spray on metallic reinforcements
8	The effects of static fatigue however induced, e.g. structural loading, thermal loading
9	The effects of blasts, fires or vandalism on the behaviour of the structure
^a The topics to be checked by site trial(s).	

Table 14 — Factors affecting durability and performance of buried soil reinforcement materials

Material	General factors	Specific factors
Metallic	Loading	Bacterial/electro-chemical composition of soil
	Water Damage	Corrosive fluids
Polymeric	Loading	Chemical/bacterial composition of soil
	Damage UV exposure Temperature	Aggressive fluids

4.3.4.2 Tests for durability

Tests for polymeric reinforcement durability are reviewed in PD 6533.

For metallic reinforcements consisting of steel, research and experience has allowed an electrochemical specification for fills to be drawn-up, see Table 4, which allow for losses in reinforcement thickness less than the limits given in Table 7. For more aggressive environments a specific study for the particular site is recommended.

It is not practical to consider all of the possible environmental hazards which could affect the performance of polymeric reinforcements although generally it is not anticipated that conventional chemical and bacteriological contents of fills and natural soils will pose any problems. The designer should consider the site specific aspects of a scheme where, for example, petrol or other chemical spillage might occur. The subsequent use of the completed works can also introduce problems if these can induce high ground temperatures or even possible combustion since this would affect the performance of polymeric reinforcing materials.

The risk of attack of aggressive fluids on both metallic and polymeric reinforcements can often be dealt with by using preventative design measures such as the incorporation of impermeable barriers and effective drainage systems.

4.3.5 Design parameters

Bond failure is not considered here. It is necessary to know how the reinforcement base strength at a time equal to the design life can be defined as the design strength with respect to tensile rupture.

For metallic reinforcements which exhibit negligible creep the reinforcement base strength at the end of the design life can be taken as the product of ultimate tensile strength and the net cross-sectional area of the member. The net cross-sectional area is the gross cross-sectional area less the area lost by corrosion (see 3.2.2.2). Provided the total axial strain in the reinforcement is not expected to exceed 1 % over the design life of the structure no further information is needed. Clearly the key factor for metallic reinforcement is knowledge of rates of corrosion.

For polymeric reinforcements, the reinforcement base strength may not be governed by tensile creep rupture. It may be governed by the strain occurring after construction or it may be governed by the load in the reinforcement after stress (or load) relaxation at constant strain. Thus the base strength of a polymeric reinforcement should be the lesser of:

- a) base strength with respect to tensile creep rupture;
- b) base strength with respect to creep strain.

In the case of basal reinforcement and clays where the gain in strength is slow (i.e. where consolidation is slow) consideration may need to be given to the concept of stress relaxation.

4.3.6 Tests for polymeric reinforcements

4.3.6.1 General

Three basic levels of testing are considered for polymeric reinforcements.

- a) *Index testing*. Testing carried out under standardized conditions used to compare the basic properties of products, (e.g. wide width tensile strength, creep under load, friction properties, etc.).
 - b) *Quality control testing*. Rapid testing to assure continuity of quality.
 - c) *Performance testing*. Testing of polymeric reinforcements in contact with a soil/fill under standardized conditions in the laboratory, to provide a better simulation of site conditions than index testing.
- Performance testing may also be carried out at full-scale on site.

The range of tests presently available is limited, and a variety of test methods from various national standards (BS, ASTM, DIN, NF, etc.) are often quoted in manufacturer's literature and occasionally used in specifications. A number of test methods have been published in the BS 6906 series, and these will be supplemented or superseded by ISO and EN tests. Future ISO and EN publications will also deal with sampling and identification. The majority of these tests will relate to index or quality control testing.

4.3.6.2 Tensile strength and creep properties

To assess the reinforcement base strength at a selected design life a knowledge of the stress and strain properties of the reinforcement is essential. The tensile test (BS 6906-1 or ISO 10319) should be used to determine the short term strength properties. Creep testing should follow the procedure given in BS 6906-5. The recommended minimum test duration of 10 000 h should be adhered to for all polymeric reinforcements having a service life in excess of 10 years. For temporary or short term works, see Table 12, the minimum creep rupture test duration should be 1 000 h. Although BS 6906-5 is intended to provide information on creep strain performance the same test method may be used, with elevated loads, to assess tensile creep rupture performance.

The testing of joints should be carried out in accordance with the principles of BS 6906-1 or ISO 10321.

Testing is normally carried out at a standard temperature of 20 °C or 23 °C but the properties of different polymeric reinforcements are sensitive to temperature to different degrees and testing should cover the range of operational temperatures likely in the structure. As a guide mean ground temperatures in the UK are about 10 °C with a range of about 0 °C to 20 °C at depth and about – 10 °C to + 35 °C near exposed surfaces in soil structures. Aspect, diurnal and seasonal fluctuations should be considered. Design strengths should be based on the maximum operational temperature.

4.3.6.3 Other properties

Testing may be necessary to determine the following properties for design purposes.

- a) *Site damage*. The susceptibility of polymeric reinforcement to damage during installation and the value of the partial material factor f_{m21} , may be determined by carefully controlled site or full-scale trials of the effect of installation on relevant properties, see 5.3.3.
- b) *In-plane flow*. This will be relevant if the designer considers that the reinforcement could act as a drainage channel for excess pore pressure removal. This may be of benefit in marginal quality fills.

4.3.7 Reinforcement samples

Where the risk category or design life of the structure makes it appropriate, it would be helpful to include provision for recovery of reinforcement samples over an extended number of years and for these to be compared with reference samples kept in darkroom conditions. This would be of value both in giving early warning of potential problems at that structure and in providing useful background on the long term performance of both similar structures and the type of reinforcement.

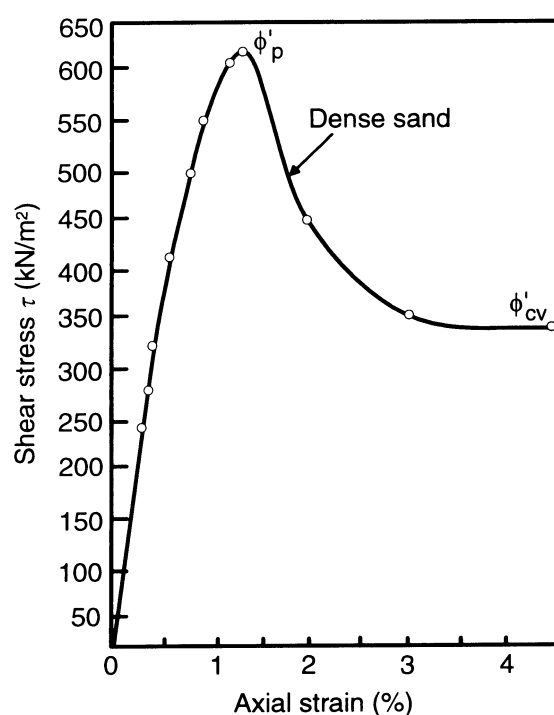
Durability to ultra-violet exposure for an appropriate duration is of particular importance, and polymeric reinforcements should be able to meet the recommendations of 9.4.2.7.

4.4 Facing units

Facing units should either be designed to the appropriate British Standards or may be assessed by testing.

4.5 Trial constructions to evaluate constructability

Construction trials should be carried out if the soil reinforcement, the reinforcement connections or reinforcement/facing connections are significantly different from those used previously or have not been previously used. A trial should also be carried out if the proposed use of the fill material falls outside previous experience with respect to the type of reinforcement under consideration, see Annex F.



Mobilized shear stress versus axial strain for Leighton-Buzzard sand from plane strain test

Figure 9 — Stress/strain relationship for sand under plane strain loading

Section 5. Principles of design

5.1 Design philosophy

The philosophy followed in this document is to design against the occurrence of a limit state (see section 2.3). The objective in taking this approach is to achieve compatibility with other related codes of practice and reference has been made to CIRIA [17].

By its nature reinforced soil is a combination of structural and geotechnical engineering. The evolution of limit state design in structural engineering has led to the definition of a number of partial load factors which are applied to loads in design combinations and material factors which are applied to the structural components. In geotechnical engineering the application of partial factors to the various geotechnical parameters has not been found practical in general design and overall factors of safety are still used.

For the purposes of reinforced soil design a limit state is deemed to be reached when one of the following occurs:

- a) collapse or major damage;
- b) deformations in excess of acceptable limits;
- c) other forms of distress or minor damage which would render the structure unsightly, require unforeseen maintenance or shorten the expected life of the structure.

The condition defined in a) is the ultimate limit state, and b) and c) are serviceability limit states. Practice in reinforced soil is to design against the ultimate limit state and check for the serviceability limit state. In reinforced soil design some of these limit states may be evaluated by conventional soil mechanics approaches (e.g. settlement). In this case design loads should be applied to the soils as for the design of a conventional structure. Other deformations may be due to excessive strain in the reinforcements and current design practice is generally to ensure that an adequate factor of safety against excessive loading of the reinforcements is available.

5.2 Service life

The service life of reinforced soil structures should be considered in design. In most applications the selected design life of the reinforcing elements is equal to the service life of the structure. In certain cases, mostly foundations to embankments, the entire structure can have a long term service life but it may only be necessary for the reinforced portion to function for a shorter time while the surrounding ground gains strength.

Table 12 gives examples for the categorization of the service life of reinforced soil for a variety of applications.

For each service category consideration should be given to:

- site investigation requirements;
- environmental and loading considerations;
- requirements for handling, storing and placing materials;
- quality control;
- safety margins appropriate to that particular category of structure;
- demolition during or at the end of service life.

5.3 Factors of safety

5.3.1 General

The recommended approach to applying factors of safety is one of partial material and load factors as recommended in CIRIA [17]. The partial factor format developed below is appropriate to reinforced soil where a variety of materials may be used for structures of various selected service lives and where the ramifications of failure depend upon geometries and end use.

It is recognized that the magnitude of a factor of safety influences structural behaviour. In the case of reinforced soil a larger factor of safety implies a stronger and stiffer structure subject to less deformation and possibly increased stresses in the reinforcement, [18].

In principle the overall factor of safety of a construction reflects the consequences of its failure. An approach to this is to consider the economic consequences of failure at some future time. This implies a reliability analysis approach to design, [19]. However, pending further work on this issue it is recommended that for routine design the consequences of failure should be taken into account in applying the partial factors described below.

5.3.2 Economic ramifications of failure factor

A partial factor, f_n , should be applied to take account of the ramifications of failure of the structure, see Table 3. Factor f_n will be common to all reinforcements and should be assigned values dependent upon the class of risk for the particular structure as given in Table 3. Examples of structures in categories 1, 2 and 3 identified in Table 3 are given in Figure 10, Figure 11, Figure 12, Figure 13 and Figure 14.

Following the recommendations of CIRIA [17], f_n could be applied to either the material factor f_m (5.3.3 and 5.3.4) or the load factor f_f (5.3.6). However, the application of increased (factored) external loads to an earth retaining structure or to a slope stability problem is not always unfavourable. This is because increased stress in a frictional soil results in an enhanced shear strength.

In addition, larger factors are generally applied to live loads than to dead loads and in the general case of reinforced soil structures the superimposed live loads are often small in comparison to dead loads. Therefore, the application of f_n to the reinforcement design strength is considered to result in a more consistent approach to a margin of safety due to increased ramifications of failure than if it were applied to the loads.

5.3.3 Partial material factors for reinforcements

5.3.3.1 General

Two basic partial material factors f_{m1} and f_{m2} are applicable to the reinforcements. The factor f_{m1} is related to the properties of the material itself whereas f_{m2} is concerned with construction effects and environmental effects. Each of these factors is made up from component factors as indicated in Table 15. Although two component factors are shown for each principal factor, the description of the intended purpose of each factor illustrates that a further subdivision of factors applies in practice, see Annex A.

Table 15 — Partial materials factors for reinforcements

Principal factor	Component factor	Intended purpose
f_{m1}	f_{m11}	<i>Manufacture</i> ; to cover the possible reductions in the capacity of the material as a whole compared with the characteristic value deduced from the control test specimens and possible inaccuracy in the assessment of the resistance of a structural element resulting from modelling errors.
	f_{m12}	<i>Extrapolation of test data</i> ; to take account of the confidence of the long term capacity assessment. This factor may vary with the required service life of the structure.
f_{m2}	f_{m21}	<i>Susceptibility to damage</i> ; to take account of damage during construction. This factor may be derived from site damage tests referred to in section 4.
	f_{m22}	<i>Environment</i> ; to take account of different rates of degradation due to environmental conditions.

Annex A details the principles that are appropriate for the determination of f_m for all reinforcements. A specific numerical value of f_m for steel metallic reinforcements is given. For other metallic reinforcements and polymeric reinforcements specific values of f_m can not be given as these materials are proprietary in nature.

The total material factor f_m is given by:

$$f_m = f_{m1} \times f_{m2}$$

where

$$f_{m1} = f_{m11} \times f_{m12}$$

and