

BS 8006-1:2010



BSI Standards Publication

# Code of practice for strengthened/reinforced soils and other fills

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ISBN 978 0 580 53842 1

ICS 93.020

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

Committee reference B/526/4

Draft for comment 09/30093258 DC

**Publication history**

First published October 2010

**Amendments issued since publication**

Date	Text affected
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**Summary of pages**

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 250, an inside back cover and a back cover.

## Foreword

### Publishing information

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

### Supersession

Together with BS 8006-2, this part of BS 8006 supersedes BS 8006:1995, which is withdrawn.

### Relationship with other publications

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

### Use of this document

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

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

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

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

### Presentational conventions

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

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

### Contractual and legal considerations

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

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



# Section 1: General

## 1.1 Scope

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

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

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

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

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

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

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

## 1.2 Normative references

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

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

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

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

- BS 1377-9, *Methods of test for soils for civil engineering purposes – In-situ tests*
- BS 1449-1, *Steel plate, sheet and strip – Specification for carbon and carbon-manganese plate, sheet and strip*
- BS 2569, *Control cable fittings and turnbarrels – Technical specification*
- BS 3416, *Specification for bitumen-based coatings for cold application, suitable for use in contact with potable water*
- BS 3692:2001, *ISO metric precision hexagon bolts, screws and nuts – Specification*
- BS 4147, *Specification for bitumen-based hot-applied coating materials for protecting iron and steel, including suitable primers where required*
- BS 4164, *Specification for coal-tar-based hot-applied coating materials for protecting iron and steel, including a suitable primer*
- BS 4449:2005, *Steel for the reinforcement of concrete – Weldable reinforcing steel – Bar, coil and decoiled product – Specification*
- BS 5930:1999+A1:2007, *Code of practice for site investigations*
- BS 6349, *Maritime structures*
- BS 7263-1:2001, *Precast concrete flags, kerbs, channels, edgings and quadrants – Part 1: Precast, unreinforced concrete paving flags and complementary fittings – Requirements and test methods*
- BS EN 771-3:2003+A1:2005, *Specification for masonry units – Part 3: Aggregate concrete masonry units (dense and light-weight aggregates)*
- BS EN 1990:2002+A1:2005, *Eurocode – Basis of structural design*
- BS EN 1997-1:2004, *Eurocode 7: Geotechnical design – Part 1: General rules*
- BS EN 1997-2, *Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing*
- BS EN 10025-2:2004, *Hot rolled products of structural steels – Part 2: Technical delivery conditions for non-alloy structural steels*
- BS EN 10080:2005, *Steel for the reinforcement of concrete – Weldable reinforcing steel – General*
- BS EN 12225, *Geotextiles and geotextile-related products – Method for determining the microbiological resistance by a soil burial test*
- BS EN 12878, *Pigments for the colouring of building materials based on cement and/or lime – Specifications and methods of test*
- BS EN 14475:2006, *Execution of special geotechnical works – Reinforced fill*
- BS EN ISO 1461, *Hot dip galvanized coatings on fabricated iron and steel articles – Specifications and test methods*
- BS EN ISO 10319, *Geotextiles – Wide-width tensile test*
- BS EN ISO 10320, *Geotextiles and geotextile-related products – Identification on site*
- BS EN ISO 10321, *Geosynthetics – Tensile test for joints/seams by wide-width strip method*
- BS EN ISO 14688-2, *Geotechnical investigation and testing – Identification and classification of soil – Part 2: Principles for a classification*

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

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

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

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

### Other publications

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

## 1.3 Terms and definitions

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

### 1.3.1 anchored earth

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

### 1.3.2 axially flexible reinforcement

reinforcement that can absorb tensile loads only

### 1.3.3 axially stiff reinforcement

reinforcement that can absorb tensile, shear and bending loads

### 1.3.4 cohesive frictional fill

fill containing at least 15% material passing a 63 mm sieve

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

### 1.3.5 extensible reinforcement

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

### 1.3.6 frictional fill

fill containing less than 15% material passing a 63 mm test sieve

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

### 1.3.7 geogrid

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

### 1.3.8 geotextile

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

### 1.3.9 inextensible reinforcement

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

<sup>1)</sup> PD 6694-1 is in preparation.



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

## 1.4 Symbols

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

$e_j$	Eccentricity of resultant vertical load at $j$ th level about centreline of reinforcement
$f_f$	Partial load factor applied to external dead loads
$f_{fs}$	Partial load factor applied to soil unit weight loads
$f_m$	Partial material factor applied to reinforcement
$f_{m1}$	Partial material factor related to material properties of the reinforcement
$f_{m2}$	Partial material factor related to construction effects and tolerances of the reinforcement
$f_{ms}$	Partial material factor applied to soil parameters
$f_n$	Partial factor associated with ramification of failure
$f_q$	Partial load factor applied to external live loads
$f_p$	Partial factor against pull-out failure of the reinforcement
$f_s$	Partial factor against sliding failure across the reinforcement or soil
$h$	Average height of fill above reinforcement
$h_j$	Height of fill above $j$ th layer of reinforcements
$h_p$	Height of the prop above toe of a structure
$h_t$	Height of fill above toe of a structure
$k_b$	Variable in <b>6.8.4.3.5</b>
$k_r$	Variable in <b>6.8.4.3.6</b>
$l$	Variable, defined in Figure 36
$n$	Number of shear planes resisting applied shear, in <b>6.8.4.3.3</b> ; or Number of slices in a moment stability analysis; or Sideslope of embankment
$p'_c$	Vertical stress on pile cap
$pH$	Value of acidity of an aqueous solution
$q_r$	Bearing pressure
$q_{ult}$	Ultimate bearing capacity of foundation soil
$r_u$	Pore pressure ratio
$s$	Pile spacing
$t_a$	Long term height of an anchor head
$t_d$	Design life
$t_t$	Test duration
$u$	Pore water pressure
$u_i$	Average pore pressure acting on the base of slice $i$ in stability analysis
$w_s$	Uniformly distributed surcharge on top of a structure
$z$	Depth measured from top of wall or slope
$z_a$	Depth of design line 1; coherent gravity method
$z_c$	Critical depth of foundation sliding block
$z_o$	6 m critical depth
$z_s$	Depth of dispersal of horizontal shear

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

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

$\alpha$	Inclination of slope supported by reinforced soil structure
$\alpha'$	Friction coefficient relating soil friction angle to soil/reinforcement bond
$\alpha'_{bc}$	Adhesion coefficient relating soil cohesion to soil/reinforcement bond
$\alpha_i$	Angle of inclination of base of soil slice $i$ .
$\beta$	Angle of inclination of backfill thrust on reinforced soil block
$\beta'$	Inclination of a potential failure plane to the vertical plane
$\beta_s$	Inclination of slope
$\gamma$	Unit weight density of soil
$\delta'$	Angle of shearing resistance between soil and reinforcement measured under effective stress conditions
$\delta_h$	Lateral strain of soil under an applied load
$\delta_v$	Axial compression under an applied load
$\varepsilon$	Reinforcement strain
$\varepsilon_{max}$	Maximum allowable strain in reinforcement
$\theta$	Angle of draw in embankment fill
$\theta_d$	Angle of slice $i$ in a log-spiral analysis
$\theta_j$	Angle of nail $j$ location in the slope in log-spiral analysis
$\theta_p$	Angle (to the vertical) between the outer edge of the outside pile cap and the shoulder of an embankment
$\lambda$	Load shedding coefficient
$\mu$	Coefficient of friction between the fill and reinforcing element derived from the identity ( $\mu = \tan \delta'$ where $\delta'$ is the angle of bond stress between the fill and the reinforcing elements – statistical mean of results obtained)
$\mu^*$	Apparent coefficient of friction
$\rho$	Rate of increase in undrained shear strength with depth of foundation
$\sigma$	Normal stress; or Statistical standard deviation of results obtained
$\sigma_{aj}$	Vertical stress acting at level $j$ in a wall in the resistant zone
$\sigma_b$	Maximum bearing pressure between connected parts
$\sigma_{bc}$	Ultimate bearing strength of connection components
$\sigma_c$	Tensile stress in connections
$\sigma_{ec}$	Extreme fibre bending stress for compression
$\sigma_{et}$	Extreme fibre bending stress for tension
$\sigma_h$	Horizontal stress on an element of soil
$\sigma_q$	Ultimate shear strength of connection components
$\sigma'_{rj}$	Normal effective confining stress acting on nail $j$ in the slope
$\sigma_t$	Ultimate tensile strength of connection components
$\sigma_v$	Vertical stress on an element of soil
$\sigma_{vj}$	Vertical stress acting at level $j$ in a wall

$\sigma'_v$	Applied vertical effective stress
$\sigma_1$	Major principal effective stress
$\sigma_3$	Minor principal effective stress
$\tau$	Shear stress
$\tau_c$	Average shear stress
$\phi$	Mohr-Coulomb angle of friction
$\phi_u$	Angle of internal friction of soil under total stress conditions
$\phi'$	Angle of internal friction of soil under effective stress conditions
$\phi'_{cv}$	Angle of constant volume shearing resistance under effective stress conditions
$\phi'_p$	Peak angle of shearing resistance under effective stress conditions
$\omega_j$	Inclination to the horizontal of nail $j$ in the slope



## Section 2: Concepts and fundamental principles

### 2.1 General

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

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

### 2.2 Limit state principles

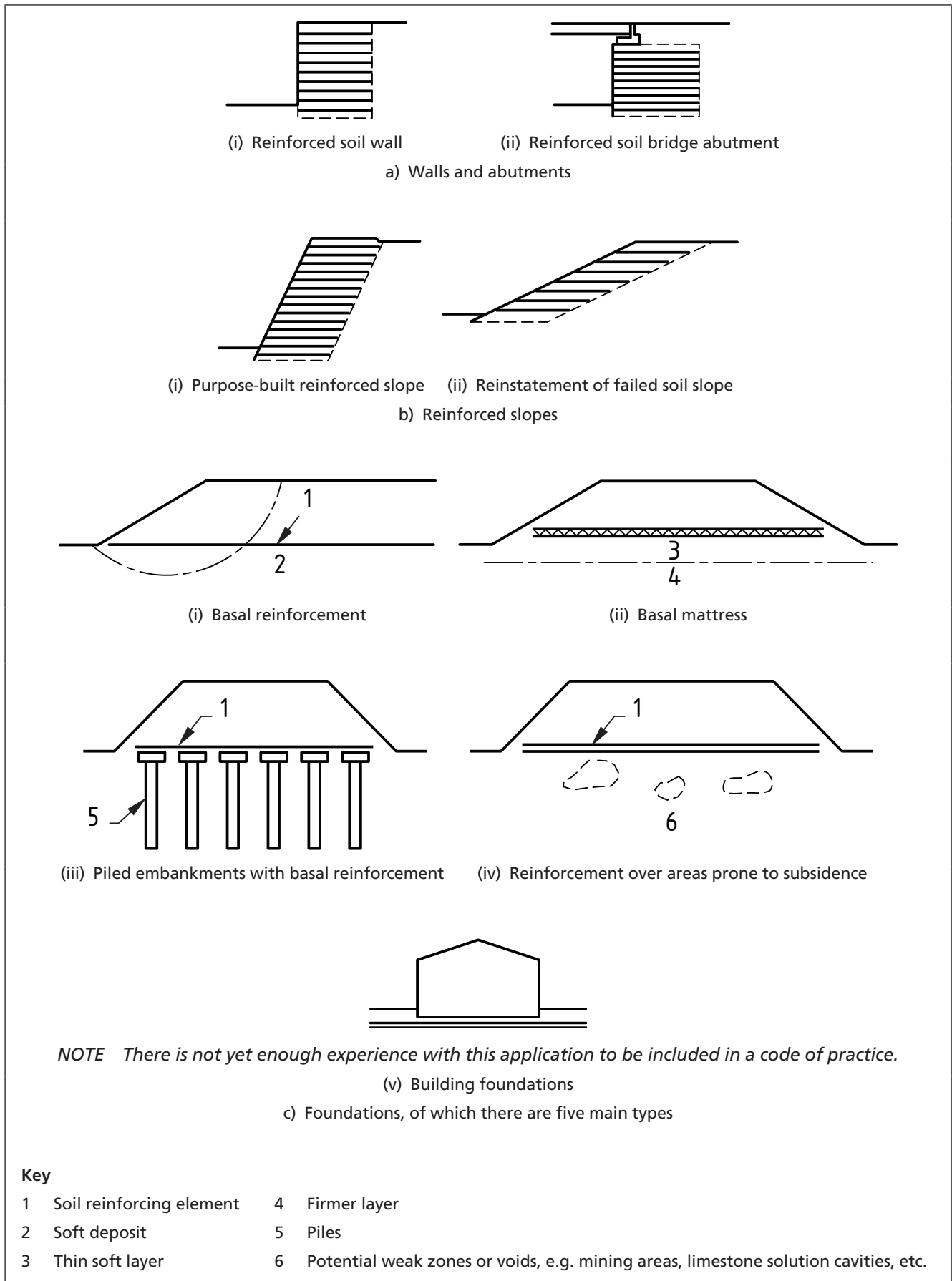
Limit state principles are applied to the design of reinforced soil walls, slopes and foundations to embankments or similarly resilient structures. The two limit states considered in design are the ultimate limit state and the serviceability limit state.

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

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

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

Figure 1 Range of applications of reinforced soil



NOTE There is not yet enough experience with this application to be included in a code of practice.

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## 2.3 Partial factors

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

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

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

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

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

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

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

## 2.4 Design loads

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

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

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

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

## 2.5 Design strengths

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

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

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

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

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

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

## 2.6 Fundamental mechanisms

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

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

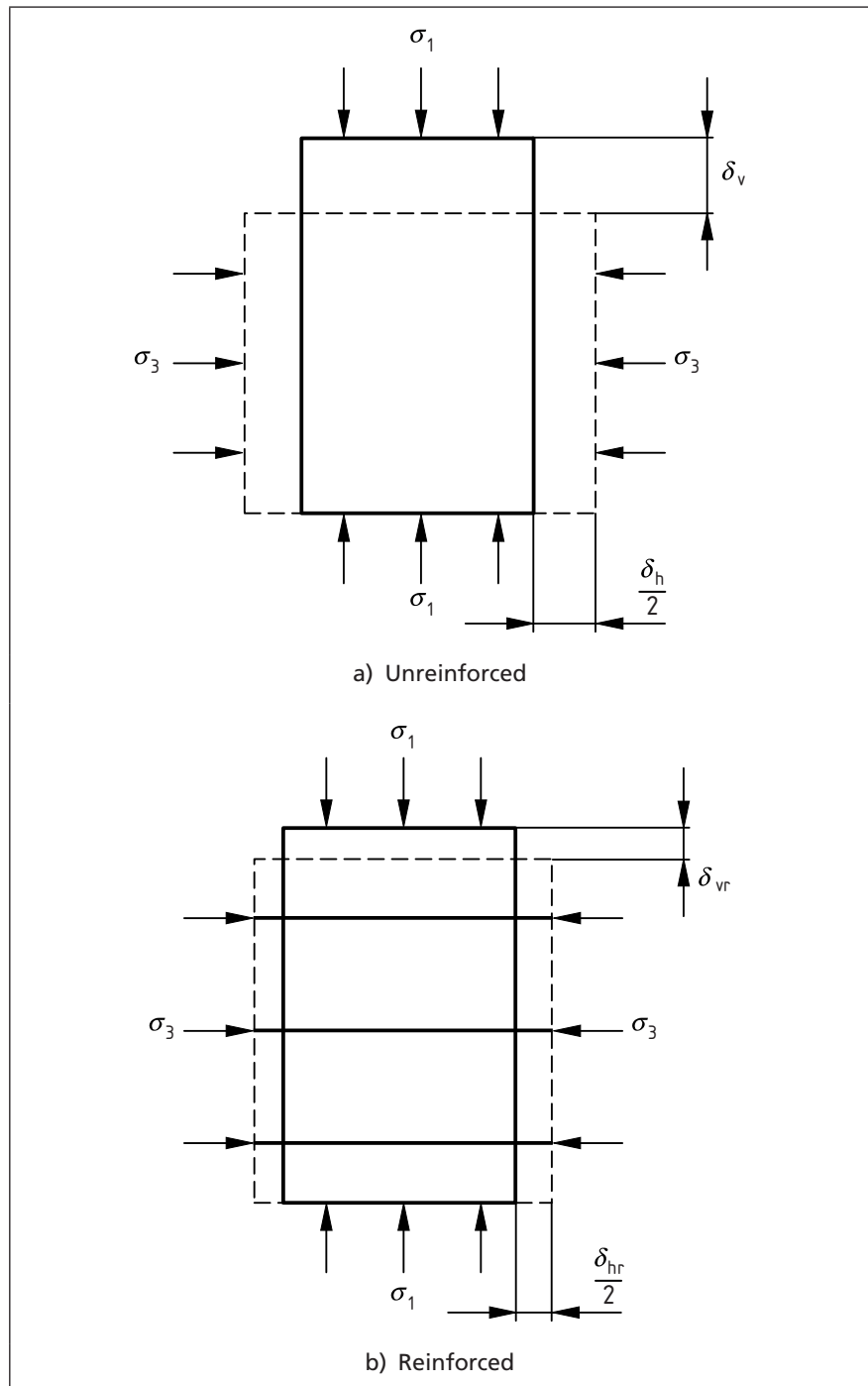
If several horizontal layers of reinforcement are inserted into the soil [see Figure 2b)] and the same external loads applied, then the resulting deformations are  $\delta_{vr}$  and  $\frac{1}{2}\delta_{hr}$  where  $\delta_{vr} < \delta_v$  and  $\frac{1}{2}\delta_{hr} < \frac{1}{2}\delta_h$ . This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

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

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

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

Figure 2 Effect of reinforcement on a soil element



In the above illustration, both reinforced and unreinforced samples are subjected to the same magnitudes of externally applied load and the addition of reinforcement decreases deformations compared to the unreinforced soil. The addition of reinforcement will also improve soil strength.

If the unreinforced soil is confined by a constant stress  $\sigma_3$  and the magnitude of  $\sigma_1$  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  $\sigma_1$  is needed to cause failure. This is because increments of  $\sigma_1$  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  $\beta_s$  to the horizontal, where  $\beta_s$  is greater than the internal angle of shearing resistance. Without the benefit of soil reinforcement the slope would collapse, however, by the incorporation of suitable soil reinforcement the slope may be rendered stable. Investigations of the basic reinforcing mechanisms reveal that the soil in the slope comprises two distinct zones. These are shown in Figure 3 as the active zone and the resistant zone. Without reinforcement the active zone is unstable and tends to move outwards and downwards with respect to the resistant zone.

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

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

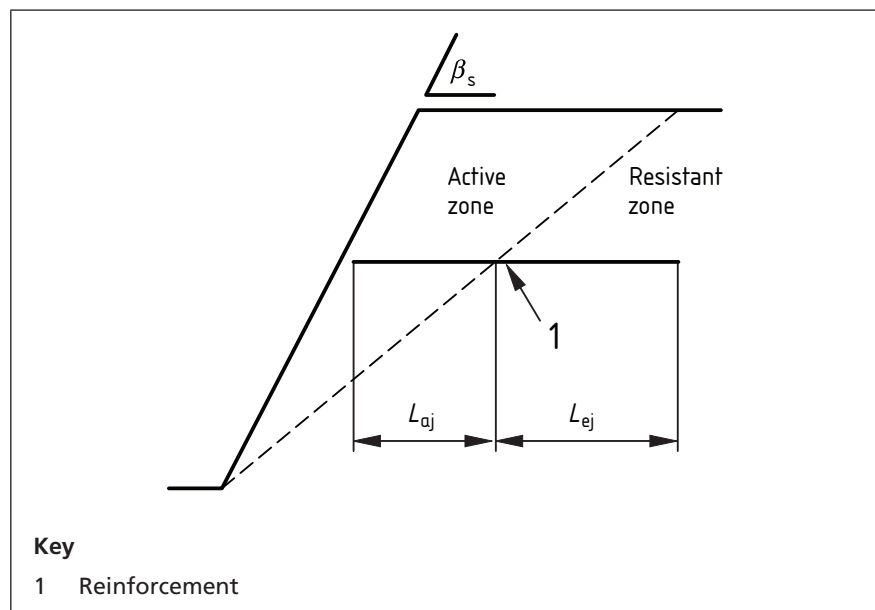
Provided the reinforcement develops an adequate bond, and has adequate tensile stiffness, it will absorb tensile strains developed in the soil in the active zone. These tensile strains are transferred from the soil, to the reinforcement, through the mechanism of soil/reinforcement bond. The tensile strains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

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



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

Figure 3 Reinforcing mechanisms in walls and slopes



## 2.8 Soil reinforcing mechanisms in embankment foundations

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

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

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

Lateral extrusion and spreading failures can be prevented by the inclusion of horizontally inclined reinforcement at the interface of the embankment fill and the foundation soil. Provided that the

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

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

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

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

## 2.9 Soil reinforcement interaction

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

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

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

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

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

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

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

## 2.10 Soil properties to be considered

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

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

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

In particular, but with the exceptions cited above, the shear strength of fill or soil incorporating multiple layers of reinforcement [see Figure 1 a) and Figure 1 b)] is considered to be purely frictional. Since the strains induced in multiple layer reinforcement such as walls and slopes are small, the frictional strength is represented by  $\phi_p$ , 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  $\phi'_{cv}$ , 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  $\phi'_{cv}$ . This value is used for short, intermediate and long term analysis with due regard for pore water pressures.

Table 1 Factors affecting performance

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

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

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

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

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

## 2.11 Reinforcement geometry

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

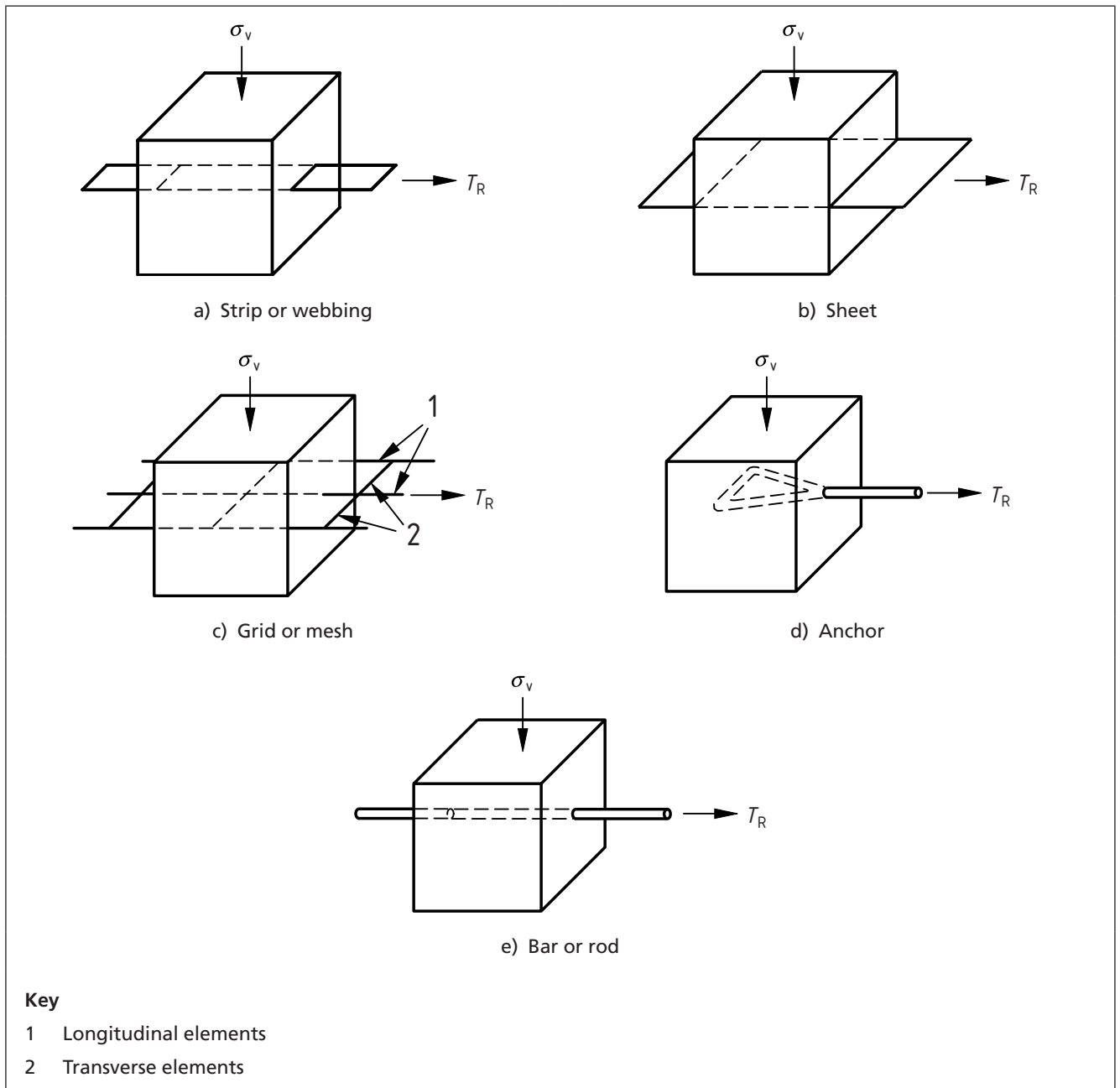
Soil reinforcement can take a variety of forms, some of which are shown in Figure 4. Grids, meshes and strips can be metallic or polymeric whilst sheet reinforcement takes the form of polymeric geotextiles. Anchored earth fill employs multiple layers of flexible steel bars or polymeric materials, which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

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

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

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

Figure 4 Forms of reinforcement



## 2.12 Reinforcement bond

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

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

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

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

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

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

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

## 2.13 Effects of flexible reinforcement axial stiffness on loads

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

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



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

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

## 2.14 Factors affecting tensile behaviour of flexible reinforcement

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

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

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

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

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

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

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

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

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

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

## Section 3: Materials

### COMMENTARY ON SECTION 3

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

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

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

	<b>Walls and abutments</b>	<b>Steep slopes (<math>\leq 70^\circ</math>, <math>&gt; 45^\circ</math>)</b>	<b>Shallow slopes (<math>\leq 45^\circ</math>)</b>	<b>Foundations</b>	<b>Comments</b>
<b>Fill materials</b>					
<b>Class or type</b>	Selected granular fills (Classes 6I/J), Selected cohesive fills (Classes 7B/C/D) and Chalk (see Note 3)	Selected granular fills (Classes 6I/J), Selected cohesive fills (Classes 7B/C/D), General granular fill (Class 1), General cohesive fill (Class 2)(see Note 4) and Chalk (see Note 3)	Selected granular fills (Classes 6I/J), Selected cohesive fills (Classes 7B/C/D), General granular fill (Class 1), General cohesive fill (Class 2), and Chalk (see Note 3)	Classes 6F2 and 6F3	Fill classes as SHW Table 6/1. Class 7B not permitted for steel reinforcing elements. Specific drainage measures may be required with cohesive fills. See also 3.1.3.2 and 3.1.3.3.
<b>Permitted constituents</b>	SHW Table 6/1	SHW Table 6/1	SHW Table 6/1	SHW Table 6/1	
<b>Grading</b>	SHW Table 6/1	SHW Table 6/1	SHW Table 6/1	SHW Table 6/1	Grading not applicable to chalk
<b>Shear strength (<math>\phi</math>) test</b>	BS 8006-1 4.2.1	BS 8006-1 4.2.1	BS 8006-1 4.2.1	BS 8006-1 4.2.1	SHW, cl 636
<b>Friction (<math>\mu</math>) test</b>	BS 8006-1 4.3.3	BS 8006-1 4.3.3	BS 8006-1 4.3.3	BS 8006-1 4.3.3	SHW, cl 639
<b>Compaction</b>	SHW Table 6/1 and Table 6/3	SHW Table 6/1 and Table 6/3	SHW Table 6/1 and Table 6/3	SHW Table 6/1 and Table 6/3	
<b>Chemical</b>	BS EN 14475:2006 Table B.1 for steel elements	BS EN 14475:2006 Table B.1 for steel elements	BS EN 14475:2006 Table B.1 for steel elements	BS EN 14475:2006 Table B.1 for steel elements	See Table 2. For polymeric, see appropriate proprietary product certification.
<b>Soil reinforcements</b>					
<b>Steel types and specifications</b>	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	See Table 3 for design properties of some materials listed in BS EN 14475:2006. See Table 4 for corrosion allowances
<b>Polymeric specifications</b>	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	BS EN 14475:2006	See appropriate proprietary product certification for design properties

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

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

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

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

**NOTE 3** Information on chalk fill is given in 6.10.2.5.

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

SHW = Specification for Highway Works [1]

## 3.1 Soils and fills

### 3.1.1 General

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

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

### 3.1.2 Electrochemical criteria

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

### 3.1.3 Non-standard fills

#### 3.1.3.1 General

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

### 3.1.3.2 General cohesive fill

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

### 3.1.3.3 Pulverized-fuel ash (PFA)

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

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

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

### 3.1.3.4 Colliery spoil

#### COMMENTARY ON 3.1.3.4

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

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

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

### 3.1.3.5 Argillaceous materials

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

#### COMMENTARY ON 3.1.3.5

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



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

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

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

### 3.1.3.6 Chalk

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

### 3.1.3.7 Friable materials

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

## 3.2 Reinforcing materials

### COMMENTARY ON 3.2

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

### 3.2.1 Metallic soil reinforcements

#### 3.2.1.1 General

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

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

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

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

### 3.2.1.2 Corrosion allowance

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

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

Table 3 Minimum properties of some different types of steel reinforcement

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

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

Design 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
10	B	0.35	0.4
	G	0	0
50	B	1.15	1.55
	G	0.3	0.55
60	B	1.35	1.68
	G	0.38	0.63
70	G	0.45	0.7
120	G	0.75	1.0

#### Key

- B black steel (ungalvanized)  
G galvanized steel

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

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

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



### 3.2.1.3 Handling

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

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

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

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

### 3.2.1.4 Storage

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

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

### 3.2.1.5 Placing

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

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

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

## 3.2.2 Polymeric reinforcements

### 3.2.2.1 General

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

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

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

### 3.2.2.2 Handling

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

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

### 3.2.2.3 Storage

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

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

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

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

### 3.2.2.4 Placing

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

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

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

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

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

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

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

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

### 3.2.3 Polymeric reinforcement joints

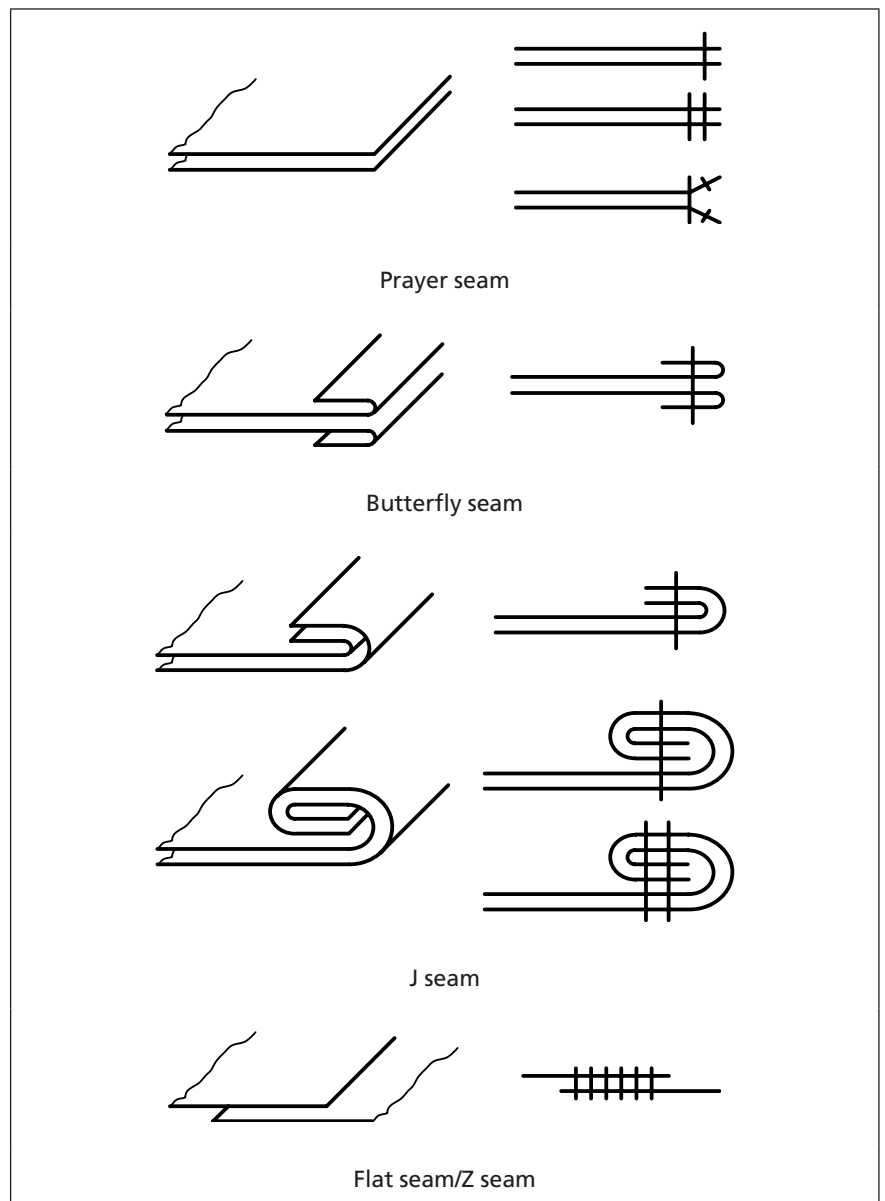
#### 3.2.3.1 General

**COMMENTARY ON 3.2.3.1**

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

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

Figure 5 Types of seams



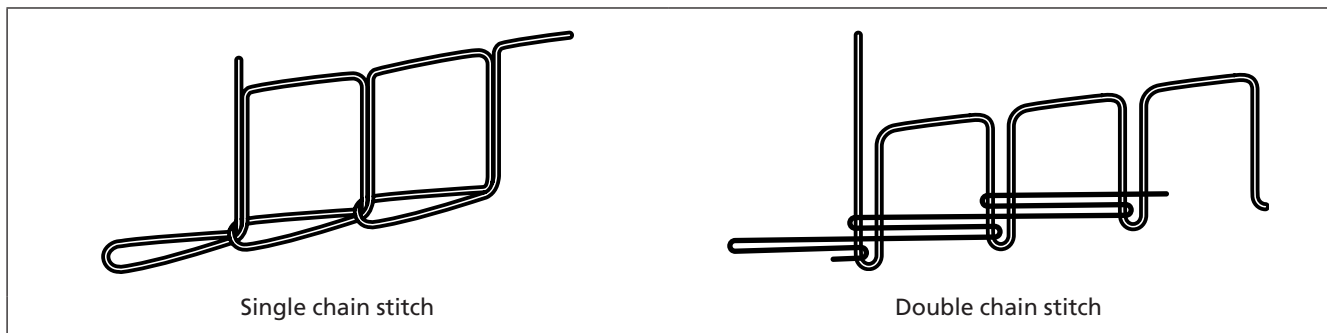
Joints should be formed to have the highest mechanical and durability efficiency possible, compared to the performance characteristics of the parent materials. Test methods used to assess joints should correspond closely to those procedures employed when determining the properties of the parent materials.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

### 3.2.3.2 Overlaps

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

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

Figure 6 Stitch configuration



### 3.2.3.3 Bodkin joints

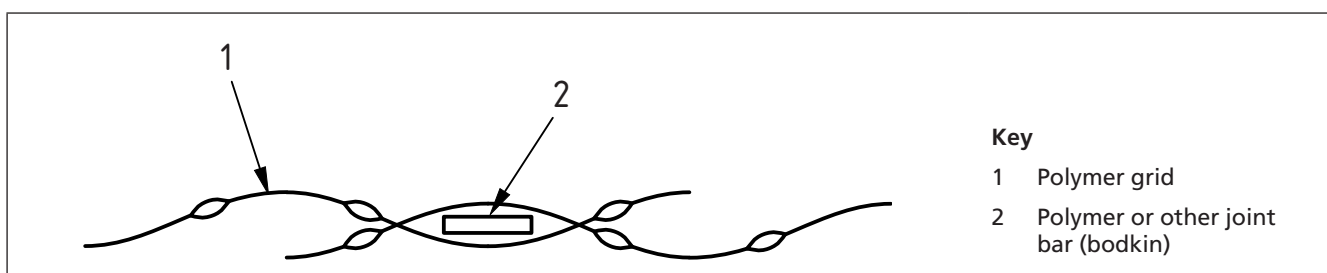
#### COMMENTARY ON 3.2.3.3

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

Care should be taken to ensure that:

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

Figure 7 Bodkin joint



#### 3.2.3.4 Stapling

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

#### 3.2.3.5 Other jointing methods

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

### 3.3 Facings

#### 3.3.1 General

##### COMMENTARY ON 3.3.1

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

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

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

#### 3.3.2 Panel facing units – Joint filler materials

##### 3.3.2.1 General

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

##### 3.3.2.2 Bedding material

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

##### 3.3.2.3 Sealing material

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

#### 3.3.3 Concrete block facing of segmental block walls

##### 3.3.3.1 General

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

Blocks should conform to BS EN 771-3:2003+A1 and the following:

- Min concrete cube strength 30 N/mm<sup>2</sup> at 28 days;
- Maximum water absorption of 6% when tested in accordance with the method of BS 7263-1:2001, Annex C;
- Minimum density: 2 100 kg/m<sup>3</sup>;
- Minimum cement content: 365 kg/m<sup>3</sup>;
- Maximum water/cement ratio: 0.5.

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

### 3.3.3.2 Tolerances for blocks

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

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

#### COMMENTARY ON 3.3.3.3

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

## 3.4 Fasteners and connections between the facing and reinforcing elements

### 3.4.1 General

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

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

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

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

Recommended steel coatings may be divided into two groups:

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

### 3.4.2 Stresses

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

## 3.5 Testing materials and components not covered by relevant specifications

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

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

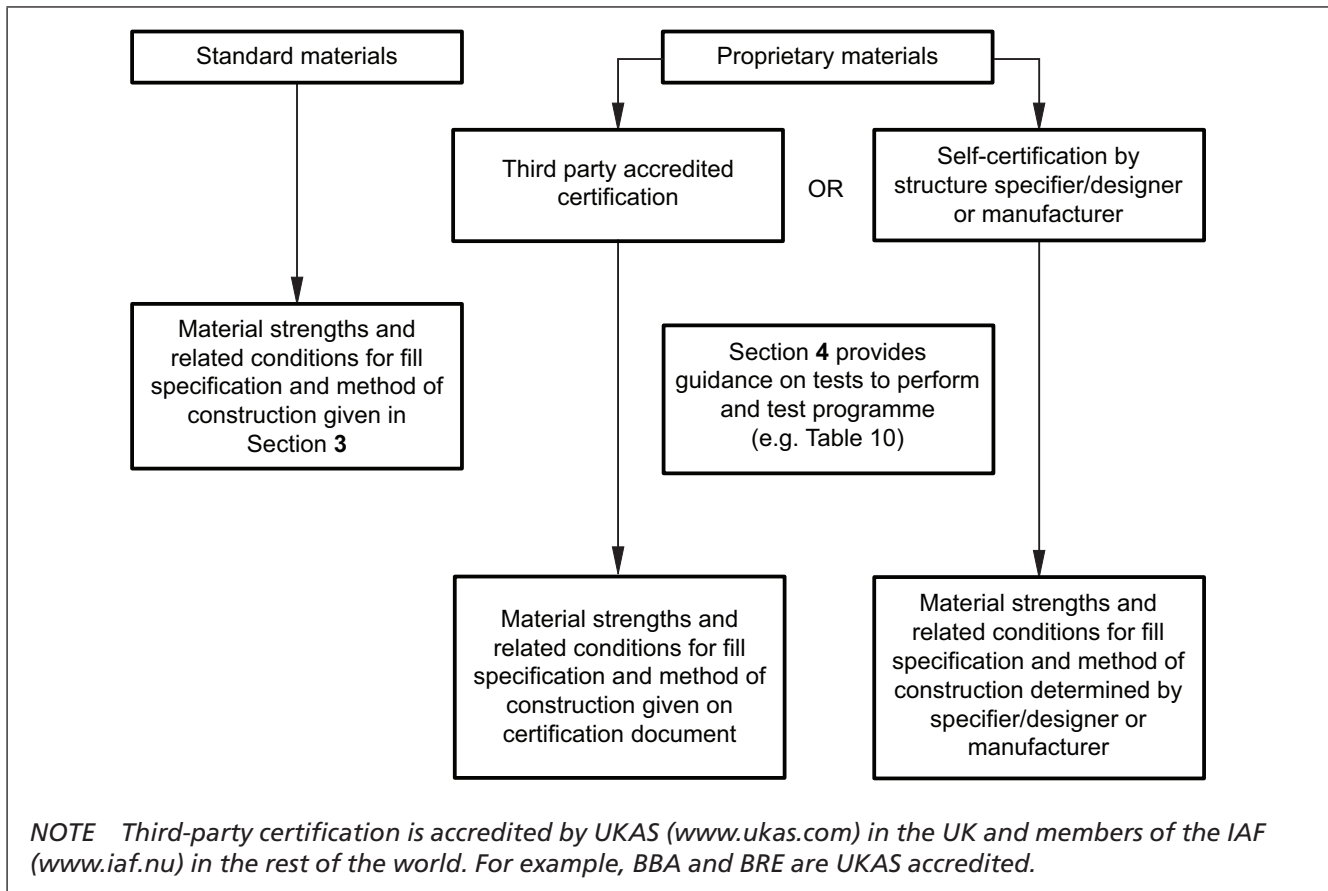
Table 5 Properties of bolts and screws up to 40 mm stock size

Material	Tensile strength	Shear strength	Bearing strength
	$\sigma_t$ N/mm <sup>2</sup>	$\sigma_q$ N/mm <sup>2</sup>	$\sigma_{bc}$ N/mm <sup>2</sup>
Alloy steel to BS 3692:2001			
grade 8.8	800	480	800
grade 10.9	1 000	600	1 000

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

Material	Tensile strength	Shear strength	Bearing strength
	$\sigma_t$ N/mm <sup>2</sup>	$\sigma_q$ N/mm <sup>2</sup>	$\sigma_{bc}$ N/mm <sup>2</sup>
Carbon steel to grade B500 of BS 4449:2005 and BS EN 10080:2005	525	315	525

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



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## Section 4: Testing for design purposes

### 4.1 General

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

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

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

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

Table 7 Examples of service life

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

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

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

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

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

## 4.2 Fill and ground

### 4.2.1 General

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

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

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

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

### 4.2.2 Mobilized soil shear strength

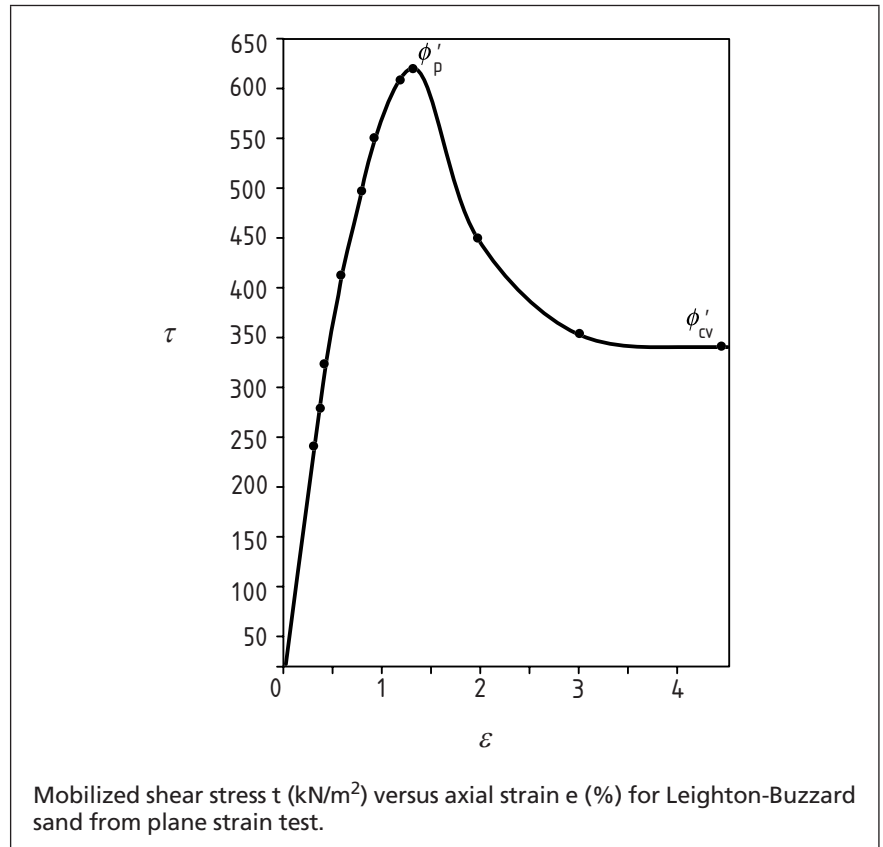
It should be noted that in soil subject to a compressive loading regime, mobilized shear strength increases with increasing axial and lateral strain until the peak shear strength is mobilized. Also, in elasto-plastic soils shear strength continues to be mobilized at the peak value when strains exceed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

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

mobilized at axial strains of 1% or less. For greater strains the mobilized friction angle may be assumed to tend towards  $\phi'_{cv}$ .

For this code of practice  $\phi'_p$  should be used for walls, abutments and steep slopes, and  $\phi'_{cv}$  should be used for fill to shallow slopes and embankments founded on weak foundations.

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



#### 4.2.3 Fill deformation and strength

Experience has shown that with inextensible reinforcements lateral stresses in the upper fill behind a wall are largely determined by compaction stresses, and may be approximated to a  $K_0$  condition. Guidance is given in Section 6, Section 7 and Section 8 on the appropriate selection of earth pressure  $k$ .

*NOTE* The mobilized shear strength and the coefficient of earth pressure  $K$  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.

#### 4.2.4 Fill strength related to reinforcement strain

The axial tensile stiffness of reinforcement materials may be classified as extensible or inextensible (see 2.5).

It should be noted that 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  $\phi'$  and effective cohesion  $c'$  may be determined by shear box or triaxial tests in accordance with clause 636 of the *Specification for Highway Works* [1], or BS 1377-7 and BS 1377-8.

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 the in situ ground may be determined using conventional techniques for site investigation and testing described in BS EN 1997-2, BS 5930:1999+A1, BS EN ISO 14688-2 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

##### 4.2.7.1 Soil chemistry

It should be borne in mind that 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, should be placed on the fill used with galvanized or ungalvanized steel (see BS EN 14475:2006, Table B.1); 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 4.

For geosynthetic reinforcement, a reduction factor should be used with respect to the aggression of a fill, or soil, which is defined in Annex A as  $RF_{chr}$ , and is a function of the specific polymeric reinforcement and in particular the specific polymer, and additives, used in the reinforcement. Consideration should be given to the fact that 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. Further guidance on the testing methods for evaluating a polymeric reinforcement may be found in 4.3.5.

##### 4.2.7.2 Soil grading and constitution

Consideration should be given to the durability of both metallic and polymeric reinforcement, which can 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 that may be anticipated will vary from one reinforcement product to another. In general surface scratches may be assumed to 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 should be expected to be more severe than for metallic reinforcement and will be dependent on the specific polymer and additives used to form each proprietary product. The susceptibility of polymeric reinforcement to damage during installation e.g. cuts, tears, splits and perforations may be assessed by site or full-scale trials to enable the value of the partial material factor  $RF_{id}$ , and other relevant properties (see 4.3) to be determined. (Further guidance can be found in Annex A.)

## 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 that should not be exceeded.

Design against collapse should consider both internal and external stability. The main influence of the soil reinforcement on external stability that should be considered 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 Section 6, Section 7 and Section 8. Similarly these sections consider various aspects of design with respect to internal stability.) However, in general, internal stability may be controlled by avoiding failure of the reinforcement through either tensile rupture or by loss of bond.

Consequently, the objective should be to devise meaningful and reproducible test methods that 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 that 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 should be assessed on the basis of direct shear testing as described in clause 639 of the *Specification for Highway Works* [1]. The effective angle of internal friction and effective cohesion of the fill to be reinforced should be determined as described in 4.2.5.

For walls and slopes bond failure should be assessed using peak bond strength, which is reduced by a partial factor ( $f_p$  or  $f_s$ ) to give a design bond strength. These partial factors are defined in Section 6 and Section 7. For reinforced foundations bond failure should be assessed using the large strain bond strength with the fill with a partial factor defined in Section 8 applied to give a design bond strength.

For certain reinforcing elements such as anchored fill systems, shear box testing should not be used. 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.

*NOTE 1 The use of the shear box to assess bond strength reflects current design practice in accordance with the Specification for Highway Works [1]. Whilst the shear box test ought to prove adequate for quality control purposes the use of laboratory (e.g. BS EN 13738:2004) and in situ pull-out tests may be appropriate to assess load-displacement characteristics with respect to serviceability.*

*NOTE 2 BS EN ISO 12957-1:2005 describes an index test method to determine the friction characteristics of geotextiles and geotextile related product in contact with standard sand.*

*NOTE 3 BS EN 13738:2004 describes a test method for measuring the pull-out resistance of geotextiles and geotextile related products.*

### 4.3.4 Durability and performance with time

#### 4.3.4.1 General

Durability may be defined as 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 checklist of factors that should be considered is given in Table 8.

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 durability of a reinforcement product will be influenced by, and therefore selected with reference to, both: general factors reflecting environments and in-service conditions which will apply to all soil reinforcements, and also specific factors applicable to the individual reinforcement product.

For metallic and polymeric reinforcing elements general factors that should be considered include design life, loading, water and installation induced damage. For polymeric reinforcements there are two additional general factors that should be considered: pre-installation UV exposure and operational temperature.

It should be recognized that for metallic reinforcements in general performance and the variation of performance with time will be affected by the corrosivity of the soil. Therefore certain limits should be set for the electrochemical parameters of fill, see BS EN 14475:2006, Table B.1. In addition to corrosive environments associated with the soil, it should be noted that there can 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. It should be noted that the effects of loading and construction damage to metallic reinforcement might not be time dependent. However, corrosion, which is related to the electrochemical nature of

the soil and the air/water requirement, is time dependent and should be considered as a reduction in net reinforcement cross-sectional area with time. For the conditions pertaining to reinforced soil, the effects of individual factors should be assumed to be additive and not interdependent; thus rates of corrosion, for example, do not vary with load intensity.

The performance of polymeric reinforcements are affected by a number of inter-related variables that should be considered; 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 should be assumed to interact. A consequence of this is that the sum of the effects of discrete environments can be less than the total effects of a compound environment, and any such synergism should be considered in design. As with metallic reinforcement there are general and specific factors which should be assumed to affect performance.

**Table 8 Checklist for investigations of reinforcement products**

<b>1</b>	<i>Physical and mechanical properties:</i>
	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.
<b>2</b>	<i>Durability:</i>
	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.
<b>3</b>	<i>Performance during installation and use:</i>
	handling, storage <sup>A)</sup> damage <sup>A)</sup> extension and movement <sup>A)</sup> effects on facing <sup>A)</sup> .
<b>4</b>	<i>The effect of the combination of load, chemical environment and construction damage:</i>
	i.e. combination of 1, 2 and 3
<b>5</b>	<i>Construction details:</i>
	fasteners: effect on 1 b) and 2
<b>6</b>	<i>Quality control procedures with production processes</i>
<b>7</b>	<i>Preconstruction exposure to an aggressive environment:</i>
	e.g. long term effect of short term UV exposure on polymeric reinforcements or ozone and marine spray on metallic reinforcements
<b>8</b>	<i>The effects of static fatigue however induced:</i>
	e.g. structural loading, thermal loading
<b>9</b>	<i>The effects of blasts, fires or vandalism on the behaviour of the structure</i>
<sup>A)</sup>	The topics to be checked by site trial(s).

Consideration should be given to how the specific factors will vary according to the product under consideration, and 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 might be degraded in a highly alkaline environment



whilst another might not. It should be noted that in addition to the additives incorporated in the polymer, the manufacturing process and the physical form of the reinforcement will affect performance. Consequently testing should be product specific. At present there are no formalized limitations on the environments in which polymeric reinforcements may be expected to operate.

#### 4.3.4.2 Tests for durability

*NOTE Tests for polymeric reinforcement durability are reviewed in PD ISO/TR 20432.*

For metallic reinforcements consisting of steel, research and experience has allowed an electrochemical specification for fills to be drawn-up, see BS EN 14475:2006, Table B.1, which should be used to allow for losses in reinforcement thickness less than the limits given in Table 4. 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 that could affect the performance of polymeric reinforcements, although generally it may be anticipated that conventional chemical and bacteriological contents of fills and natural soils will not pose any problems.

The designer should consider the site specific aspects of a scheme where, for example, petrol or other chemical spillage might occur.

It should be noted that 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 may often be dealt with by using preventative design measures such as the incorporation of impermeable barriers and effective drainage systems.

#### 4.3.4.3 Design parameters

The reinforcement base strength at a time equal to the design life should be known and may 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 may 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 3.2.1.2).

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.

It should be noted that, for polymeric reinforcements, the reinforcement base strength might not be governed by tensile creep rupture; instead, it might be governed by the strain occurring after construction or it might be governed by the load in the reinforcement after stress (or load) relaxation at constant strain.



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 should be given to the concept of stress relaxation.

#### 4.3.5 Tests for polymeric reinforcements

Tests that may be used for evaluating polymeric reinforcement are reviewed in PD ISO/TR 20432, which covers the determination of a product's long term (creep) strain behaviour, tensile strength, creep rupture strength and the effects of installation damage, weathering, chemical and biological degradation on the product's long term properties. Annex A should be used for the determination of the appropriate partial safety factors for design.

British Standard test methods may be used to measure a wide range of properties of reinforcing materials; the results of the tests as published in manufacturer's literature are normally "typical or average" values. Characteristics taken from manufacturer's CE accompanying documentation, given as the mean and tolerance, may be used to provide upper and lower bound values.

The following three basic levels of testing should be 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).
- 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.

*NOTE* In-plane flow may be assumed to be relevant if the designer considers that the reinforcement could act as a drainage channel for excess pore pressure removal; this can be of benefit in marginal quality fills. BS EN ISO 12958 details a test procedure that may be used to determine the in-plane flow capacity of a geotextile.

#### 4.3.6 Reinforcement samples

It is recommended that where the risk category or design life of the structure makes it appropriate, specifications should make provision for the recovery of reinforcement samples over an extended number of years to provide ongoing information on the structure's long term performance and the evolution of metal corrosion or polymer degradation. These retrieved samples should ideally be compared to reference samples kept in darkroom conditions.

For walls with metal reinforcement, corrosion is likely to be the main concern and stressing of the sample during retrieval is of secondary importance, so test pieces can be independent of the working structure. Access to retrieve buried samples can be through panels or facing units near the base of the wall.

For polymer reinforcement, degradation is likely to be more closely stress-related, but difficulties in obtaining appropriately stressed samples for testing without affecting the working structure or overstressing the sample upon retrieval might prove problematic.

Durability to ultra-violet exposure for an appropriate duration is of particular importance, and polymeric reinforcements should be able to meet the recommendations of 4.3.4.1.

#### 4.4 Facing units

Facing units should either be designed to the appropriate British Standards, or to a third-party accredited certification, or 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. The trial should be monitored for the following aspects of performance: ease of handling, constructability and installation accuracy repeatability, damage, and deformation or relative movements of components, including those of the facing units.

## Section 5: Principles of design

### 5.1 Design philosophy

#### COMMENTARY ON 5.1

*The philosophy followed in this document is to design against the occurrence of a limit state (see Section 2). The objective in taking this approach is to achieve compatibility with other related codes of practice and reference has been made to CIRIA R063M [20].*

*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 some applications. BS EN 1997-1:2004 now applies partial factors in geotechnical design to provide compatibility with structure design, although this standard is not applicable to the design of reinforced soil (see the note to Section 1).*

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

BS EN 1997-1:2004 does not cover the design and execution of reinforced soil structures; the values of partial factors and load factors given in BS EN 1997-1:2004 have not been calibrated for reinforced soil structures. BS EN 1997-1:2004 should not be used in the design and execution of reinforced soil. In the UK, the design and execution of reinforced fill structures should be carried out in accordance with BS 8006-1 and BS EN 14475:2006. The partial factors set out in BS 8006-1 should not be replaced by similar factors in BS EN 1997-1:2004.

For the purposes of reinforced soil design a limit state may be 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 that 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 should be 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 might be due to excessive strain in the reinforcements and the current design practice, which should be followed, 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 may be taken as 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 be designed to function for a shorter time while the surrounding ground gains strength.

Table 9 gives examples for the categorization that should be followed for the service life of reinforced soil for a variety of applications.

For each service category, consideration should be given to:

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

Table 9 Category of structure depending upon ramification of failure

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

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

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

## 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 R063M [20] and adopted in BS EN 1997-1:2004. 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 should be 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 (see Murray [21]).

In principle the overall factor of safety of a construction should reflect the consequences of its failure. An approach to this that may be followed is to consider the economic consequences of failure at some future time; this implies a reliability analysis approach to design (see Smith [22]).

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 as follows.

### 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 9. 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 9.

*NOTE* Examples of structures in categories 1, 2 and 3 identified in Table 9 are given in Figures 10 to 14.

Following the recommendations of CIRIA R063M [20],  $f_n$  may be applied to either the material factor  $f_m$  (A.2, A.3 and A.4) or the load factor  $\gamma_{fl}$  (E.2.5).

The application of increased (factored) external loads to an earth retaining structure or to a slope stability problem may not always be unfavourable; this is because increased stress in a frictional soil results in enhanced shear strength.

In addition, larger factors should generally be applied to live loads than to dead loads, and in the general case of reinforced soil structures the superimposed live loads may often be small in comparison to dead loads.

Therefore, the application of  $f_n$  to the reinforcement design strength (rupture or pullout) may be 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.

Figure 10 Examples of structures in category 1 – Applicable to walls and slopes

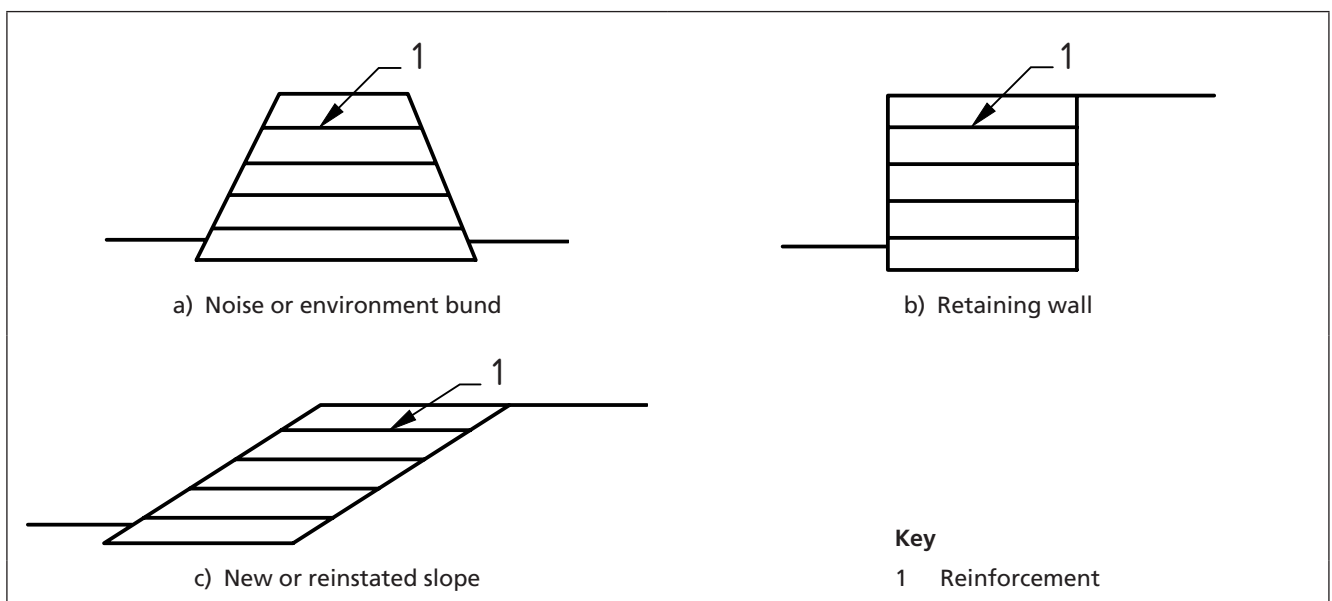


Figure 11 Examples of structures in category 2 – Applicable to walls and slopes

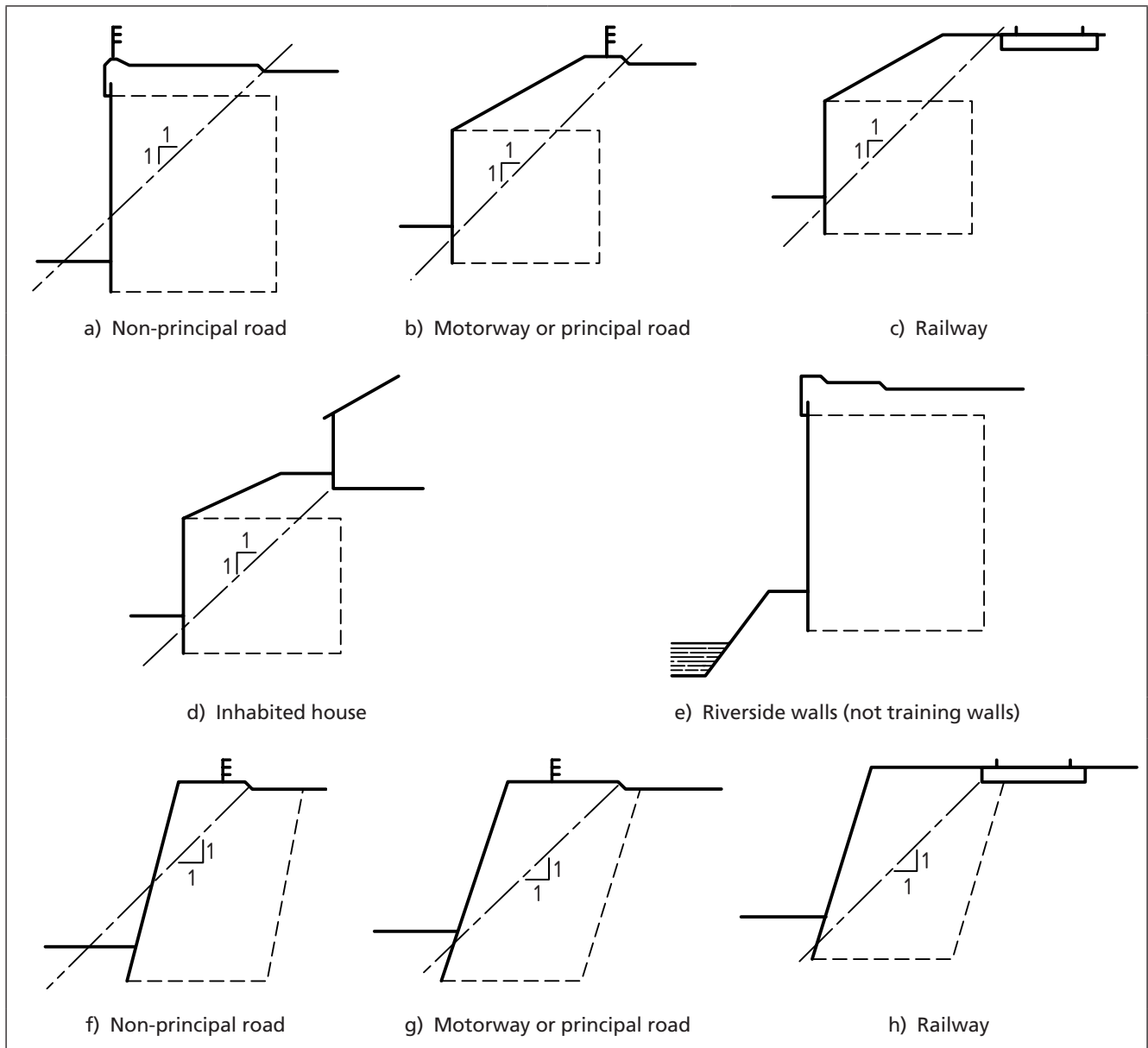


Figure 12 Examples of foundations in category 2

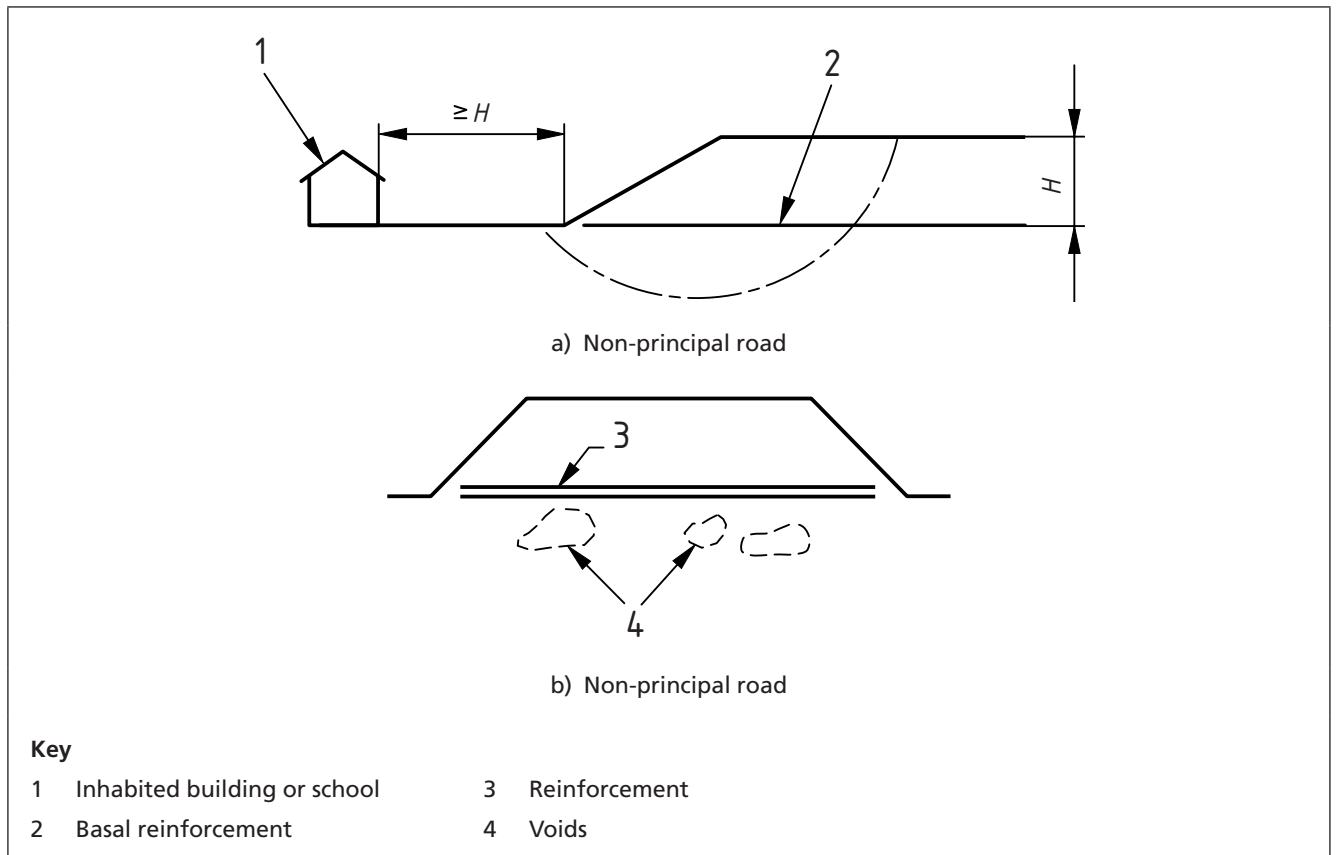
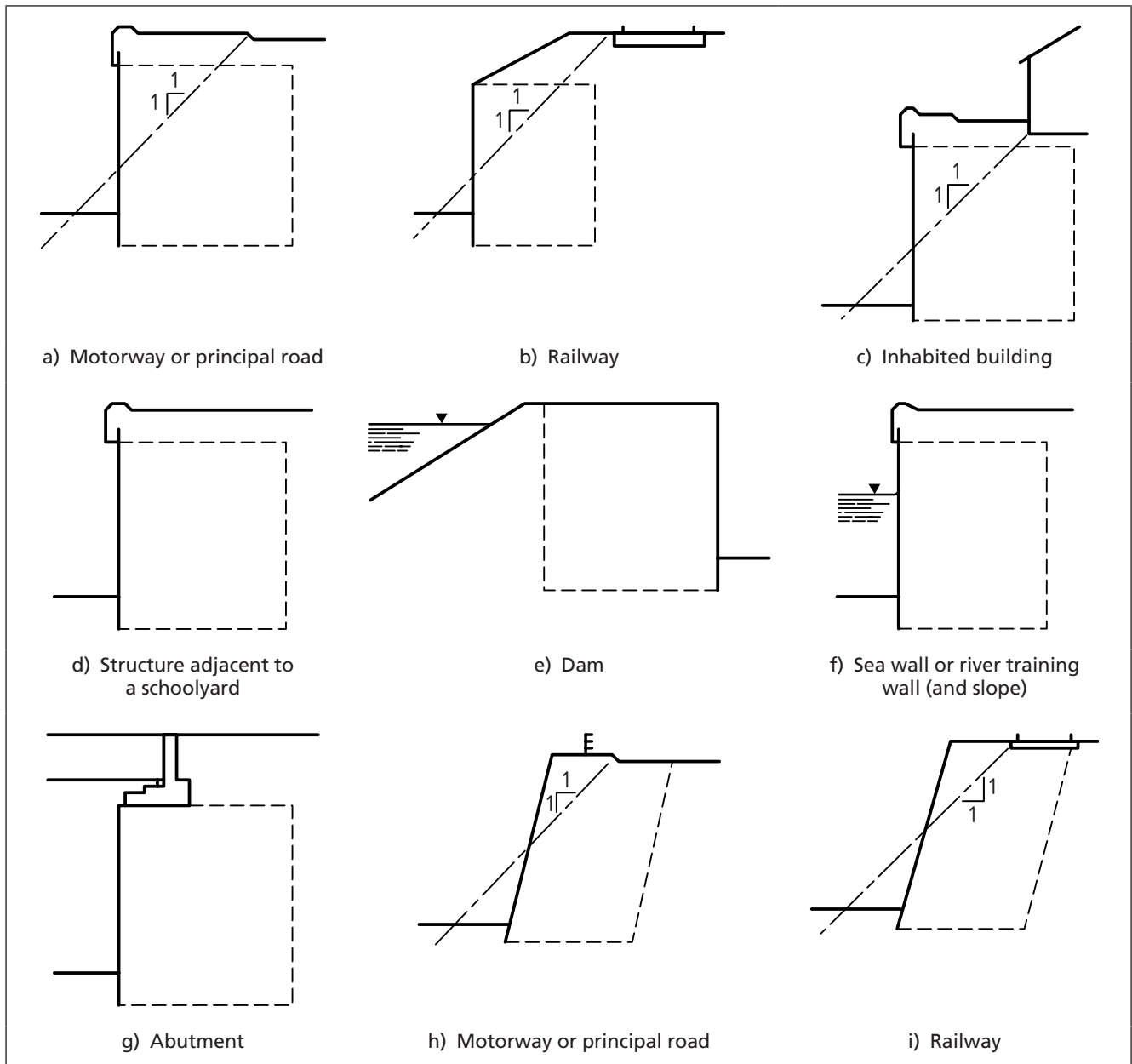


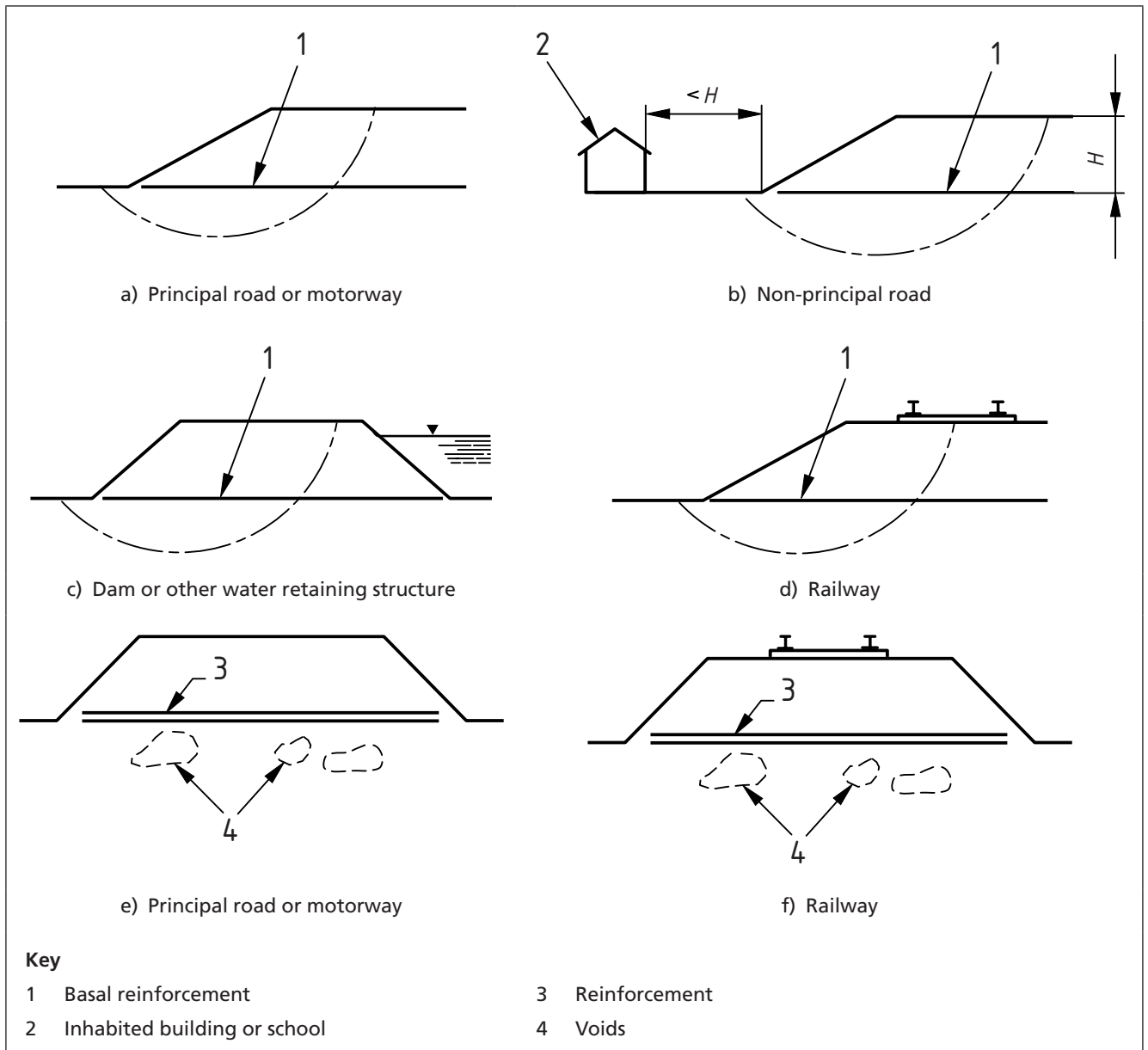
Figure 13 Examples of structures in category 3 – Applicable to walls and slopes



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Figure 14 Examples of foundations in category 3



### 5.3.3 Partial material factors for reinforcements

#### 5.3.3.1 General

For metallic reinforcements, two basic partial material factors  $f_{m1}$  and  $f_{m2}$  should be used; the factor  $f_{m1}$  is related to the properties of the material itself whereas  $f_{m2}$  is concerned with construction effects and environmental effects.

For metallic reinforcements, each of these factors should be made up from component factors as indicated in Table 10.

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 should be applied in practice, as described in Annex A.

Table 10 Partial materials factors for metallic 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 should be followed for the determination of  $f_m$  for all reinforcements. A specific numerical value of  $f_m$  that should be used for steel metallic reinforcements is given. For other metallic reinforcements and polymeric reinforcements specific values of  $f_m$  that should be used cannot be given as these materials are proprietary in nature.

The total material factor  $f_m$  for metallic reinforcements should be taken to be:

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

Where:

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

$$f_{m2} = f_{m21} \times f_{m22}$$

For polymeric reinforcements, the relationship between the components of  $f_m$  should be as shown in A.3.

### 5.3.3.2 Partial material factors for metallic reinforcements

Recommendations covering the assessment of the partial material factors for metallic reinforcements are detailed in A.2.

The design tensile strength of metallic reinforcements should be:

$$T_D = \frac{T_u}{f_m}$$

where

$T_D$  is the design strength of the reinforcement;

$T_u$  is the ultimate tensile strength of the reinforcement (its base strength);

$f_m$  is the partial material factor for the reinforcement.

### 5.3.3.3 Partial material factors for polymeric reinforcements

The following principles should be applied in assessing the design tensile strength of materials which exhibit long term creep behaviour; the procedure for evaluating the factors is described in A.3.

The design loads which may be applied to the reinforcement will be given by an ultimate limit state, governed by the tensile strength of the reinforcement, or by a serviceability limit state governed by prescribed limiting strains in the reinforcement. The design strengths of the reinforcing materials should be derived on the basis of the following two principles and both limit states should be satisfied in the design.

- a) Reinforcement should not exceed its ultimate limit state during the design life of the structure, i.e. the reinforcement should not rupture.
- b) Reinforcement should not exceed its serviceability limit state during the design life of the structure, i.e. creep in the reinforcement should remain within prescribed limits.

For walls, steep slopes and embankments, the ultimate limit state design strength of the reinforcement  $T_D$  should be taken as  $T_{CR}/f_m$  where  $T_{CR}$ , the reinforcement base strength, is the tensile creep rupture strength of the reinforcement, at the appropriate times and design temperature, and  $f_m$  is the partial material factor for the reinforcement for the same times and design temperature. The ultimate limit state design load  $T_j$  should be calculated using prescribed load factors, and loading cases, to ensure that  $T_j \leq T_D$  at all times during the design life.

For walls, steep slopes and embankments, the serviceability limit state design strength of the reinforcement  $T_D$  should be taken as  $T_{CS}/f_m$  where  $T_{CS}$ , the reinforcement base strength, is the tensile load in the reinforcement, demonstrated in Figure 43, which induces the prescribed limiting value of post construction strain in the reinforcement given in 6.6.3.2 for walls and 7.4.6 for slopes. The average serviceability limit state design load,  $T_{avj}$ , should be calculated using prescribed load factors to ensure that  $T_{avj} \leq T_D$  at all times during the design life.

As set out in 6.6.5 post-construction strain may be related to the average load in the reinforcement.

The average load in the  $j$ th level  $T_{avj}$  may be related to the maximum load in the reinforcement  $T_j$  by a factor  $k$  such that  $T_{avj} = T_j/k$ . The factor  $k$  may have a minimum value of unity and generally falls in the range 1.0 to 2.0.

The average tensile load  $T_{avj}$  may be calculated by dividing the area of the load distribution diagram along the reinforcement by the loaded length of the reinforcement. Where the distribution of tensile load along the loaded length of the reinforcement is not proven by field measurements  $k$  should be taken as unity.

For embankments with reinforced soil foundations on poor ground the serviceability limit strain should be taken to be the maximum strain; in 8.3.2.2, 8.3.3.3 and 8.4.3  $T_{CS}$  is the maximum tensile load in the reinforcement that does not cause the prescribed serviceability limit state strain to be exceeded during the design life. Where loads in the reinforcement, and  $T_{CS}$ , decrease over time,  $T_{CS}$  may be assessed using isometric creep plots. For embankments on piled foundations, 8.3.3.3 also prescribes a limited value of long term reinforcement strain from which a value of  $T_{CS}$  may be determined using the principles demonstrated in Figure 43. The serviceability limit state design strength of the reinforcement  $T_D$  should be taken as  $T_{CS}/f_m$  and the serviceability limit state design load should be calculated using prescribed load factors to check that nowhere does it exceed  $T_D$  at any time during the design life.

### 5.3.4 Partial material factors for soils

Partial material factors are prescribed for soil parameters to account for uncertainty. The design values for soil parameters should either be assessed directly or should be derived from characteristic values, see 2.5, using the relationship:

$$X_d = \frac{X_k}{f_{ms}}$$

where:

- $X_d$  is the soil parameter design value;
- $X_k$  is the characteristic value of the soil parameter;
- $f_{ms}$  is the prescribed partial factor for the soil parameter.

*NOTE* Values of  $f_{ms}$  for the various soil parameters are listed in Section 6, Section 7 and Section 8.

### 5.3.5 Partial factors for soil/reinforcement interaction

There are two possible soil/reinforcement interaction mechanisms that should be considered:

- a) soil/reinforcement bond where a potential failure surface crosses a layer of reinforcement; the soil/reinforcement interaction mechanism in this case is one of pull-out resistance; and
- b) soil/reinforcement bond where the potential failure surface coincides with a layer of reinforcement; the soil/reinforcement interaction mechanism in this case is one of sliding resistance.

For both of the above mechanisms, the length of reinforcement necessary to maintain equilibrium conditions should be determined. Values for the partial factor  $f_p$  prescribed for pull-out resistance and for the partial factor  $f_s$  prescribed for sliding resistance should be taken from Sections 6, Section 7 and Section 8. The magnitudes of  $f_p$  and  $f_s$  may also depend on whether peak or large strain soil shear strength parameters are used for design.

### 5.3.6 Partial load factors

There are three types of partial load factors that may be used in compliance with this code:

- a) partial load factors prescribed for soil self weight,  $f_{fs}$ ;
- b) partial load factors prescribed for external dead loads,  $f_f$ ;
- c) partial load factors prescribed for external live loads,  $f_{fq}$ .

Load factors should be applied as follows:

$$F_d = f_f \times F_k$$

where

- $F_d$  is the design load;
- $F_k$  is the characteristic disturbing load;
- $f_f$  is the partial load factor.

Load factors for external dead loads and live loads should normally be the same for each reinforced soil application. However it should be

noted that the load factors for soil self-weight might differ depending on the reinforced soil application. Values of the appropriate partial load factors should be taken from Sections 6 to 9. Partial load factors should have values greater than unity when assessing the ultimate limit state and values of unity when assessing the serviceability limit state.

Unusual loadings may be included in design. Where low probability loads occur they may be allowed for in the partial load factors applied. Where loads occur that are well defined and of low variability, e.g. convoy loads, they may be associated with a lower partial load factor [see BS EN 1991(all parts)].

## 5.4 Fasteners and connections

Fasteners and connections may often be necessary in reinforced soil structures, particularly where reinforcing elements are connected to some form of facing.

Appropriate material factors should be applied to the strength of a connection in the same way as for reinforcing elements.

## 5.5 Serviceability

The concept of serviceability depends very much on the end use of the structure; normally serviceability limits for reinforced soil should be prescribed in terms of acceptable deformations; deformations of reinforced soil structures are influenced as much by the construction process as by the design.

Post-construction deformations of the structure may be assumed to be caused by several factors including:

- a) external:
  - 1) foundation settlement;
  - 2) loadings not considered in design.
- b) internal:
  - 1) creep strain of polymeric reinforcements;
  - 2) creep of fine grained soil fill;
  - 3) presence of a layer of wet fill;
  - 4) compression of fill; deterioration of the reinforcement due to metal corrosion or polymer degradation.

Foundation settlements should be calculated by conventional soil mechanics approaches, see Section 6, Section 7 or Section 8.

Design of structures using polymeric reinforcement should take account of the creep properties of the material, see Section 4.

Unforeseen corrosion and degradation can lead to excessive deformations and therefore long term monitoring may be included in particularly sensitive structures or structures for which the ramifications of failure are serious to allow advance warning of potential problems to be obtained.

## 5.6 Design information

### 5.6.1 Site investigation

#### 5.6.1.1 General

The use of reinforced soil may be considered:

- a) as an integral part of the design concept of a project;
- b) as an alternative to the use of reinforced concrete or other structural solutions either on the grounds of economy or as a result of ground conditions;
- c) to act as temporary works;
- d) as remedial or improvement works to an existing configuration.

The knowledge of ground conditions at the time of proposing a reinforced soil solution should depend on the application and on the state of advance of the design.

The techniques available for ground investigations are described in BS 5930:1999+A1; the principles to be followed in collecting geotechnical data are described in BS EN 1997-1:2004. Stages of investigation recommended can be conveniently summarized as:

- 1) initial desk and field study;
- 2) main field and laboratory investigation;
- 3) investigation during construction.

If the use of reinforced soil is envisaged from the start of a project the initial desk and field study and the main field and laboratory investigation should be implicitly designed with this in mind.

Reinforced soil may often be used on areas of weak soil where conventional, more rigid structures would suffer distortion and damage unless supported on piles. The inherent flexibility of reinforced soil may be used to accommodate the effects of settling and consolidation of the subsoil without structural damage; however, this type of application requires a thorough study of the foundation soil including the effects due to short term loads during construction, and the effect of long term loads as consolidation proceeds.

The site investigation should provide information of settlement (total and differential), the rate of such settlement and the evaluation of the foundation soil strengths together with a review of the bearing capacity and rotational slip stability (see Smith and Worrall [23]).

An investigation based on the intended use of conventional structures may not adequately identify the data needed for reinforced soil construction; for example, in conventional structures the concept of the allowable bearing pressure is often governed by the normal limits for differential settlement of concrete or masonry structures.

Reinforced soil can tolerate differential settlements of one order of magnitude greater and separate considerations should be given to foundation shear failure and settlement.

Where necessary the scope of the site investigation work should consider the possible chemical or biological effects of the soil or fill environment on the proposed reinforcements. The specification of materials and the testing recommendations given in Section 3 and Section 4 should be considered when planning the site investigations.

### 5.6.1.2 Initial desk and field study

Initial desk and field studies should conform to BS 5930:1999+A1, 6.2 and 6.3 and BS EN 1997-1:2004.

The availability and characteristics of the potential local fill materials should be assessed together with details of local drainage. Where appropriate, the possible build up of potentially corrosive or detrimental chemicals should be considered. Where reinforced soil is to be used to retain ground adjacent to or below existing structures or land, the condition of the surrounding land, buildings, highways and services should be established. An assessment should be made to determine whether embankments or other fill to be retained by the permanent reinforced soil structure contains soluble salts that affect the durability of the reinforcements, facings and connections. Unless it can be shown that the presence of such materials do not introduce a further durability hazard, additional drainage facilities to minimize the hazard should be incorporated into the reinforced soil structure.

### 5.6.1.3 Ground investigation

#### 5.6.1.3.1 Extent of investigation

Ground investigations should conform to BS 5930:1999+A1, Section 2 and BS EN 1997-1:2004.

In many cases reinforced soil may be constructed over ground conditions that would be unsuitable for less flexible construction systems. Where a structure is constructed over poor foundations, information of the medium and longer term behaviour of the foundation strata under the imposed loads should be obtained, particularly if deformation considerations are important as in the case of a bridge abutment or an urban retaining structure.

#### 5.6.1.3.2 Methods of investigation and sampling

For recommended methods of investigation and sampling, see BS 5930:1999+A1, BS EN 1997-1:2004 and BS EN 1997-2.

Appropriate methods of determining the geotechnical parameters of the foundation strata and of the retained fill should be used. With foundation conditions in granular materials standard penetration tests may often be adequate. Where soft clay deposits underlie the site continuous undisturbed sampling techniques allied to penetrometer testing may be appropriate in evaluating settlement behaviour and to assess construction time or post-construction movements.

### 5.6.1.3.3 Ground water

Ground water should be tested in accordance with BS 5930:1999+A1, Clause 23.

The following should be considered:

- Ground water conditions are of importance to reinforced soil structures.
- The pH and chemical content of the ground water may affect the durability of reinforcing elements, fasteners and facings.
- Fluctuations in the ground water regime may affect the overall structural behaviour.

(Testing for ground water chemistry is considered in Section 4.)

The groundwater investigation should be designed to provide knowledge of the permeability of the fill or ground to be reinforced as well as the underlying strata in order to define long term drainage patterns which affect three unrelated aspects of water flow as follows:

- a) possible build up of pore water pressures within the reinforced structure (stability);
- b) possible build up of deleterious materials within the reinforced zone (durability);
- c) consolidation characteristics (settlement/serviceability).

Data regarding ground water conditions should be collected in accordance with the principles of BS EN 1997-1:2004 and BS EN 1997-2. Ground water measurements and sampling should be conducted in accordance with BS EN ISO 22475-1.

### 5.6.1.3.4 Data presentation and reporting

Field data and strata descriptions should be presented in accordance with BS 5930:1999+A1.

The ground investigation report should be prepared following the principles of BS EN 1997-1:2004 and should contain the relevant characteristic values of parameters for the appropriate structure as detailed in Section 6, Section 7 and Section 8. Index testing and particle size distribution results from each soil type together with short and long term strength parameters and where applicable consolidation parameters, should be included.

The fill or ground proposed to be used in the structure should be subject to the testing recommendations of Section 3 and Section 4. Any limitation of the testing procedures should be stated.

Recommendations regarding the design report are given in 5.6.4.

### 5.6.1.3.5 Investigation during construction

Monitoring of settlement and of pore water dissipation should be undertaken with construction over soft foundations where the rate of loading needs to be controlled to ensure stability.

Where the structure retains in situ material or forms a repair or stabilization measure, the retained or stabilized material should be inspected as it is uncovered. The results of this inspection should be compared to the findings of the ground investigation and the design assumptions, and the design checked against any variations.

*NOTE* See BS EN 14475:2006, 7.4 and Clause 9.



## 5.6.2 Environmental considerations

### 5.6.2.1 General

If relevant, the effects of impact or seismic loads should be considered using static methods. Loads due to water pressures including seepage pressures, buoyancy and lateral pressures should be considered, together with increased allowances for reinforcement deterioration where applicable.

### 5.6.2.2 Chemical and biological considerations

*NOTE* Materials commonly used in reinforced soil are classified in Section 3 and testing recommendations are given in Section 4.

The future use of the structure should be considered during design including the possible concentration of biological or chemical material or heat. Potential problems associated with salt run-off from highway gritting operations should be considered. Adequate details should be provided to safeguard fill, reinforcement, facing and other components forming the reinforced soil system.

### 5.6.2.3 Post-construction damage

The implication of post-construction damage should be considered in design.

Examples of post-construction damage that should be considered are accidental loadings such as vehicle impact, accidental damage by utilities contractors, etc., vandalism, fire and flooding.

Generally, reinforced soil is resilient to normal impact loadings and damage is often superficial and may be repaired without affecting the integrity of the primary structural components.

Other post-construction damage that should be considered can be due to superimposed strains such as those due to ground movements resulting from the collapse of underground mine workings or other cavities, or movement along faults. However it has been shown that mining induced strains do not generally affect reinforced fill (see Moulton et al [24], Jones [25] and Murray et al [26]).

Potential problems of this nature should be identified during the ground investigation phase.

### 5.6.2.4 Adjacent structures

If the reinforced soil structure is adjacent to or part of any other structure, then possible interactions should be considered; for example, an adjacent bridge deck or piled structure can impose limits on acceptable lateral movement resulting from differential settlement.

## 5.6.3 Load combinations

The most adverse loads likely to be applied to the structures should be considered in design. Consistent with the approach described in 5.1 load factors should be applied to each component of load.

*NOTE* The applications of reinforced soil are diverse and it is not practical to set out here all possible loadings.

The designer should therefore ensure that all possible loadings are considered for design.

#### 5.6.4 Design record

Sufficient design records should be maintained to enable review of the structure in the future. The design report of the reinforced soil structure may form part of or be referenced in the Geotechnical Design Report. See 2.8 of BS EN 1997-1:2004.

## Section 6: Walls and abutments

### 6.1 General

Walls and abutments of the type and form shown in Figure 15 reinforced by anchors, bars, grids, sheets, or strips of the form shown in Figure 4, should be designed in accordance with this section. Common facings that may be used with these structures are shown in Figure 16 and the following methods apply to all types. The design of structures with segmental block facings may require additional procedures as given in 6.6.6.

Structures that are within 20° of the vertical may be designed as vertical structures.

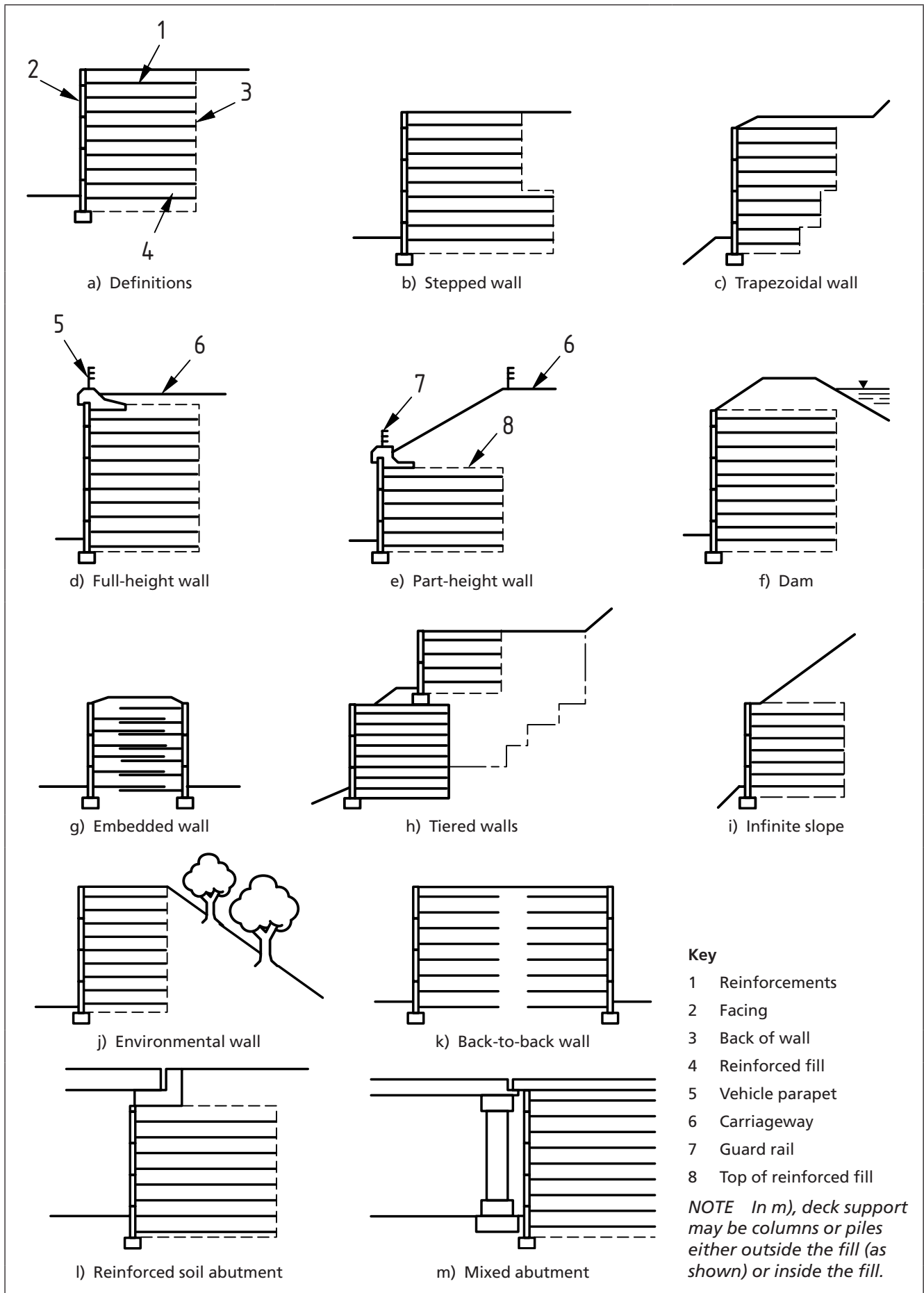
Reinforced soil slopes and reinforced embankments should be designed and implemented in accordance with Section 7 and Section 8.

This section should be followed for abutments for conventional articulated bridge decks, with expansion joints, where forces from the bridge deck are transmitted, through bearings supported on the bankseat, directly into the abutment backfill as shown on Figure 15). This section may also be applied to abutments for semi-integral bridges where thermally induced cyclic movements from jointless decks are accommodated by bearings supported on the bankseat. This section may be applied to abutments for fully integral bridges where thermally induced movements of the jointless deck which might be transmitted into the abutment fill are less than  $\pm 20$  mm (see PD 6694-1<sup>2)</sup>).

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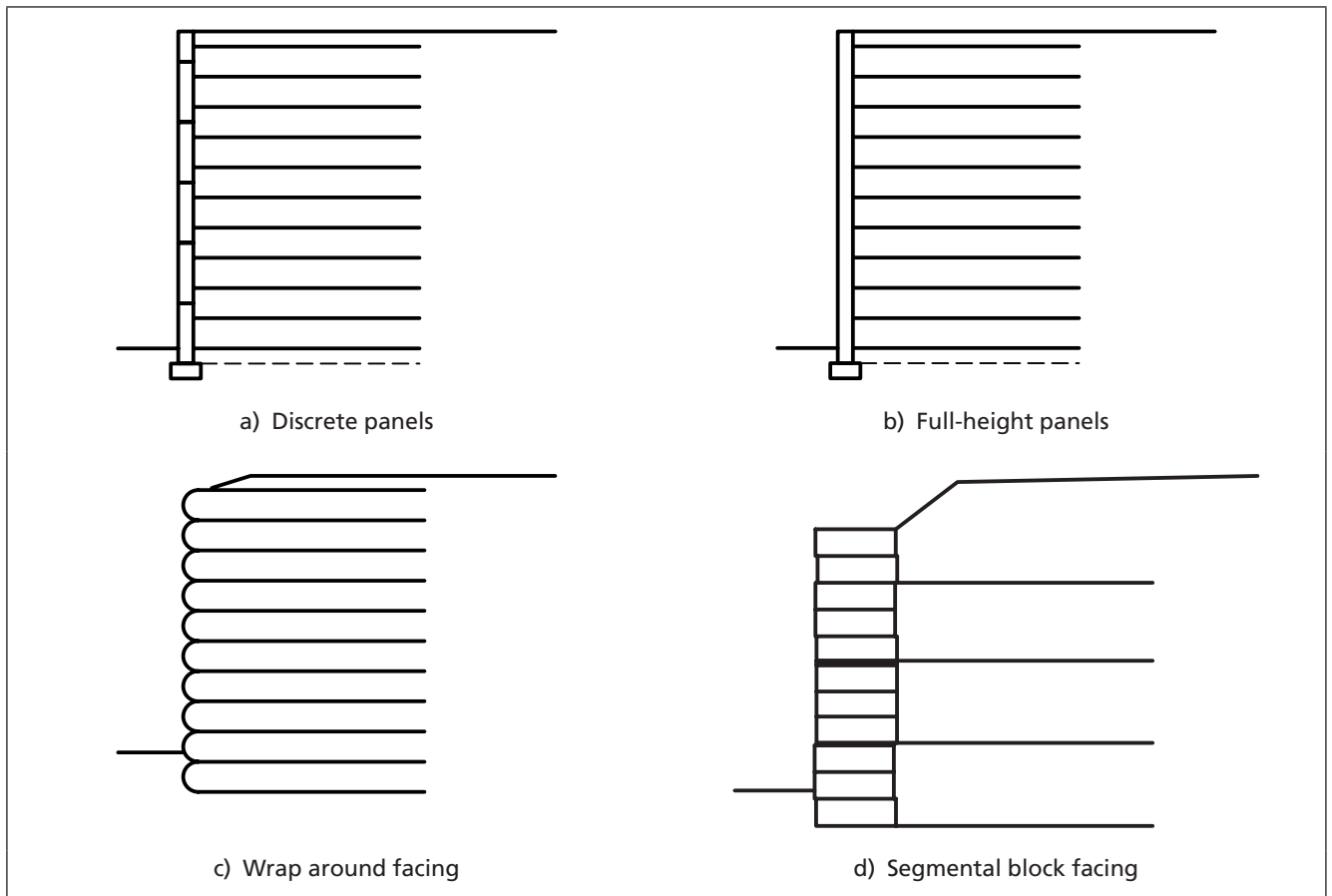
<sup>2)</sup> PD 6694-1 is in preparation.

Figure 15 Definitions and types of walls and abutments



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Figure 16 Common facings used with structures



## 6.2 Partial factors used in this section

### 6.2.1 General

The limit state design philosophy for reinforced soil walls and abutments should be implemented by increasing soil weight and live loading by the appropriate partial load factors and reducing the soil properties and reinforcement base strength by appropriate partial material factors.

The design principles established in Section 2 should be used as a basis for the procedures contained in Section 6, using the partial factors appropriate to this section, listed in Table 11.

### 6.2.2 Load factors

#### 6.2.2.1 General

The soil unit weight to which the specific load factor is ascribed should be the characteristic value, see 2.5, and should take into account variations in specific gravity, grading and compaction. The external loads to which the specific load factors are ascribed should be the characteristic values in their original unfactored state.

Table 11 Summary of partial factors to be used in Section 6

Partial factors		Ultimate limit state	Serviceability limit state
Load factors	Soil unit weight density e.g. wall fill	The appropriate value of $f_{fs}$ to be chosen according to Table 12 and Table 13 for the particular load combinations	
	External dead loads e.g. line or point loads	The appropriate value of $f_f$ to be chosen according to Table 12 and Table 13 for the particular load combinations	
	External live loads e.g. traffic loading	According to Table 12 and Table 13 for the particular load combinations	
Soil material factors:	to be applied $\tan \phi'_p$	$f_{ms} = 1.0$	$f_{ms} = 1.0$
	to be applied to $c'$	$f_{ms} = 1.6$	$f_{ms} = 1.0$
	to be applied to $c_u$	$f_{ms} = 1.0$	$f_{ms} = 1.0$
Reinforcement material factor:	to be applied to the reinforcement base strength	The value of $f_m$ should be consistent with the type of reinforcement to be used and the design life over which the reinforcement is required (see 5.3.3 and Annex A)	
Soil/reinforcement interaction factors	Sliding across surface of reinforcement	$f_s = 1.3$	$f_s = 1.0$
	Pull-out resistance of reinforcement	$f_p = 1.3$	$f_p = 1.0$
Partial factors of safety	Foundation bearing capacity: to be applied to $q_{ult}$	$f_{ms} = 1.35$	NA
	Sliding along base of structure or any horizontal surface where there is soil-to-soil contact	$f_s = 1.2$	NA

### 6.2.2.2 Load combinations

The most adverse loads likely to be applied to the structure should be considered in design. Consistent with the approach described in 5.1 load factors should be applied to each component of load. The partial factors that should be applied to each component of load for different load combinations are listed in Table 12 and Table 13.

Table 12 Partial load factors for load combinations associated with walls

Effects	Combinations		
	A	B	C
Mass of the reinforced soil body	$f_{fs} = 1.5$	$f_{fs} = 1.0$	$f_{fs} = 1.0$
Mass of the backfill on top of the reinforced soil wall	$f_{fs} = 1.5$	$f_{fs} = 1.0$	$f_{fs} = 1.0$
Earth pressure behind the structure	$f_{fs} = 1.5$	$f_{fs} = 1.5$	$f_{fs} = 1.0$
Traffic load: on reinforced soil block behind reinforced soil block	$f_q = 1.5$	$f_q = 0$	$f_q = 0$
	$f_q = 1.5$	$f_q = 1.5$	$f_q = 0$

*NOTE* The following descriptions of load cases identify the usual worst combination for the various criteria but are for guidance only. All load combinations should be checked for each layer of reinforcements within each structure to ensure the most critical condition has been found and considered.

Table 12 Partial load factors for load combinations associated with walls (continued)

Combination A This combination considers the maximum values of all loads and therefore normally generates the maximum reinforcement tension and foundation bearing pressure. It may also determine the reinforcement requirement to satisfy pull-out resistance although pull-out resistance is usually governed by combination B.

Combination B This combination considers the maximum overturning loads together with minimum self mass of structure and superimposed traffic load. This combination normally dictates the reinforcement requirement for pull-out resistance and is normally the worst case for sliding along the base.

Combination C This combination considers dead loads only without partial load factors. This combination is used to determine foundation settlements as well as generating reinforcement tensions for checking the serviceability limit state.

Table 13 Partial load factors for load combinations associated with abutments

Effects		Combinations		
		A	B	C
Dead load of the structure		$f_{fs} = 1.5$	$f_{fs} = 1.0$	$f_{fs} = 1.0$
Dead load of the fill on top of the structure		$f_{fs} = 1.5$	$f_{fss} = 1.0$	$f_s = 1.0$
Dead load of bridge and bank seat		$f_f = 1.2$	$f_f = 1.0$	$f_f = 1.0$
Backfill pressure behind the bank seat		$f_{fs} = 1.5$	$f_{fs} = 1.5$	$f_{fs} = 1.0$
Backfill pressure behind the structure		$f_{fs} = 1.5$	$f_{fs} = 1.5$	$f_{fs} = 1.0$
Horizontal loads due to creep and shrinkage		$f_f = 1.2$	$f_f = 1.2$	$f_f = 1.0$
Traffic loading		Over the entire structure, $f_q = 1.5$	Behind the reinforced zone, $f_q = 1.5$	
Bridge vertical live load	HA	$f_q = 1.5$	$f_q = 1.5$	
	HA and HB	$f_q = 1.3$	$f_q = 1.3$	
Braking dynamic load	HA	$f_q = 1.25$	$f_q = 1.25$	
	HA and HB	$f_q = 1.1$	$f_q = 1.1$	
Temperature effects		$f_q = 1.3$	$f_q = 1.3$	

NOTE 1 The following descriptions of load cases identify the usual worst combination for the various criteria but are for guidance only. All load combinations should be checked for each layer of reinforcements within each structure to ensure the most critical condition has been found and considered.

NOTE 2 The designations HA and HB are currently under review by the Highways Agency.

NOTE 3 Details of the traffic loads to be used when evaluating the traffic surcharge pressures are given in NA to BS EN 1991-2.

Combination A This combination considers the maximum values of all loads and therefore normally generates the maximum reinforcement tension and foundation bearing pressure. It may also determine the reinforcement requirement to satisfy pull-out resistance although pull-out resistance is usually governed by combination B.

Combination B This combination considers the maximum overturning loads together with minimum self mass of structure and superimposed traffic load. This combination normally dictates the reinforcement requirement for pull-out resistance and is normally the worst case for sliding along the base.

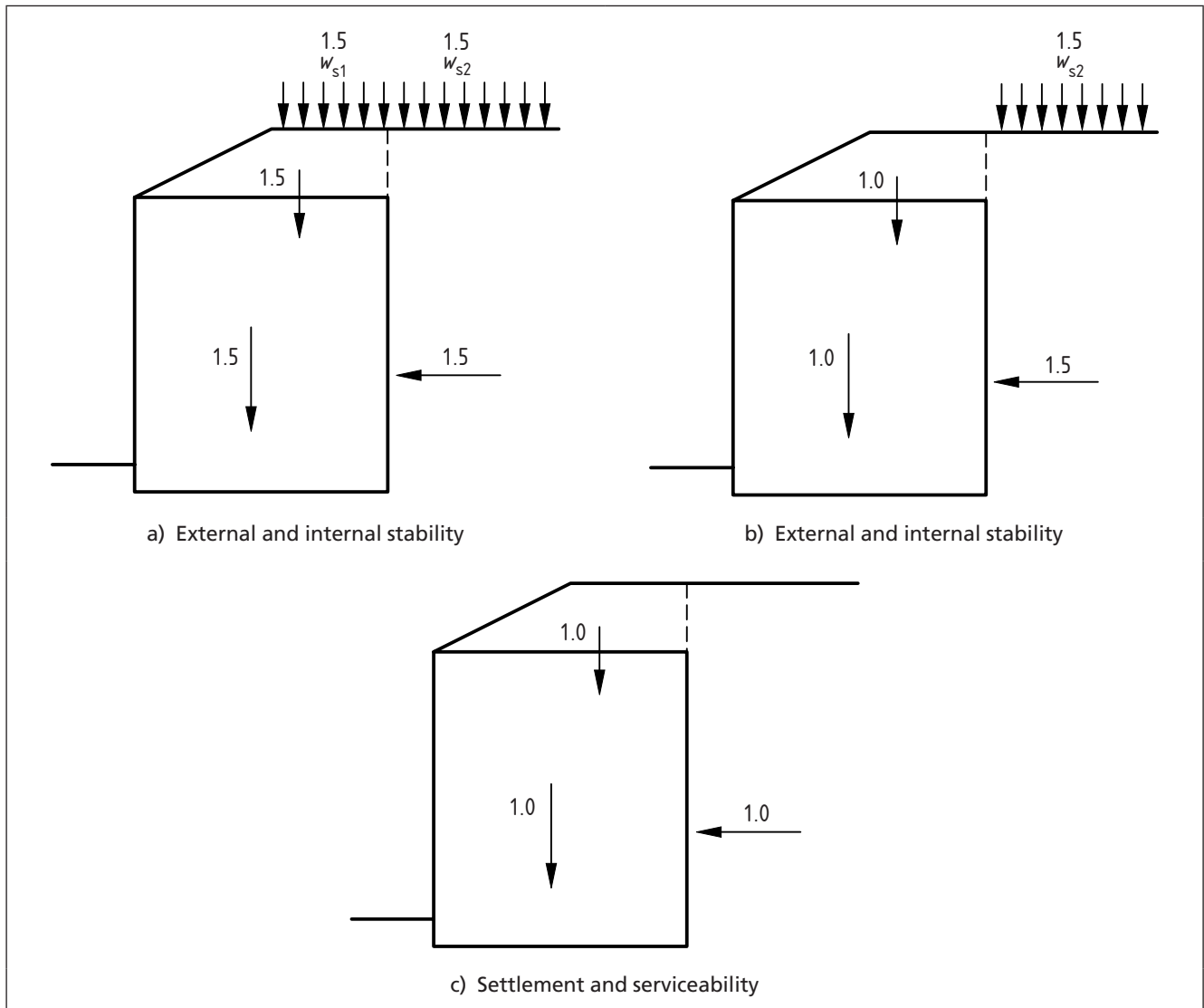
Combination C This combination considers dead loads only without partial load factors. This combination is used to determine foundation settlements as well as generating reinforcement tensions for checking the serviceability limit state.

6.2.2.3 Land based structures

The three basic load combinations which should be considered for the design of land based vertical structures are shown in Figure 17 and set out in Table 12 and Table 13

When considering the stability of tiered walls [Figure 15h)], the influence of the loading of the upper tier on the lower tier should be taken into account. For embedded walls [Figure 15g)], reinforcement overlapping in the central part of the structure should be separated in plan and/or elevation, to avoid significant face to face contact between strip or geotextile sheet reinforcements. For environmental walls [Figure 15j)], if the distance from the structure to the motorway, principal road, railway, inhabited building, school building/yard, or similar feature is at a distance less than the mechanical height, the structure should be considered as category 3 of Table 9 and  $f_n$  should be taken as 1.1.

Figure 17 Load combinations showing partial load factors (see Table 12)





#### 6.2.2.4 Maritime or river structures

Loading considerations for structures constructed as sea or river walls should include the effects of varying water levels, tidal lag, buoyancy of soil, wave impact and impact of vessels (see Wu and Smith [27]) Scour of the toe should be considered (see BS 6349). The durability of the reinforcements and the facing units, particularly within the tidal zone or zone of seasonal rise and fall of river flows should be considered.

#### 6.2.3 Materials factors

The soil material factors relating to the peak values of  $\phi'_p$ ,  $c'$ , and  $c_u$  all have values greater than or equal to unity at the ultimate limit state. At the serviceability limit state these materials factors are set to unity.

The reinforcement material factor should be assessed in accordance with the procedures described in 5.3.3 and Annex A, taking due regard of the type of reinforcement and the appropriate design life.

#### 6.2.4 Soil/reinforcement interaction factors

##### COMMENTARY ON 6.2.4

*In reinforced soil walls and abutments there are two main interfaces where the soil and the reinforcement interact:*

- *soil sliding across the surface of the reinforcement;*
- *pull-out of the reinforcement from the resistant zone.*

These parameters are based on peak values and hence the partial factors should be unity or greater in the ultimate limit state, as shown in Table 11.

#### 6.2.5 Partial factors of safety related to soil parameters

Two partial factors of safety should be applied to reinforced soil walls; these are foundation bearing capacity and sliding stability where there is soil-to-soil contact (as distinct from reinforcement-to-soil contact) along the base of structures. At the ultimate limit state these partial factors should be greater than unity, as shown in Table 11.

### 6.3 Basis for design

The design of reinforced soil walls and abutments should follow the principles involved in conventional earth retaining structures, however, reinforced soil structures require additional consideration with regard to soil/reinforcement interaction. For convenience analysis should usually be considered in two main parts covering external and internal stability, but also consider 6.5.6.

It should be noted that external stability covers the basic stability of the reinforced soil structure as a unit, whilst internal stability covers all areas relating to internal behaviour mechanisms, consideration of the stress within the structure, arrangement and behaviour of the reinforcements and backfill properties.

There are two methods that may be used for the design of reinforced soil structures (as shown in Figure 18), which are referred to as the tie back wedge method and the coherent gravity method:

- a) the tie back wedge method (6.6.3) follows basic design principles currently employed for classical or anchored retaining walls. It has evolved from the use of all forms of permitted reinforcements;
- b) the coherent gravity method (6.6.4) is based on the monitored behaviour of structures using inextensible reinforcements and has evolved over a number of years from observations on a large number of structures, corroborated by theoretical analysis.

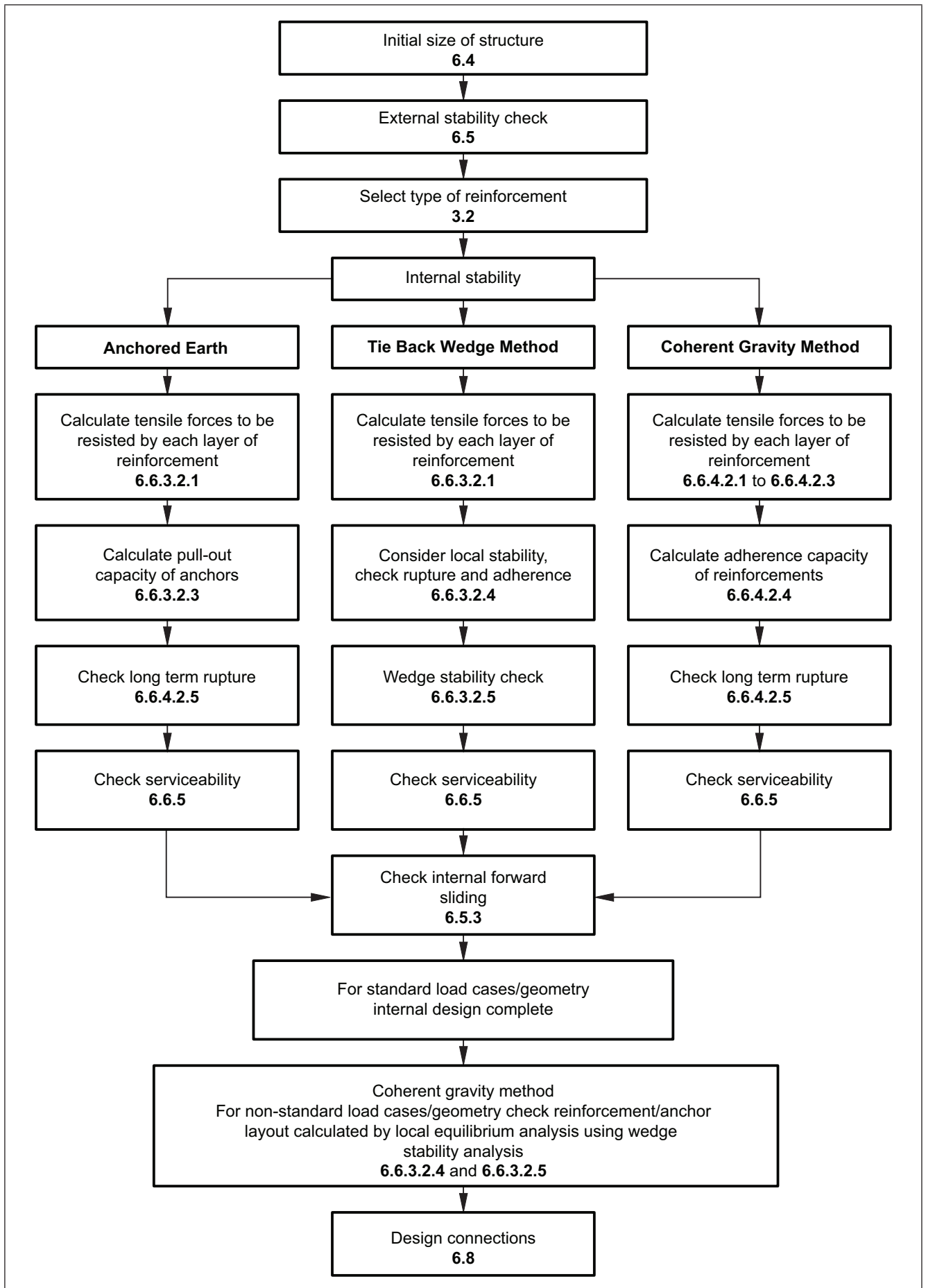
Reinforced soil structures should be designed to conform to two limit states, see 5.1.

- 1) *Ultimate limit state*. The limit state wherein relevant potential collapse mechanisms are identified and considered together with limit state factors.
- 2) *Serviceability limit state*. The limit state wherein relevant working conditions are identified and the structure checked to ensure that it will retain the characteristics necessary for it to fulfil its function throughout its life without the need for abnormal maintenance.

Two design methods are identified for internal stability and the design procedure should follow that shown in Figure 18. Field observations have shown that lateral earth pressures in the upper reaches of a wall will be influenced by the axial tensile stiffness of the reinforcement. For inextensible reinforcement, lateral earth pressures approximate to  $K_0$  pressures and such walls should be designed using the coherent gravity method. Unless shown otherwise by field observations, active earth pressure may be assumed to act upon walls with extensible reinforcement and such walls are designed by the tie back wedge method. The ultimate limit state and serviceability limit state should be checked in both methods. Normal procedure, which should be followed, is to design for the ultimate limit state and check the serviceability limit state (see 2.15). Both methods consider the design of reinforced soil and anchored earth structures.

Design should usually be based upon the assumption of a two dimensional plane strain condition.

Figure 18 Design procedure for reinforced soil walls



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## 6.4 Dimensions of the structure

### 6.4.1 General

Prior to considering external stability the overall geometry of the wall or abutment should be selected.

Consideration of either the external or the internal stability may require the dimensions of the structure to be increased from the initial size. The initial dimensions of the structure should not be less than the minimum specified in Table 14 unless it can be satisfactorily demonstrated by previous experience that smaller values are adequate.

The geometrical size of a structure should be based upon a concept of mechanical height  $H$ , which is defined as the vertical distance from the toe of the structure to the point where a line at arc tan 0.3 to the vertical outcrops the upper ground line above the wall. Figure 19 and Figure 20 give details of the initial sizing of structures of the form shown in Figure 15 and referred to in Table 14. Walls with a trapezoidal cross section should only be considered where foundations are formed by excavation into rock or when good foundations exist.

Table 14 Dimensions of walls and abutments

Structure type	Minimum reinforcement length
Walls with normal retaining function	$0.7H$ (3 m minimum)
Bridge abutments	The greater of $(0.6H + 2)$ m or 7 m
Trapezoidal walls and abutments <sup>A)</sup>	$0.7H$ for reinforcements in top half of structure, $0.4H$ for reinforcements in bottom half of structure or 3 m minimum
Stepped walls and abutments	$0.7H$ in top half of structure, see Figure 20c) for longer strips at base
Walls subject to low thrust from retained fill such as negative backslope or embedded walls, see Figure 15g) and Figure 15j)	$0.6H$ or 3 m minimum
Low height walls i.e. less than 1.5 m	Subject to particular considerations

<sup>A)</sup> For trapezoidal walls the vertical spacing of the reinforcements should obey the following:

$$L/H < 0.55 : S_v/H \leq 0.125$$

$$0.55 \leq L/H < 0.65 : S_v/H \leq 0.167$$

$$0.65 \leq L/H < 0.75 : S_v/H \leq 0.222$$

where

$S_v$  is the vertical spacing of reinforcements;

$L$  is the length of reinforcement at any level;

$H$  is the height of structure defined in Figure 20b).

Figure 19 Initial sizing of structures

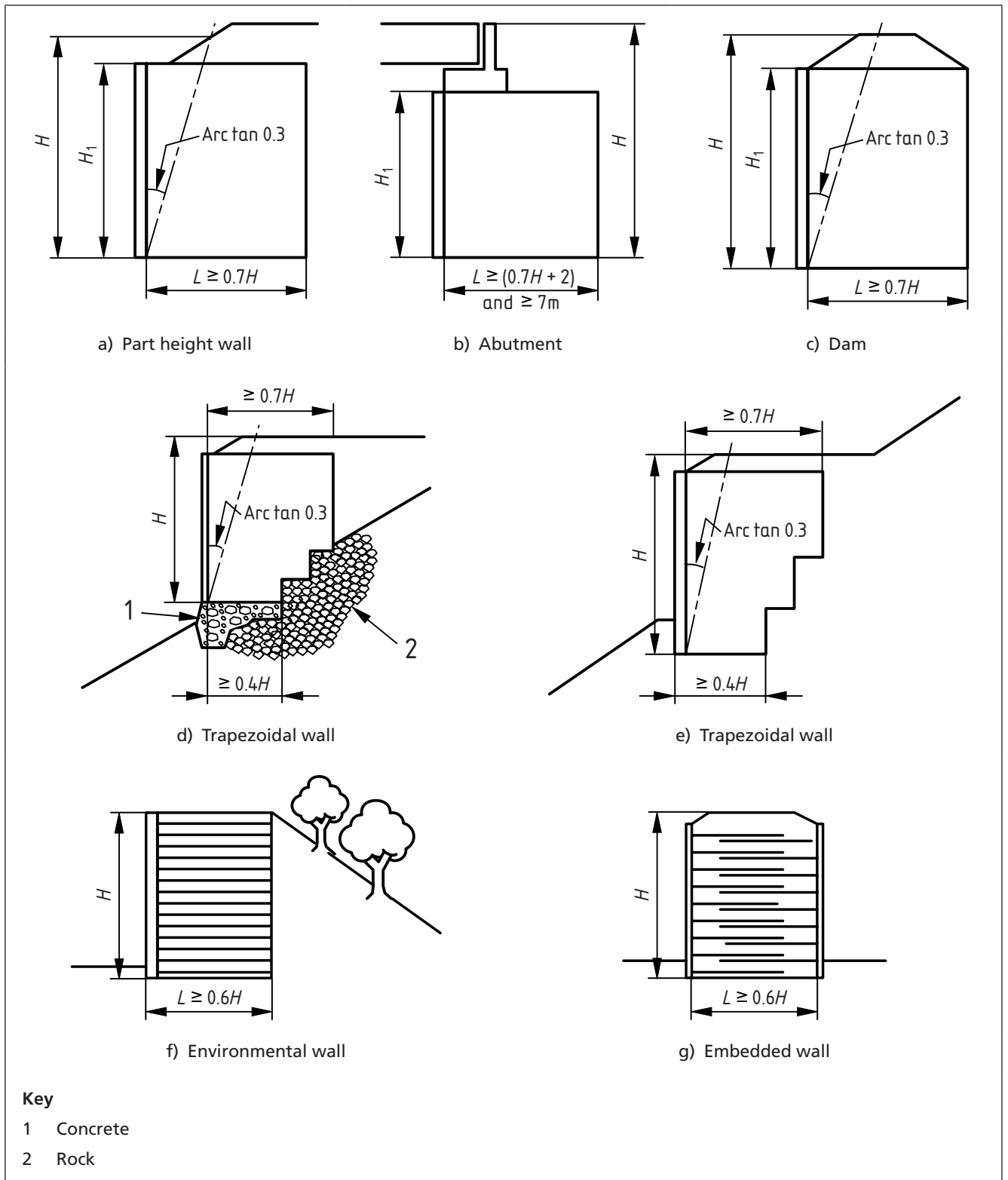
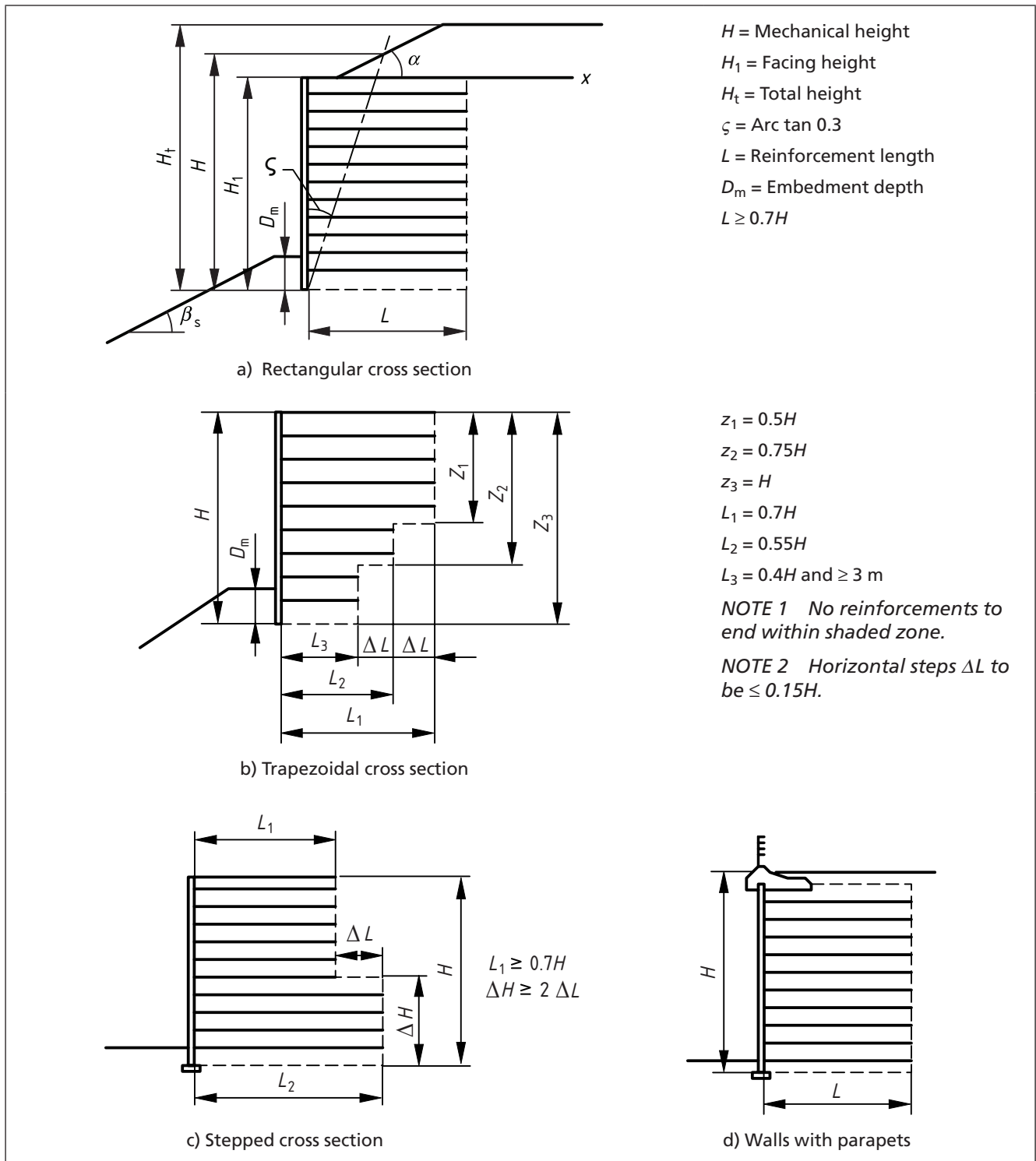


Figure 20 Sizing of walls with various geometries



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### 6.4.2 Embedment

The toe of the structure should be embedded below ground surface; the definition of embedment is provided in Figure 21. Embedment is recommended to avoid local failure by punching in the vicinity of the facing and to avoid the phenomenon of local soil flow similar to piping within the structure. The amount of embedment that should be used depends on various factors which include:

- pressure imposed by the structure on its foundation;
- frost depth (usually taken as 0.45 m in the UK);
- risk of piping if a water head builds up behind the facing in river and sea walls;
- risk of exposure of the toe due to subsequent excavation;
- risk of scour at the toe of river training walls and sea walls.

Structures should have an embedment depth of at least the commonly adopted frost penetration depth of 0.45 m unless they are founded on a rock or structural base such as a raft, mattress or old pavements.

The minimum embedment should not be less than that given in Table 15, which is applicable to a structure slenderness ratio of not less than  $L/H = 0.7$  and for good ground conditions. On sites where the foundation is weak or soft, greater embedment should be considered. In Table 15 the minimum embedment depth expressed in terms of the mechanical height of the wall provides a conservative value and should generally be used. The minimum embedment depth expressed in terms of the factored bearing pressure at the base of the wall may be used to provide a more rigorous solution.

For structures subject to water action by river or sea, anti-scour precautions, rip-rap or gabion mattresses should be provided to ensure stability. In these cases an embedment depth greater than the minimum defined in Table 15 should be considered.

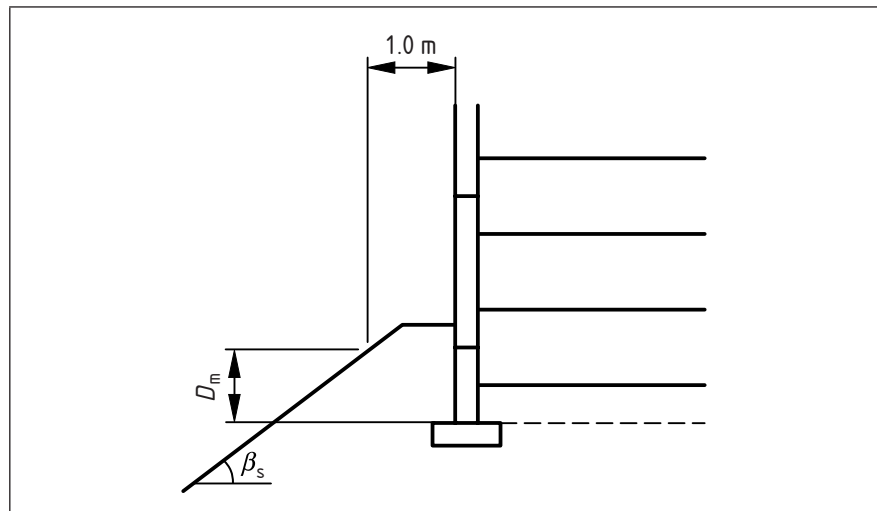
Where the embedment depth based on Table 15 is less than that of drains or services adjacent to the toe of a structure, consideration may be given to increasing the embedment to below the depth of the drains or services.

Table 15 Determination of the minimum embedment as a function of the mechanical height  $H$  in metres and the factored bearing pressure  $q_r$  in  $\text{kN/m}^2$

Slope of the ground at toe $\beta_s$		Minimum embedment $D_m$ m	Minimum embedment factor $D_m/q_r$ $\text{m}^3/\text{kN}$
$\beta_s = 0$	Walls	$H/20$	$1.35 \times 10^{-3}$
$\beta_s = 0$	Abutments	$H/10$	$1.35 \times 10^{-3}$
$\beta_s = 18^\circ$ (cot $\beta_s = 3/1$ )	Walls	$H/10$	$2.7 \times 10^{-3}$
$\beta_s = 27^\circ$ (cot $\beta_s = 2/1$ )	Walls	$H/7$	$4.0 \times 10^{-3}$
$\beta_s = 34^\circ$ (cot $\beta_s = 3/2$ )	Walls	$H/5$	$5.9 \times 10^{-3}$

NOTE 1 For definition of notation see Figure 21.

NOTE 2  $D_m \geq 0.45 \text{ m}$ .

Figure 21 Definition of embedment,  $D_m$ 

## 6.5 External stability

### 6.5.1 General

The effects of dead loads and other loads and forces acting on the structure should be considered when assessing external stability. Stability should be checked for bearing and tilt failure, forward sliding, and slip circle failure (Figure 22) as well as settlement of the structure [Figure 23a)]. The definitions of soil properties for the reinforced soil, retained fill and foundation, together with the principal superimposed loads considered in stability calculations are shown in Figure 24. Both short and long term soil properties should be considered to allow for the construction and in-service conditions and changes in pore water pressures. Passive earth pressures exerted on the foot of the wall or structure below ground level should be ignored when considering stabilizing forces.



Figure 22 Ultimate limit states – External stability

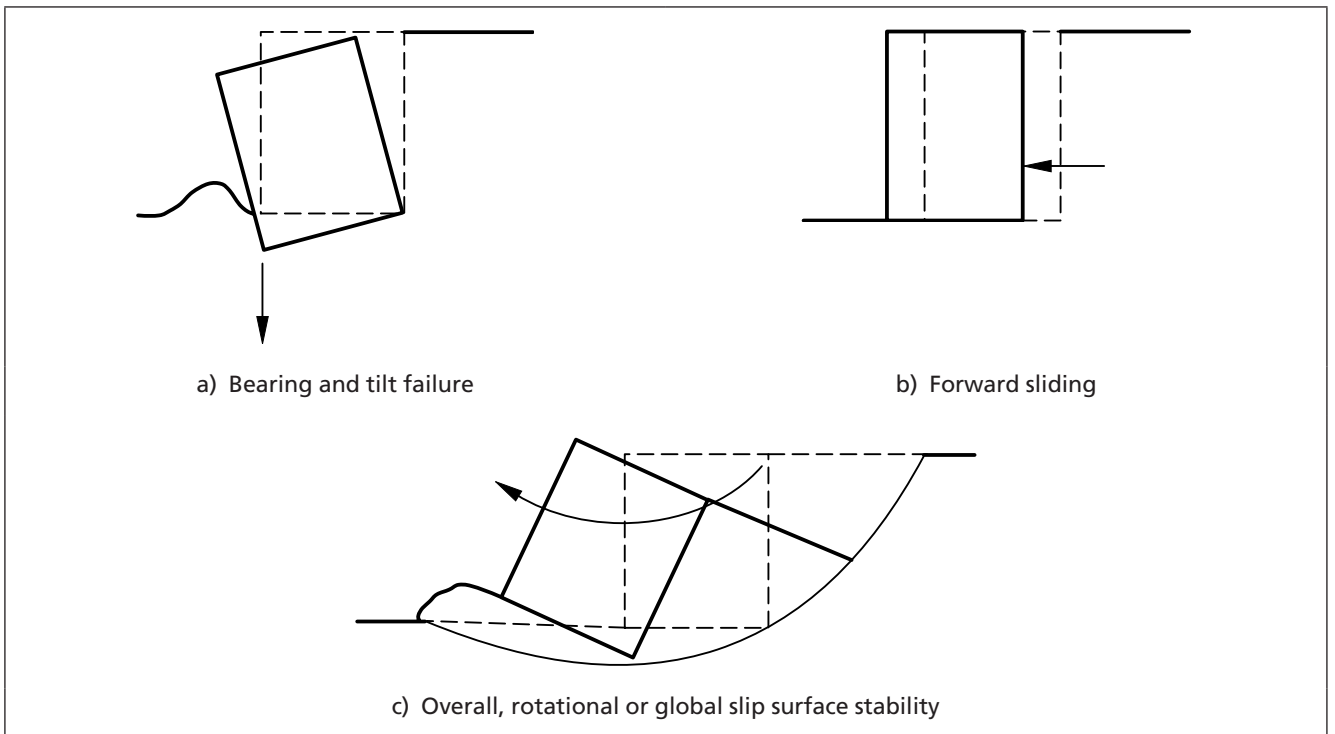


Figure 23 Serviceability limit states – External and internal stability

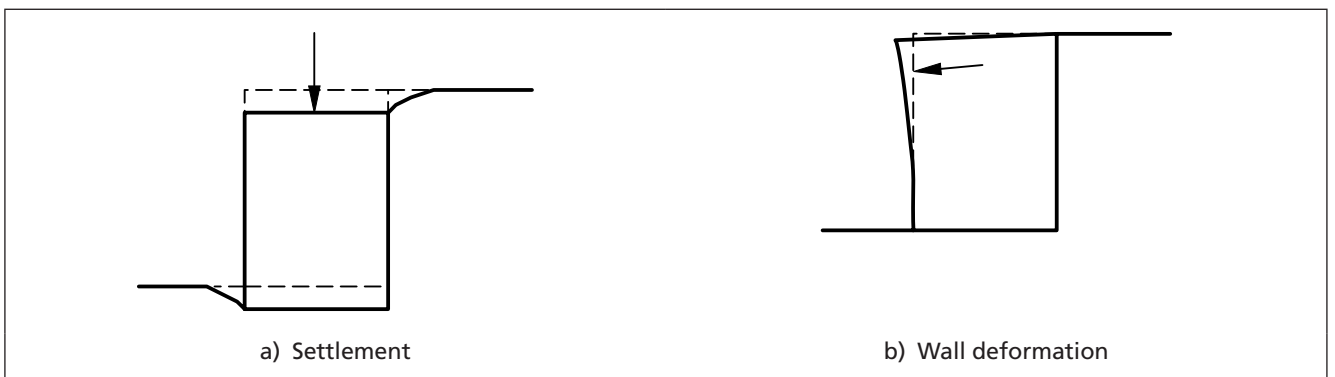
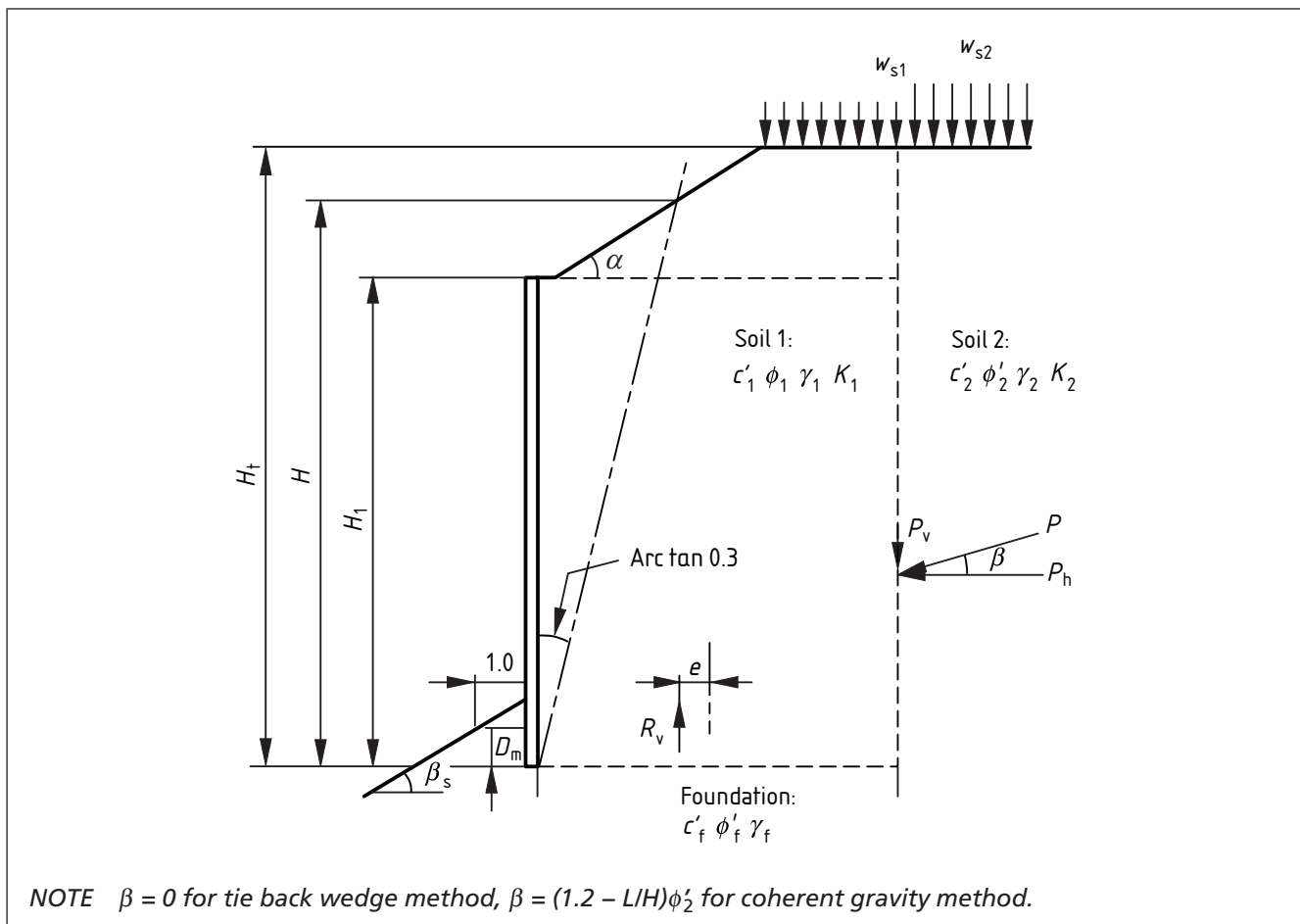


Figure 24 Definition of soil properties and loads



### 6.5.2 Bearing and tilt failure

The typical bearing pressure imposed by a reinforced soil structure on the foundation strata, is shown in Figure 25a); for design, a bearing pressure  $q_r$  based upon a Meyerhof distribution may be assumed [see Figure 25b)].

$$q_r = \frac{R_v}{L - 2e}$$

where

$q_r$  is the factored bearing pressure acting on the base of the wall;

$R_v$  is the resultant of all factored vertical load components (load factors from Table 12 and Table 13 as applicable to each load case);

$L$  is the reinforcement length at the base of the wall;

$e$  is the eccentricity of resultant load  $R_v$  about the centre line of the base of width  $L$ .

The imposed bearing pressure  $q_r$  should be compared with the ultimate bearing capacity of the foundation soil as follows:

$$q_r \leq \frac{q_{ult}}{f_{ms}} + \gamma D_m$$

where

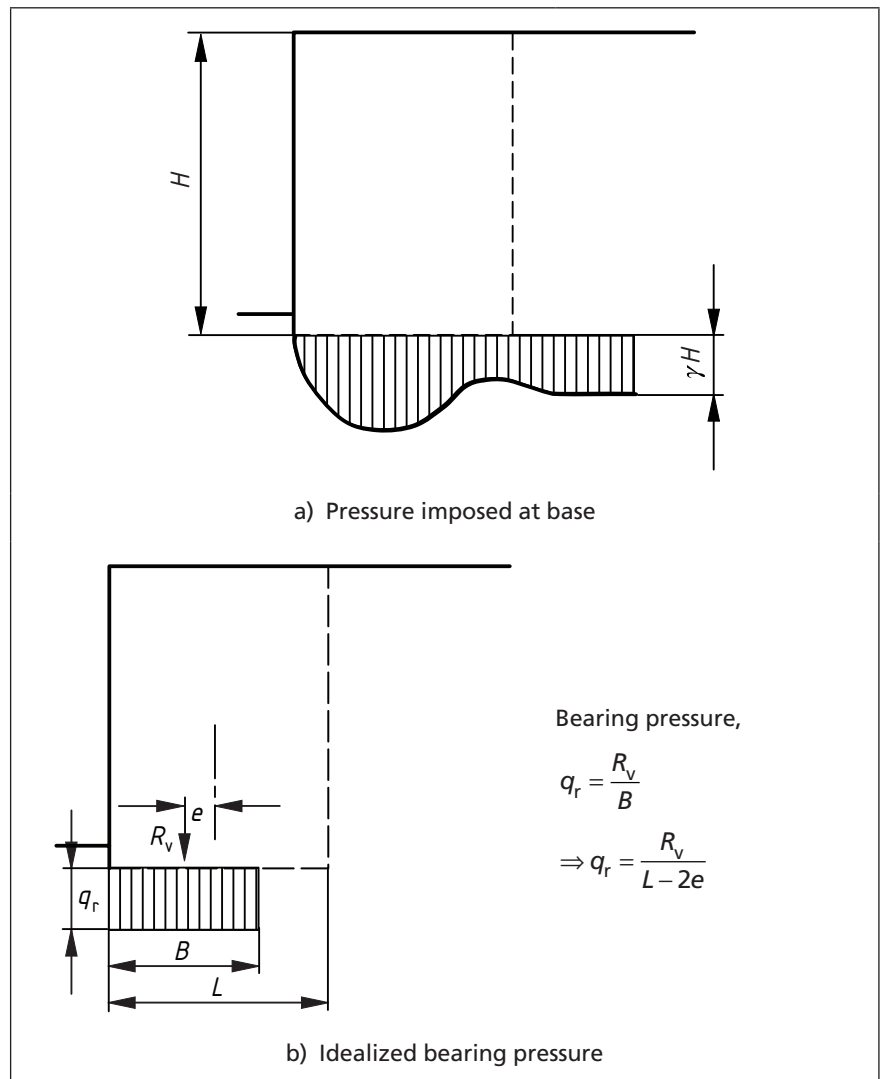
$q_{ult}$  is the ultimate bearing capacity of foundation soil;

$\gamma$  is the foundation soil density;

$D_m$  is the wall embedment depth;

$f_{ms}$  is the partial material factor applied to  $q_{ult}$  (see Table 11).

Figure 25 Pressure distribution along base of wall



### 6.5.3 Sliding along the base

The stability against forward sliding of the structure at the interface between the reinforced fill and the subsoil should be considered. The resistance to movement should be based upon the properties of either the subsoil or the reinforced fill, whichever is the weaker, and consideration should be given to sliding on or between any reinforcement layers used at the base of the structure, using the following:

for long term stability where there is soil-to-soil contact at the base of the structure;

$$f_s R_h \leq R_v \frac{\tan \phi'_p}{f_{ms}} + \frac{c'}{f_{ms}} L$$

for long term stability where there is reinforcement-to-soil contact at the base of the structure;

$$f_s R_h \leq R_v \frac{a' \tan \phi'_p}{f_{ms}} + \frac{a'_{bc} c'}{f_{ms}} L$$

for short term stability where there is soil-to-soil contact at the base of the structure;

$$f_s R_h \leq \frac{c_u}{f_{ms}} L$$

for short term stability where there is reinforcement-to-soil contact at the base of the structure;

$$f_s R_h \leq \frac{a'_{bc} c_u}{f_{ms}} L$$

where

- $R_h$  is the horizontal factored disturbing force (load factors from Table 12 and Table 13 as applicable to load case);
- $R_v$  is the vertical factored resultant force (load factors from Table 12 and Table 13 as applicable to load case);
- $\phi'_p$  is the peak angle of shearing resistance under effective stress conditions;
- $c'$  is the cohesion of the soil under effective stress conditions;
- $c_u$  is the undrained shear strength of the soil;
- $L$  is the effective base width for sliding;
- $f_{ms}$  is the partial materials factor applied to  $\tan \phi'_p$ ,  $c'$  and  $c_u$  see Table 11;
- $f_s$  is the partial factor against base sliding;
- $a'$  is the interaction coefficient relating soil/reinforcement bond angle with  $\tan \phi'_p$ ;
- $a'_{bc}$  is the adhesion coefficient relating soil cohesion to soil/reinforcement bond.

Where a layer of reinforcement coincides with the base of the wall the value of  $f_s$  listed in Table 11 for soil/reinforcement sliding should be used. Where reinforcement does not coincide with the base of the wall the value of  $f_s$  listed in Table 11 for soil-to-soil sliding should be used.

## 6.5.4 Settlement

### 6.5.4.1 General

The total settlement of a reinforced soil structure should be taken as the combined effect of the settlement of the foundation soil under the influence of the pressures imposed by the reinforced soil structure, and the internal compression of the reinforced backfill.

Reinforced soil structures built on good ground behave in a manner similar to conventional earth retaining structures and do not undergo significant settlement. However, due to their general ability to withstand the effects of large settlements of poor foundation soils they may be used with great effect in these situations (see Smith [22], Smith and Worrall [23], Rodrigues and Villadrruel [28], Brady [29], Brady [30], Worrall [31]). To obtain maximum economy the supporting ground and the reinforced soil structure should be considered as a whole (see Jones and Edwards [32], Kempton et al [33]).

Where reinforced soil walls are built adjacent to other new structures consideration should be given to the possible interaction of the structures. To ensure a compatible response of both structures it may be appropriate to combine the structures as an integral unit rather than use two different construction forms.

Thus a mixture of reinforced soil wing walls and piled abutments may often require greater care than the use of reinforced soil for both the wing walls and the abutment of a bridge.

It should be understood that total settlement can influence the serviceability of the wall or abutment, e.g. loss of headroom in the case of a bridge deck supported on reinforced soil abutments. It can also affect the serviceability of drains and services.

### 6.5.4.2 Settlement of foundation soil

Virtually all foundation soils settle when subject to increased overburden pressures as is the case when supporting reinforced soil structures; however due to the inherent ability of such structures to accommodate foundation movements, 6.5.4.3 and 6.5.4.4 should be considered as being more relevant to significant settlements.

The significance of any settlement is a matter for the designer, but will be mainly determined by the type of structure and any adjacent structures or services sensitive to foundation movements.

*NOTE The actual pressures imposed on foundations by reinforced soil structures are lower and more evenly distributed than conventional concrete structures and this normally acts to reduce foundation settlements.*

### 6.5.4.3 Internal settlement of reinforced soil fill

It should be understood that the amount of settlement (compression) within the reinforced volume and to be accommodated by a reinforcement system depends upon the properties of the fill, its compaction, and the vertical pressures within the fill; these pressures are largely a function of structure height and surcharge loading. For walls and abutments the specified fills, when properly compacted, produce relatively small internal settlements.

The movement capacities detailed in Table 16 should be considered as a minimum to be provided.

*NOTE Additional information is provided in BS EN 14475:2006, C.3.2.*

Sliding connections may be used with full height panels. Particular care should be taken with connections to blockwalls and non-sliding connections to full height panels where the fill may settle relative to the facing and cause increased load at connections.

Table 16 **Typical vertical movement capacities required for facing systems to cope with vertical internal settlement of reinforced fill**

Structural form	Typical vertical movement capacity of system
Discrete panels	Joint closure of 1 in 150 relative to panel height
Full height panels and blockwalls	Required vertical movement capacity of connections increase with wall height
Semi-elliptical facings	Vertical distortion of 1 in 150 relative to panel height
Geotextile wrap-around facings	No specific limit except for appearance or serviceability considerations

#### 6.5.4.4 Differential settlement

Whereas the total settlement of a structure can interfere with some special aspect of its function, e.g. loss of clearance to bridge deck in the case of an abutment, it is differential or relative settlement which in general may be expected to produce the most severe effects on the completed structure itself. It should be understood that reinforced soil is tolerant of large differential settlements and it is often the facing which determines the limits to settlement.

Where large differential settlements are anticipated, as in the case of mining subsidence, special slip joints may be incorporated into the facing (see Jones [25], Murray et al [26]). Tolerance of reinforced soil structures to differential settlements along the line of the facing should be considered as listed in Table 17. (Further guidance is given in BS EN 14475:2006 C.3.3.)

Reinforced soil bridge abutments are able to accommodate differential settlements significantly in excess of the established tolerable movement criteria for bridge decks (see Moulton et al [24]; in these conditions special structural precautions should be used with regard to the bridge superstructure (see Worrall [31], *The Inspection Manual for Highway Structures* [34], Murray [35], Jewell et al [36]).

It should be understood that reinforced soil structures are also tolerant to movements due to mining subsidence (see Moulton et al [24], Lawson [37]).

Table 17 Guide to the effects of settlement

Maximum differential settlement	Comment
1 in 1000	Not normally significant
1 in 200	Full height panels may be affected by joints closing or opening. Normal safe limit for segmental blockwalls.
1 in 100	Normal safe limit, without special measures, for discrete concrete panel facings
1 in 50	Normal safe limit for semi-elliptical steel face elements. Discrete concrete panels may suffer closed joints if special measures not included
1 in < 50	Soft facings might suffer distortion affecting their retaining ability

*NOTE There is no intended firm limit between categories. This is a preliminary guide only.*

## 6.5.5 Construction tolerances and serviceability limits

### 6.5.5.1 General

Reinforced soil structures deform during construction; consideration should be given to provide the necessary clearances to permit the structure to attain a stable configuration and also to ensure that construction and post-construction movements are within acceptable limits.

The serviceability of a structure may usually be taken as dependant upon the deformations which evolve during its service life. These deformations are due normally to the creep of the reinforcement material under service load, compression of the backfill and settlement of poor foundation soil under service load.

The creep in metals is very small and insignificant for the load levels found in these structures; however for polymeric reinforcements the value of the strains during the creep phase should be assessed.

Creep can also be evident, and should be watched for, when soils with a high fines content are used particularly when saturated. The design and testing recommendations related to strain and creep of reinforcements are given in Section 4.

Deformations in the face and top surface of the structure should be kept within acceptable limits.

The following considerations may be used to determine these limits.

- a) The wall face should be visually acceptable and free from bulges, overhangs and erratic alignment.
- b) All tops should follow smooth curves or straights.
- c) Construction sequence can be critical in ensuring that abutments should not deform thus causing movement of supported bank seats, closing of deck joints and axial loading of bridge decks in excess of those allowed for in the design (see Figure 48).
- d) Wall faces should not deform and cause damage to the facing material. In the case of concrete facings this damage could include closure of joints, spalling of panel edges and panel cracking.

The values in Table 18 should be understood as being indicative of the construction tolerances which are commonly achieved, or the deformations which are seen after construction.

Table 18 Construction tolerances commonly achieved for faces of retaining walls and abutments

Feature	Tolerance
Location of plane of structure	± 50 mm
Verticality	± 5 mm per metre height (1 in 200)
Bulging (vertical) and bowing (horizontal)	± 25 mm in 4.0 m template

*NOTE 1* The face of segmental block walls are usually built at a batter often by setting back the face of a row of blocks from the previous layer of blocks. The tolerance for verticality in the table is to be read as the tolerance from the intended designed face batter.

*NOTE 2* This is a guide only and structures with greater tolerances can often be satisfactory.

### 6.5.5.2 Serviceability limits

Post-construction movements of reinforced soil structures that should be considered can result from:

- foundation settlements (6.5.4.2);
- internal compression of fill (6.5.4.3);
- internal creep strain of reinforcement;
- uniform or differential settlements resulting from mining or closure of voids beneath the structure (6.5.4.4);
- creep strain of backfill with a high fines content.

While these post-construction movements can generally be avoided by good construction practices the internal creep strain of the reinforcement should be limited to the values shown in Table 19, or as specified by the designer, using the method described in Figure 43.

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

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

### 6.5.5.3 Staged construction

Staged construction may be used in order to allow consolidation of the foundation and an increase in shear strength prior to the subsequent construction stages. When major settlements are anticipated with reinforced soil abutments, staged construction may be appropriate and arrangements for the subsequent jacking of the deck should be considered.



Consideration should be given to how stage construction and consolidation of the foundations can affect construction, and that the placing of discrete panels in a high structure can be difficult following consolidation settlement of an earlier stage.

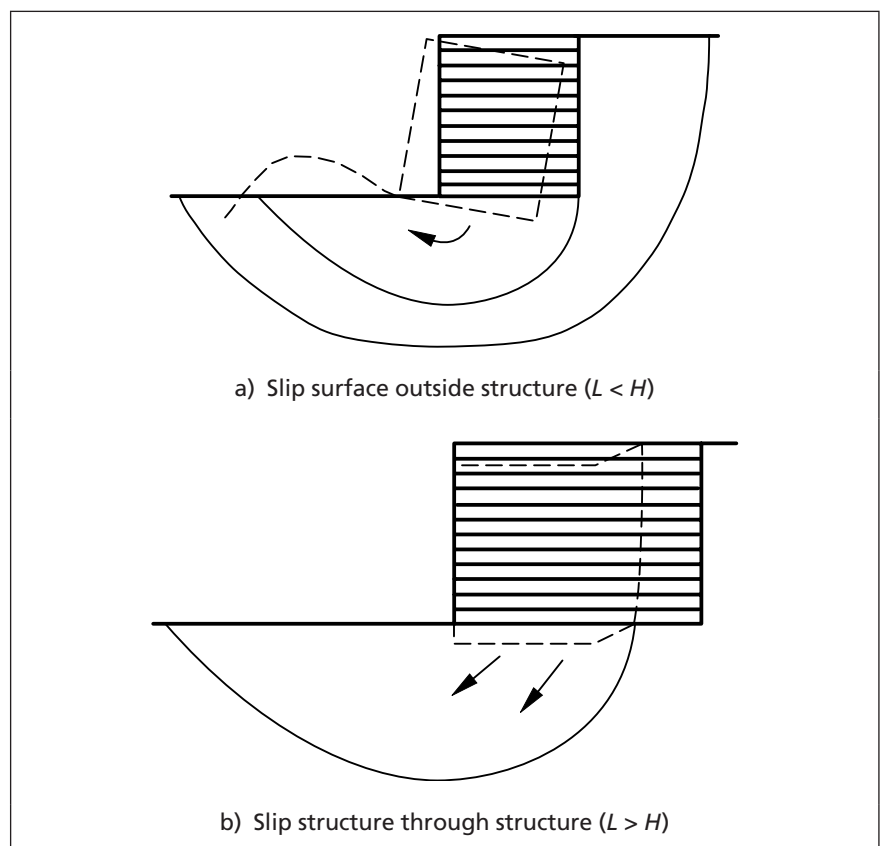
The influence of consolidation should be considered in the detailing of the facings.

### 6.5.6 External slip surfaces

All potential slip surfaces should be considered, including those passing wholly external to the structure [see Figure 26a)]. If residual shear surfaces are present then appropriate soil parameters should be used. The appropriate analysis method and the partial factors used should conform to BS EN 1997-1:2004.

Potential slip surfaces passing partly through the structure and partly external to the structure [see Figure 26b)] should also be considered. In the case of a potential failure surface passing partly through the structure, the resistance to failure provided by the reinforcement crossing the failure surface should be considered. The analysis should use the load factors and partial factors as given elsewhere in this standard.

Figure 26 Types of slip surface failure



## 6.6 Internal stability

### 6.6.1 General

It should be noted that stability within a reinforced structure is achieved by the reinforcing elements carrying tensile forces and transferring them by friction, friction and adhesion, or friction and bearing. In addition forces can be transferred through fill trapped by the elements. The fill is then able to support the associated shear and compressive forces. In the case of anchored earth, stability within a structure is achieved by the anchor elements carrying tensile forces and transferring these by friction along the anchor shaft or anchor loop and bearing of the anchor to the surrounding fill.

It should be noted that internal stability is concerned with the integrity of the reinforced volume; the structure has the potential to fail by the rupture or loss of bond of the reinforcements.

Consideration should be given to the local stability of individual layers of elements, sliding on horizontal planes and the stability of wedges.

The design recommendations below are generally applicable to all fills conforming to 3.1 except that special consideration should be given to amending the design equations where it is anticipated that pore water pressures will affect the short term soil properties of cohesive frictional fill.

The arrangement and layout of reinforcing elements should be chosen to provide stability and to suit the size, shape and detail of the facing. For simplicity a uniform distribution of identical reinforcing elements may be used throughout the height of the wall. However, it may be economical to divide the height of the wall into a number of zones and to design appropriate reinforcing elements for each zone.

### 6.6.2 Collapse mechanisms

#### 6.6.2.1 General

The following potential collapse mechanisms should be considered:

- a) stability of individual elements;
- b) resistance to sliding of upper portions of the structure;
- c) stability of wedges in the reinforced fill.

The following factors which influence stability should be included in the design check:

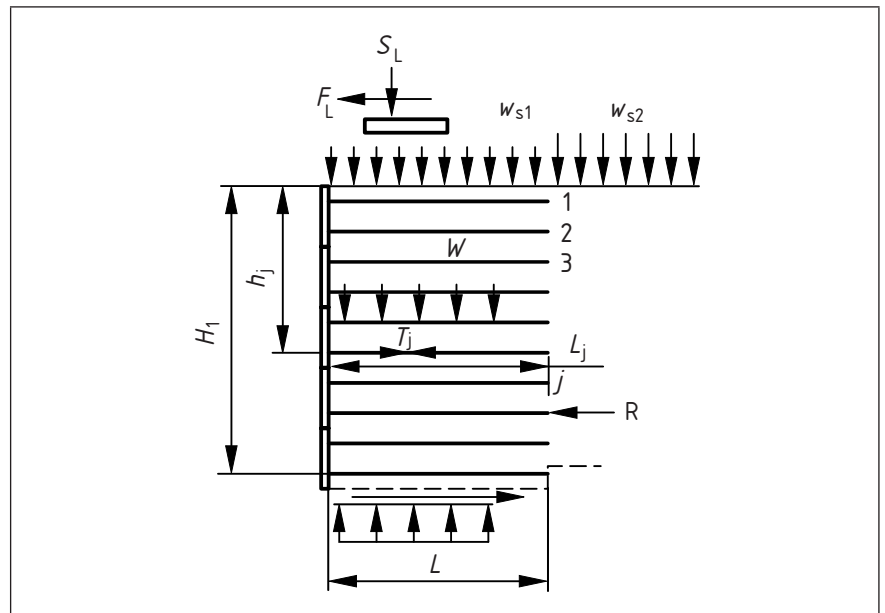
- 1) the capacity to transfer shear between the reinforcing elements;
- 2) the tensile capacity of the reinforcing elements;
- 3) the capacity of the fill to support compression.

The ultimate limit state should be modelled with the following assumptions.

- i) The soil shear strength is based upon  $\phi'_p$  using the appropriate partial material factor contained in Table 11.
- ii) Partial load factors are applied to the characteristic loads in accordance with 5.1 using the values contained in Table 12 or Table 13 for walls and abutments respectively to model collapse.

- iii) Loads are distributed throughout the reinforced soil block in accordance with the Meyerhof distribution, see Figure 27.
- iv) Partial reinforcement material factors  $f_m$  for the limit states and selected design life, are applied to the materials base strength in accordance with Table 10.
- v) The economic ramifications of collapse are considered by applying a partial factor  $f_n$  in accordance with Table 9.
- vi) The tensions in the reinforcement are based on the stresses assumed to occur in the soil at a short distance from the face of the wall.

Figure 27 Stability – Effects to be considered



### 6.6.2.2 Forward sliding of any portion of a wall on any horizontal plane

The stability against this mode of failure should be considered at the following interfaces where applicable:

- a) fill on fill within any layer;
- b) sheet reinforcement on any layer of fill;
- c) reinforcement elements and fill on any layer of fill.

### 6.6.3 Tie back wedge method for determining internal stability

#### 6.6.3.1 Coefficient of earth pressure

The coefficient of earth pressure should be taken as the active condition  $K_a$  for both the ultimate limit state and the serviceability limit state.

### 6.6.3.2 Ultimate limit state

#### 6.6.3.2.1 Local stability of a layer of reinforcing elements

The maximum ultimate limit state tensile force  $T_j$  to be resisted by the  $j$ th layer of elements at a depth of  $h_j$ , below the top of the structure, may be obtained from the summation of the appropriate forces as follows (see Figure 27);

$$T_j = T_{pj} + T_{sj} + T_{fj} \quad \text{or frictional fill}$$

and

$$T_j = T_{pj} + T_{sj} + T_{fj} - T_{cj} \quad \text{or cohesive friction fill}$$

where  $T_{pj}$ ,  $T_{sj}$ ,  $T_{fj}$  and  $T_{cj}$  are derived as follows.

- a) Vertical loading due to self weight of fill plus any surcharge and bending moment caused by external loading acting on the wall as shown in Figure 28.

$$T_{pj} = K_a \sigma_{vj} S_{vj}$$

where

$T_{pj}$  is the tensile force per metre "run";

$K_a$  is the coefficient of earth pressure within the reinforced volume;

$\sigma_{vj}$  is the factored vertical stress acting on the  $j$ th level of reinforcements according to the Meyerhof distribution [see Figure 25b)];

$S_{vj}$  is the vertical spacing of reinforcements at the  $j$ th level in the wall.

$$\sigma_{vj} = \frac{R_{vj}}{L_j - 2e_j}$$

where

$R_v$  is the resultant factored vertical load acting on the  $j$ th layer of reinforcements;

$L_j$  is the length of the reinforcements at the  $j$ th level in the wall;

$e_j$  is the eccentricity of resultant vertical load at the  $j$ th level in the wall.

For a uniform surcharge the expression for  $T_{pj}$  becomes (see Figure 28):

$$T_{pj} = \frac{K_{a1}(f_{fs}\gamma_1 h_j + f_q w_s) S_{vj}}{\left( \frac{K_{a2}(f_{fs}\gamma_2 h_j + 3f_q w_s) \left(\frac{h_j}{L}\right)^2}{1 - \frac{3(f_{fs}\gamma_1 h_j + f_q w_s)}{3(f_{fs}\gamma_1 h_j + f_q w_s)}} \right)}$$

where  $f_{fs}$  and  $f_q$  are partial load factors taken for the appropriate load combination and given in Table 12 and Table 13, and  $K_{a1}$  and  $K_{a2}$  are the active earth pressure coefficients of the reinforced zone and the fill behind respectively and  $w_s = w_{s1} = w_{s2}$ .

- b) Vertical strip loading  $S_L$  applied to a strip contact area of width  $b$  on top of the wall (see Figure 29).

For the purpose of deriving the magnitude only of the tensile force  $T_{sj}$  dispersal of the vertical load  $S_L$  from the contact area on top of the wall, may be taken at a slope of 2 vertically to 1 horizontally as shown in Figure 29.

$$T_{sj} = K_a S_{vj} \frac{f_f S_L}{D_j}$$

where

$$D_j = (h_j + b) \text{ if } h_j \leq (2d - b)$$

$f_f$  is the partial load factor for external concentrated dead loads, Table 11.

$$= (h_j + b)/2 + d \text{ if } h_j > (2d - b)$$

The tensile force obtained from the equation above should be taken as not less than that derived from the bending moment caused by the vertical loading  $S_L$  alone acting on the wall treated as a rigid body.

- c) Horizontal shear  $F_L$  applied to a strip contact area of width  $b$  on top of the wall, see Figure 30. For the purpose of deriving the magnitude only of the tensile force  $T_{fj}$  dispersal of the load  $F_L$  from the contact area on top of the wall may be taken as shown in Figure 30.

$$T_{fj} = 2 S_{vj} f_f F_L Q (1 - h_j Q)$$

where

$$Q = \frac{\tan(45^\circ - \phi'_p / 2)}{d + b/2}$$

and  $f_f$  is the partial load factor for external concentrated dead loads, see Table 11.

The tensile force obtained from the equation above should be taken as not less than that derived from the bending moment caused by the horizontal loading  $F_L$  alone acting on the wall treated as a rigid body.

- d) Effect of cohesion on force in reinforcement  $T_{cj}$

$$T_{cj} = 2 S_{vj} \frac{c'}{f_{ms}} \sqrt{K_a}$$

where

$c'$  is the cohesion under effective stress conditions;

$f_{ms}$  is the partial material factor applied to  $c'$  see Table 11.

Care should be taken that  $c'$  can be relied upon in design; for granular soils the value determined can be affected by the difficulty of fitting linear failure envelopes to Mohr circles; for cohesive soil, the value can be affected by failure to equalize pore pressures in the laboratory or the long term development of fissuring in the field.

For cohesive frictional fill

$$T_{pj} - T_{cj} \geq 0.5\gamma_w S_{vj} \left( h_j + \frac{f_{fs} W_s}{\gamma_1} \right)$$

where

$\gamma_w$  is the unit weight of water;

$h_j$  is the depth of the elements below the top of the structure;

$f_{fs}$  is the partial load factor applied to surcharge dead loads, Table 11;

$w_s$  is the surcharge dead load;

$\gamma_1$  is the unit weight of the soil (see Figure 28).

To avoid taking a too great and unsafe reduction in earth pressure due to the cohesive effect of fine grained backfill, the pressure should not be less than that due to a fluid with half the unit weight of water.

Figure 28 Stresses imposed due to self-weight, surcharge and retained backfill

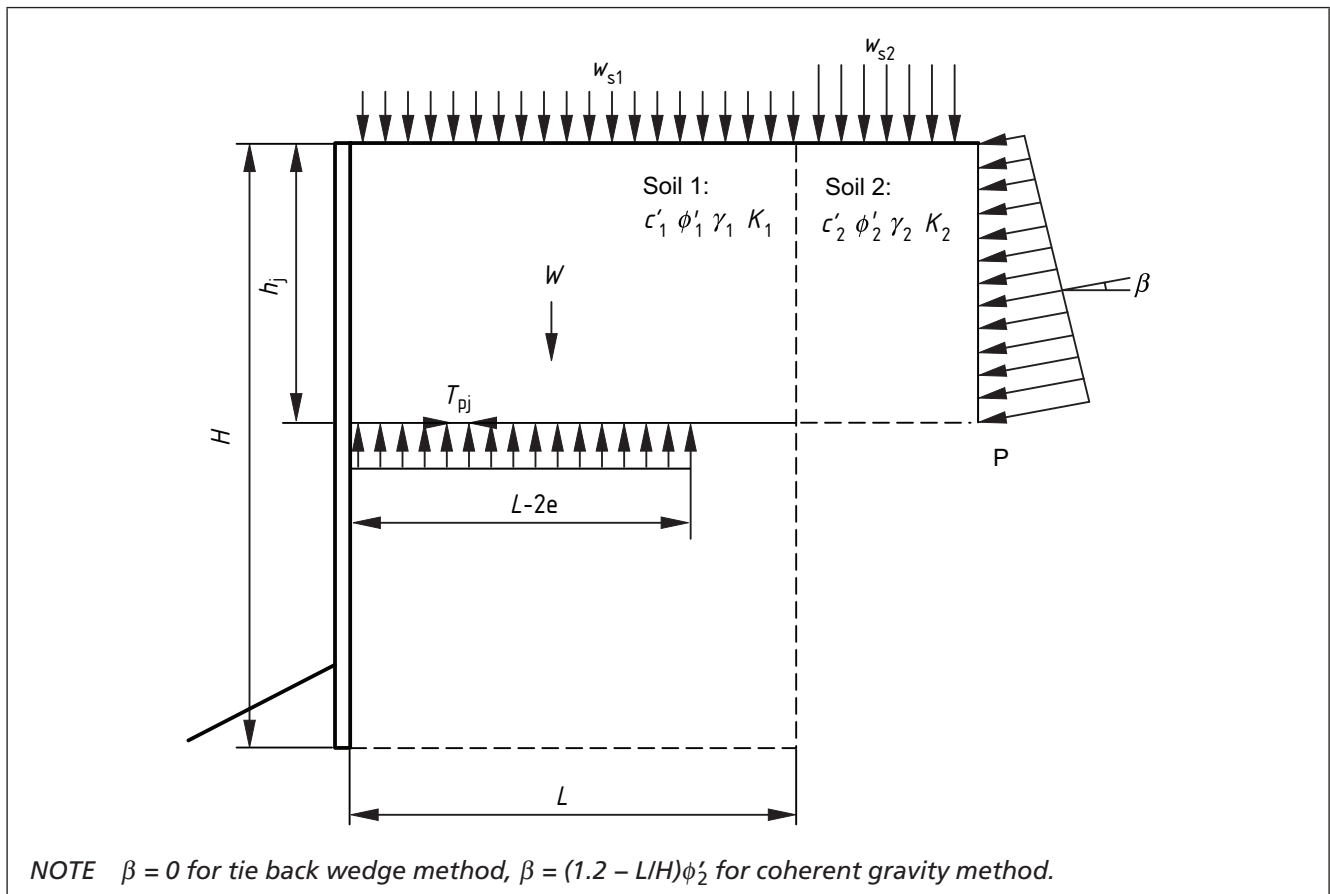


Figure 29 Dispersal of vertical strip load through reinforced fill – Tie back wedge method

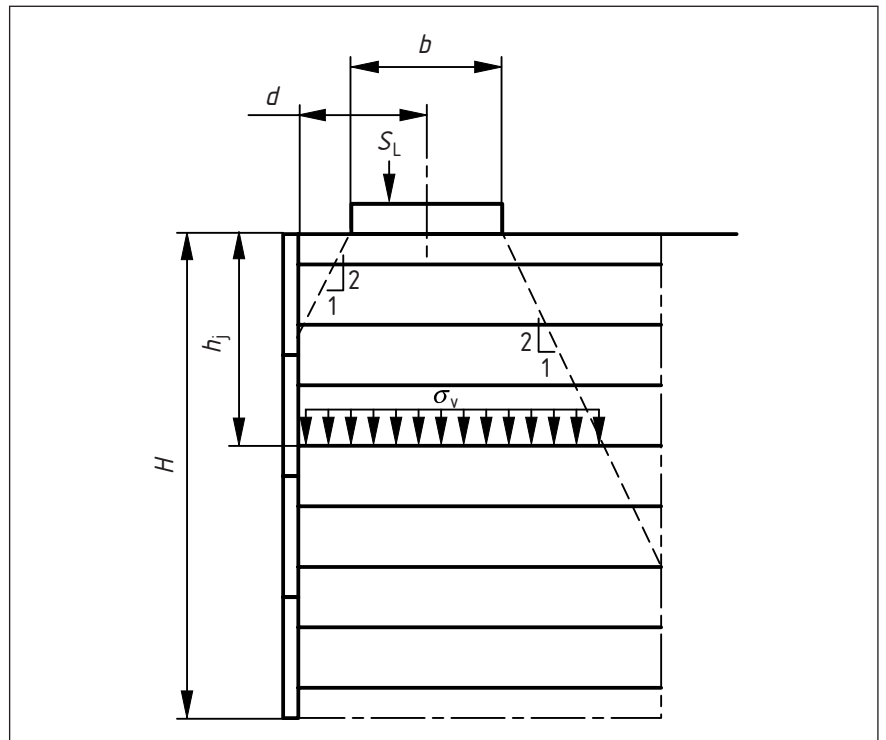
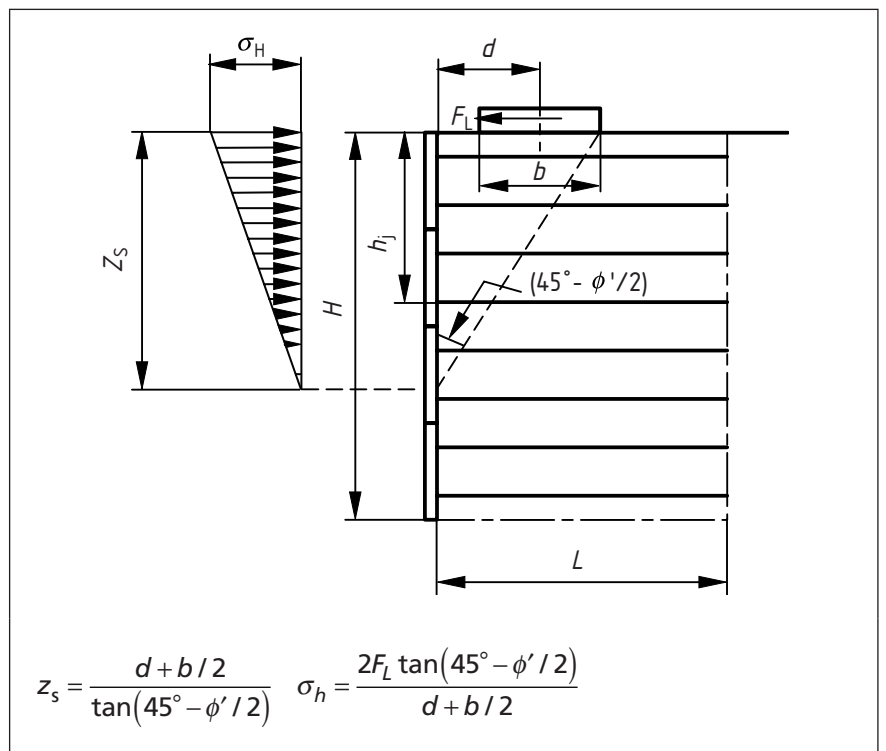


Figure 30 Dispersal of horizontal shear through reinforced fill – Tie back wedge method



### 6.6.3.2.2 Local stability check

The resistance of the  $j$ th reinforcing element should be checked against rupture and adherence failure whilst carrying the factored loads, as follows.

- a) *Rupture*. The tensile strength of the  $j$ th layer of reinforcing elements needed to satisfy local stability considerations is:

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

where

$T_j$  is the maximum value from 6.6.3.2.1;

$T_D$  is the design strength of the reinforcement calculated in accordance with 5.3.3;

$f_n$  is the partial factor for economic ramifications of failure, see Table 9.

- b) *Adherence*. The perimeter  $P_j$  of the  $j$ th layer of reinforcing elements needed to satisfy local stability considerations is:

$$P_j \geq \frac{T_j}{\frac{\mu L_{ej}(f_{fs}\gamma_1 h_j + f_f w_s)}{f_p f_n} + \frac{a'_{bc} c' L_{ej}}{f_{ms} f_p f_n}}$$

where

$P_j$  is the total horizontal width of the top and bottom faces of the reinforcing element at the  $j$ th layer, per metre "run";

$T_j$  is the maximum value from 6.6.3.2.1;

$f_{fs}$  is the partial load factor applied to soil self weight taken from the same load combination as  $T_j$ , see Table 11;

$f_f$  is the partial load factor applied to surcharge dead loads taken from the same load combination as  $T_j$ , see Table 11;

$\mu$  is the coefficient of friction between the fill and reinforcing elements;

$L_{ej}$  is the length of reinforcement in the resistant zone outside failure wedge, at the  $j$ th layer of reinforcements, see Figure 31;

$w_s$  is the surcharge due to dead loads only;

$f_p$  is the partial factor for reinforcement pull-out resistance, see Table 11;

$f_n$  is the partial factor applied to economic ramifications of failure, see Table 9;

$a'_{bc}$  is the adhesion coefficient between the soil and the reinforcement;

$c'$  is the cohesion of the soil measured under effective stress conditions;

$f_{ms}$  is the partial material factor applied to  $c'$ , see Table 11.



For convenience it may be assumed

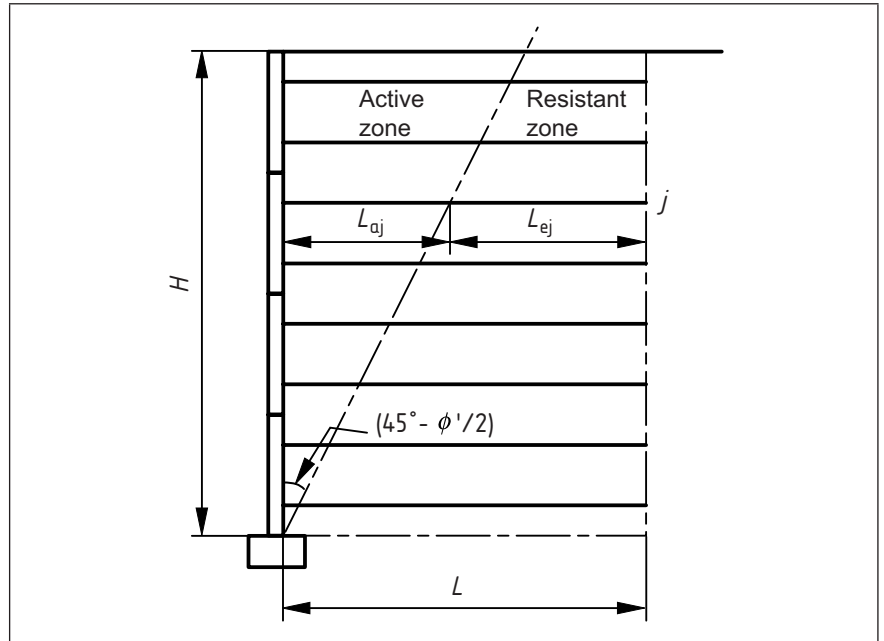
$$\mu = \frac{a' \tan \phi'_p}{f_{ms}}$$

where

$a'$  is the interaction coefficient relating soil/reinforcement bond angle with  $\tan \phi'_p$ ;

$f_{ms}$  is the partial material factor applied to  $\tan \phi'_p$ , Table 11.

Figure 31 Determination of adherence capacity of the reinforcement – Tie back wedge method



6.6.3.2.3 Anchored earth

There are a variety of different anchored earth systems that may be used (for examples see Figure 32). The tensile forces generated in the anchor should be calculated in accordance with 6.6.3.2.1. Local stability in terms of rupture should be considered in accordance with 6.6.3.2.2 or 6.6.4.2.5. The pull-out capacity of anchor reinforcing elements to satisfy local stability considerations may be determined from:

$$\frac{P_{uj}}{f_p f_n} \geq T_j$$

where

$P_{uj}$  is the ultimate pull-out resistance of the anchor;

$f_p$  is the partial factor for reinforcement pull-out resistance, see Table 11;

$f_n$  is the partial factor applied to economic ramifications of failure, see Table 9;

$T_j$  is the maximum value of the  $j$ th level of reinforcements from 6.6.3.2.1.

The ultimate pull-out resistance of an anchor element of the *j*th layer may be determined from:

$$P_{uj} = P_{sj} + P_{aj}$$

$$P_{sj} = 2 \mu B_s \sigma_{aj} L_{ej}$$

and

$$P_{aj} = 4K_p B_a t_a \sigma_{aj}$$

where

$P_{sj}$  is the shaft or loop resistance developed by friction beyond the potential failure plane, at the *j*th layer of anchors;

$P_{aj}$  is the bearing resistance at the *j*th layer of anchors;

$\mu$  is the coefficient of soil/reinforcement friction and is determined according to the relationship given in 6.6.3.2.2;

$B_s$  is the long term horizontal projection area of shaft or loop;

$K_p$  is the horizontal passive earth pressure coefficient;

$B_a$  is the long term width of anchor head;

$t_a$  is the long term height of anchor head;

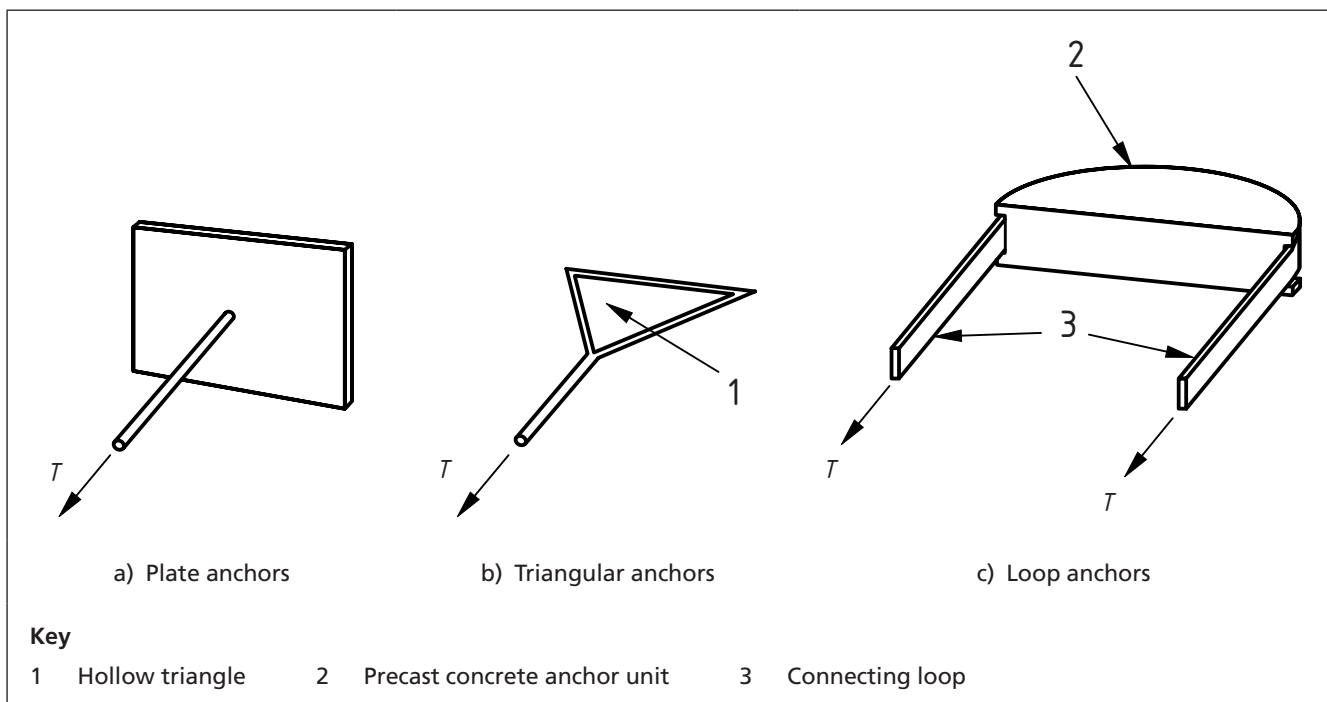
$\sigma_{aj}$  is the vertical applied pressure at the *j*th layer of anchors in the resistant zone;

$L_{ej}$  is the length of the anchor shaft beyond the potential failure plane.

*NOTE* When threaded end connections are used, the cross-sectional area of the anchor shaft should be based upon the tensile stress area.

Grouted anchor elements should be treated as ground anchors, and the ultimate pull-out resistance should be determined from the relations given in BS 8081.

Figure 32 Examples of some types of reinforced soil anchors



#### 6.6.3.2.4 Wedge stability

Wedges are assumed to behave as rigid bodies and may be of any size and shape. Stability of any wedge may be maintained when friction forces acting on the potential failure plane in conjunction with the tensile resistance/bond of the group of reinforcements or anchors embedded in the fill beyond the plane is able to resist the applied loads tending to cause movement, see Figure 33.

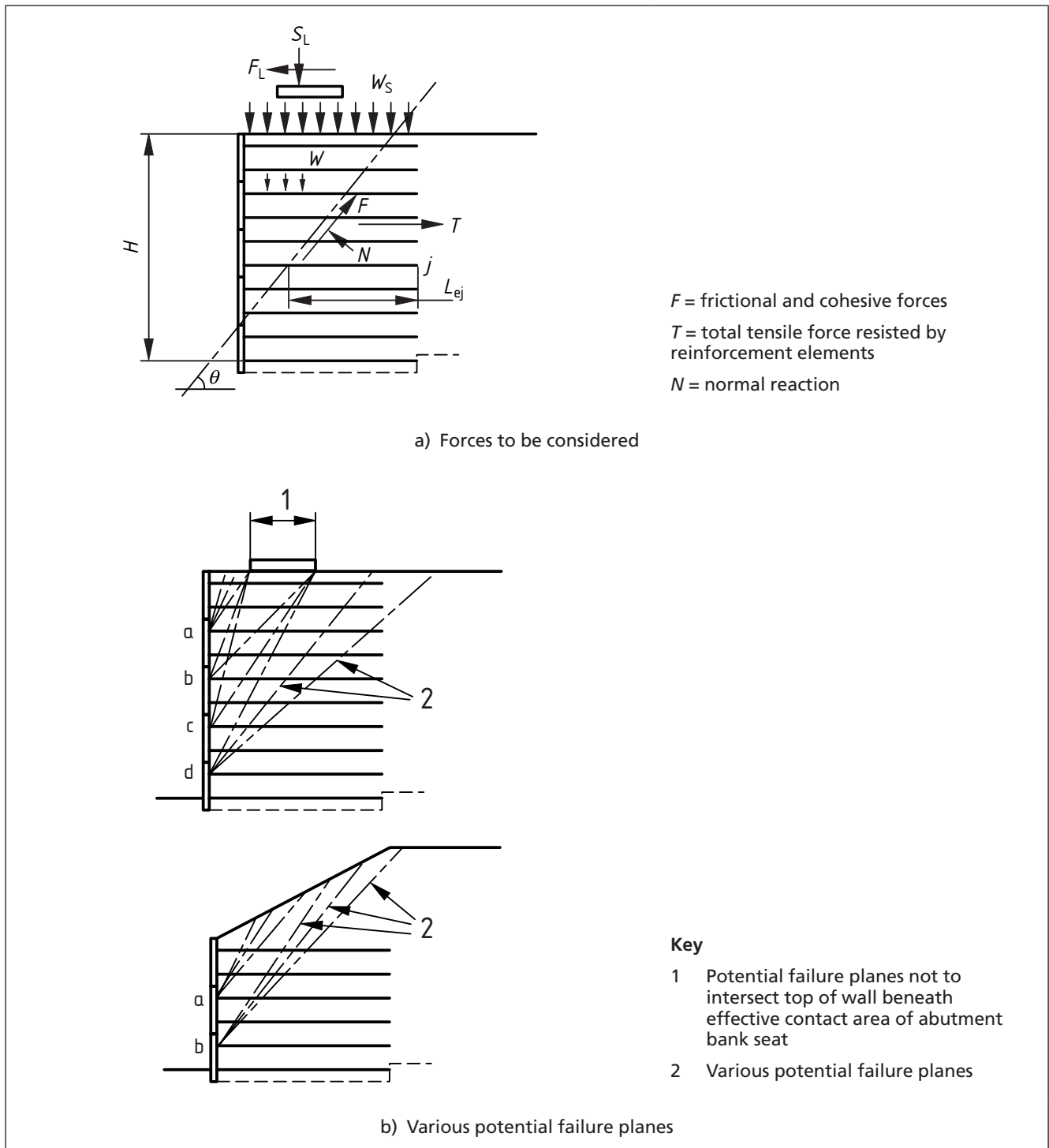
The following loads, factored in accordance with combinations in Table 11, and forces should be considered:

- a) self weight of the fill in the wedge;
- b) uniformly distributed surcharge loads,  $w_s$ ;
- c) vertical strip loading,  $S_L$ ;
- d) horizontal shear,  $F_L$ ;
- e) frictional and cohesive forces acting along the potential failure plane;
- f) the normal reaction of the failure plane.

A selection of potential failure planes should be investigated for each of the typical points a, b, c, etc., shown in Figure 33b). The forces acting on each wedge should be resolved into two mutually perpendicular directions. Since the forces are assumed to be in equilibrium the two equations may be solved simultaneously to yield the value of the gross tensile force  $T$  to be resisted by reinforcing elements or anchors. For each of the typical points the maximum value of  $T$  should be established by analysing the forces acting on a number of different wedges. The maximum value of  $T$  and the corresponding value of  $\beta'$  should be used to calculate the frictional/tensile capacity of the group of elements anchoring the wedge, see 6.6.3.2.5 and Figure 34.

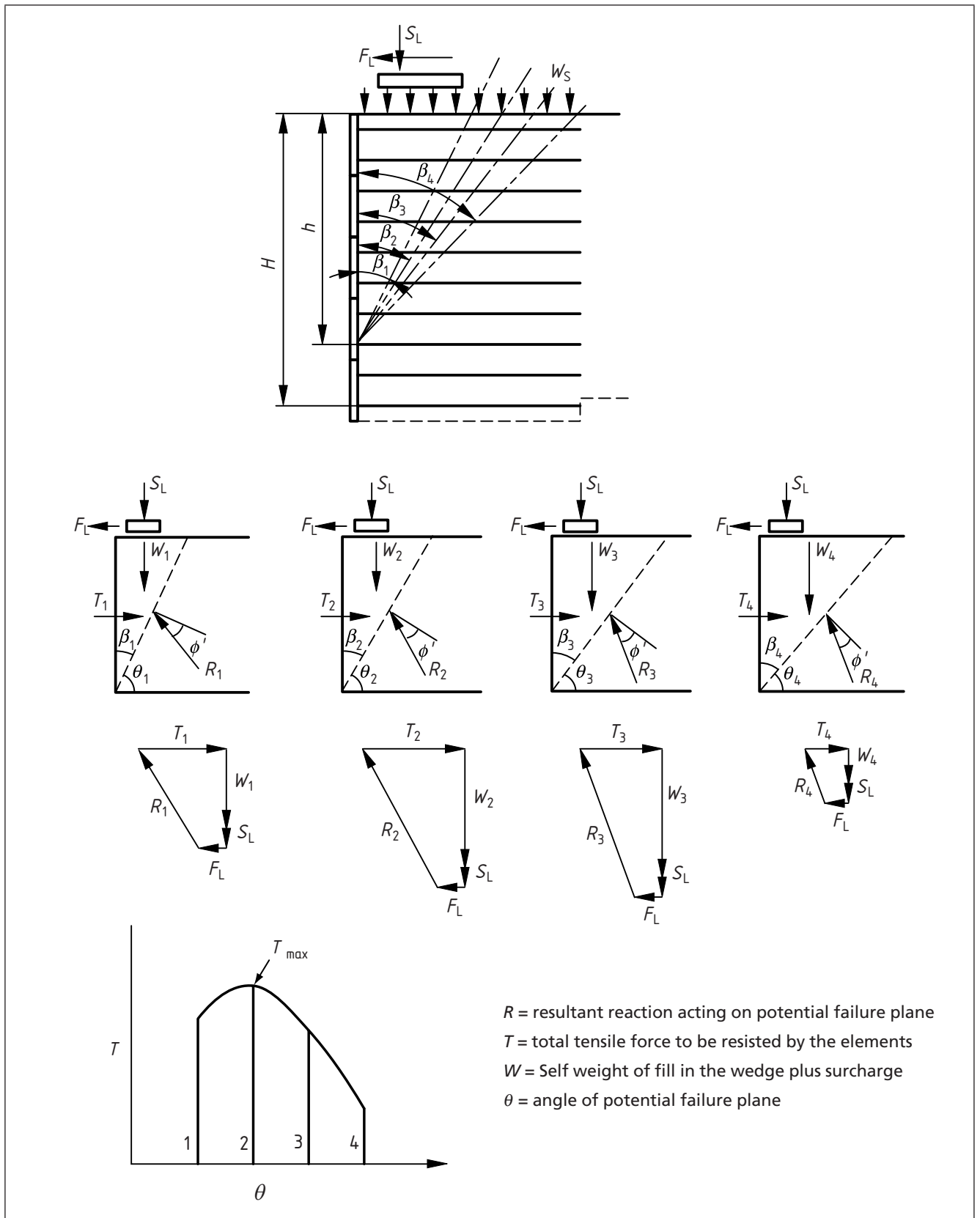
For the case of a wall with a level top containing frictional fill and which supports uniform surcharge only the inclination of the potential failure plane may be taken as  $\beta' = (45^\circ - \phi'_p / 2)$ . However in the more complex general case it is not possible to give any guidance on either the angle of the potential failure plane which produces the maximum value of  $T$  or on the number of points which should be checked. These should be determined for each structure. It may be assumed that no potential failure plane will pass through the strip contact area representing a bridge bank seat. When the facing consists of a structural element formed in one piece the shear resistance offered by the rupture of the facing may be considered.

Figure 33 Internal wedge stability



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Figure 34 Internal wedge stability analysis of simple problem



### 6.6.3.2.5 Wedge stability check

The resistance provided by an individual layer of reinforcing elements should be taken to be the lesser of either:

- the frictional resistance of that part of the layer embedded in the fill beyond the potential failure plane or, in the case of anchored earth, the pull-out resistance of the part of the anchors embedded in the fill beyond the potential failure plane (which should be neglected when the distance between the potential failure plane and the start of the anchorage is less than one metre); or
- the tensile resistance of the layer of elements.

For reinforced soil the total resistance of the layers of elements anchoring the wedge is satisfied by:

$$\sum_{j=1}^m \left[ \frac{T_{Dj}}{f_n} \right] \geq T \text{ or}$$

$$\sum_{j=1}^m \left[ \frac{P_j L_{ej}}{f_p f_n} \left( \mu f_{fs} \gamma h_j + \mu f_f w_s + \frac{a'_{bc} c'}{f_{ms}} \right) \right] \geq T$$

where

the lesser value for each layer should be used in the summation;

$T_{Dj}$  is the design strength of the reinforcements at  $j$ th level in wall, see 5.3.3;

$f_n$  is the partial factor applied to economic ramifications of failure, see Table 9;

$P_j$  is the total horizontal width of the top and bottom faces of the reinforcing element;

$L_{ej}$  is the length of reinforcement in the resistant zone outside potential failure wedge, see Figure 33;

$f_p$  is the partial factor for reinforcement pull-out resistance, see Table 11;

$w_s$  is the surcharge due to dead loads only;

$a'_{bc}$  is the adhesion coefficient relating soil cohesion to soil/reinforcement bond;

$c'$  is soil cohesion measured under effective stress conditions;

$f_{ms}$  is the partial material factor applied to  $c'$ , see Table 11.

## 6.6.4 Coherent gravity method

### 6.6.4.1 Coefficient of earth pressure within the structure

For both the ultimate limit state and the serviceability limit state the coefficient of earth pressure should be taken as  $K_o$  at the top of the wall reducing linearly with depth to a value of  $K_a$  at a depth of 6 m below the top of the structure as set out below and as shown on Figure 35.

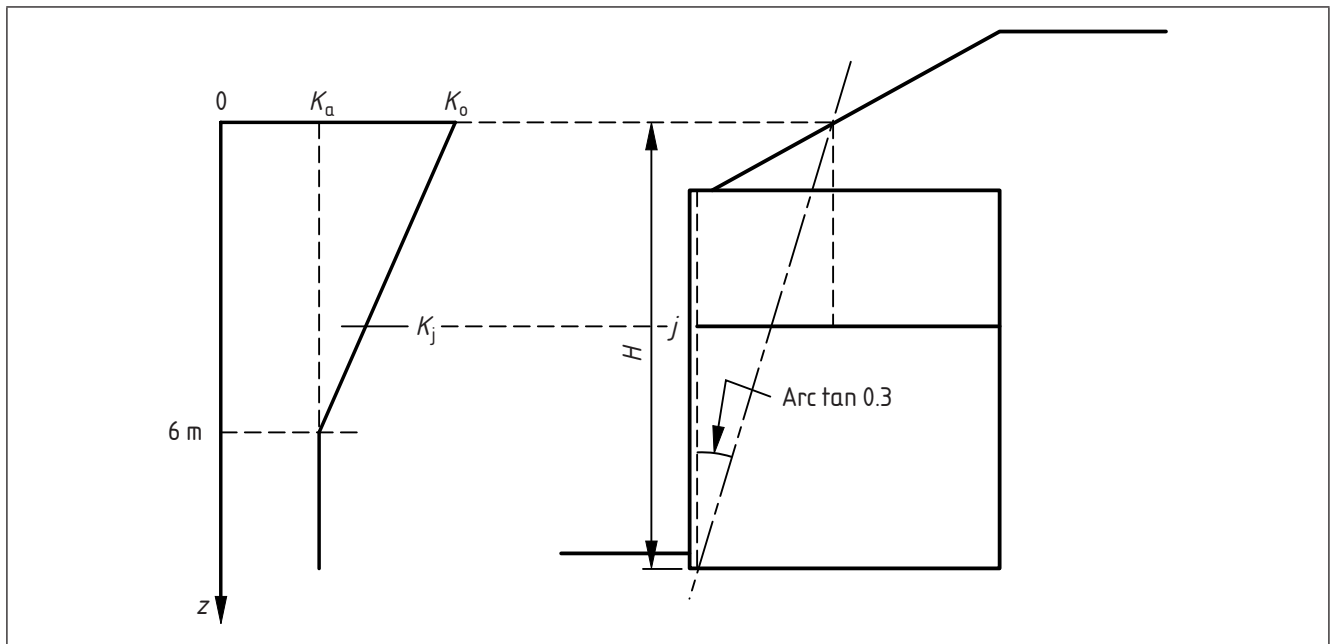
$$K = K_o(1 - z/z_o) + (K_a z/z_o) \text{ for } z \leq z_o = 6 \text{ m}$$

$$K = K_a \text{ for } z > z_o$$

where

$z$  is the depth measured from the upper level of the mechanical height  $H$ .

Figure 35 Variation of coefficient of earth pressure with depth – Coherent gravity method



#### 6.6.4.2 Ultimate limit state

##### 6.6.4.2.1 Local stability of a layer of reinforcing elements

The maximum tensile force  $T_j$  to be resisted by the  $j$ th layer of elements at a depth  $h_j$  below the top of the wall may be obtained from the summation of the appropriate forces as follows:

$$T_j = T_{pj} + T_{sj} + T_{fj} \quad \text{for frictional fill}$$

$$T_j = T_{pj} + T_{sj} + T_{fj} - T_{cj} \quad \text{for cohesive frictional fill}$$

where  $T_{pj}$ ,  $T_{sj}$ ,  $T_{fj}$  and  $T_{cj}$  are derived as follows and measured in terms of load per metre "run".

The force in each reinforcement layer may be derived by calculation of the component due to the various load effects as shown on Figure 27. These should be summed to give the total load to be resisted as follows.

- a) Vertical loading due to self weight plus any surcharge and bending moment caused by external loading acting on the wall (see Figure 28):

$$T_{pj} = K\sigma_{vj}S_{vj}$$

where

$K$  is the coefficient of earth pressure within reinforced volume, see 6.6.4.1;

$\sigma_{vj}$  is the vertical stress on the  $j$ th level of reinforcements;

$S_{vj}$  is the vertical spacing of reinforcements at the  $j$ th level in the wall;

and

$$\sigma_{vj} = \frac{R_{vj}}{L_j - 2e_j}$$

where

- $R_{vj}$  is the resultant factored vertical load excluding external strip loads acting on the  $j$ th layer of reinforcements;
- $L_j$  is the length of the reinforcements at  $j$ th level in the wall;
- $e_j$  is the eccentricity of resultant vertical load at  $j$ th level of the wall.

To avoid taking a too great and unsafe reduction in earth pressure due to the cohesive effect of fine grained backfill, the pressure should not be less than that due to a fluid with half the unit weight of water.

The bending moment arising from self weight, surcharge and external loading should include the effects of the external strip loads as  $S_L l$  and  $F_L h_j$  (see Figure 36 and Figure 37), where  $l$  is the offset dimension between centre of pressure diagram below strip load and centre of pressure diagram on the  $j$ th layer of reinforcement.

b) Vertical loading  $S_L$  applied to a strip contact area (see Figure 36):

$$T_{sj} = K \sigma_v(h_j, d') S_{vj}$$

where

$$\sigma_v(h_j, d') = f_f \frac{Q}{2} \left[ F_B \left( \frac{d' + b'}{h_j} \right) - F_B \left( \frac{d' - b'}{h_j} \right) \right]$$

where  $F_B$  is a function and is equal to

$$F_B = \frac{2}{\pi} \left[ \frac{X}{1 + X^2} + \tan^{-1}(X) \right],$$

with  $\tan^{-1}(X)$  in radians

where

- $X$  is  $(d' + b')/h_j$  and  $(d' - b')/h_j$  as shown in the above equation for  $\sigma_v(h_j, d')$ ;
- $Q$  is the pressure beneath the strip footing as shown in Figure 36;
- $f_f$  is the partial load factor for external loads, see Table 11;
- $S_{vj}$  is the vertical spacing of reinforcements at  $j$ th level in the wall.

At each level of  $h_j$ , the above expressions may be used to calculate the value of  $\sigma_v$  for various values of  $d'$  as the vertical stress varies along the reinforcement of length  $L$ . The relevant value of  $\sigma_v(h_j, d')$  may be used in the above expression for calculating  $T_{sj}$  at the corresponding locations along the reinforcement. The variation in  $\sigma_v$  along the reinforcement may be used to determine the adherence capacity of the reinforcements, see 6.6.4.2.4.

*NOTE* The equations are derived from Boussinesq for half of an imaginary load centred on the front face of the wall.

Alternatively,  $\sigma_v$  may be calculated in a simpler way by assuming that the dispersal of the vertical load  $S_L$  from the contact area  $b'$  on top of the wall, may be taken at a slope of 2 vertically to 1 horizontally similar to Figure 29.



- c) Horizontal shear  $F_L$  applied to a strip contact area of width  $b$  on top of the wall (see Figure 37):

$$T_{fj} = \frac{2f_f F_L S_{vj}}{d + \frac{b}{2}} \left( 1 - \frac{h_j}{d + \frac{b}{2}} \right)$$

where

$f_f$  is the partial load factor applied to external loads, see Table 11.

The values for  $T_{sj}$  and  $T_{fj}$  do not consider any longitudinal diffusion parallel to the face of the structure.

A more rigorous analysis may be performed when:

for  $0 < h_j < 0.75S_{L1}$  longitudinal diffusion at 1 vertical to 4 horizontal;

for  $h_j > 0.75S_{L1}$  longitudinal diffusion at 3 vertical to 4 horizontal;

where

$S_{L1}$  is the length of strip load.

- d) Effect of cohesion on force in reinforcement  $T_{cj}$ :

$$T_{cj} = 2S_{vj} \frac{c'}{f_{ms}} \sqrt{K}$$

where

$c'$  is the cohesion under effective stress conditions;

$f_{ms}$  is the partial material factor applied to  $c'$ , see Table 11;

$K$  is the coefficient of earth pressure, see 6.6.4.1.

$$T_{pj} - T_{cj} \geq 0.5\gamma_w S_{vj} \left( h_j + \frac{f_{fs} w_s}{\gamma_1} \right)$$

where

$\gamma_w$  is the unit weight of water;

$h_j$  is the depth of the elements below the top of the structure;

$f_{fs}$  is the partial load factor applied to surcharge dead loads (see Table 11);

$w_s$  is the surcharge dead load;

$\gamma_1$  is the unit weight of the soil (see Figure 28).

Care should be taken that  $c'$  can be relied upon in design; for granular soils the value determined can be affected by the difficulty of fitting linear failure envelopes to Mohr circles; for cohesive soil the value can be affected by failure to equalize pore pressures in the laboratory or the long term development of fissuring in the field.

Figure 36 Dispersal of vertical strip load through reinforced fill – Coherent gravity method

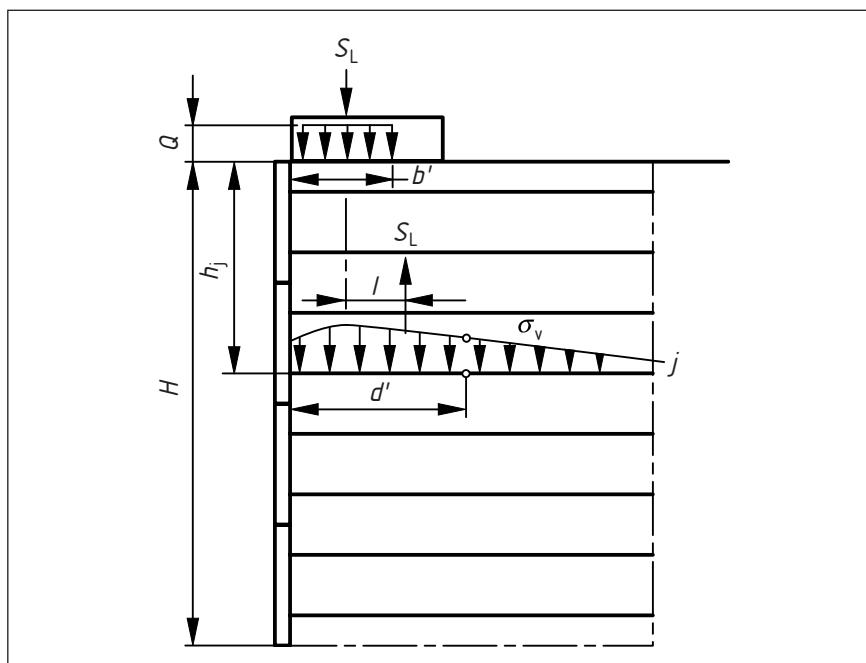
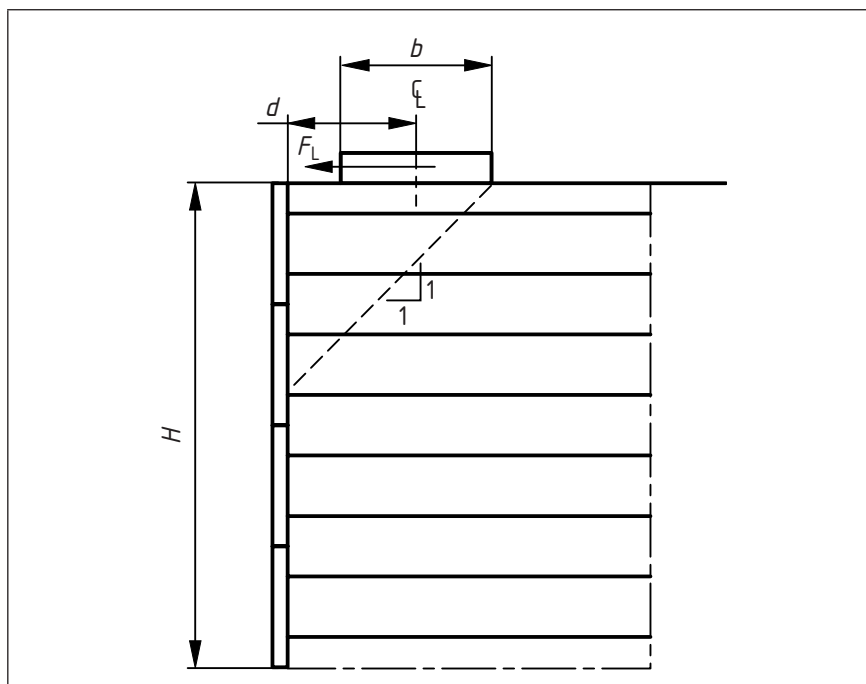


Figure 37 Dispersal of horizontal shear through reinforced fill – Coherent gravity method



### 6.6.4.2.2 Lines of maximum tension

The line of maximum tension for a retaining wall may be assumed to follow a log spiral (see Figure 38). For calculation purposes this line may be assumed to be as shown in Figure 39, referred to as maximum tension line 2. It should be noted that when a structure supports a superimposed strip load then the influence of the strip loads may affect the location of line 2.

When the strip load is located beyond the position of line 2 defined in Figure 39 the upper 1:6 portion of line 2 should be assumed to coincide with the rear of the strip load. However, line 2 does not go beyond that defined by a structure of equivalent height  $H_m$ .

Where  $H_m$  is the greater of:

- $H$  [see Figure 19b)]; or
- $H_1 + Q_m/\gamma_1$

where  $Q_m$  is the average pressure over an area of  $0.5H_1$  behind the facing.  $Q_m$  is calculated by the Meyerhof method [38] and with all load factors set to 1.0.

When a structure is subject to superimposed strip loads a second line of maximum tension should be considered in addition to the maximum tension line 2 defined above, defined as maximum tension line 1.

Both potential maximum tensions lines 1 and 2 are shown in Figure 40, the maximum tension is assumed where the reinforcement crosses either line 1 or line 2. For calculation purposes the definition of line 1 may be rationalized to the lines shown in Figure 41.

Figure 38 Line of maximum tension for retaining wall – Coherent gravity method

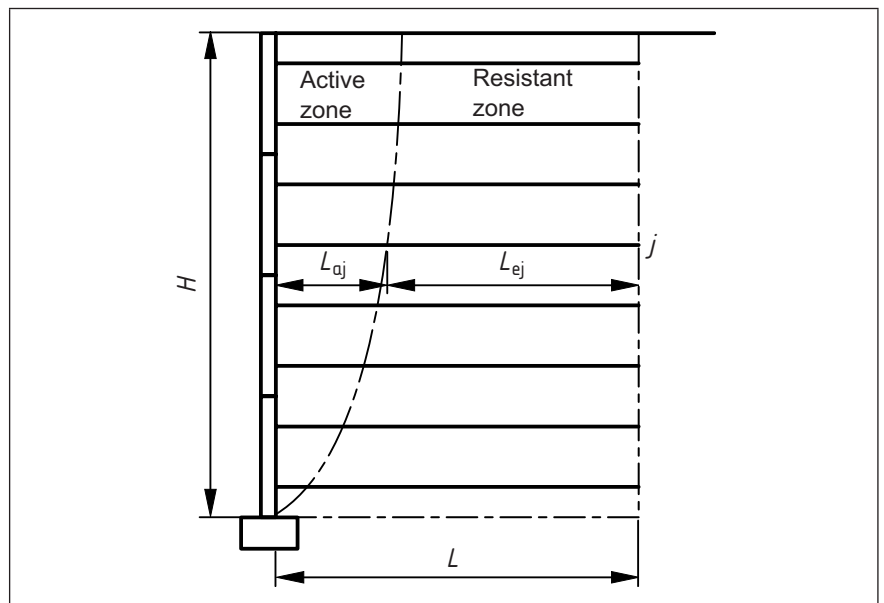


Figure 39 Definition of maximum tension line 2 (retaining wall without superimposed strip loads) – Coherent gravity method

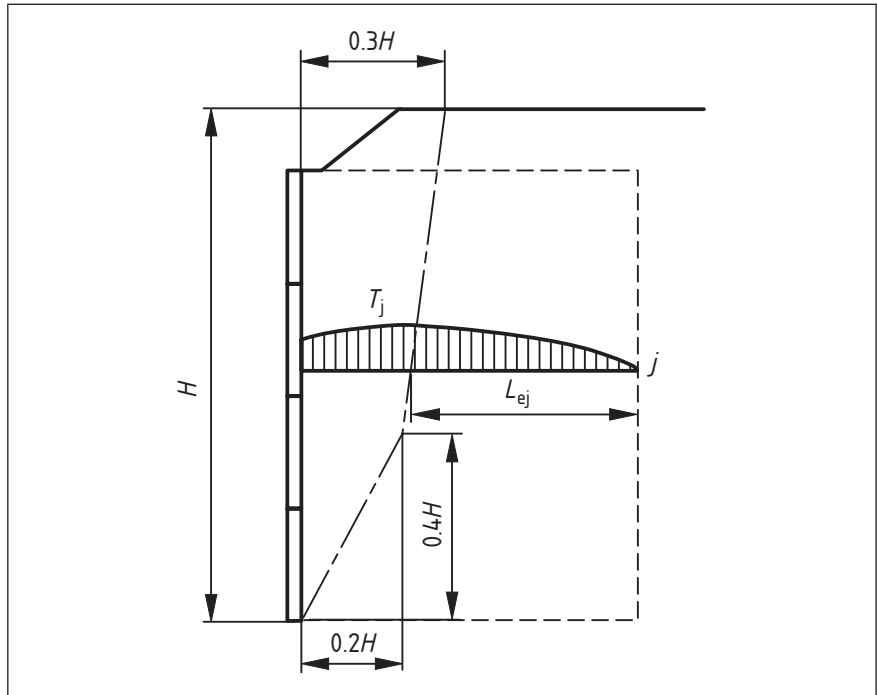
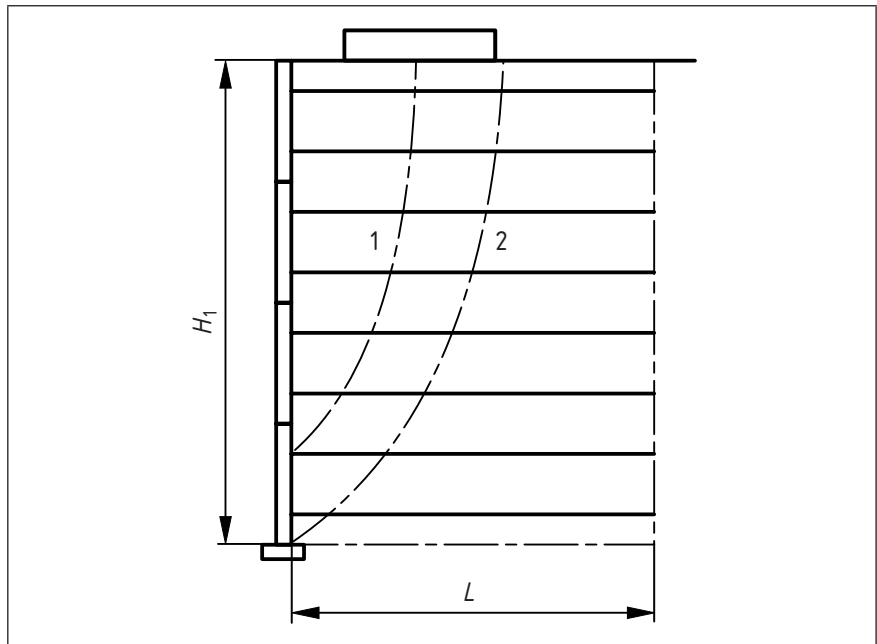
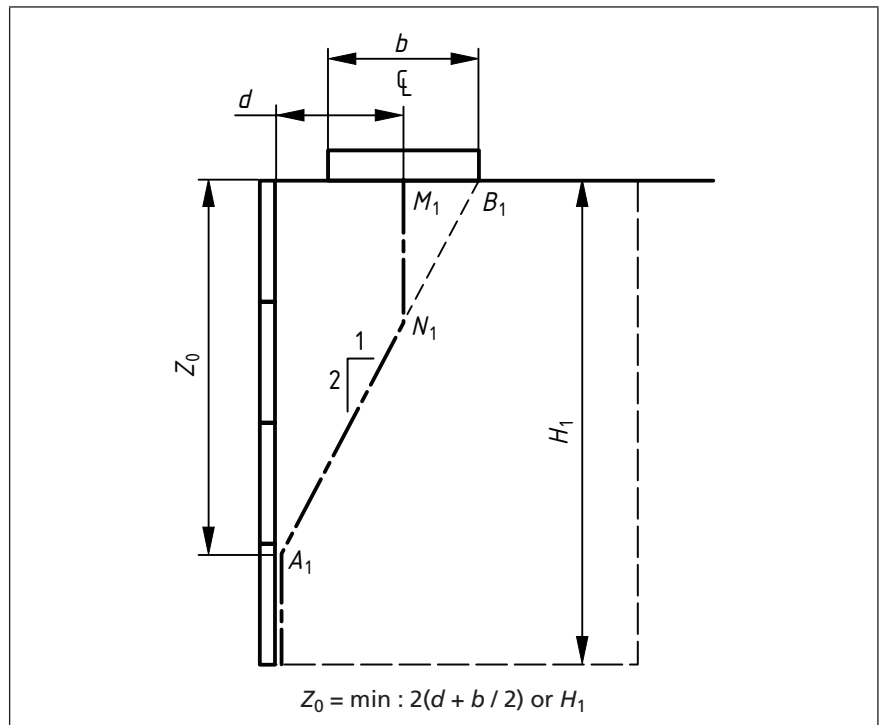


Figure 40 Lines of maximum tension for structures with strip loads – Coherent gravity method



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Figure 41 Definition of line 1 – Coherent gravity method



#### 6.6.4.2.3 Tension in the reinforcements

The tensile loads should be calculated at three positions:

- a) at the facing;
- b) along maximum tension line 1;
- c) along maximum tension line 2.

The values of  $T_j$  at each level calculated in accordance with 6.6.4.2.1 may be taken to be the maximum loads imposed in the reinforcements due to the sum of all the various loading effects.

The load in a reinforcement varies along its length and factors should be applied to determine tensile loads at various positions.

For frictional fill:

- at facing,  $T_j = a_0 T_{pj} + T_{sj} + T_{fj}$
- at line 1,  $T_j = a_1 T_{pj} + T_{sj} + T_{fj}$
- at line 2,  $T_j = T_{pj} + T_{sj} + T_{fj}$ .

For cohesive frictional fill, calculate  $T_{pj}$  as in the three preceding equations but reduce by the value of  $T_{cj}$ .

$$T_{pj} = (T_{pj} - T_{cj});$$

where

$a_0$  = variable;

$a_1$  = variable.

For an articulated face:

$$a_0 = 0.85 \text{ if } h_j \leq Z_2;$$

$$a_0 = 1 - 0.15 (H_1 - h_j)/(H_1 - Z_2) \text{ if } h_j > Z_2;$$

$$a_1 = 1 \text{ if } h_j \leq Z_1;$$

$$a_1 = a_0 + (1 - a_0)(Z_0 - h_j)/(Z_0 - Z_1) \text{ if } Z_1 < h_j < Z_0;$$

$$a_1 = a_0 \text{ if } h_j \geq Z_0.$$

where

$Z_0$  is the minimum of  $2(d + b/2)$  and  $H_1$ ;

$Z_1$  is the width  $b$  of the strip;

$$Z_2 = 1.5H_1 - 3X;$$

$X$  is the width of active zone at underside of strip footing.

*NOTE*  $T_{sj}$  in the above equations is that value applicable to the point along the reinforcement being considered.

#### 6.6.4.2.4 Adherence capacity of the reinforcement

For inextensible reinforcement the line between the active and resistant zones is as shown in Figure 38 but for design purposes the equivalent line in Figure 39 may be adopted. For the structures with strip loads the adherence should be checked beyond lines 1 and 2 (see Figure 40 and Figure 41), and compared with the relevant reinforcement tension at each of these points.

The adherence capacity  $T_j$  of each layer of reinforcement is given by:

$$T_j \leq \frac{2B\mu}{f_p f_n} \int_{L-L_{aj}}^L f_{ts} \sigma_v(x) dx$$

where

$f_p$  is the partial factor for reinforcement pull-out resistance, see Table 11;

2 is for two faces of the reinforcement;

$B$  is the width of the reinforcement;

$L$  is the total length of reinforcement;

$L_{aj}$  is the length of reinforcement beyond the line of maximum tension considered at the  $j$ th level;

$\mu$  is the value of the coefficient of friction, or  $\mu^*$  as appropriate, at the vertical stress level;

$\sigma_v(x)$  is the vertical stress along length  $x$  of the reinforcement;

$f_n$  is the partial factor for economic ramifications of failure, see Table 9;

$f_{ts}$  is the partial load factor, see Table 12 or Table 13.

#### 6.6.4.2.5 Long term rupture capacity of reinforcement or anchors at the end of the service life

The capacity of the reinforcing element at each layer should satisfy the following expression:

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

where

$T_j$  is the maximum value from 6.6.4.2.3;

$T_D$  is the design strength of the reinforcement calculated per metre run of the structure in accordance with 5.3.3;

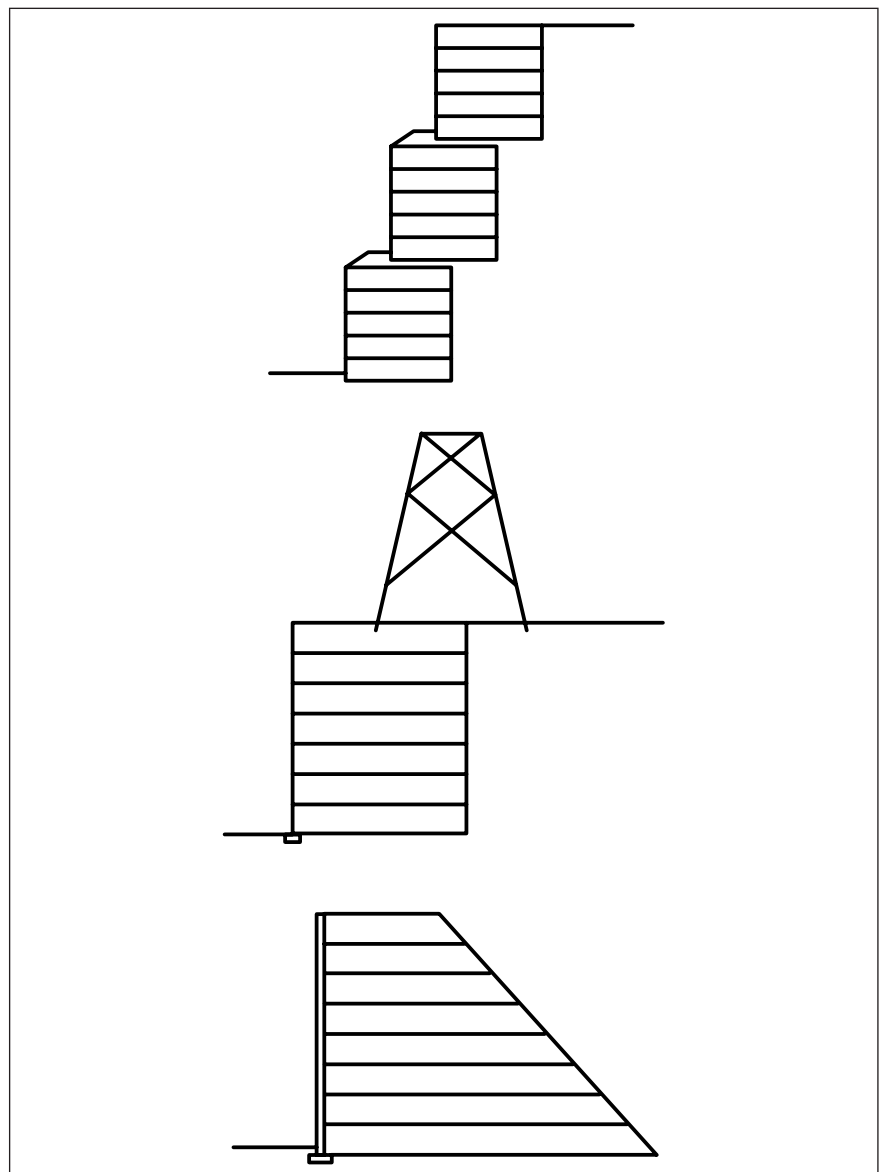
$f_n$  is the partial factor for economic ramifications of failure, see Table 9;

### 6.6.4.3 Global internal stability analysis

Structures should be designed to provide local stability to each layer of reinforcing elements in accordance with 6.6.4.2.1 to 6.6.4.2.5. In the general case additional stability analysis is not necessary.

Where the structure is of unusual geometry or supports concentrated loads that are not specifically covered in the code the local equilibrium method above might not be sufficient and a slip circle analysis or a global wedge stability check should be performed, see 6.6.3.2.4. Examples of structures requiring a global analysis are shown in Figure 42.

Figure 42 Examples of structures requiring global stability analysis – Coherent gravity method



### 6.6.5 Serviceability limit state

The potential mechanism of post-construction internal movements should be considered [see Figure 23b)].

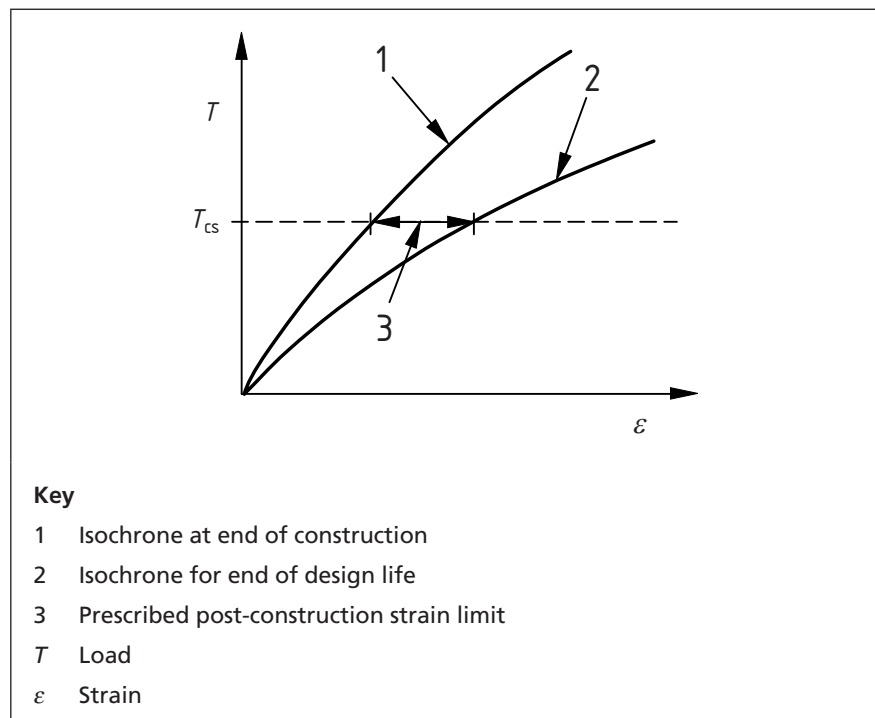
The following factors, which can influence serviceability should be included in the design check where appropriate:

- a) post-construction internal creep strain of polymeric reinforcements;
- b) post-construction internal creep strain of saturated fine grained soils used with reinforced soil.

For a polymeric reinforcement where the short term axial tensile stiffness decreases with time through the agency of creep, the strain occurring between the end of construction and the end of the selected design life may be estimated from isochronous load strain curves for these two times; Figure 43 demonstrates this procedure, where  $T_{CS}$  is the capacity of the reinforcement at a prescribed limiting value of post construction strain. To conform to the serviceability limit state the post-construction strain should not exceed the values given in Table 19.

For metallic reinforcements, or anchors, creep is negligible but for anchors it may be necessary to evaluate the creep of the anchor itself by reference to the foundation settlement theory for an elastic soil.

Figure 43 Assessment of serviceability limit state base strength



### 6.6.6 Segmental block walls

#### 6.6.6.1 Vertical spacing of soil reinforcement layers

The vertical spacing of soil reinforcements should not be too great and the interface shear connection between blocks should be adequate to prevent bulging due to excessive shear deformation or shear failure between successive courses of blocks (see Figure 44 and NCMA [39]).



Horizontal spacing of reinforcements should not be too great so that the segmental block wall bulges.

### 6.6.6.2 Un-reinforced face heights

The vertical spacing of soil reinforcements should not be too great:

- a) during structure erection before the next reinforcement is fixed; or
- b) at the top of the face above the last upper layer of reinforcement.

The blocks should be checked against overturning and sliding-off (see Figure 45 and NCMA [39]).

Figure 44 Check that facing does not bulge

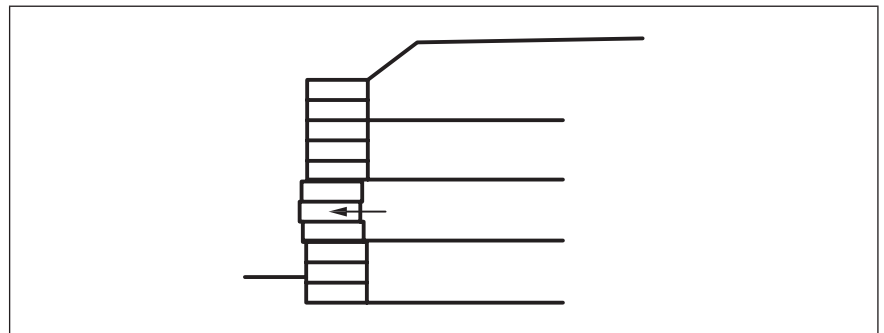
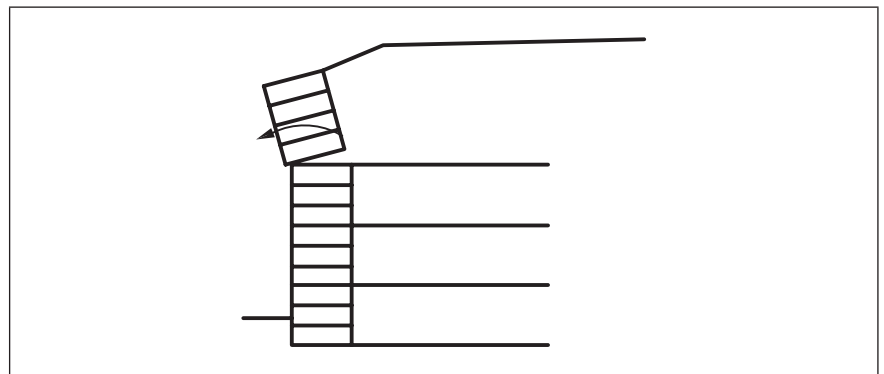


Figure 45 Check of unreinforced facing height



### 6.6.6.3 Shear strength of joints between blocks and between blocks and reinforcing elements

The structural integrity of the block facing should be achieved by the shear connection between successive courses, either by the friction between blocks and more commonly formed by shear keys, leading/trailing lips, pins/clips or similar. The connection between the soil reinforcements and the blocks may take various forms such as the friction between blocks and soil reinforcement and more commonly by pins/clips, shear connectors or keyways. The shear strength between blocks and connection strength between blocks and reinforcing element may be determined by calculation for metallic connections, and should be determined by full scale testing on other materials. The suitability of the connection type to the structure/application should be considered in the design of the

blockwall, i.e. some structures may be expected to experience more face movements than others (e.g. foundation settlements, seismic action, etc.) and in such cases a block type with a more positive connection to the reinforcement may be preferable.

## 6.7 Facings

### 6.7.1 General

It should be understood that in reinforced soil walls and abutments the primary load bearing function of the structure is provided by the interaction of the reinforcements and the soil.

A facing to the structure should:

- a) give external form to the structure;
- b) provide an aesthetically acceptable finish;
- c) prevent ravelling of the soil fill caused by weathering;
- d) provide local support to the soil between reinforcement layers;
- e) contributes to anchoring the reinforcement in the active zone.

The facing should be robust, durable and able to fulfil its function during the life of the structure, see Table 7.

### 6.7.2 Structural form

Facings may take a variety of forms dependent upon the function of the structure. They may be formed from concrete, timber, steel or polymers in the form of discrete panels, full height units or geotextiles. The form and construction requirement of a range of facings are considered in 6.10.

### 6.7.3 Structural loads on facing

Facings should be designed to accommodate the loads and effects resulting from:

- a) horizontal soil pressures and the corresponding reinforcement tension reactions developed in the connections between the facing and the reinforcement;
- b) forces arising from superimposed facing units;
- c) vertical shear forces developed as a result of relative movement between the facing and the fill together with any additional tensile reactions generated;
- d) any externally applied loads (temporary or permanent);
- e) possible differential longitudinal settlement (see 6.7.4).

#### COMMENTARY ON 6.7.3

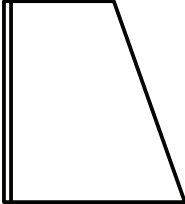
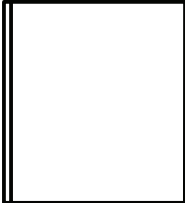
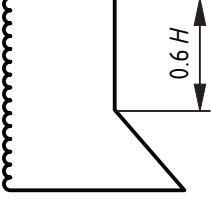
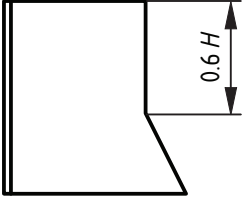
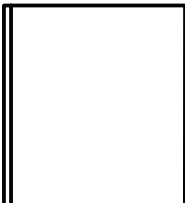
*Facings for reinforced soil walls and abutments are generally thin sections and are not intended nor can they be designed to resist direct impact loads due to collision by vehicles on their front faces. The facings are secondary components being only cladding units carrying local loads of backfill and might not carry the full force in the soil reinforcements. The impact on the front of the facing is resisted by the inertia of the mass of fill behind the facing, which is much greater than the impacting vehicle. Experience has shown that such impacts only cause minor surface damage*

to the facing, which may be repaired by local methods without removal of the face unit. If a collision impact requires a panel to be replaced then procedures are available to allow this to be done while the structure remains in service and without loss of fill at the panel location.

**6.7.4 Settlement of the facing and tolerances**

Guidance for the tolerance of facings to differential settlements and internal movements are given in 6.5.4.3, 6.5.5.2, and Tables 17 to 20.

Table 20 Connection loads for the ultimate and serviceability limit states

<b>Tie back wedge method</b>		
All facings with movement capacity or movement capacity at connections		$T_{conn} = 75\%T_j$
		$T_{conn} = 100\%T_j$
	Toe	
Stiff face, e.g. segmental block walls and full height panels with no movement capacity at connections		$T_{conn} = 100\%T_j$
	Toe	
<b>Coherent gravity method</b>		
Flexible face, e.g. metal U-shape elements		$T_{conn} = 75\%T_j$
		$T_{conn} = 100\%T_j$
	Toe	
Articulated face, e.g. discrete concrete panels		$T_{conn} = 85\%T_j$
		$T_{conn} = 100\%T_j$
	Toe	
Stiff face, e.g. segmental blockwalls and full height panels with no movement capacity at connections		$T_{conn} = 100\%T_j$
	Toe	

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## 6.8 Connections

### 6.8.1 General

The reinforcement of a structure whether in the form of geotextiles, geogrids, linear elements or anchors should normally be connected to the facing. In the case of geotextile reinforced walls or abutments the material may be selected to provide both the reinforcement and the facing.

### 6.8.2 Loads in connections

The load at the connection should be taken from Table 20.

### 6.8.3 Design of connections

In the case of metallic connections between the facing and metallic reinforcements, the connections should be designed in accordance with 6.8.4.

Metallic parts of a combined metallic/polymeric connection between the facing and reinforcements should be designed in accordance with 6.8.4 and the polymeric parts should be designed in accordance with 3.2.2 and confirmed by testing.

The connections between polymeric reinforcements and facings should be designed in accordance with 3.2.2 and confirmed by testing.

### 6.8.4 Metallic connections

#### 6.8.4.1 General

When calculating the load capacity of a steel metallic connection, allowance should be made for corrosion except where corrosion protection is assured for the full design life of the connection as follows.

- a) A sacrificial thickness in accordance with Table 4 should be deducted from each external surface of all component parts of the connection in contact with the soil.
- b) A sacrificial thickness of 0.5 times the value in Table 4 should be deducted from each internal surface of all component parts in close metal-to-metal contact or wholly enclosed within the connection.

All section properties for the component parts of the connection should be based upon the dimensions obtained after deducing the sacrificial thicknesses.

#### 6.8.4.2 Spacing of holes

##### 6.8.4.2.1 Minimum pitch

The distance between centres of fasteners or other components passing through the steel metallic member should not be less than 2.5 times the nominal diameter of the shank of the fastener or other component passing through the member.

**6.8.4.2.2 Maximum pitch**

The recommendations for maximum pitch are as follows.

- a) *In any direction.* Except as noted in b), the distance between centres of two adjacent fasteners should not exceed  $32t$  or 300 mm, whichever is the lesser, where  $t$  is nominal thickness of the thinner part joined.
- b) *In the direction of stress.* Except as noted in **6.8.4.2.3**, the distance between centres of two consecutive fasteners in a line lying in the direction of stress and sharing the applied load should not be greater than  $16t$  or 200 mm, whichever is the lesser, if the parts are joined in tension or shear.
- c) *Adjacent to an edge.* Except as noted in **6.8.4.2.3**, the distance between centres of two adjacent fasteners in a line adjacent to and parallel to an edge of an outside connected part should not be greater than  $(100 + 4t)$  or 200 mm, whichever is the lesser.

**6.8.4.2.3 Staggered spacing**

Where fasteners are staggered at equal intervals, and the gauge is not greater than 75 mm, the maximum distance between centres of fasteners as given in **6.8.4.2.2b)** and **6.8.4.2.2c)** should be increased by 50%.

The gauge being defined as the minimum lateral distance between centre lines of holes in adjacent rows.

**6.8.4.2.4 Edge and end distance**

The distance from the centre of a fastener to the edge of a part should not be less than  $1.2d$ , or such larger distance as may be needed to meet the provisions of **6.8.4.3.5**, where  $d$  is the nominal diameter of the fastener, or other component passing through the member.

**6.8.4.3 Strength of steel components in the connection****6.8.4.3.1 General**

Steel connections should be designed for long term rupture at the ultimate limit state.

**6.8.4.3.2 Strength of components subjected to axial tension**

In a component subjected to applied axial tension, the tensile stress  $\sigma_c$  should be taken as:

$$\sigma_c = \frac{T_c}{a_c} \leq \frac{\sigma_t}{f_m f_n}$$

where

- $T_c$  is the applied maximum tensile load on the component;
- $a_c$  is the tensile stress area calculated in accordance with **6.8.4**;
- $\sigma_t$  is the appropriate ultimate tensile strength from Table 3, Table 5 and Table 6;
- $f_m$  is the appropriate partial material factor calculated in accordance with **5.3.3** and Annex A;
- $f_n$  is the partial factor for economic ramifications of failure.

**6.8.4.3.3 Strength of components subjected to shear only**

In a component subjected to shear, the average shear stress  $\tau_c$  should be taken as:

$$\tau_c = \frac{V_c}{na_q} \leq \frac{\sigma_q}{f_m f_n}$$

where

- $V_c$  is the applied maximum load on the component;
- $a_q$  is the sectional area of the shear plane resisting the applied shear calculated in accordance with **6.8.4**;
- $n$  is the number of shear planes resisting the applied shear;
- $\sigma_q$  is the appropriate shear strength for the component given in Table 3, Table 5 and Table 6;
- $f_m$  is the appropriate partial material factor calculated in accordance with **5.3** and Annex A.

**6.8.4.3.4 Strength of components subjected to tension and shear**

Components subjected to combined tensile and shear forces should be in accordance with **6.8.4.3.2** and **6.8.4.3.3** and the tensile stress and the shear stress in combination should be such that:

$$\sqrt{\left(\frac{\sigma_c}{\sigma_t}\right)^2 + 2\left(\frac{\tau_c}{\sigma_q}\right)^2} \leq \frac{1}{f_m f_n}$$

where

- $\sigma_c$ ,  $\sigma_t$ ,  $\tau_c$  and  $\sigma_q$  are defined in **6.8.4.3.2** and **6.8.4.3.3**;
- $f_m$  is the appropriate partial material factor calculated in accordance with **5.3** and Annex A.

**6.8.4.3.5 Strength of components in bearing**

The maximum bearing pressure  $\sigma_b$  between connected parts:

$$\sigma_b = \frac{V_c}{a_{eb}} \leq \frac{k_b \sigma_{bc}}{f_m f_n}$$

where

- $V_c$  is the maximum load transmitted to each connected part;
- $a_{eb}$  is the area resisting the applied load, calculated in accordance with **6.8.4**;
- $k_b$  is 4.00 when the end distance  $\geq 3d_c$  and 1.92 when the end distance is  $1.2d_c$  (values for  $k_b$  for end distances between these values may be linearly interpolated);
- $d_c$  is the nominal diameter of the fastener passing through the member;
- $\sigma_{bc}$  is the appropriate bearing strength from Table 3, Table 5 and Table 6;
- $f_m$  is the appropriate partial material factor calculated in accordance with **5.3** and Annex A.

For components adjacent to an edge (in the direction of stress) where the edge distance is less than  $3d_c$  the reduced capacity applies only

to components adjacent to the edge. Subject to the provisions of 6.8.4.3.6 the total bearing capacity of the components should be the sum of the full bearing capacity of components away from the edge and the reduced capacity of those adjacent to the edge.

#### 6.8.4.3.6 Long connections

Where the distance  $L$  between centres of the end fasteners of a connection, measured in the direction of the load transmitted and sharing the applied load, is more than  $15d_c$  the strength of all the components determined in accordance with 6.8.4.3.2 to 6.8.4.3.5 should be reduced by a multiplying factor,  $k_r$ :

$$k_r = 1 - (L - 15d_c)/200, \text{ but } k_r \geq 0.75$$

### 6.8.4.4 Components in bending

#### 6.8.4.4.1 General

Connection components subjected to applied forces that result in bending stresses may be assumed to be fully restrained against lateral buckling when wholly enclosed by soil provided the soil is considered to provide full lateral restraint.

The section modulus and second moment of area for a component subjected to bending stresses should be calculated in accordance with 6.8.4 taking due account of any holes or other reductions in component size.

#### 6.8.4.4.2 Strength of components in uniaxial bending

In a component subjected to uniaxial bending the bending resistance  $M_d$  of the component should be calculated using:

$$M_d = \frac{\sigma_t Z}{f_m f_n} \geq 1.0$$

where

- $M$  is the maximum bending moment in the component;
- $\sigma$  is the appropriate tensile strength from Table 3, Table 5 and Table 6;
- $Z$  is the section modulus calculated in accordance with 6.8.4.4;
- $f_m$  is the appropriate partial material factor calculated in accordance;
- $f_n$  is the partial factor for economic ramifications of failure.

#### 6.8.4.4.3 Strength of components in biaxial bending

Where a component is subjected to bending about two axes:

$$\frac{M_{x, \max}}{M_{d,x}} + \frac{M_{y, \max}}{M_{d,y}} \geq 1.0$$

where

$M_x, M_y$  are the co-incident maximum bending moments about the X-X and Y-Y axes respectively;

$M_{d,x}, M_{d,y}$  are the corresponding bending resistances calculated in accordance with 6.8.4.4.2.

#### 6.8.4.5 Components in combined bending and axial tension

At any section, the following should be satisfied:

$$\frac{T_c}{a_c} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \leq \frac{\sigma_t}{f_m f_n}$$

where

$T_c$  and  $a_c$  are defined in **6.8.4.3.2**;

$M_x, M_y$  are the co-incident maximum bending moments about the X-X and Y-Y axes respectively;

$Z_x, Z_y$  are the section moduli of the component about the X-X and Y-Y axes calculated in accordance with **6.8.4.4**;

$\sigma_t$  is the appropriate tensile strength from Table 3, Table 5 and Table 6;

$f_m$  is the appropriate partial material factor calculated in accordance with **5.3** and Annex A.

#### 6.8.4.6 Components in combined bending and shear

At any section, the following may be assumed:

$$\sqrt{\sigma_{et}^2 + 3\tau^2} \leq \frac{\sigma_t}{f_m f_n}$$

or

$$\sqrt{\sigma_{ec}^2 + 3\tau^2} \leq \frac{\sigma_t}{f_m f_n}$$

where

$\sigma_{et}, \sigma_{ec}$  are the extreme fibre maximum bending stresses for tension and compression;

$\tau$  is the maximum co-existent shear stress;

$\sigma_t$  is the appropriate tensile strength from Table 3, Table 5 and Table 6;

$f_m$  is the appropriate partial material factor calculated in accordance with **5.3** and Annex A.

#### 6.8.4.7 Components in combined bending, bearing and shear

At any section, the following may be assumed;

$$\sqrt{\sigma_{et}^2 + \sigma_b^2 + \sigma_{et}\sigma_b + 3\tau^2} \leq \frac{\sigma_t}{f_m f_n}$$

or

$$\sqrt{\sigma_{ec}^2 + \sigma_b^2 + \sigma_{ec}\sigma_b + 3\tau^2} \leq \frac{\sigma_t}{f_m f_n}$$

where

$\sigma_{et}, \sigma_{ec}, \sigma_b$  and  $\tau$  are the co-existent maximum bending, bearing and shear stresses;

$\sigma_t$  is the appropriate tensile strength from Table 3, Table 5 and Table 6;

$f_m$  is the appropriate partial material factor calculated in accordance with **5.3** and Annex A;

$f_n$  is the partial factor for economic ramifications of failure.



#### 6.8.4.8 Determination of strength of steel components in the connection and load tests

The load capacity of components in connections may be determined by load testing and statistical analysis of an adequate number of samples.

### 6.9 Superimposed structures and loads for walls and abutments

#### 6.9.1 Superimposed structures such as bridge decks

Small horizontal and vertical movements of the fill and the facing should be anticipated during and after construction, see 6.5.4 and BS EN 14475:2006.

Superimposed structures should be relatively flexible and should be designed to accommodate such small movements. If the reinforced soil structure is built on poor foundation soil that is expected to experience significant settlement, the effect on the superimposed structure may be reduced by allowing sufficient time to elapse for the greater part of the total settlement to occur, and/or possibly by using a surcharge, prior to building part or whole of the superimposed structure (see Smith and Worrall [23]).

Vertical loads should be transferred directly to the reinforced fill. Base slabs of significant superimposed structures, such as bank seats for bridge decks, should not be fully or partly supported directly by the facing.

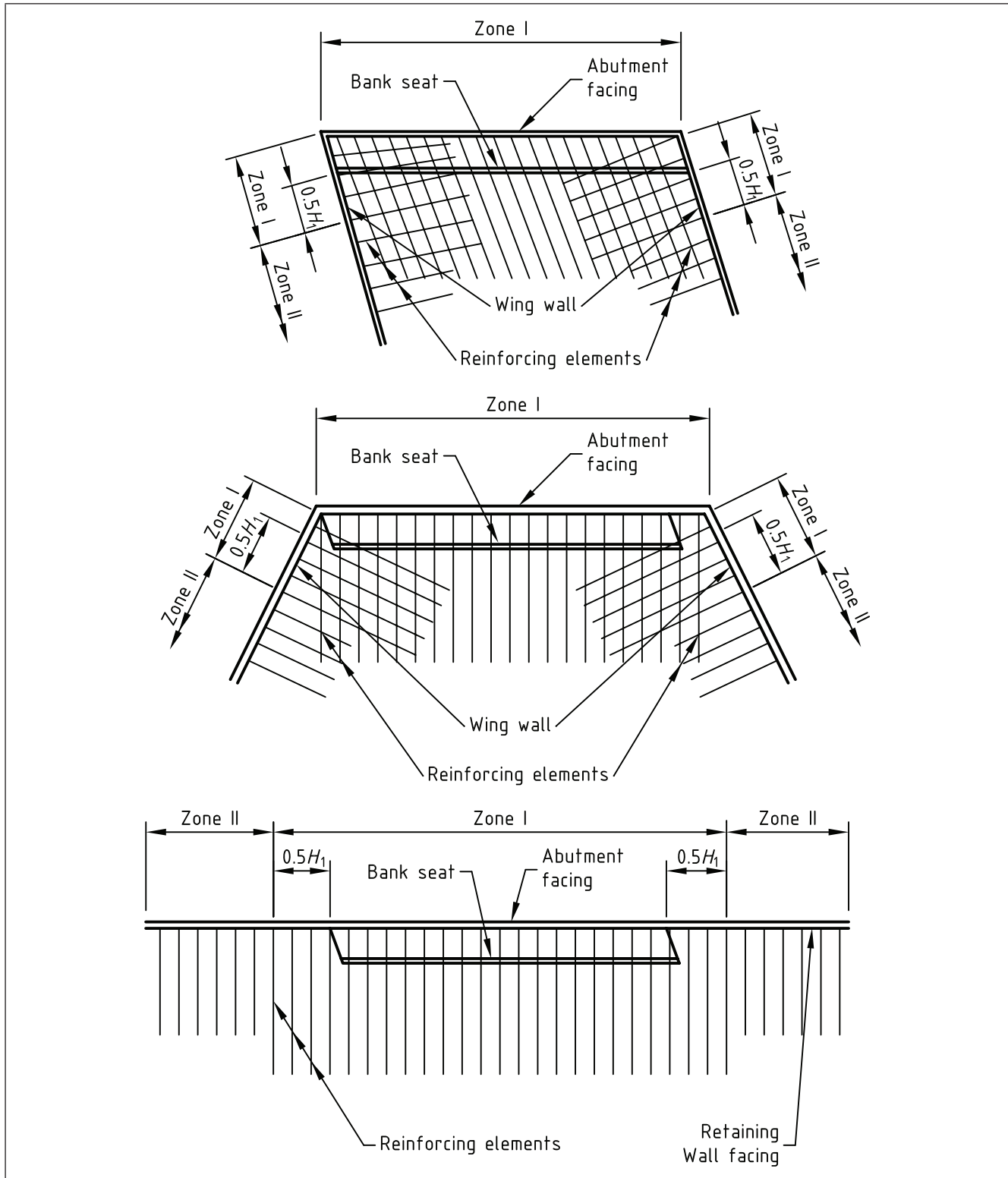
The resistance of the bank seat of a superimposed structure to horizontal loads should not be increased by attaching reinforcing elements as some movement of the bank seat is required before loads are transmitted to the reinforcing elements. Base slabs should not be attached to friction slabs or a system of ties with a ground beam or anchor blocks behind the reinforced zone, except where methods of analysis have been subject to specific study and client approval.

When a reinforced soil wall is used as a bridge abutment, then for simplicity the design of abutments may be considered in two parts, as shown in Figure 46. Zone I should be designed assuming loading includes that of the bank seat and its applied loads. Zone II should be designed as a retaining wall ignoring any loading derived from the bank seat. The load from the bank seat should be assumed to diffuse downwards at  $2v:1h$  so that the width of Zone I increases linearly from bank seat level to be  $0.5H_1$  greater in width at foundation level.

The construction sequence for the construction of a bridge abutment should be carefully considered: a bridge abutment is built in several stages, including erection of the reinforced soil mass, followed by the construction of the bankseat, and then the installation of the deck. The reinforced soil design should take account of the weight of the bankseat, the dead weight of the deck and the live loads acting on the deck. However, there might be other load cases that also have to be taken into account such as the reinforced soil mass carrying the weight of the bankseat and perhaps with the soil above the reinforced soil mass not yet up to final road level, which will not provide the same restoring overburden over the soil reinforcements for their pull-out resistance as in the final load case. The deck is likely to be installed before the road pavement layers are completed and, again, the overburden over the soil reinforcements will be less than

in the final load case. The design should consider all the possible load cases and state clearly on the working drawings the arrangement and level of fill to be in place above the reinforced soil mass before construction of the bankseat and also before installation of the deck (see Figure 48). For these construction load cases, the strength of the soil reinforcement may be taken as that appropriate for the service life of temporary works as given in Table 7.

Figure 46 Bridge abutments – Typical layout plans for strengthening elements



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### 6.9.2 Design of base slabs supporting vehicle parapets

The design of base slabs supporting vehicle parapets should be in accordance with Annex E unless the parapet supporting system is incorporated in a current third-party certificate<sup>3)</sup>. A contribution to base sliding resistance may be provided by attaching reinforcing elements to the base slab but care should be taken to ensure that these are effective.

## 6.10 Construction and maintenance of walls and abutments

### COMMENTARY ON 6.10

*Factors affecting the performance of reinforced soil walls and abutments are detailed in Table 1.*

*Factors that affect the construction of reinforced soil walls and abutments include (see Moulton et al [24]):*

- *foundation soils;*
- *fill material;*
- *reinforcement;*
- *facings;*
- *connections;*
- *drainage;*
- *site constraints;*
- *end use;*
- *erection rate.*

*BS EN 14475:2006, Table 2 illustrates some possible aspects of design output.*

### 6.10.1 Foundations

The foundation of the wall or abutment is the total width of the surface prepared to accept the length of the lowest layer of reinforcement.

The depth of the foundation below the finished ground level at the foot of the wall should conform to **6.4.2**.

Where the foundation is placed on natural ground it should be given several passes of a dead weight roller before the placing of any fill material. Soft spots should be removed and replaced with well graded granular fill. An additional trench excavation should normally be provided at foundation level for a mass concrete levelling pad, either precast or formed in situ, beneath hard facings in order to facilitate erection. (For segmental block walls, see **6.10.4.4**.)

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<sup>3)</sup> Third-party certification is accredited by UKAS ([www.ukas.com](http://www.ukas.com)) in the UK and members of the IAF ([www.iaf.nu](http://www.iaf.nu)) in the rest of the world. For example, BBA and BRE are UKAS accredited.

## 6.10.2 Fill material

### 6.10.2.1 General

Reinforced soil walls and abutments are normally designed to use fill material that will allow for easy and quick erection. This fill should conform to 3.1.

*NOTE Further guidance is given in BS EN 14475:2006, 6.2.*

### 6.10.2.2 Placing and compaction of fill

The fill should be deposited, spread, levelled and compacted in horizontal layers of appropriate thickness as described in the Clause 622 within the *Specification for Highway Works* [1]

*NOTE Further guidance is given in BS EN 14475:2006, 8.5.4.*

### 6.10.2.3 Quality control of fill during construction

The selection, placing and compaction of the fill should conform to the general requirements of the *Specification for Highway Works* [1] and BS EN 14475, 8.5.4.

### 6.10.2.4 Restrictions on fill

#### COMMENTARY ON 6.10.2.4

*Fills that might be suitable for embankment construction and reinforced slopes might not be suitable for use in reinforced soil walls e.g cohesive soils.*

Frictional or cohesive frictional fills may be used in reinforced soil walls and abutments; these soils are easy to compact and are also relatively free draining avoiding the need for special drainage layers.

For land based structures the fill should conform to 3.1. In cases where the fines content is greater than 10% the effects of pore water pressure both during construction and during the service life of the structure should be considered.

*NOTE If significant pore pressures are created during compaction the soil tends to flow, and in reinforced soil this can lead to face deformations.*

Very fine soils should not be used for the construction of walls in marine or river environments unless special precautions are taken and unless permanent drainage layers are provided. Construction under water using hydraulic fill may be used but in these cases the structure should incorporate special measures to allow for the settlement of the fill without compaction (see Wu and Smith [27])

### 6.10.2.5 Use of chalk fill

Chalk having an intact lump dry density,  $IDD > 1.55 \text{ Mg/m}^3$  (i.e. medium and/or high density chalk) and natural moisture content of up to 25% may be allowed for use as fill. Soft chalk ( $IDD < 1.55 \text{ Mg/m}^3$ ) should not be used as fill for permanent works. Chalk of adequate quality may be used in reinforced soil structures but a similar degree of care, as is necessary for general chalk earthworks, should be exercised.

*NOTE The quality of chalk specified is that which was used in a full scale trial of walls at Paulsgrove [40].*

The chalk as delivered should not contain lumps greater than 600 mm and should be deposited and compacted by bulldozer and smooth

dead weight roller to ensure that no particles larger than 125 mm remain within the body of the fill. Any large lumps of chalk within 2.0 m of the face of a structure should be broken down with a power rammer or pulled back into the body of the structure and crushed. The compaction plant should be chosen to suit the grade of chalk.

The fill within 2.0 m of the face of the wall should be compacted in accordance with 6.10.2.2. Compaction of chalk should not take place when it is in such a condition that it turns to a slurry. It is recommended that the method of working should be approved with a trial before work proceeds (see Griffiths [40]).

A 300 mm wide granular drainage and frost blanket or similar measure should be used against the back face of any facing units. Where appropriate this material should be specified as frictional fill in accordance with 3.1. A geotextile wrapping or fin drain may also be adopted.

Although the recommended limit of IDD and moisture content is given above, special care should be taken in placing and compacting chalks that approach these limiting values. In particular it is necessary to ensure that:

- a) the facing panels are not displaced by heavy compaction plant operating close to the back of the panels; and
- b) the compaction plant can effectively break down large fragments to achieve the recommended compaction standard; the compaction plant necessary to achieve this objective is dependent on the properties of the chalk but the provision of a power rammer is recommended. (Further guidance is given within Clause 622 in the *Specification for Highway Works* [1].)

### 6.10.2.6 Use of other materials

#### 6.10.2.6.1 General

Other fills including argillaceous materials, pulverized-fuel ash and colliery spoil may be used in accordance with 3.1.

It should be noted that the suitability of these materials depends upon their frictional strength and on their aggressiveness to the intended reinforcement, facing and connections.

Their suitability should be determined beforehand by tests on a representative number of samples. To ensure quality during construction frequent site checks on both strength and the chemical properties should be carried out.

#### 6.10.2.6.2 Argillaceous materials

Argillaceous material (e.g. shales, mudstones) used in reinforced soil applications should be carefully selected Rainbow [15]). Particular care should be taken in assessing the chemical characteristics of these materials, and their variability, to ensure compatibility with the reinforcement (see 3.1.2). Many argillaceous materials can be friable in nature and in such circumstances they should not be used (see 3.1.3.5).

#### 6.10.2.6.3 Pulverized-fuel ash

Pulverized-fuel ash, used as fill in reinforced soil in accordance with 3.1.3.3 is the resultant ash from pulverized coal burnt in power

stations. The material used as fill in any structure should be obtained from one source and should have a maximum particle size of 3 mm.

Pulverized-fuel ash should be compacted by vibrating rollers at a moisture content not exceeding optimum.

*NOTE Some pulverized-fuel ash fills might not conform to the electrochemical limits of fill in respect of metallic reinforcement.*

Only non-metallic reinforcing elements, e.g. polymeric reinforcement, should be used with pulverized-fuel ash fill.

A layer of frictional fill not less than 500 mm thick should be placed on top of the PFA and below the road formation level. This layer should connect with the vertical drainage layer described in 6.10.5.2.

A 300 mm wide granular drainage blanket complying with SHW [1] Clause 622.5 (ii) should be used against the facing units. Reinforcement connections buried within the drainage blanket should be considered in accordance with 3.2.1 and Table 3.

#### 6.10.2.6.4 Colliery spoil

Generally material from a spoil heap should be preferred as this will have undergone a degree of physical and chemical weathering both during placing and whilst in place on the tip. Material taken direct from a mine or coal preparation plant may be used but as the material may vary a high level of quality control should be undertaken to ensure the materials characteristics remain consistent during use (see 3.1.1).

Minestone within a spoil heap is generally well graded and at optimum moisture content for compaction and selected as-dug material may be used as fill. Compaction should be achieved using vibrating smooth wheeled rollers.

### 6.10.3 Reinforcement elements

Metallic reinforcement should be prefabricated and delivered to site ready for installation into the structure. Stiff polymeric reinforcement should be prefabricated and delivered to site ready for installation into the structure. Flexible reinforcement including geotextile sheets, meshes, grids and strips should be prefabricated and delivered to site in secure rolls, if appropriate.

*NOTE Further guidance is given in BS EN 14475:2006, 6.3.*

### 6.10.4 Facing

#### 6.10.4.1 General

Only hard or flexible/deformable units should be used for walls.

##### COMMENTARY ON 6.10.4.1

*The visual appearance of a structure is affected by the final shape of the facing. The surface can vary from the theoretical plane due to several causes including the following.*

- a) Poor workmanship. *The effect of poor workmanship will be evident as construction proceeds and may be noticed by the erratic inclination of the facing. Face construction should be supervised and checked for alignment as work progresses.*
- b) Extension of the reinforcements during and immediately after construction. *Extension of the reinforcements under load can give*

*rise to wall face deformation, the amount of which will be dependent upon the axial stiffness of the reinforcement and the extent of the restraint mobilized in the soil as the composite soil/reinforcement system is put under the load.*

- c) Creep in reinforcement. *These are limited to the serviceability limits specified in 6.5.5.2 and Table 19.*
- d) Creep in fine grained soils. *This can occur due to high moisture content (see 6.5.5.1).*

*Further guidance is given in BS EN 14475:2006 6.4 and Annex B.*

#### 6.10.4.2 Hard facing

##### COMMENTARY ON 6.10.4.2

*Hard facing units are usually produced in precast concrete. They can be full height panels, partial height panels, sloping panels, planter units or segmental blocks.*

*Examples are shown in Figure 16a), Figure 16b) and Figure 16d).*

Full height panels should be propped during the filling operation and until the connected reinforcements support the facing. The toe of the wall facing should be restrained to prevent forward movement before the bottom layer of reinforcement is able to act; an estimate of prop force is needed in order to size up the prop and its foundation.

#### 6.10.4.3 Flexible facing

##### COMMENTARY ON 6.10.4.3

*Flexible or deformable facings are as shown in Figure 16c).*

*They are formed of metal or polymeric material such as steel welded wire mesh, gabion baskets or tyres, or wrap-around polymeric construction.*

*In all reinforced soil applications there is a small compression of the fill during erection. In the case of discrete panels, the movement is accommodated by the use of compressible joints, with flexible metal facing the curved cross section of the facing unit flexes and with soft facings the face distorts.*

*In the case of full height facing panels of any significant height any relative displacement between fill (and the embedded reinforcing element) and facing may be accommodated by permitting the reinforcements to slide or move relative to the facing; various methods have been used including the use of grooves, slots, vertical poles, lugs or bolts. A sliding connection will permit the reinforcement to transmit horizontal load and yet slide downwards as filling progresses without loss or gain in load. In the case of relatively short full height panels, reinforcements such as geotextiles may be fixed into the facing panel. With this arrangement, deformation in the region of the face connections can occur. An assessment of the additional load imposed by this deformation may be included in the design load to be carried.*

#### 6.10.4.4 Segmental block wall facing

To aid alignment of the facing, the blocks should be laid on a leveling pad, which is usually concrete, although granular fill may also be used; the leveling pad is not a foundation but a means of providing a flat level surface for the first course of blocks.

If concrete is used it is usually mass concrete of low strength and about 150 mm thick and extending beyond the front and rear faces of the blocks; the blocks may be laid on a mortar bed on the concrete leveling pad.



A vertical layer of granular drainage fill, typically class 6H, with a width of 300 mm should be used behind the block facings. This layer may prevent the loss of fines from the structural fill through the joints between the blocks and also reduces the risk of block displacement during compaction of the fill. The drain may also channel any water seepage to the toe of the wall where it may exit through the normal dry joints between the blocks to the front of the wall where a longitudinal drain may be laid, see 6.10.5.3.

Traditional earth retaining walls in reinforced concrete or masonry should be built with a slight batter to off-set the slight forward rotation as backfill is placed against their stem and causing the stem to move towards vertical; if such a wall was built with a vertical stem its face would move forward of vertical and look over-hanging and have a disturbing visual effect.

The discrete panel face units of reinforced soil walls, having facings which are designed to be vertical upon completion of construction, should be placed in position in the wall and inclined inwards slightly and allowed to rotate out to vertical as the backfill is placed against them, as the load is taken up by the soil reinforcement.

It should be noted that segmental blockwalls are similar to traditional masonry walls and so their faces are usually battered back by one degree or more by either the front face of the block being battered or by setting the front of a block slightly inwards compared with the course below – the amount is called the set-back. It is common for nibs and lips formed on the upper and lower faces of the blocks to automatically form the set-back.

### 6.10.5 Drainage

#### COMMENTARY ON 6.10.5

*In reinforced soil structures drainage is an important consideration. If the structure is allowed to become waterlogged the tensile forces in the reinforcing elements increase and the properties of the fill and retained ground can change. The force on the wall can increase and any pore water pressures can reduce the effective overburden pressure on the reinforcements thus reducing pull-out capacity.*

*Water can enter a structure in two ways.*

*Water can percolate from the upper surface unless effective sealing details are provided.*

*Ground water can flow into the structure from the retained ground. This is usually only significant in cases of structures supporting roads or railways on side-long ground where water can emerge from the cutting at the uphill side.*

*Consideration has to be given to drainage during construction.*

*Further guidance is given in BS EN 14475:2006, 8.4.*

#### 6.10.5.1 Drainage at the top of a wall

For walls supporting roads, the use of a sealed kerb and drainage channel at the back of the hard shoulder should normally be sufficient. Where there is no hard shoulder, a channel with flexibly sealed joints should be provided at the back of the hard strip/edge of carriageway [see Figure 47a)]. In addition, for part-height walls a drainage system should be provided at the top of the facing behind the panel top or coping, if used, in order to remove water running



off the side slope [see Figure 47b)]. This may consist of a simple drain channel leading surface water along the wall top to discharge beyond the end of the wall.

For all structures, details should be used to avoid significant water penetration from the upper surface and means of collecting and leading away rain water should be provided. Abutment bank seats should include means of collecting any seepage from a faulty joint between the curtain wall and the deck (see Figure 48).

Figure 47 Reinforced soil retaining walls

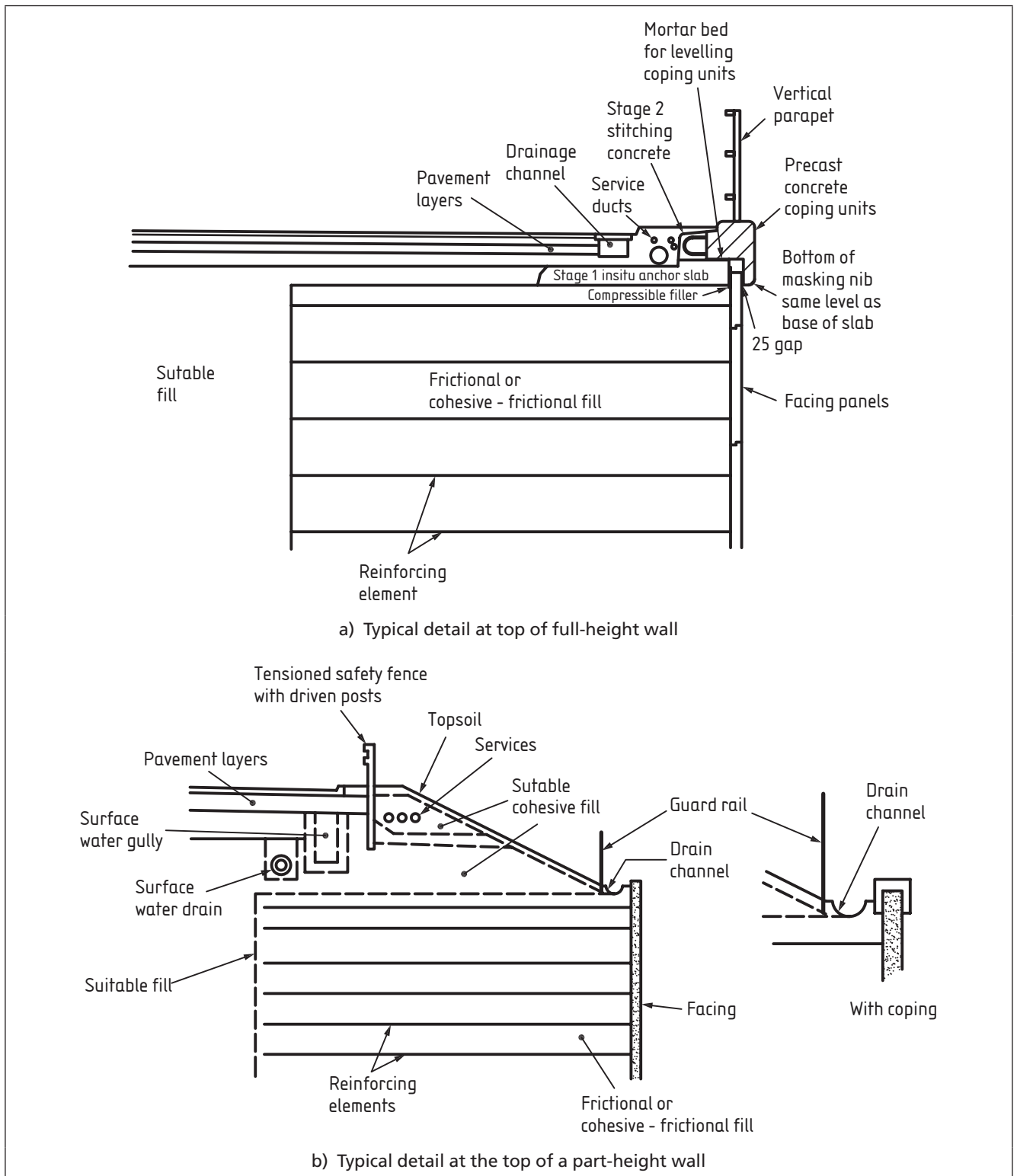
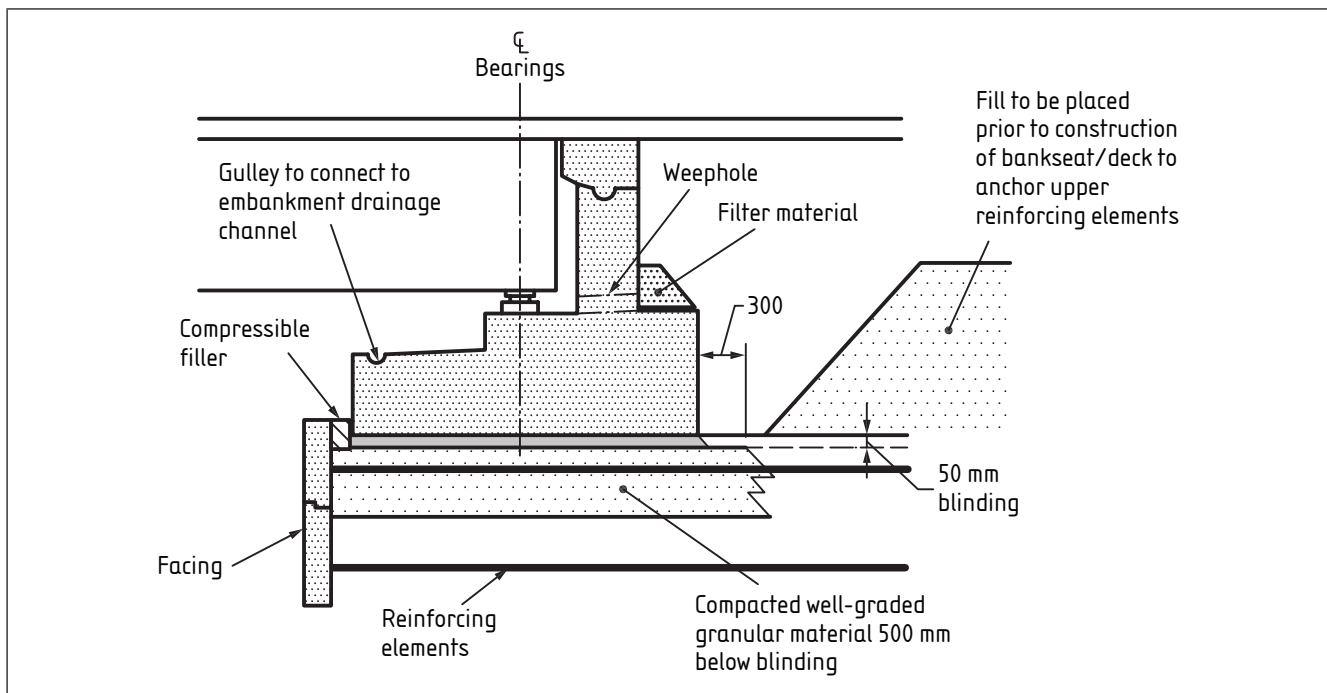


Figure 48 Typical drainage detail for abutment bankseat



### 6.10.5.2 Drainage of the wall

#### COMMENTARY ON 6.10.5.2

The normal considerations of drainage of conventional structures apply equally to reinforced soils. However due to the design considerations requiring the reinforced fill to have frictional properties means that it is also relatively permeable compared to fill retained behind conventional structures. Even where the reinforced fill is at the fine end of the specified range it will be relatively permeable and will have a large width (usually at least  $0.7H$ ).

In many circumstances this reinforced mass is effective as a drain without the use of other means, see Figure 49.

If the structure is located on a permeable foundation soil above the water table any small water seepage will pass into the foundation soils and a drain layer/pipe should not be necessary.

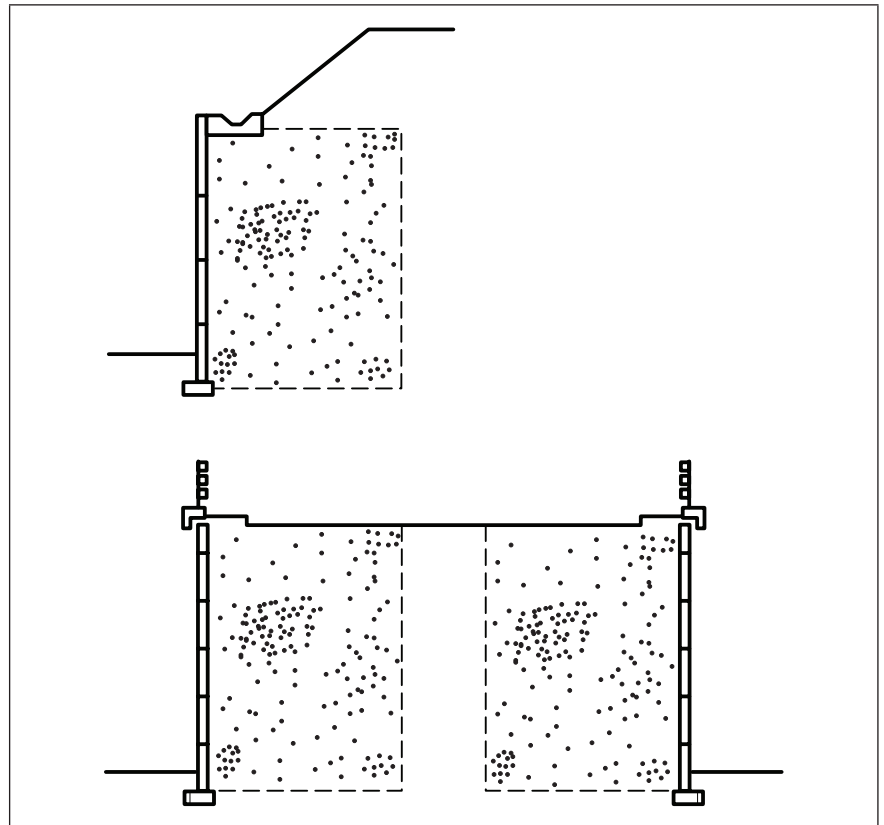
In other situations a longitudinal porous or open jointed pipe of not less than 150 mm diameter should be used at the front toe of the structure to collect water and bring it into the site drainage system. This pipe should be laid in front of the face panel where it will be accessible for future maintenance. To enable any seepage to pass through a hard facing, weep holes may be located in selected panels. For discrete facings the drain path may be more easily provided by omission of the vertical joint filler between all panels at the foot of the wall in the embedded depth, see Figure 50.

**NOTE** A pipe located in front of the facing allows reinforced soil construction to commence without the interruption of drain laying amongst the reinforcements. If the pipe is laid behind the panels there can be difficulty providing adequate falls due to the adjacent reinforcements. Access is also more difficult and substantial facilities for rodding the pipe are necessary.

A continuous drain at the base of the structure may be required in situations where capillary rise of deleterious ground water might need to be prevented. The layer should connect with the drainage system at the base of the structure.

A horizontal drainage layer 450 mm thick should be placed beneath the PFA in conformance with SHW [1] Clause 622.5 (ii). This layer should connect with the drainage system at the base of the structure. This horizontal drainage layer may be omitted if the underlying soil is sufficiently permeable to ensure the fill is adequately drained.

Figure 49 Reinforced soil mass acting as drain



### 6.10.5.3 Drainage of walls supporting cuttings

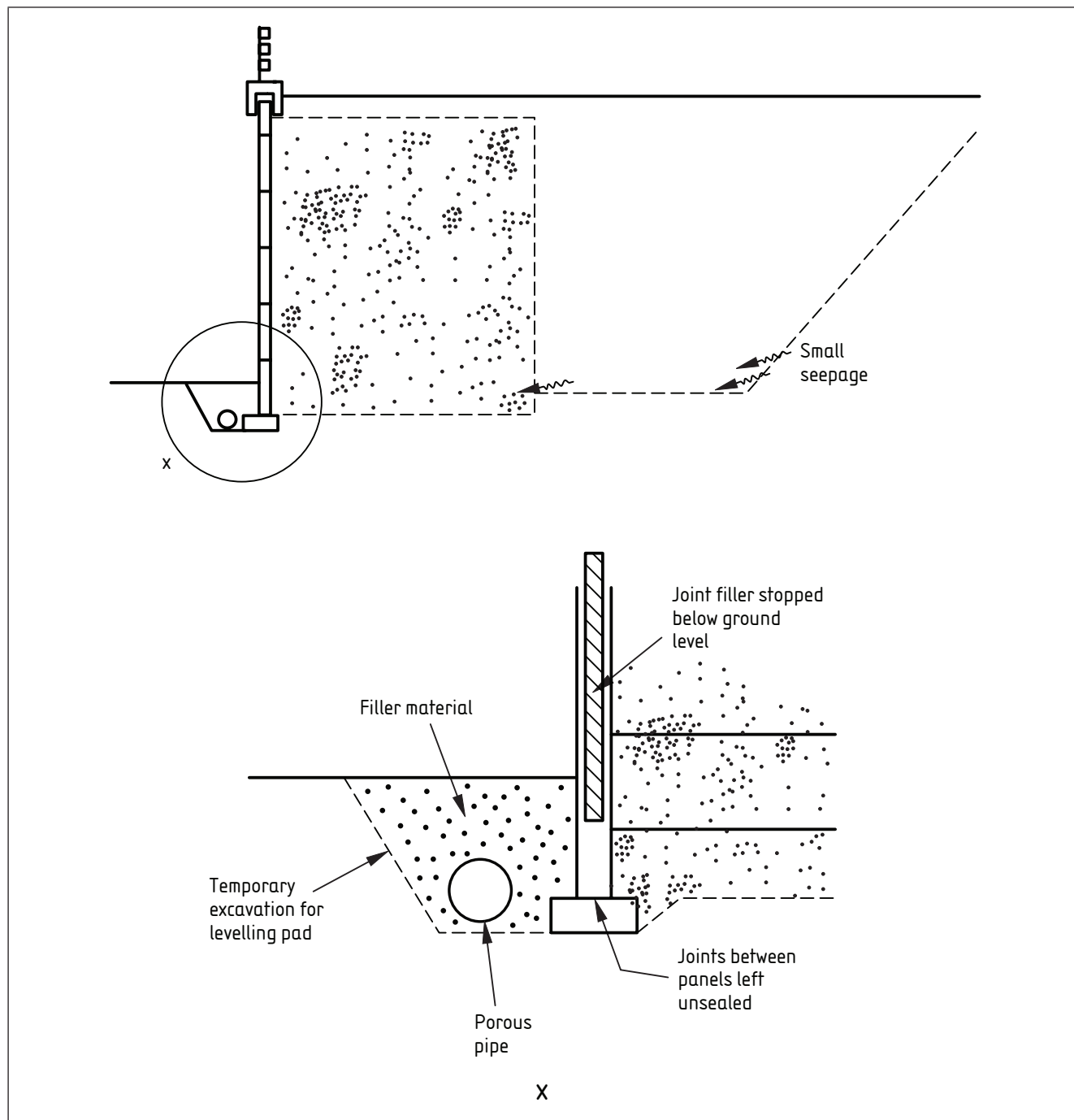
For locations where water flow is expected from the retained soil, drainage trenches typically 300 mm thick and 1 000 mm wide, should be placed at intervals along the wall [see Figure 51a)].

In cases of significant water flows, a drainage blanket typically 300 mm thick may be constructed below the reinforced soil wall and discharged beyond the toe. If necessary this blanket may be continued up along the face of the temporary excavation for as high as is needed [see Figure 51b)].

The details shown in Figure 51 may be varied to suit the conditions met during construction without change to the reinforced soil details. For cases where downhill discharge is not possible a toe collector pipe may be used. The dimensions of the drainage trenches and blanket should be designed to suit the anticipated conditions.

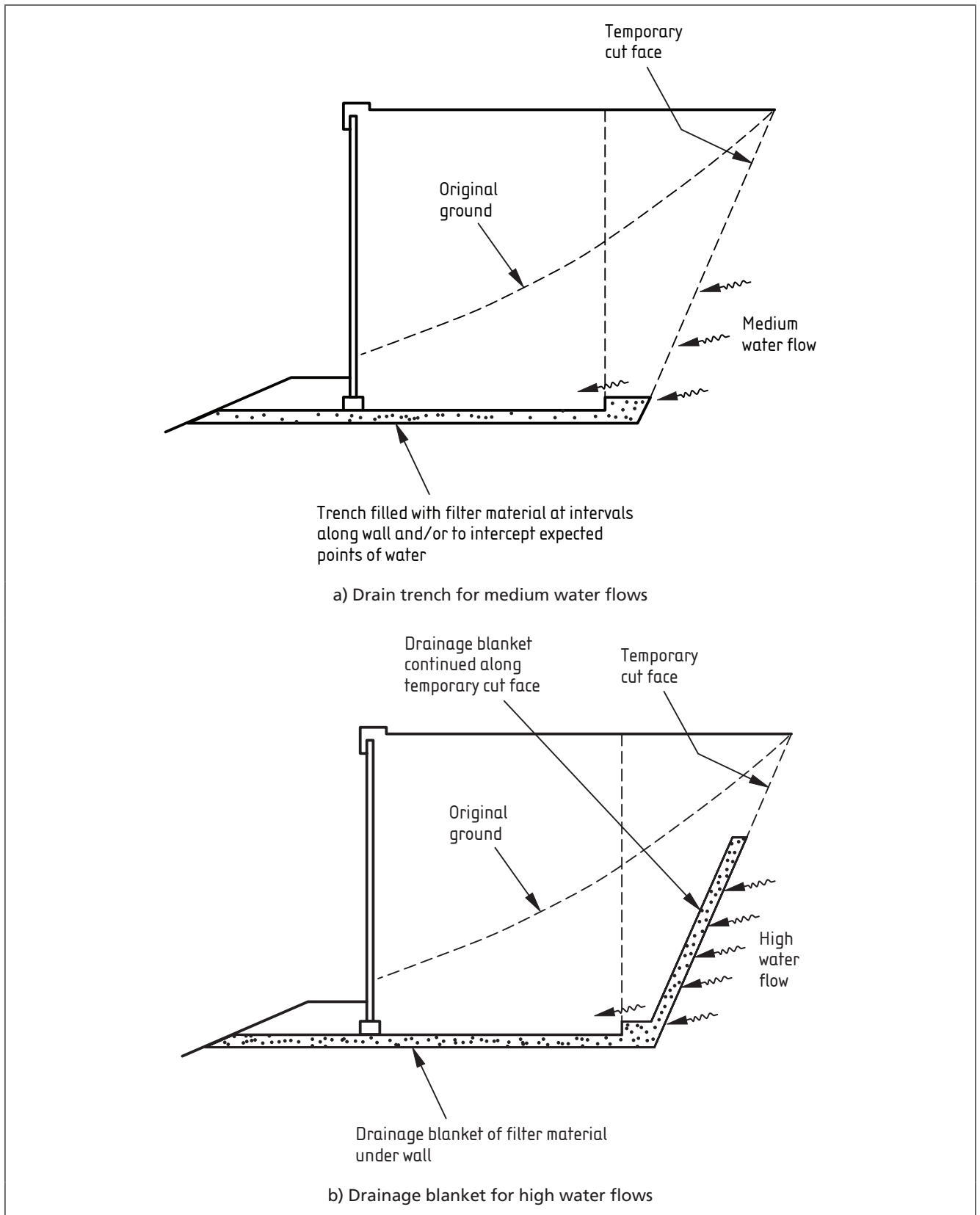
In all cases the under-structure drainage filter material should be designed to avoid loss of reinforced fill or adjacent soil into the drain.

Figure 50 Porous pipe at wall face



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Figure 51 Drainage details for walls supporting cuttings



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### 6.10.6 Services

It should be noted that, owing to the multiplicity of services and their possible configurations it is not possible to give more than some general guidance for design.

The possible effects arising from the presence of services in, on, over or in the vicinity of a permanent structure should be examined. In addition, the installation, maintenance and removal of services under normal and failure conditions should be considered. The appropriate authorities should be consulted at the design stage.

In full height walls, services should be carried in ducts or in a service bay above the reinforced zone, preferably incorporated in the parapet base detail, if applicable. In part height walls, services should preferably be carried in the embankment above the reinforced soil structure.

If possible, water mains should be located clear of permanent structures. Where this is not possible, every effort should be made to avoid the potentially disastrous effects of a burst water main. Suitable expedients, e.g. sleeving, should be agreed with the appropriate authority at the design stage.

### 6.10.7 Maintenance

All reinforced soil structures should be subjected to a regular programme of inspection and maintenance and records of the inspections and any maintenance carried out should be kept.

*NOTE* Guidance on the frequency and purpose of inspections and the form of records to be kept is contained in *The Inspection Manual for Highway Structures* [34].

It is particularly important that reinforced soil structures should be inspected for indications of:

- a) excessive settlements, either even or differential;
- b) horizontal displacements of the facings;
- c) damage to the facings;
- d) evidence of drainage problems in, around or under the reinforced soil mass;
- e) corrosion of metallic elements (rust staining);
- f) opening of facing joints or joints between one structure and another;
- g) cracking within the earthworks adjacent to the structure.

## Section 7: Reinforced slopes

### 7.1 General

The reinforcement of slopes may be undertaken for a number of applications including:

- reinforcement of fill in new construction [see Figure 52a)];
- reinforcement of failed slopes [see Figure 52b)].

Slopes with inclinations less than vertical should be designed in accordance with the recommendations of this section; guidance is also provided. Slopes with face angles within 20° of the vertical may be designed in accordance with the procedures in Section 6 if desired.

*NOTE* The applications illustrated in Figure 52 require different approaches and will be dealt with separately in the following clauses.

The density of reinforcement in a reinforced soil structure with a steep face will generally result in a stiff reinforced structure and hence the soil pressures acting on the reinforced block should be taken into account; as the angle of the face declines from the vertical the influence of the retained soil reduces and the proportion of the stability provided by the reinforcement decreases.

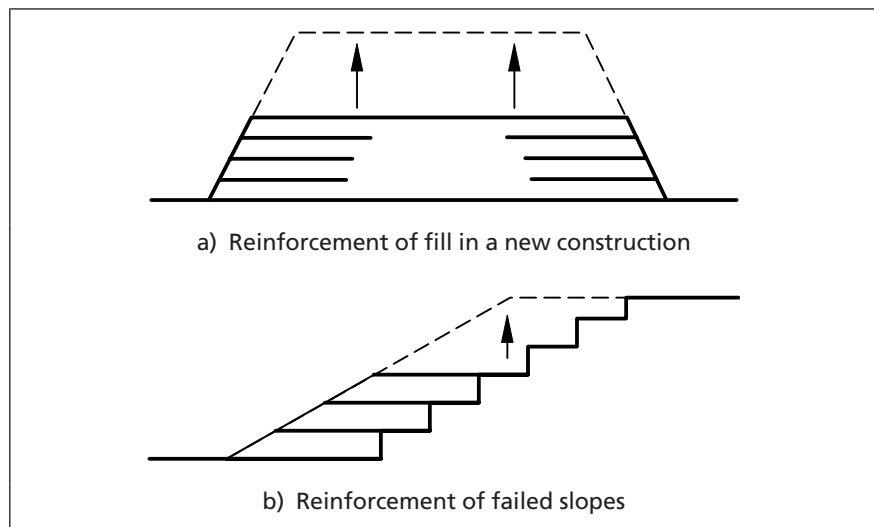
It should be noted that the design methods for limit state are derived from limit equilibrium methods and the most common methods are described. Other limit equilibrium methods that are valid for unreinforced slopes may also be applicable to a reinforced slope although the assumptions made in the analysis should be carefully evaluated to confirm their suitability.

The angle of the slope will have some influence on the method of analysis that may be employed, but most importantly will determine the type of facing that should be employed and the method of construction that should be used (see 7.5.3). A distinction should therefore be made between "steep slopes" (slope angles greater than 45° to the horizontal), and "shallow slopes" (slope angles less than or equal to 45° to the horizontal). Some form of facing should generally be provided for steep slopes to enable anchorage of the reinforcement in the active zone and to provide erosion protection.

For shallow slopes it may usually be possible to establish vegetation for long term erosion protection. Some suitable fills have adequate stability at 45° providing resistance against shallow slips thus structural facings may not be needed.

Lighter intermediate layers of reinforcement may also be used to ensure the face stability of less competent fills. Therefore a face slope may be placed, compacted and trim filled to 45° without permanent or temporary support of the face, although face-erosion matting or topsoil confinement system should be considered.

Figure 52 Examples of slope reinforcement



## 7.2 Partial factors used in the design of reinforced slopes

### 7.2.1 General

It should be noted that, in limit state design the philosophy for reinforced slopes involves increasing the soil weight and external loading by the appropriate partial load factors and reducing the soil properties and reinforcement base strength by appropriate partial material factors.

The design principles established in Section 2 should be used as a basis for the procedures contained in Section 7. Table 21 contains the partial factors appropriate to this section.

Table 21 Summary of partial factors to be used in Section 7

Partial factors		Ultimate limit state	Serviceability limit state
Load factors	Soil unit weight, e.g. slope fill,	$f_{fs} = 1.5$	$f_s = 1.0$
	External dead loads, e.g. line or point loads	$f_f = 1.2$	$f_f = 1.0$
	External live loads, e.g. traffic loading	$f_q = 1.3$	$f_q = 1.0$
Soil material factors	To be applied $\tan \phi'_p$	$f_{ms} = 1.0$	$f_{ms} = 1.0$
	To be applied to $c'$	$f_{ms} = 1.6$	$f_{ms} = 1.0$
Reinforcement material factor	To be applied to the reinforcement base strength	The value of $f_m$ should be consistent with the type of reinforcement to be used and the design life over which the reinforcement is required (see 5.3 and Annex A)	
Soil/reinforcement interaction factors	Sliding across surface of reinforcement	$f_s = 1.3$	$f_s = 1.0$
	Pull-out resistance of reinforcement	$f_p = 1.3$	$f_p = 1.0$
Partial factors of safety	Sliding along base of structure where there is soil-to-soil contact	$f_s = 1.2$	NA



### 7.2.2 Load factors

The soil unit weight to which the partial load factor is ascribed should be the characteristic value (see 3.5), and should take into account variations in specific gravity, grading and compaction. The external loads to which the partial load factors are ascribed should be the characteristic values in their original unfactored state.

### 7.2.3 Materials factors

The soil material factors relating to the peak values of  $\phi'_p$  and  $c'$  should have values in accordance with those in Table 21.

The reinforcement material factor should be assessed in accordance with the procedures described in 5.3.3 and Annex A, taking due regard of the type of reinforcement and the selected design life.

### 7.2.4 Soil/reinforcement interaction factors

In reinforced soil slopes there are two main interfaces where the soil–reinforcement interaction properties should be characterized:

- soil sliding across the surface of the reinforcement;
- pull-out of the reinforcement from the resistant or active zone.

### 7.2.5 Economic ramifications of failure

The partial factor for economic ramifications of failure  $f_n$  should be applied to the reinforcement design strength in accordance with Table 9.

### 7.2.6 Pore pressures

In situations where the pore pressure coefficient  $r_u$  is greater than 0.15, or where there is a specific phreatic surface within the reinforced soil block, the analysis should be carried out using conventional limit equilibrium methods satisfying a minimum safety factor of 1.3.

## 7.3 Areas of application

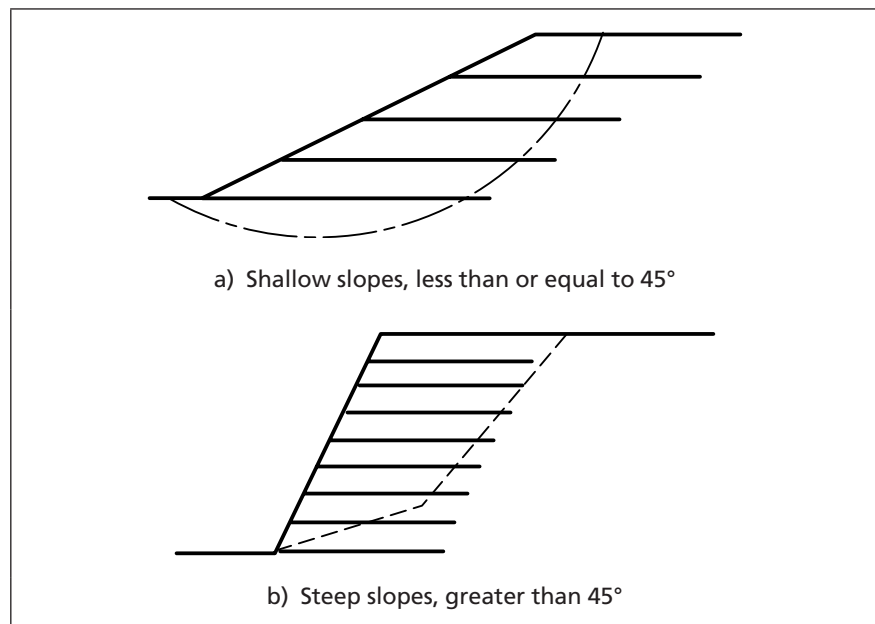
The method of constructing reinforced soil fill slopes should involve the reinforcement of fill materials, new or excavated and replaced, by reinforcement which is placed horizontally within the compacted layers of fill [see Figure 53a) and Figure 53b)].

## 7.4 Reinforcement of fill materials

### 7.4.1 Basis for design

The recommendations for stability of reinforced slopes may have much in common with those for reinforced soil walls described in Section 6, depending on the angle of the face. However as the slope changes the relative importance of the external and internal stability can change and the critical aspects of a design may tend towards internal modes of failure (see Figure 53). Nevertheless, both internal and external stability should still be checked.

Figure 53 Design basis for reinforced slopes



### 7.4.2 Limit states

The ultimate limit states that should be considered are as follows:

- a) external stability:
  - 1) bearing and tilt failure [see Figure 54a)];
  - 2) forward sliding [see Figure 54b)];
  - 3) slip failure around the reinforced soil block [see Figure 54c)].
- b) internal stability:
  - 1) tensile failure of the individual reinforcement elements [see Figure 55a)];
  - 2) bond failure of the individual reinforcement elements [see Figure 55b)].
- c) compound stability:
  - 1) tensile failure of the individual reinforcement elements [see Figure 56a)];
  - 2) bond failure of the individual reinforcement elements [see Figure 56b)].

The maximum load carried by the reinforcement in considering the ultimate limit state condition should be:

$$T_j \leq \frac{T_D}{f_n}$$

where:

- $T_j$  is the maximum reinforcement tensile load at any level  $j$  in the slope;
- $T_D$  is the design tensile strength of the reinforcement calculated in accordance with 5.3.3;
- $f_n$  is the partial factor governing the economic ramifications of failure, see Table 9.

The serviceability limit states which should be considered are:

- 1) external stability:
  - settlement of the slope foundation [see Figure 57a)].
- 2) internal stability:
  - post-construction strain in the reinforcement [see Figure 57b)];
  - post-construction creep strain of saturated fine grained soils used with reinforced soil [see Figure 57c)].

Consideration should be given to the possibility of an ultimate limit state assessment of external stability leading to a problem of unserviceability rather than collapse, such as a bearing capacity problem leading to deformation, not collapse.

Figure 54 **Ultimate limit states – External stability**

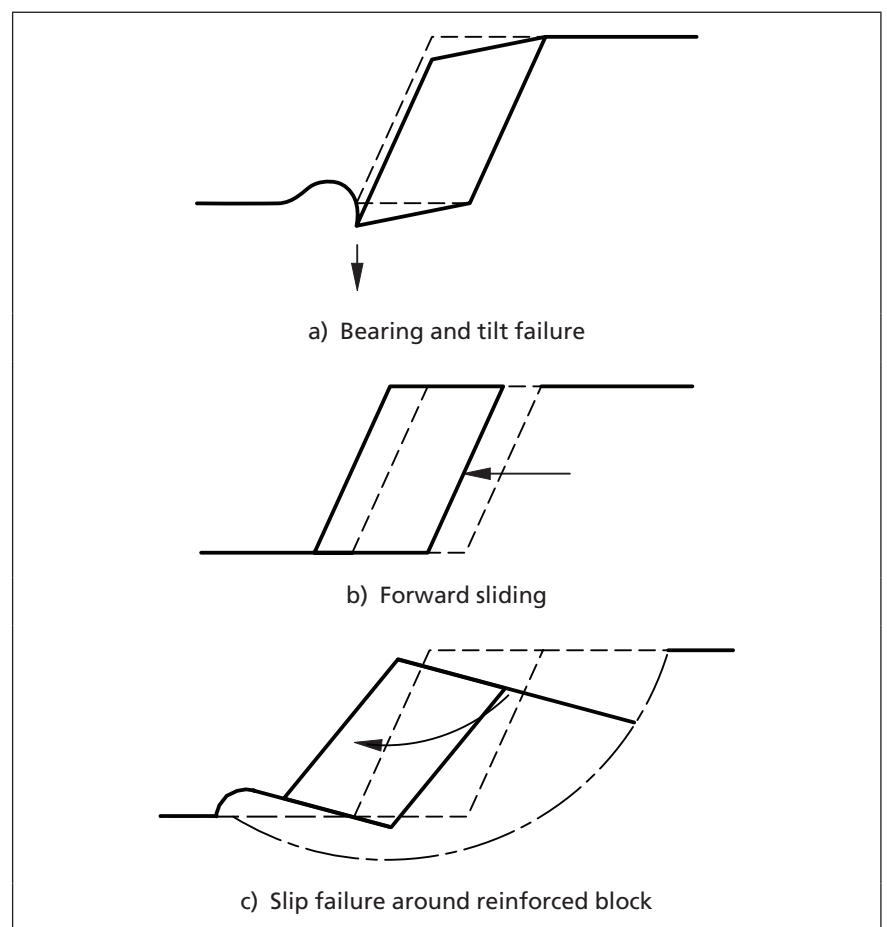


Figure 55 Ultimate limit states – Internal stability

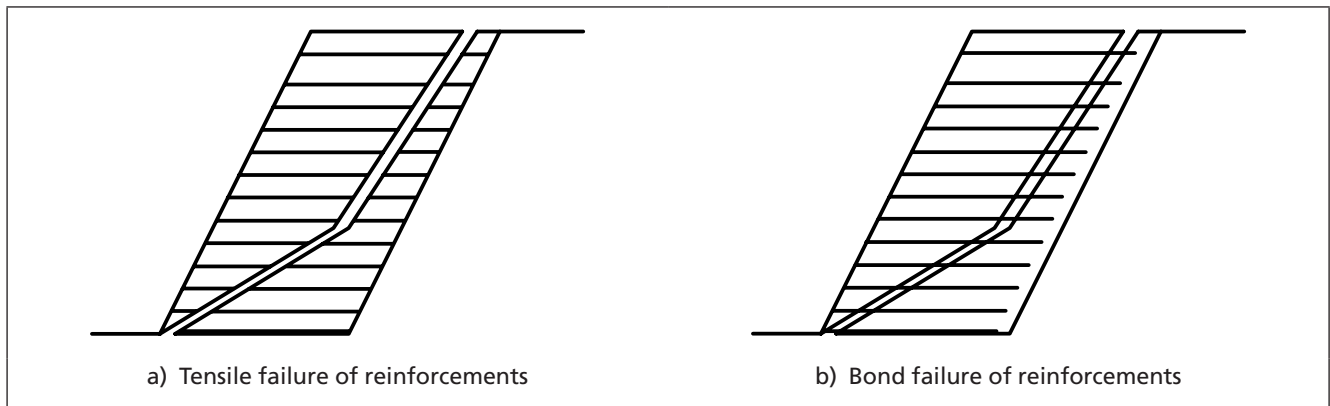


Figure 56 Ultimate limit states – Compound stability

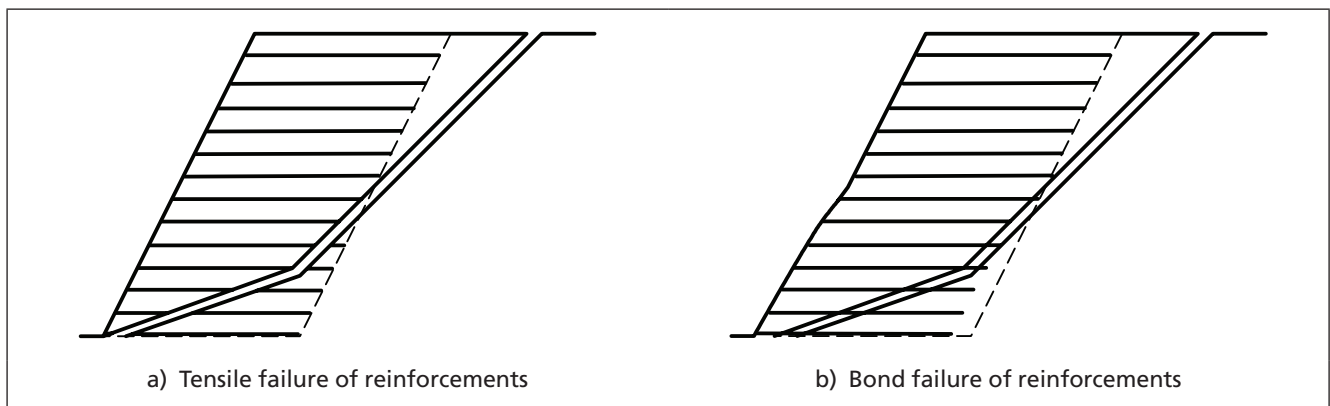
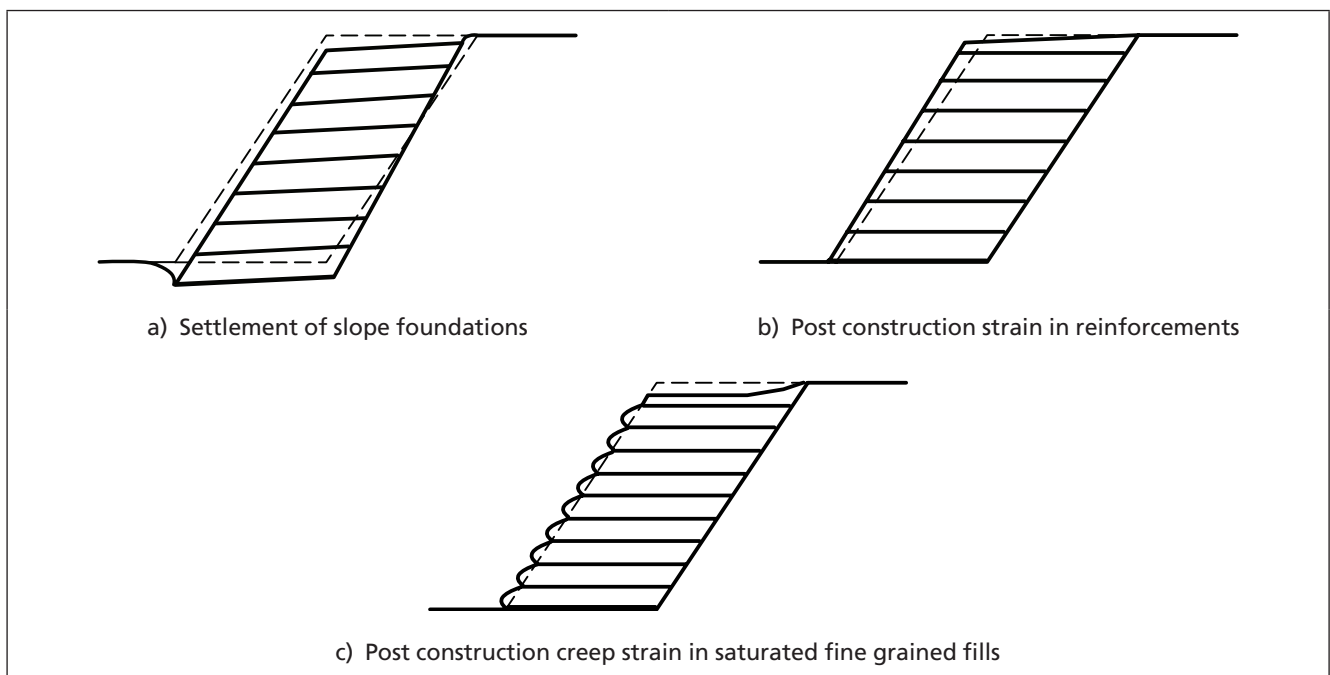


Figure 57 Serviceability limit states



### 7.4.3 External stability

The assessment of external stability for steep slopes should be based on the usual procedures adopted for reinforced soil retaining walls, 6.5.

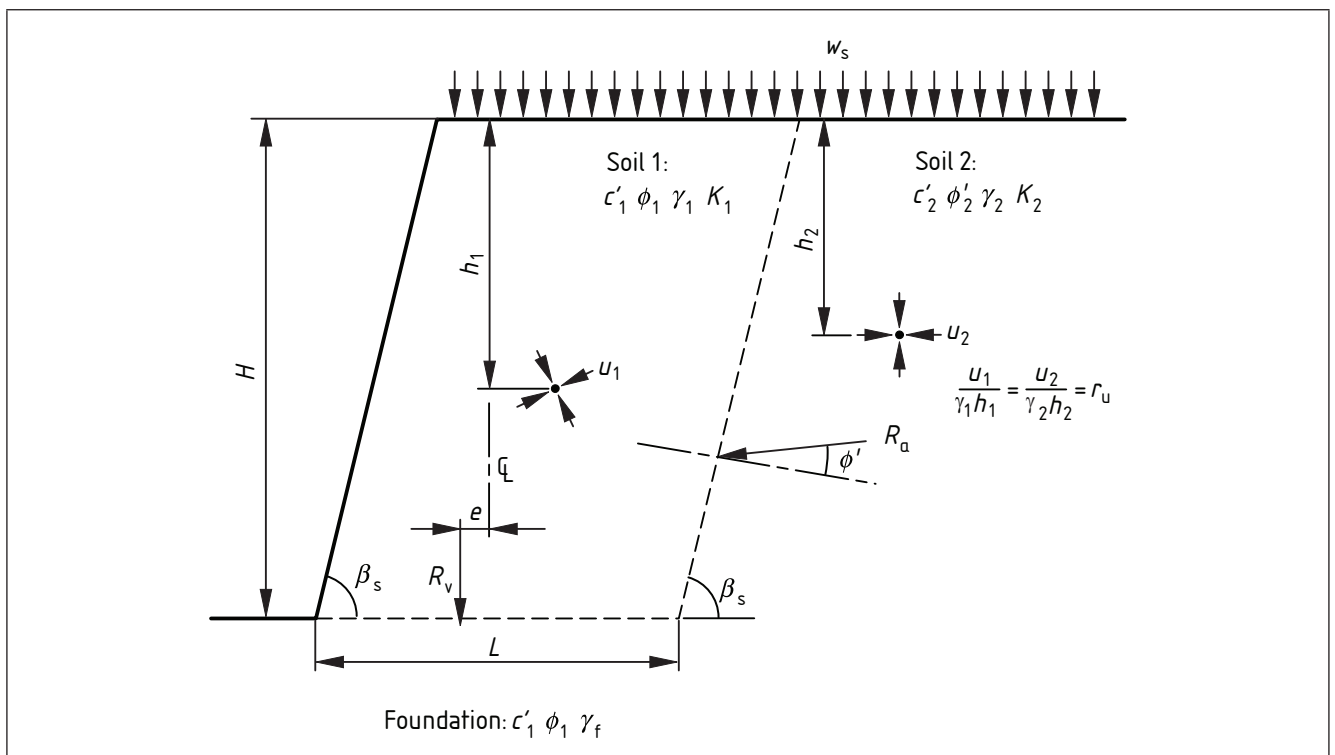
The soil properties and principal loads shown in Figure 58 should be taken into account when assessing external stability. For shallow slopes external stability should normally be analysed using slip circle techniques. Where these stability assessments indicate that one or more modes of potential collapse exist, several options may be available:

- reduce the slope angle;
- increase the width of the reinforced zone;
- use better quality fill;
- enhance the foundation with ground treatment;
- employ a counterweight such as a berm;
- use lightweight fill (e.g. PFA);
- incorporate reinforcement at formation level, see Section 8;
- introduce drainage to reduce pore water pressures.

For steep slopes, each of the ultimate limit state modes shown in Figure 54 should be assessed. To ensure the limit states of bearing failure [see Figure 54a)], forward sliding [see Figure 54b)], and external slip circle failure do not occur the reinforced slope should have sufficient dimensions in terms of reinforcement length  $L$  (see Figure 58).

All potential slip surfaces should be considered, including those passing wholly external to the structure, similar to those in Figure 26a). The appropriate analysis method and the partial safety factors used should conform to BS EN 1997-1:2004, with due consideration of NA to BS EN 1997-1:2004.

Figure 58 Definition of soil properties and principal loads for reinforced steep fill slopes



## 7.4.4 Internal stability

### 7.4.4.1 General

For internal stability of the reinforced slope the reinforcements should be able to resist the loads imposed upon them.

The stability assessment should generally be carried out on the basis of limit equilibrium methods adjusted by the use of appropriate partial factors in accordance with the limit state being considered.

*NOTE 1 A large number of methods are available to choose from. These include two-part wedge analyses (see Murray [35], Jewell et al [36]); circular or non-circular analyses (see Murray et al [41]); log-spiral failure analyses (see Leschinsky and Volk [42]); and coherent gravity method (see Segrestin et al [43]).*

*NOTE 2 Proprietary software is available for the calculation of internal stability.*

*NOTE 3 Nevertheless inaccuracies in determining material properties are likely to have a much greater influence on the design than differences in the types of analysis, which may be regarded as of secondary importance in most circumstances.*

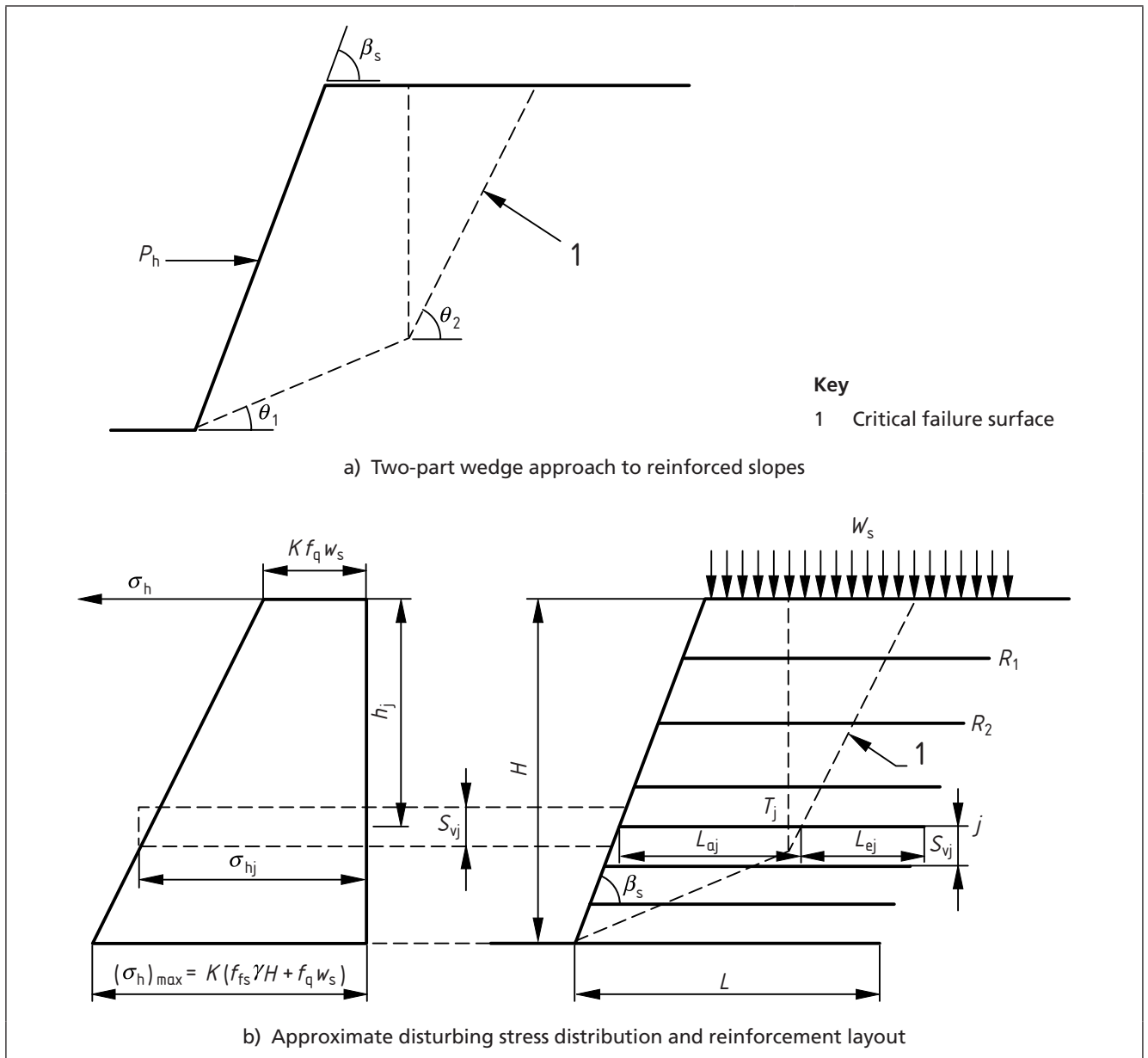
### 7.4.4.2 Two-part wedge analysis

It should be noted that the two-part wedge analysis assumes a bilinear failure surface [see Figure 59a)]. This has been shown to provide a reasonable representation of the potential failure surfaces for slopes (see Jewell et al [36], HA 68/94 [44]). This is a logical extension of the Coulomb wedge approach for vertical walls. As the angle of the face reduces from vertical, the critical mechanism is taken to be a two-part wedge.

The analysis considers various trial surfaces and then considers the equilibrium of the mass of soil above the selected surface; this may be carried out in a number of ways depending on the assumed interface conditions between the two parts of the wedges. The critical potential failure surface yields the maximum gross disturbing force that should be resisted to ensure limit states do not occur [see Figure 59a)].

For slopes where the fill is finished with a horizontal surface, the gross disturbing force may be considered to be the resultant of lateral earth pressures, which increase approximately linearly with depth over the slope height [see Figure 59b)] and an example design approach with details of stability equations may be found in HA 68/94 [44].

Figure 59 Two-part wedge analysis for internal stability of reinforced fill slopes



### 7.4.4.3 Slices method for circular slip analysis

For more general slopes of varying geometry and multiple soil strata the method of slices has been well established and may be used for the analysis of the stability of unreinforced and reinforced slopes [see Figure 60a)]. In the case of reinforced slopes the assumption is that the interslice forces may be ignored because of the complexity of the reinforcement influencing these forces and because the presence of the reinforcement will mean that there is little distortion of the soil mass under consideration. It may also be assumed that the reinforcement layers are horizontal and only considered where they intersect the assumed failure surface on a particular slice.

The restoring moment of the combined effects of soil and reinforcement should not be less than the disturbing moment due to the weight of the soil. The moments should be calculated about the centre of rotation of the disturbed mass. For equilibrium:

$$M_D \leq M_{RS} + M_{RR}$$

where

$M_D$  is the disturbing moment due to the weight of the soil plus surcharge;

$M_{RS}$  is the restoring moment due to the shear strength of the soil;

$M_{RR}$  is the restoring moment due to the presence of the reinforcement in the slope.

Using Figure 60a):

$$M_D = \sum_{i=1}^n [(f_{fs}W_i + f_q b_i w_{si}) \sin \alpha_i] R_d$$

and

$$M_{RS} = \sum_{i=1}^n \frac{\left[ \frac{c' b_i}{f_{ms}} + (f_{fs}W_i + f_q b_i w_{si})(1 - r_u) \frac{\tan \phi'_p}{f_{ms}} \right] \sec \alpha_i R_d}{1 + \frac{\tan \phi'_p \tan \alpha_i}{f_{ms}}}$$

and

$$M_{RR} = \sum_{j=1}^m T_j Y_j$$

where

$f_{fs}$  is the partial factor applied to soil unit weight (see Table 21);

$f_q$  is the partial load factor applied to external surcharge loads (see Table 21);

$w_{si}$  is the external surcharge acting on slice  $i$ ;

$c'$  is the cohesion of the fill measured under effective stress conditions;

$u_i$  is the pore water pressure acting on the slip surface of slice  $i$ ;

$\phi'_p$  is the peak angle of shearing resistance of the fill;

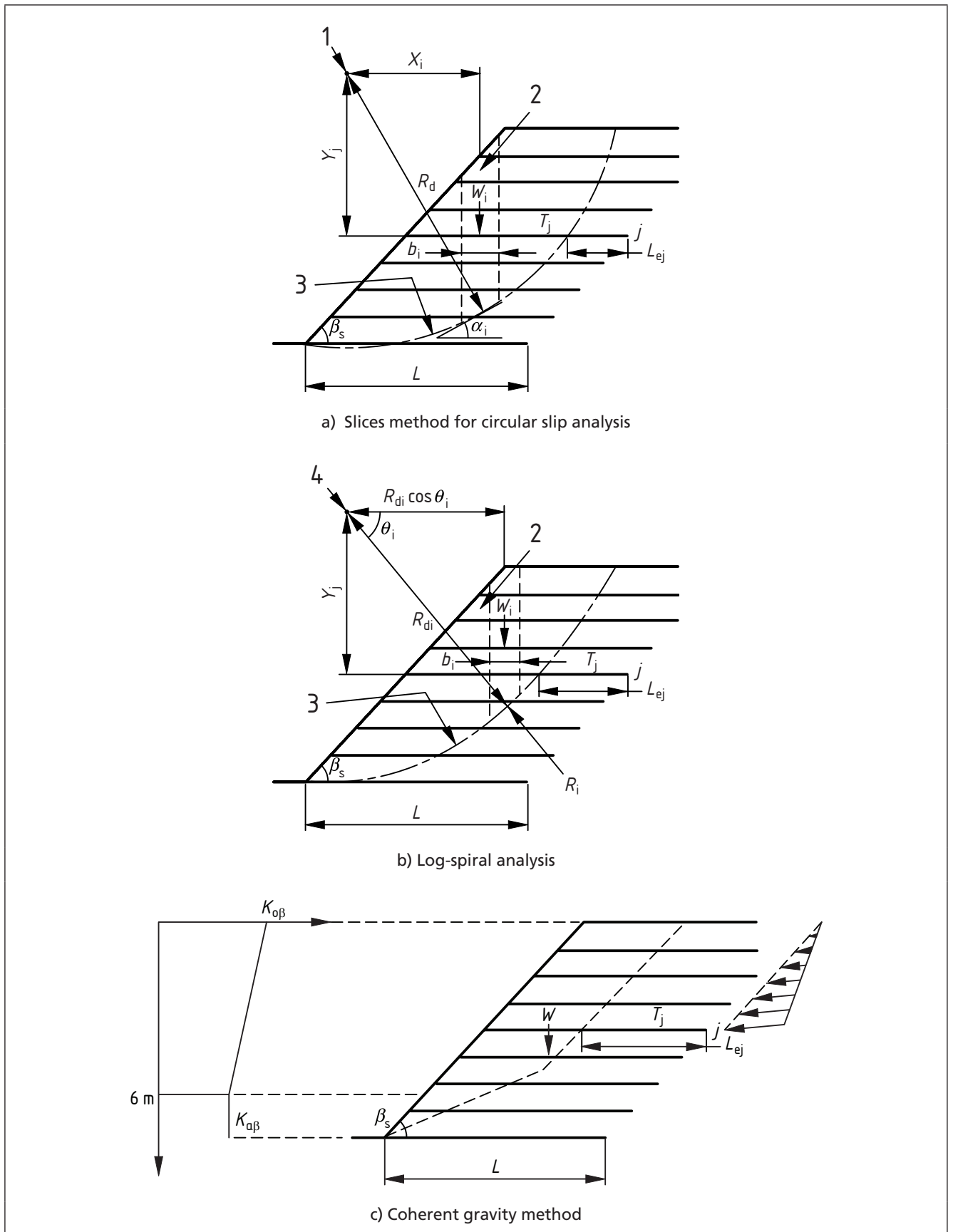
$f_{ms}$  are the partial material factors applied to  $\tan \phi'_p$  and  $c'$  (see Table 21).

To ensure the ultimate limit state governing reinforcement bond failure is not attained the reinforcement bond length  $L_{ej}$  may be determined using the relationship in 7.4.4.2.

*NOTE This method has also been used to analyse the reinforcement of shallow cohesive slopes (see Greenwood [45], and Greenwood [46]). A similar approach has been adopted for non-circular slip analyses.*



Figure 60 Other methods of internal stability analysis of reinforced fill slopes



#### 7.4.4.4 Other methods of analysis

*NOTE* There are a number of other methods of analysis for reinforced slopes. These are based on the need to achieve either moment or force equilibrium.

##### 7.4.4.4.1 Conjugate stress analysis

In the method of conjugate stress analysis a relatively simple form of failure surface is assumed and stresses acting on this surface may be determined on the basis of conjugate stress theory and Mohr's circle of stress analysis (see Taylor [47]).

*NOTE* The results of adopting this method are presented in Murray et al [41].

##### 7.4.4.4.2 Log-spiral analysis

Analysis may also be based on log-spiral slip surfaces [see Figure 60b)]. The assessment of moment equilibrium assuming such surfaces has been studied by Leschinsky and Boedecker [48].

To ensure the ultimate limit state governing reinforcement bond failure is not attained, the reinforcement bond length  $L_{ej}$  may be determined using the relationship in 7.4.4.2.

##### 7.4.4.4.3 Coherent gravity method

A modification of the coherent gravity method used for walls may be used for the design of slopes [see Figure 60c)]. This should also include a two-part wedge failure mechanism with modifications to lateral earth pressures and lines of maximum tension, due to the inclination of the structure (see Segrestin et al [43]). This method may be applied to steep slopes reinforced with inextensible reinforcements.

##### 7.4.4.5 Shallow slopes

The analysis of the internal stability of shallow slopes may follow the same procedure as steep slopes.

However, it should be noted that reinforcement anchorage considerations are more complex for shallow slopes because in many instances no facing is used; in these instances the reinforcement load carrying capacity can be limited by the bond length available near the surface of the slope.

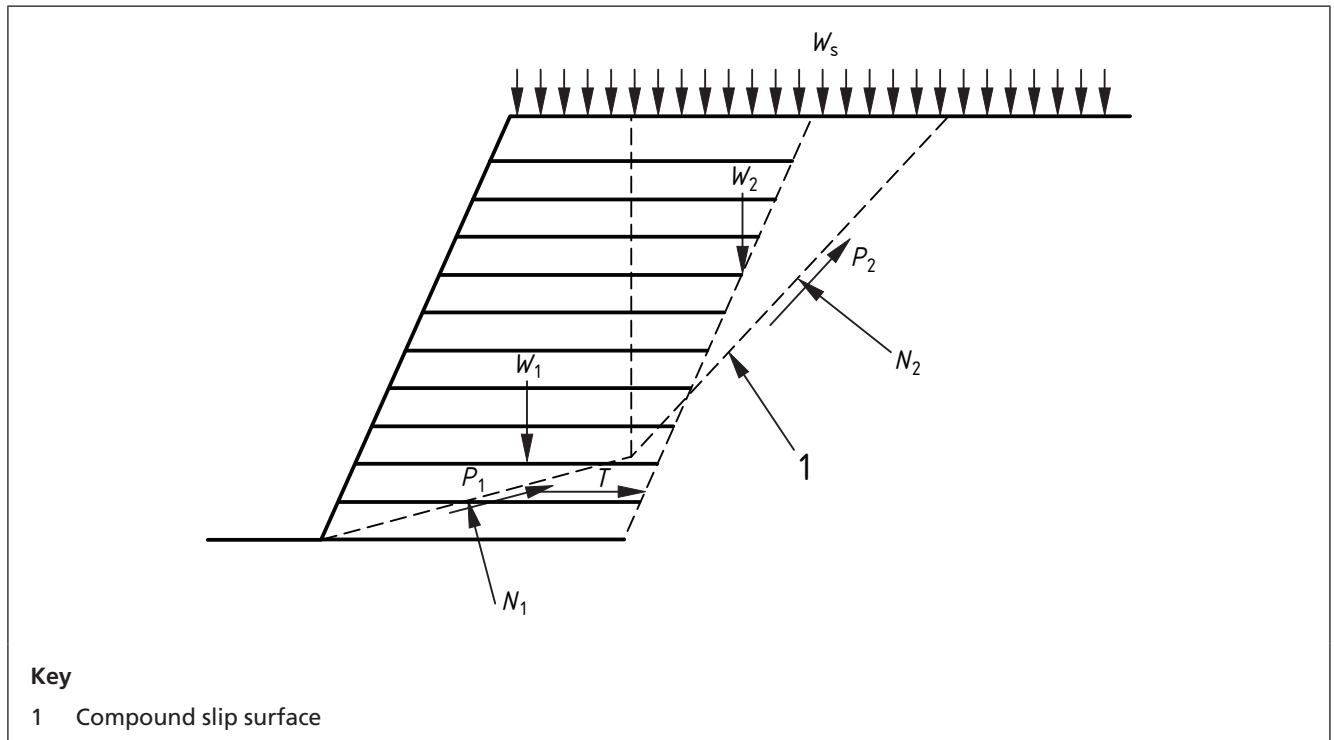
If reinforcement anchorage considerations are found to be important when analysing the stability close to the face consideration should be given to the value of the angle of friction of soil at low confining pressure near the slope surface as this can be much higher than at high confining pressures well within the slope.

#### 7.4.5 Compound stability

A complete search for potential failure surfaces may result in compound stability being the limiting mode of failure where the potential failure surface passes partially through the reinforced zone and partially through the backfill (see Figure 61).

The effect of the reinforcement should be considered on that part of the potential failure surface that intersects the reinforcement layers. Both reinforcement rupture and bond should be considered. The various methods used to analyse internal stability may be extended to analyse compound stability.

Figure 61 Force components in two-part wedge analysis of compound stability



## 7.4.6 Serviceability limits

### 7.4.6.1 General

There are three overall serviceability limit states which should be satisfied (see Figure 57). For the majority of applications involving reinforced slopes serviceability limits should not be critical, except where the slope is designed to special tolerances to support external dead loads. In this case the serviceability limits applied to the reinforced slope should be similar to that for walls listed in 6.5.5.2.

### 7.4.6.2 Settlement of slope foundation

Settlement of the slope foundation [see Figure 57a)], is not normally critical, however, consideration should be given to the added stresses imparted to the reinforcement due to the deformation of the structure as a whole.

### 7.4.6.3 Post-construction strain in reinforcement

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

#### 7.4.6.4 Post-construction creep strain in saturated fine grained fills

This is very difficult to calculate; consequently where this is considered to comprise a limit state, consideration should be given to providing good drainage and/or sealing of the reinforced zone. Preferably, better quality fill should be used.

#### 7.4.6.5 Construction tolerances

Details of acceptable construction tolerances should be taken from BS EN 14475:2006, Annex C.

#### 7.4.7 Slip repairs

A slip takes place on the first plane on which the factor of safety reduces below 1.0; it is therefore likely that deeper surfaces will still have an unacceptably low factor of safety and a full analysis of a slope should be performed prior to repairs being undertaken.

A drainage layer installed on the base and back of the excavation before replacing the fill may lower the water pressures to such an extent that the factor of safety is acceptable throughout the slope without additional excavation.

The material removed from a failed slope may usually be replaced together with reinforcement to provide a slope with an acceptable factor of safety.

*NOTE Methods of analysis have been developed specifically for design of slip repairs (see Greenwood [45], Murray [50]).*

#### 7.4.8 Wrap-around faces

*NOTE Wrap-around facing is one technique that may be used when the slope face is steeper than 45°.*

For the wrap-around face, the force applied to the reinforcement tail (see Figure 62) may be approximated as:

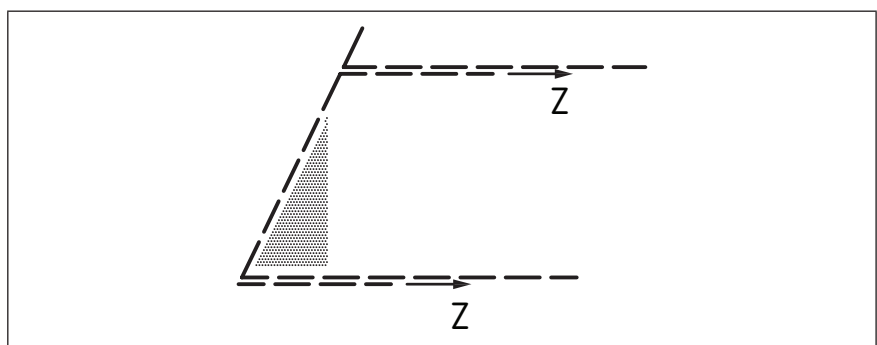
$$Z = T_j/3$$

where  $T_j$  is the load in the reinforcement layer under consideration.

Any connection between the tail of the wrap around face reinforcement and the next layer of main reinforcement above it should be able to resist the force  $Z$ . The facing part should also be able to resist the force  $Z$ .

Where the tail of the wrap-around face reinforcement is not connected to the next layer of main reinforcement above it the tail should have sufficient anchorage into the fill to resist the force  $Z$ .

Figure 62 Force applied to the reinforcement tail of a wrap-around face



### 7.4.9 Surficial stability

The stability of the face area of shallow slopes is generally influenced by the infiltration of water into the surface soils and the potential generation of a shallow slip parallel to the slope surface should be assessed. Short lengths of reinforcement should be designed to intercept these near surface slips using an appropriate model (see Thielen and Collin [51]).

### 7.4.10 Design of facings

It should be noted that there is a difference in the approach to slope facing design depending on the form of the reinforcement and facing being used.

Wrap-around facings are usually a continuation of the sheet reinforcement and the load in the reinforcement, at the connection with the face, may be assumed to be carried round the wrap-around face material; 7.4.8 provides guidance on the design anchor length of a non-continuous face sheet.

Pre-formed facing may be designed using the methods used for design of rock support/capture steel mesh which are anchored to a rock face and an example design approach with details of stability equations can be found in Ruegger et al [52] and CIRIA C637 [53].

*NOTE Pre-formed facings are usually flat or bent sheets of steel mesh or grid which are attached to the ends of the reinforcements. These facings are flexible and do not span between reinforcements but act similarly to rock capture nets where the design criteria is that of the reinforcement punching out of the mesh.*

The design of gabion faces should be carried out using methods provided in gabion design literature.

## 7.5 Construction and maintenance of slopes

### 7.5.1 General

Table 1 shows the factors which should be considered in assessing the performance of a reinforced soil slope.

The construction of reinforced slopes entails many of the techniques inherent in the construction of conventional earthworks and the factors that affect the construction of reinforced soil walls and abutments should also be considered see Section 6. See BS EN 14475:2006, 6.4 and Annex C.

### 7.5.2 Foundations

Although differential settlement is of less significance than for structures the importance of compaction to the integrity of the slope does require some foundation preparation.

Site preparation should be carried out as described in 8.5.2.5.

*NOTE Further guidance is given in BS EN 14475:2006, 8.3.2.*

### 7.5.3 Reinforcing elements and fill

Fill materials should conform to 3.1 and be placed and compacted in accordance with the *Specification for Highway Works* [1]. Further guidance is given in BS EN 14475:2006, 8.5.4.

Recommendations and guidance on the quality and testing of the fill is given in Section 3 and Section 4.

Sheet reinforcement should be laid in the direction of the principal loading, that is perpendicular to the face of the fill. Any jointing or connections across the direction of principal load should be as detailed in 3.2.3.

In the transverse direction, it is recommended that geotextiles should overlap by 200 mm to 300 mm (or 500 mm if significant movements are expected). Geogrid materials are normally laid with minimal overlap, however overlaps similar to that for geotextiles should be used if significant movements are to be expected.

Construction traffic should not pass over the reinforcements before a minimum thickness of 100 mm of fill has been placed.

*NOTE Further guidance given in BS EN 14475:2006, 8.5.3.*

### 7.5.4 Facing

#### 7.5.4.1 Wrap-around facings

*NOTE 1 A widely used soft facing unit is wrap-around facing as illustrated in Figure 63.*

Wrap-around facings may be constructed without the use of shoring or formwork if the front slope is less than about 1:1 or with the use of formwork for steeper slopes with the following recommendations.

- a) *Slopes shallower than or equal to 1:1.* These do not require wrap-around faces. A typical construction sequence is as follows.
  - 1) Lay the reinforcement on the previously compacted layer of fill.
  - 2) Fill and compact to the edge of the slope profile.
  - 3) Complete laying of reinforcement, placement of fill and compaction.
  - 4) Trim slope face to necessary profile.
  - 5) Topsoil and seed to encourage rapid development of vegetation and prevent erosion (alternatively provide an erosion control geotextile).
- b) *Slopes steeper than 1:1.* Steep reinforced fill slopes will generally require some form of formwork and/or face containment. The details of the formwork may vary but generally the result will be a plane slope in the range 60° to 80° [see Figure 64b)].

Where geotextiles are used for reinforcement, a stepped slope or an inclined plane slope should usually be considered adequate as the material has a tendency to deform and the aesthetics are not significantly improved by using sloping formwork. Where geogrid reinforcement is used, an inclined plane face should generally be formed.

The sequence of construction for the steep slope face should be as follows.

- 1) Prepare a level foundation.
- 2) Erect temporary formwork to the face angle needed.
- 3) Cut and position the base layer of reinforcement with an allowance at the face for the wrap-around and the turn back into the fill.
- 4) Place a liner, e.g. geotextile or turf, if needed, inside the wrap around face to prevent loss of fill through the face.
- 5) Fill and compact over the reinforcement in accordance with 6.10.2.
- 6) Wrap the free end of the reinforcement around the face of the fill to encapsulate it.
- 7) Either anchor the free end of the reinforcement into the fill with an anchorage length or connect it to the next layer of reinforcement, tensioning the face wrap-around to hold the face tight.
- 8) Continue to full height.

Whilst variations on this method have been described by Jones et al. [54], Jones [55], Jones [56], Paul [57], Göbel et al [58] and Rügger [59], up-to-date information should be sought from the appropriate manufacturers.

Formwork may be used in the construction of stepped-slope facings. The construction sequence should be similar to that given above and the formwork should generally be related to the fill layer-height, [see Figure 64b)].

Reinforcement should be protected against damage from ultra-violet rays or vandalism. Recommendations on protective measures including seeding and planting are given in 7.4.10.6.

Figure 63 **Wrap-around facing**

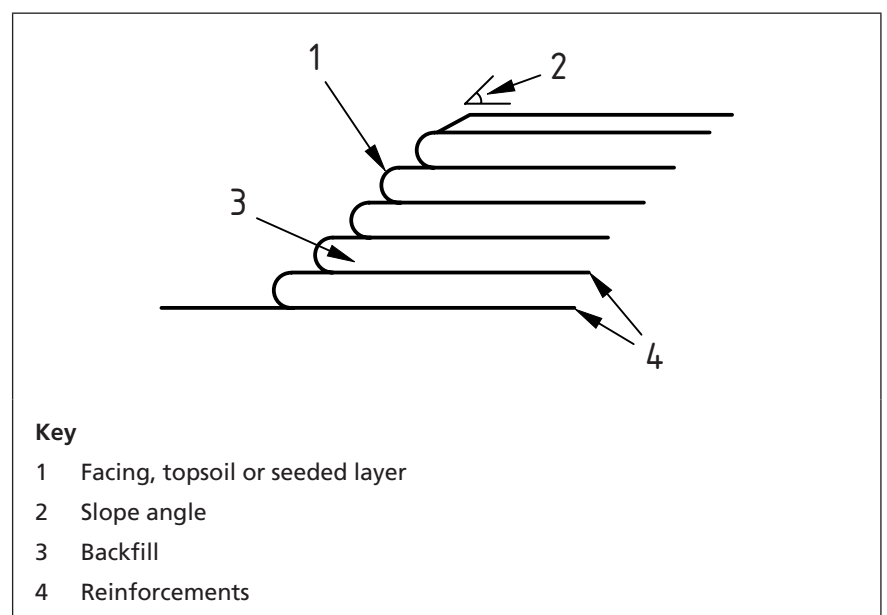
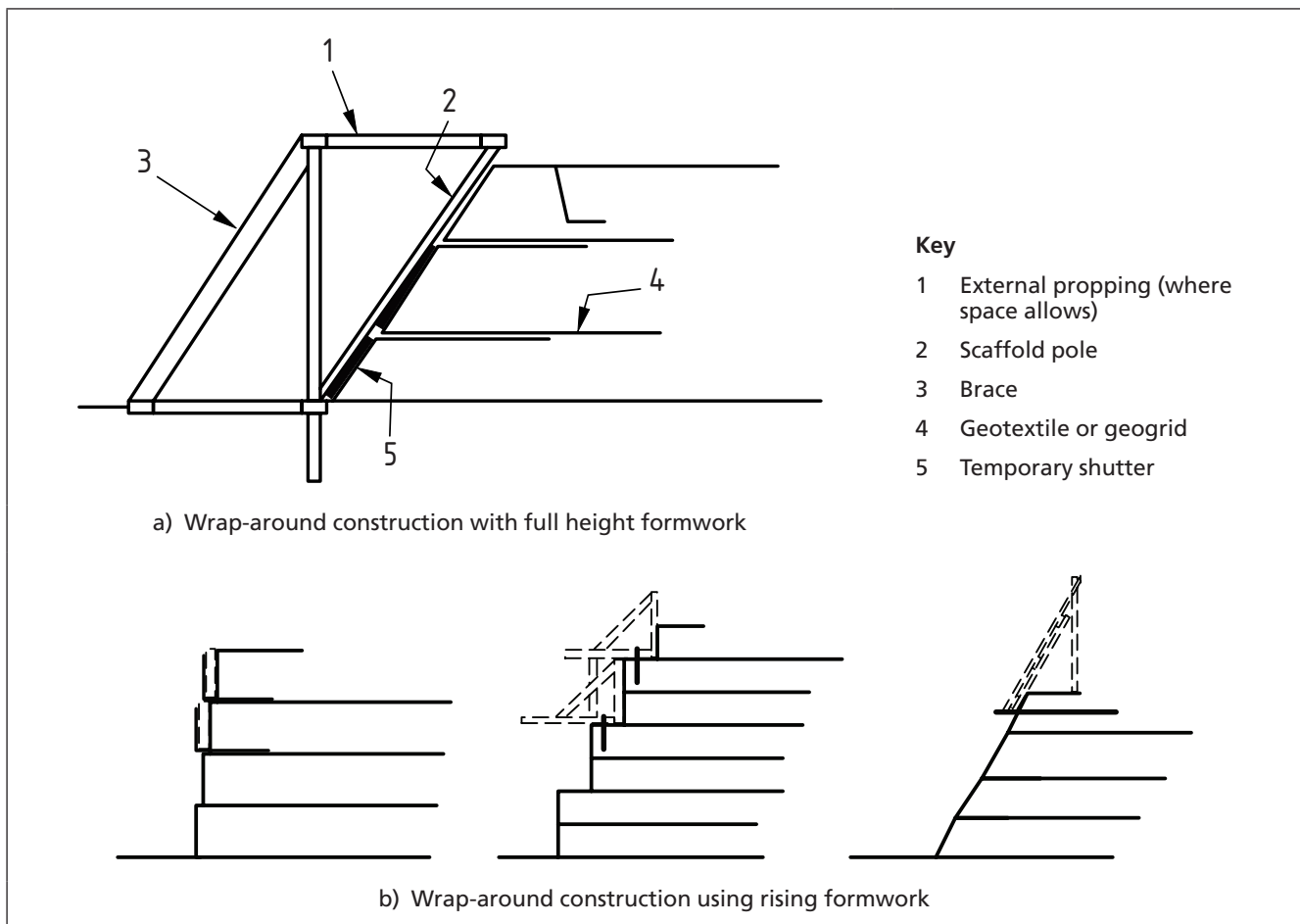


Figure 64 Wrap-around construction techniques



#### 7.5.4.2 Gabion and bag type facings

Combined facing and formwork may be provided using gabions or bags formed of geosynthetic material.

Steel mesh gabions may also be used, as can other types of formwork such as filled car tyres (see Jones [56] and Figure 65).

Where gabions or bags are used the construction technique should be in accordance with 7.4.4.1.1 with the gabions or bags acting as the formwork (see Jones et al [54]).

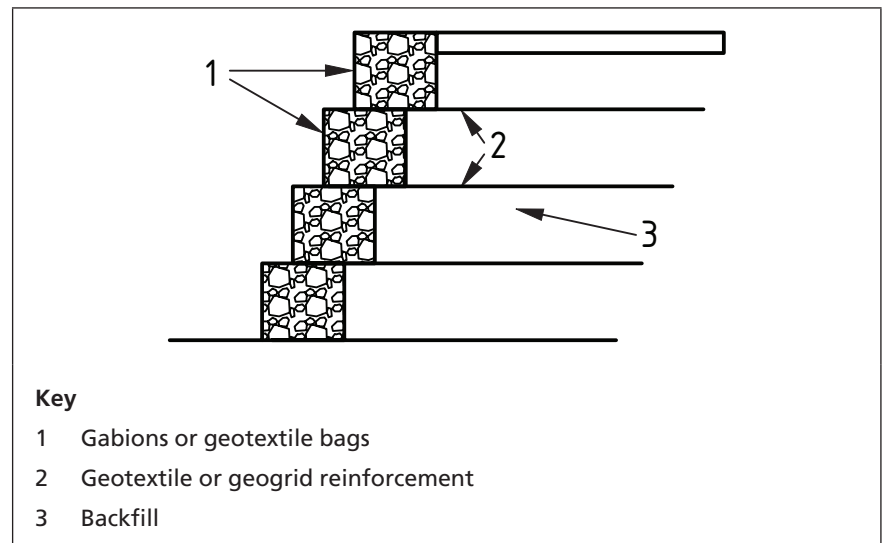
Geotextile gabions and bags will deform easily and a technique of connecting the sheet reinforcement to the bags is not recommended in view of likely local overstressing of the geotextile material. The connection should normally be achieved by maintaining the gabions or bags within the wrap-around facing of the geogrid reinforcement.

Polymer grid or steel mesh gabions form a stiffer structure and in this case the main reinforcement should be attached to the base of the gabion basket and laid back into the fill (see Figure 65). In this application, the gabion baskets should be filled when in position. Individual units should be placed side by side (end on) and tied together using braid, steel wire or bodkins to form a course up to 10 m or 12 m long. The shape of the gabions should be maintained during filling by applying tension along the "string" of empty gabions.



This may be done using a tensioning device and an arrangement of steel straining rods that are inserted in the end panel of the end gabion. The gabions should be filled by hand or mechanical plant. Where possible a fair face of large flat stone should be placed at the exposed faces only; this enhances the appearance and reduces the risk of construction-induced damage. For gabions used as facing units, cross ties should be installed at one-third and two-thirds heights for 1 000 mm high gabions and at half-height only for 500 mm high units. The operation should be completed by slightly over filling the gabions ideally using the finer fractions of the fill. The lids of the gabions should then be closed and secured to the top of the vertical walls of the unit using braid or steel wire (see Clause 626 of the *Specification for Highway Works* [1]).

Figure 65 Reinforced gabions



#### 7.5.4.3 Pre-formed facing panels

Steel mesh or grid facing may be used for face slope angles steeper than 45° and these may be either flat sheets or shaped meshes. (see Smith [60], Beazant [61]).

#### 7.5.4.4 Superficial facings

Reinforced fill slopes should be protected against degradation due to natural or man-made causes including the effects of ultra-violet light and vandalism. The most usual method of protection that may be used is the seeding of the slope. In this case, a layer of topsoil should be placed either in front of or behind the in situ facing.

The topsoil layer should be placed over a granular layer and local stabilization measures taken to ensure surface stability before germination of the seeds.

Hydroseeding may also be used to apply seed directly to the outside of a geotextile wrap-around facing, in which case topsoil may be provided behind the facing to give sustenance to the grass roots penetrating through the geotextile and providing an effective protective layer.

An alternative that may be used, is to apply a seed-impregnated layer just behind the facing.

Hard superficial facings, such as shotcrete or concrete panels, clipped onto or nailed into the facings may also be used. When using shotcrete the possible effects of post-construction movements in causing cracking and spalling of the applied facing should be considered.

BS EN 14475:2006 should be referenced for more detail.

### 7.5.5 Drainage

Measures should be taken to ensure that the fill does not become waterlogged, or that any water pressures assumed in design are not exceeded. The considerations of drainage should be similar to the case of walls (see 6.10.5 and BS EN 14475:2006, 8.4.2).

### 7.5.6 Reinstatement of fill slopes

#### 7.5.6.1 General

A common application of soil reinforcement is in the repair of slopes, see Figure 66. Design and construction should proceed in accordance with 7.4. Further guidance is given in HD 41/03 [62], CIRIA C591 [63] and CIRIA C592 [64].

The use of the in situ failed material, modified if appropriate, in the reinstatement using reinforced soil generally includes the incorporation of improved drainage and should be considered as an alternative to the conventional approach of using imported granular fill.

Repairs are often carried out in confined spaces, e.g. at the side of a road or railway; in planning the work consideration should be given to possible problems of access and storage of materials.

A typical arrangement of benched excavations are shown for a slope in Figure 66c) where, generally, the longitudinal extent of the benches should either be limited to 30 m or other smaller amount appropriate for temporary slopes.

#### 7.5.6.2 Construction

The recommended construction sequence for slope reinstatement includes the following.

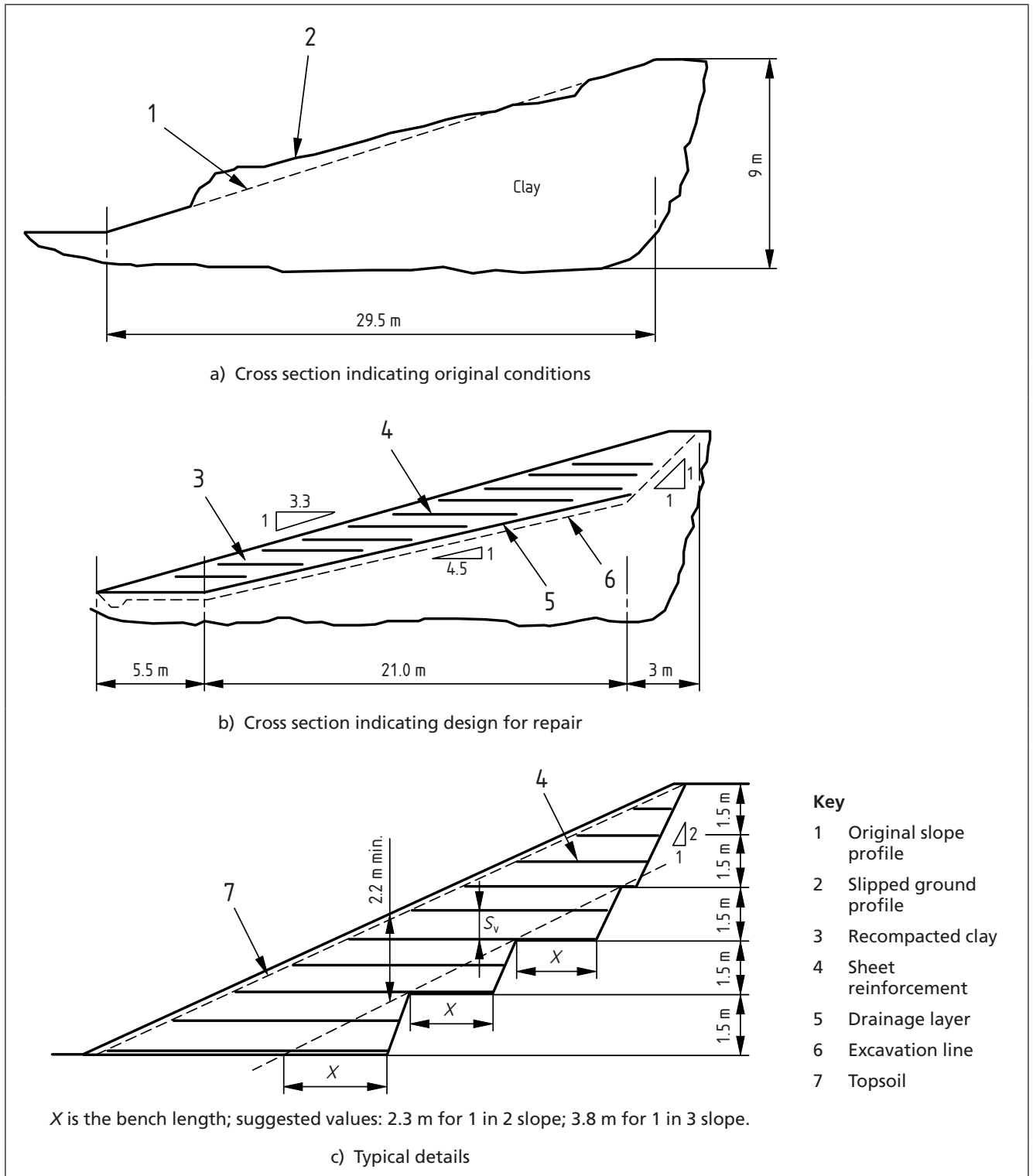
- a) Strip topsoil and remove to stock pile.
- b) Excavate to the minimum depth shown on the drawings with the maximum temporary backslope shown on the drawings (subject to safety). The cut should be benched as shown in the drawings [see for example Figure 66c)].
- c) Place reinforcement transverse to the embankment face on the lowermost ground level. Adjacent sheets need not be overlapped.
- d) Replace the excavated material in layers and compact in accordance with 6.10.2.2. If the material is excessively wet or plastic, quicklime sufficient to improve the workability of the soil should be added provided this is in accordance with 3.1.
- e) At the appropriate level as shown on the drawings place the next reinforcement sheet on a compacted layer and continue backfilling and placing reinforcement.
- f) Replace topsoil.

It is not necessary for the reinforcement to extend beyond the face.

During construction existing drainage should be inspected and checked for damage. If necessary the toe drain should be replaced by a french drain and downslope french drains installed or replaced. Any seepage encountered should be drained to prevent further softening.

Normally a maximum length of 30 m of slope should be treated at one time, or less if local conditions dictate.

Figure 66 Reinstatement of failed slopes



## 7.5.7 Vegetation on slope faces

### 7.5.7.1 Introduction

With few exceptions, vegetation should provide the most acceptable finish to an earthwork slope and it can make a significant contribution to the stability of such slopes. Vegetation may include plants, grasses and small bushes but should not be plants that will become large trees and damage the face.

The experience gained from the successful greening of unreinforced slope at angles between 25° and 30° in the hostile environment of vehicle spray and pollution should be referenced when considering the steeper slopes that have been introduced in the last 20 years.

Similar considerations to those applicable to traditional un-reinforced slopes should be made, and in particular care should be taken in the choice of vegetation type, the angle of the face, method of seeding and the soil medium at the face into which the roots of the vegetation grows.

Vegetation has been established on slopes with face angles up to about 75°. With slopes steeper than this, the use of creepers that spread over the surface to create a green face should be considered. The following clauses, which are essentially an extension of the recommendations for unreinforced slope faces (see CIRIA C708 [65]), with the focus on slopes adjacent to highways may also be applied to slopes in less hostile environments.

### 7.5.7.2 Role of vegetation

The various roles of vegetation may be categorized as mechanical, hydrological, thermal, and ecological or environmental with the beneficial effects of vegetation being protection by armouring and sheltering, stabilization by root reinforcement, and water removal.

Vegetation may be chosen to enhance the performance of a strengthened earthwork through one or more of the following:

- a) providing shade against UV radiation to any geosynthetics used to form the face;
- b) controlling erosion by shielding against the impact of rain, water flow and wind, by acting as a reservoir to store intercepted rainfall, as a soil binder through root action, and a barrier to downward movement of debris;
- c) through evapotranspiration, reducing the weight of any potentially unstable mass of soil;
- d) increasing the shear strength of the retained soil by reducing pore water pressures, i.e. by generating suction through plant evapotranspiration;
- e) increasing the shear strength of retained fills by reinforcement of the plant roots.

### 7.5.7.3 Plant characteristics

The suitability for particular types of grass should be considered with regard to the ability of the vegetation to perform its function at a particular site and is dependent on a number of characteristics:

- a) vigour;
- b) resistance to impact, burial, and erosive forces;
- c) the ability to withstand drought;
- d) the ability to grow in infertile and poorly structured soils – which is important for the establishment of vegetation on newly formed slopes;
- e) the ability to grow in low sunlight conditions.

Root architecture, strength and bond, and good root/stem ratios should also be addressed and, for watercourse and shoreline protection applications, the ability to withstand water-logging is also essential.

### 7.5.7.4 Site appraisal

It is vital that the relevant characteristics and conditions of a site should be taken into account when designing a vegetation scheme, in particular the vegetation (local and of the habitat indicated). In particularly difficult or hostile environments the following additional information may be required:

- a) site observations: geology, geomorphology, hydrology, climate/microclimate;
- b) site investigation and analysis: soil grading, particle shape, pH, organic matter and nutrient levels, presence of contaminants, preliminary slope stability checks.

### 7.5.7.5 Plant types

The most frequently used plant type that may be adopted for the face of reinforced slopes is grass, although other types of vegetation are increasing in their use. Grass species that are drought resistant and low growing may generally be used for the application, although as the face gets steeper, and the availability of water reduces, the use of creeping plants that are rooted at the bottom of the face become more attractive. A mixture of different plant species may increase the probability of successful establishment, i.e. grasses and legumes (clovers) to give rapid cover and the associated erosion protection.

Low height plants, pioneer plants and evergreens all have their place in the vegetation selection and advice from a specialist consultant on the particular plant selection is recommended.

### 7.5.7.6 Planting and seeding

#### 7.5.7.6.1 Grass

The establishment of grass may be achieved in a number of ways.

- a) Turf can be supplied with many different seed mixes and in many different sizes to suit the particular site conditions and construction method. Reinforced slope up to 45° face angle do not generally require a structural face and in those situations

the turf can be placed on a prepared topsoil bed directly on the surface in large pieces or rolls. As the face angle increases the turf is placed behind a face mesh/grid, polymeric or steel, and hence is required to be in smaller, handleable, sections.

- b) Seeded topsoil with added nutrients, either placed on the surface of slopes within erosion matting materials, contained behind a mesh or in degradable bags behind a wrap-around face is also common.
- c) Pre-seeded mats, blankets or meshes are also used to give short term protection to the face until the vegetation establishes and takes over that function. Some mesh materials will remain durable and provide longer term assistance to the vegetation root mat in situations where the erosion forces are high, e.g. water courses, steepened slopes up to 45° where no wrap-around face is used.
- d) Hydroseeding, with careful specification and installation of a sufficient thickness of carrier material, has been very successfully used in a number of countries. Specialist organizations are usually employed for this application.

#### 7.5.7.6.2 Other plants, shrubs, etc.

The planting of individual plants on reinforced soil slopes may be achieved by locally cutting the face, taking out the required plug of fill, and planting in prepared holes. Any face mesh or grid can then be folded back to its original position and tied.

Willow brush layering may also be retro-fitted to a reinforced soil slope.

#### 7.5.7.7 Design considerations

A number of factors should be considered when deciding on the vegetation selection for a particular reinforced soil slope. These factors are an extension of those that should be considered for shallower slopes although their significance may be greater as the slope angle increases.

- a) *Direction of slope.* The direction in which a slope faces is significant as, in the UK, south and west facing slopes receive direct sunlight and so ambient temperatures can be high and vegetation growth might be more difficult. North and east facing slopes tend to be wetter and at lower ambient temperature encouraging growth, however lower light conditions might be a problem for some species.
- b) *Prevailing wind.* The direction of the prevailing wind and its moisture carrying capacity also need to be considered. The volume and type of traffic is also significant as the effective "wind" caused by high sided vehicles can mask the effect of the prevailing wind but still have its own effect.
- c) *Drought.* All the vegetation solutions require water and nutrients to sustain their growth. The correct selection of species with particular attributes can minimize the requirements. Some species have longer roots, which will penetrate deeper in to the soil in search of moisture and these will survive drought, or extreme conditions better than shallow rooted species. In particular circumstances irrigation may form part of the solution and allow a wider range of vegetation to be used, although there are cost implications.

- d) *Growing season.* The growing season for vegetation in the more northern areas of the UK may require the installation of the permanent vegetation scheme to be carried out at a different time to the main engineering works. This may influence the construction programme and require temporary measures to be considered.
- e) *Soil type.* The type of soil in constructed slopes has a large influence on the types of vegetation that could thrive on them. Sandy soils are prone to desiccation and may need particular measures to ensure moisture is present whereas clay soils can be resistant to root penetration.

#### 7.5.7.8 Maintenance

Whilst it is generally held that vegetative ground covers require little maintenance and are self-repairing, the factors that should be considered in terms of maintenance are ensuring continuity of vegetation cover and cutting of vegetation during the life of the slope.

For reinforced slopes the most important recommendation is to have a reasonable continuity of vegetation cover, which assists in maintaining integrity of the soil at the face zone. There are various methods of vegetating reinforced slopes that may be used, such as hydroseeding, seeding the topsoil during installation, placing pre-seeded topsoil and placing turfs at the face. The degree of maintenance should be influenced by the vegetation technique.

The factors that should be considered regarding the need to cut the vegetation are similar to those for un-reinforced slopes where it is usual only to cut vegetation at the bottom of slopes to ensure traffic sight lines are not infringed.

## Section 8: Design of embankments with reinforced soil foundations on poor ground

### 8.1 General

The design of embankments with reinforced soil foundations should be fully explained and detailed in a Geotechnical Design Report prepared in accordance with BS EN 1997-1:2004.

For the purposes of this code of practice, poor ground describes ground conditions that, if left untreated or unreinforced, would impair either the serviceability or ultimate limit state conditions of the proposed embankment; reinforcement may be used in the foundation to enhance the resistance of embankments to avoid failure through excessive deformation or shear in the foundation.

Two areas of application are considered in this section:

- a) embankments over soft and very soft foundation soils (see 8.3);
- b) embankments over areas prone to subsidence (see 8.4).

*NOTE* The scope of this section is limited to foundations for earthworks because of the present limited experience related to other structures.

### 8.2 Partial factors used in the design of embankments with reinforced soil foundations on poor ground

#### 8.2.1 General

Much of the design and analysis of reinforced soil foundations has utilized a limit equilibrium approach where a global factor of safety should be satisfied. Because these methods are based on equilibrium considerations they may be simply restated in a limit state format by increasing the soil weight and live loading by appropriate partial load factors and reducing the soil properties and reinforcement strength by appropriate partial material factors.

The design principles established in Section 2 and 5.3 should be used as a basis for the procedures contained in this section. Table 22 contains the partial factors appropriate to this section.

#### 8.2.2 Load factors

The soil unit weight to which the partial load factor is ascribed should be the characteristic value (see 2.5) and should take into account variations in specific gravity, grading and compaction. The external loads to which the partial load factors are ascribed should be the characteristic values in their original unfactored state.



### 8.2.3 Material factors

Material properties such as reinforcement capacity or soil properties should be reduced by dividing by prescribed material factors (greater than unity) to produce design material properties.

The soil material factors relating to the values of  $c'$  and  $c_u$  should have values greater than or equal to unity when assessing the ultimate limit state. The soil material factor relating to  $\phi'_{cv}$  should be unity because this parameter already relates to the ultimate limit state condition.

The reinforcement material factors are applied to the base strength of the reinforcement and should have a value consistent with the type of reinforcement to be used and the design life over which the reinforcement is needed. The method of determining the appropriate value of  $f_m$  should be in accordance with the procedure established in 5.3.3 and Annex A.

Table 22 Summary of partial factors to be used in Section 8

	Partial factors	Ultimate limit state	Serviceability limit state
Load factors	Soil unit mass, e.g. embankment fill	$f_{fs} = 1.3$	$f_{fs} = 1.0$
	External dead loads, e.g. line or point loads	$f_f = 1.2$	$f_f = 1.0$
	External live loads, e.g. traffic loading	$f_q = 1.3$	$f_q = 1.0$
Soil material factors	To be applied to $\tan \phi'_{cv}$	$f_{ms} = 1.0$	$f_{ms} = 1.0$
	To be applied to $c'$	$f_{ms} = 1.6$	$f_{ms} = 1.0$
	To be applied to $c_u$	$f_{ms} = 1.0$	$f_{ms} = 1.0$
Reinforcement material factor	To be applied to the reinforcement base strength	The value of $f_m$ should be consistent with the type of reinforcement to be used and the design life over which the reinforcement is required (see 5.3.3 and Annex A)	
Soil/reinforcement interaction factors	Sliding across surface of reinforcement	$f_s = 1.3$	$f_s = 1.0$
	Pull-out resistance of reinforcement	$f_p = 1.3$	$f_p = 1.0$

### 8.2.4 Soil/reinforcement interaction factors

There are two soil/reinforcement interaction parameters that should be considered in this section:

- soil sliding across the surface of the reinforcement;
- pull-out of the reinforcement from the soil.

### 8.2.5 Economic ramifications of failure

The partial factor for economic ramifications of failure  $f_n$  should be applied to the reinforcement design strength in accordance with Table 9.

## 8.3 Reinforced embankments over soft and very soft foundation soils

### 8.3.1 Areas of application

For the construction of reinforced embankments over soft and very soft foundation soils the techniques in use may be divided into one of two categories:

- a) those techniques where the reinforcement is used to control stability of the embankment, without controlling settlement (see Figure 67);

*NOTE Techniques include basal reinforcement alone [see Figure 67a)]; basal reinforcement with vertical drains [see Figure 67b)]; and basal mattress reinforcement [see Figure 67c)].*

- b) those techniques where the reinforcement is used as part of a foundation stabilization system to control stability and prevent or limit settlement of the embankment (see Figure 68).

Figure 67 Reinforcement used to control only stability of embankment

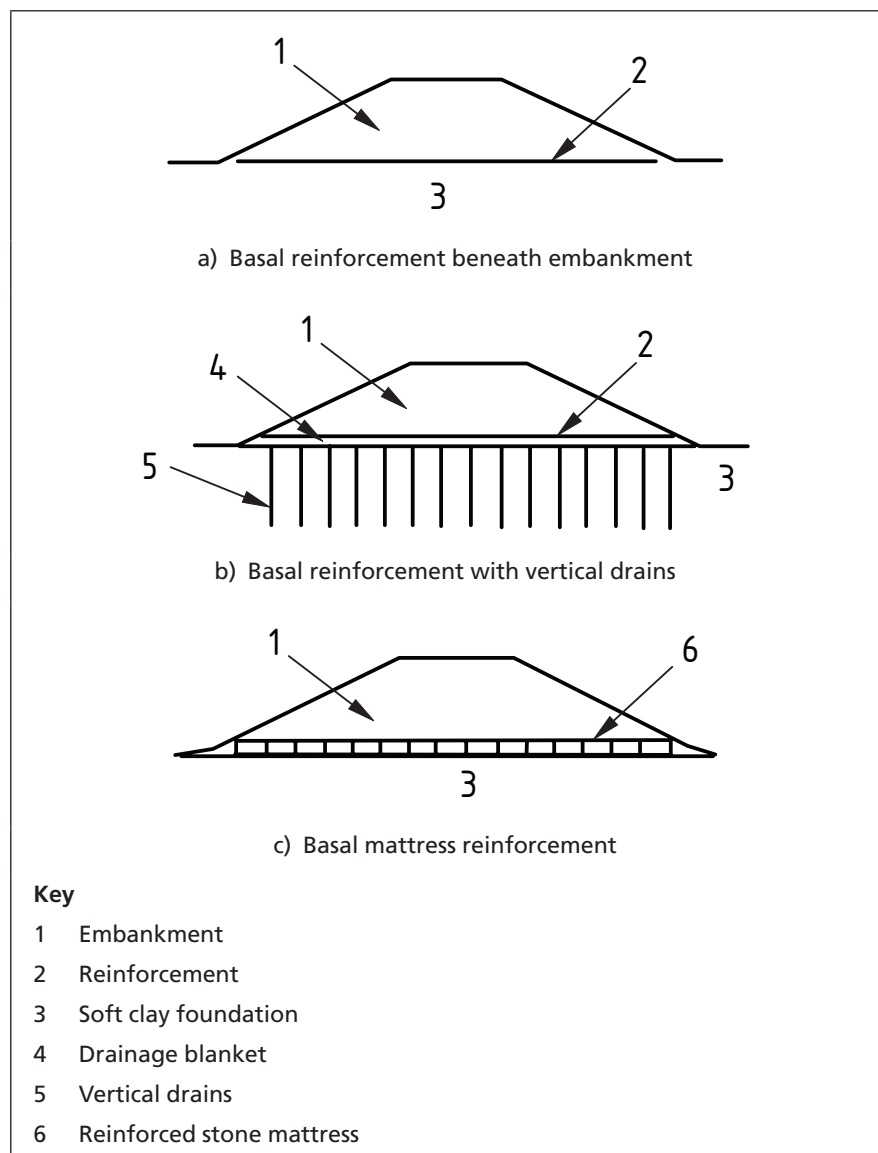
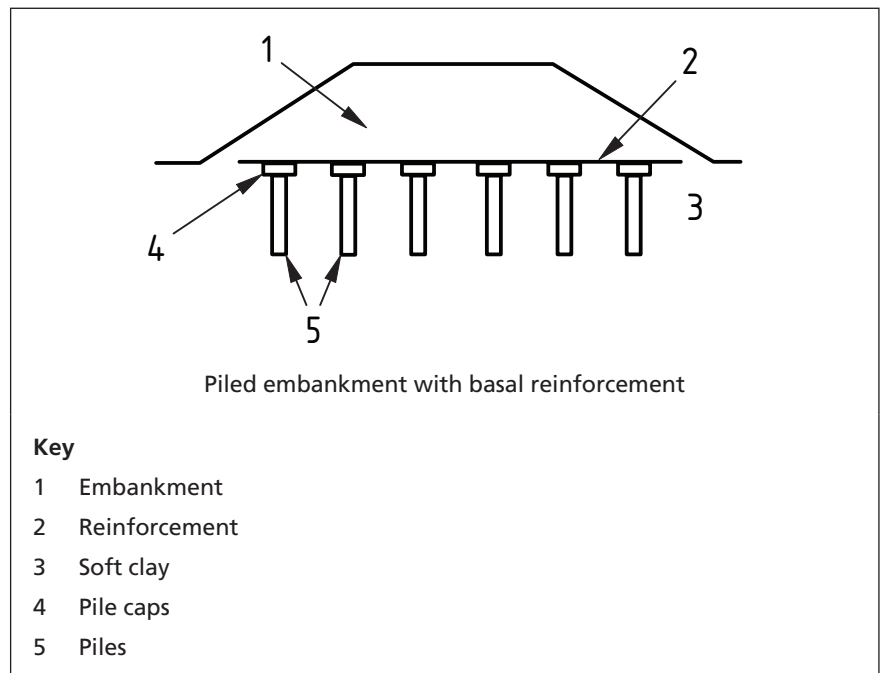


Figure 68 Reinforcement used to control both stability and settlement of embankment



### 8.3.2 Reinforcement used to control embankment stability

#### 8.3.2.1 Basis for design

The stability of an embankment constructed on soft soil is governed mostly by the shearing resistance of the foundation, and the construction of an embankment on soft soil may be limited to a problem of bearing capacity.

Reinforcement may be placed at foundation level to prevent shear failure both in the embankment fill and in the foundation soil, any reduction in differential settlement is of secondary importance.

It is important to consider that the stability of an embankment on soft soil is most critical during construction, because the relatively low permeability of the soft foundation does not permit full consolidation in the normal time scale of construction.

At the end of construction the embankment loading has been applied, but the gain in shearing resistance of the foundation due to consolidation might be insufficient for stability. It should be noted that, these problems can be compounded by quicker/shorter construction periods.

Once consolidation has occurred, the resulting improvement in shearing resistance in the foundation should usually remove the need for the reinforcement to improve stability. Thus during the period between the end of construction and consolidation of the foundation, the fundamental strength requirement of the reinforcement should be that at any instant in time the factored reinforcement design strength equals or exceeds the design load.

**COMMENTARY ON 8.3.2.1**

*Basal reinforcement stabilizes an embankment over soft ground by preventing lateral spreading of the fill, extrusion of the foundation and overall rotational failure. This stabilizing force is generated in the reinforcement by shear stresses transmitted from the foundation soil and fill which place the reinforcement in tension.*

**8.3.2.2 Limit states**

The ultimate limit states that should be considered are as follows:

- a) local stability of the embankment fill [see Figure 69a)];
- b) rotational stability of the embankment [see Figure 69b)];
- c) lateral sliding stability of the embankment fill [see Figure 69c)];
- d) foundation extrusion stability [see Figure 69d)];
- e) overall stability [see Figure 69e)].

The serviceability limit states that should be considered are:

- 1) excessive strain in the reinforcement [see Figure 70a)];
- 2) settlement of the foundation [see Figure 70b)].

*NOTE These limit states are covered in detail in the following subclauses.*

The maximum limit state tensile force  $T_r$  to be resisted by the basal reinforcement should be the greater of:

- i) the maximum tensile force needed to resist the rotational limit state  $T_{ro}$  per metre run (see 8.3.2.5); or
- ii) the sum of the maximum tensile force needed to resist lateral sliding  $T_{ds}$  per metre run (8.3.2.6) and the maximum tensile force needed to resist foundation extrusion  $T_{rf}$ , per metre "run" (8.3.2.7).

That is:  $T_r = T_{ro}$  or  $T_r = T_{ds} + T_{rf}$ , whichever is the greater.

To ensure the ultimate limit state governing reinforcement rupture is not attained over the design life of the reinforcement the following condition should be observed:

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

where

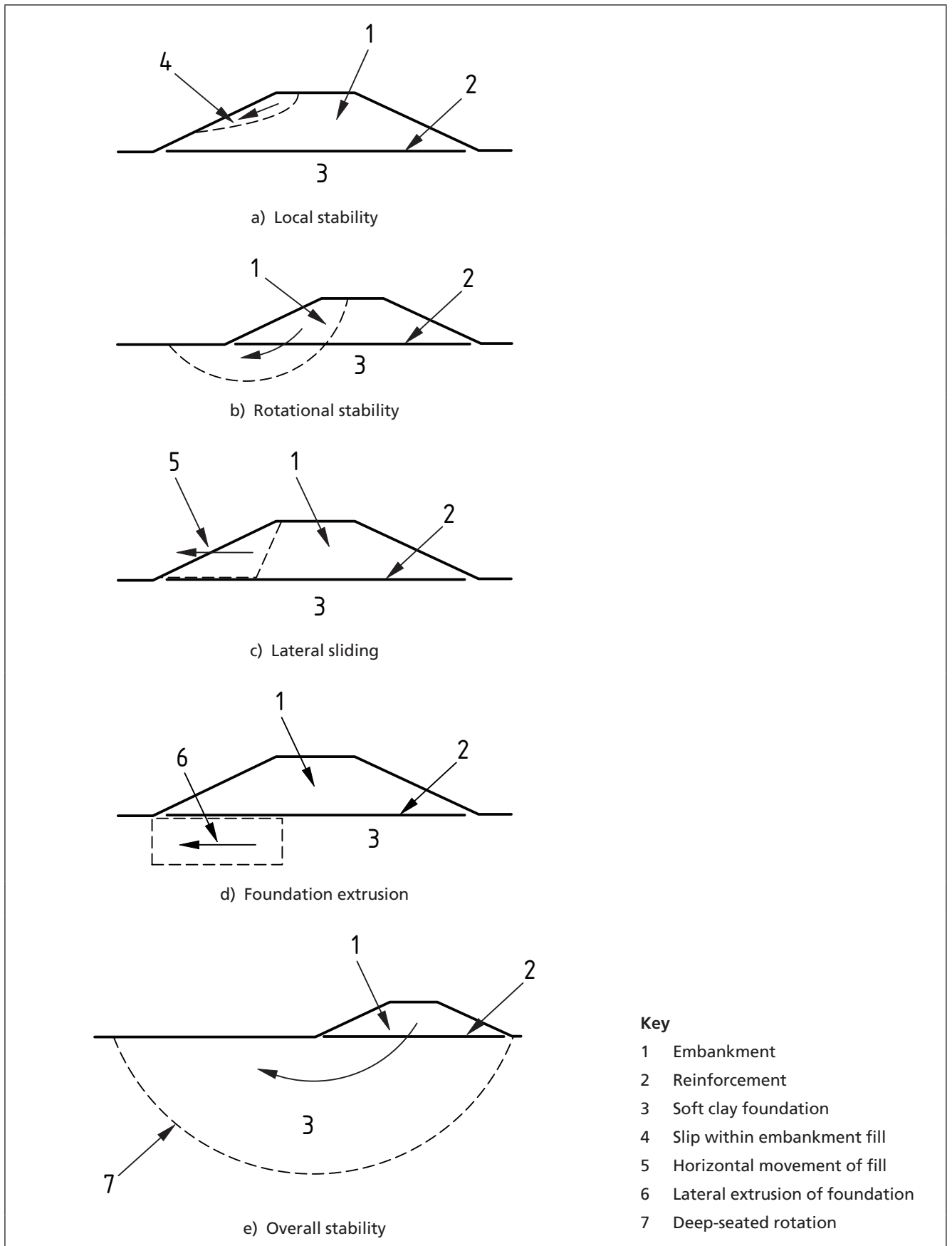
$T_D$  is the design strength of the reinforcement calculated in accordance with 5.3.3;

$f_n$  is the partial factor governing the economic ramifications of failure, see Table 9.

To ensure that the ultimate limit state tensile force can be developed along the basal reinforcement, adequate bond should be ensured between the reinforcement and the adjacent soil. For each of the limit state tensile forces to be determined ( $T_{ro}$ ,  $T_{ds}$  and  $T_{rf}$ ) the associated reinforcement bond should also be checked to ensure the limit state tensile load can be generated in the reinforcement.

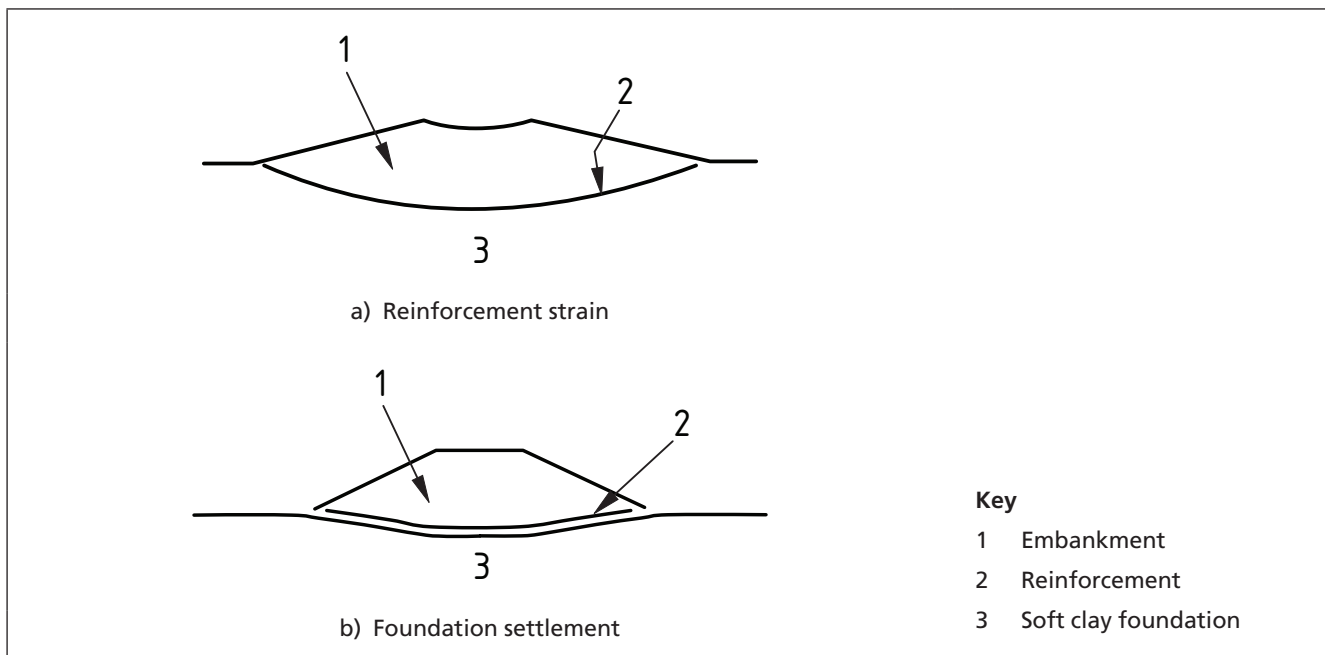
The maximum allowable serviceability limit state strain in the reinforcement  $\varepsilon_{max}$  should be as stated in 8.3.2.11.

Figure 69 Ultimate limit states for basal reinforced embankments



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Figure 70 Serviceability limit states for basal reinforced embankments



### 8.3.2.3 Long term stability

The first stage of the design should normally be to determine an embankment geometry consistent with long term stability using conventional methods (see Taylor [66], Bishop and Morgenstern [67]).

The shear strength of both the fill and the foundation soil may be modelled using appropriately factored effective shear strength parameters  $c'$  and  $\phi'$  with due regard to the long term pore water pressure regimes acting in the fill or foundation soil.

In cases where the foundation soil is very soft, a side slope based on long term stability geometry might be too steep when analysed for short term stability, even with the inclusion of the reinforcement; in this situation the slope should be flattened to satisfy shorter term stability and then re-analysed for the long term condition to ensure that stability conditions are achieved.

### 8.3.2.4 Local stability

The local stability of the embankment sideslope [see Figure 69a)], should be checked as follows:

$$\frac{H}{L_s} \leq \frac{\tan \phi'_{cv}}{f_{ms}}$$

where

$H$  is the height of fill in the embankment;

$L_s$  is the horizontal length of the sideslope of the embankment;

$\phi'_{cv}$  is the large strain angle of friction of the embankment fill under effective stress conditions;

$f_{ms}$  is the partial material factor applied to  $\tan \phi'_{cv}$ , see Table 22.

### 8.3.2.5 Rotational stability

#### 8.3.2.5.1 General

The rotational stability of the embankment [see Figure 69b)] may be analysed by a number of techniques.

*NOTE* The three most common are slip surface analyses, plasticity solutions, and finite element and finite difference techniques.

#### 8.3.2.5.2 Slip surface analyses

Slip surface analysis is the most common technique that may be used to analyse the rotational stability of basal reinforced embankments.

The general principles involved are shown in Figure 71; the reinforcement may be considered to provide an additional restoring moment to enhance overall stability of the embankment.

The procedure should involve a slip surface analysis search along the base of the embankment to determine the profile (locus) of the tensile load in the reinforcement that is necessary to provide an adequate margin of stability [see Figure 71a)].

Care should be exercised in the choice of shape of the potential slip surfaces to account for occurrences of shallow foundation depths and soil layers of varying strengths within the soft foundation soil. The analysis may be carried out using effective stress parameters taking account of pore water pressures [see Figure 71a)], however an analysis based on undrained conditions simplifies the analysis and generally provides a more accurate solution to short term stability. The appropriate undrained strength parameters may be substituted into the relationships contained in Figure 71a). The reinforcement force  $T_{roj}$  needed per metre run at any location  $j$  along the base of the embankment [see Figure 71a)] may be determined from:

$$T_{roj}Y_j = M_{RRj} = M_{Dj} - M_{RSj}$$

where

$Y_j$  is the vertical moment arm for the critical slip surface at location  $j$  along the base of the embankment;

$M_{RRj}$  is the maximum restoring moment due to the reinforcement at location  $j$  along the base of the embankment;

$M_{Dj}$  is the factored maximum disturbing moment at location  $j$  along the base of the embankment;

$M_{RSj}$  is the factored maximum restoring moment due to the soil at location  $j$  along the base of the embankment.

*NOTE 1* No partial material factor on soil unit weight is used in the determination of  $M_{RSj}$ .

A plot of values of  $T_{roj}$  across the base of the embankment yields the locus of forces shown in Figure 71a).

*NOTE 2* The maximum reinforcement force  $T_{ro}$  needed is where  $T_{roj}$  reaches a maximum.

For the majority of embankment geometries it may only be necessary to carry out slip surface analyses on one side of the embankment to arrive at  $T_{ro}$ . However for very low, wide embankments, slip surface analyses should continue beyond the embankment centreline in order to determine  $T_{ro}$ .

Besides the method shown in Figure 71a), the methods of Bishop [68] and Janbu [69] may be modified to determine  $T_{roj}$ .

The reinforcement should achieve an adequate bond with the adjacent soil to ensure the load  $T_{roj}$  can be generated. This bond should be achievable along the reinforcement both within and beyond the potential slip surface [see Figure 71b)]. Within the slip surface:

$$f_n f_p T_{roj} \leq \gamma h \frac{a' \tan \phi'_{cv}}{f_{ms}} L_j + \frac{a'_{bc} c_u}{f_{ms}} L_j$$

where

- $f_n$  is the partial factor governing the economic ramifications of failure, see Table 9;
- $f_p$  is the partial factor for reinforcement pull-out resistance (see Table 22);
- $T_{roj}$  is the reinforcement load per metre "run" needed to maintain stability at location  $j$  along the base of the embankment;
- $\gamma$  is the unit weight of the embankment fill;
- $h$  is the average height of the embankment fill over the reinforcement length  $L_j$ ;
- $a'$  is the interaction coefficient relating the soil/reinforcement bond angle to  $\tan \phi'_{cv}$ ;
- $\phi'_{cv}$  is the large strain angle of friction of the embankment fill under effective stress conditions;
- $f_{ms}$  is the partial materials factor applied to  $\tan \phi'_{cv}$ , and  $c_u$  (see Table 23);
- $L_j$  is the necessary bond length of the reinforcement per metre run within the arc of the slip surface;
- $a'_{bc}$  is the interaction coefficient relating the soil/reinforcement adherence with  $c_u$ ;
- $c_u$  is the undrained shear strength of the soft foundation soil adjacent to the reinforcement.

The reinforcement bond length needed beyond the slip surface may be determined by substituting  $B - L_j$  for  $L_j$  in the above equation, where  $B$  is the total length of reinforcement across the embankment [see Figure 71b)].

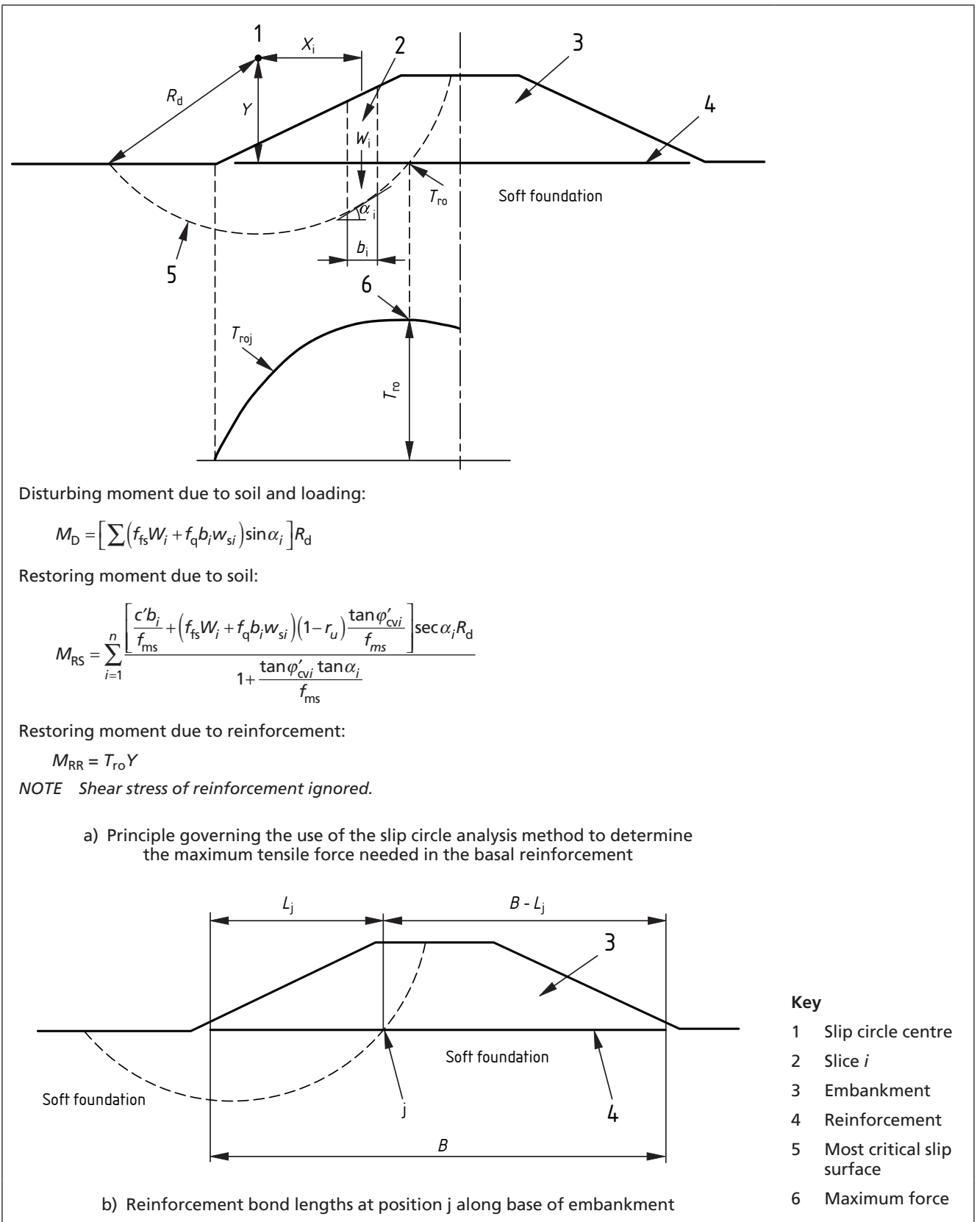
The bond length recommended is dependent on the interaction at the fill/reinforcement and foundation soil interfaces. The reinforcement bond length may be reduced by consideration of a granular regulating layer above and below the reinforcement to improve the bond strength.

### 8.3.2.5.3 Plasticity solutions

Plasticity analyses may be employed to obtain a preliminary design geometry based on foundation shear strength. Analytical procedures have been developed to consider both the case of shear strength increasing linearly with depth, and that of limited foundation depth with constant shear strength, and both cases are included in following subclauses (see Jewell [70]).



Figure 71 Procedure for assessing rotational stability by slip circle analysis



#### 8.3.2.5.4 Finite element and finite difference methods

Where the scale of the project is of such size, or where the engineering properties of the foundation soils involved are particularly complex (e.g. peat soils), a more sophisticated analysis may be warranted.

##### COMMENTARY ON 8.3.2.5.4

*The stability of walls, slopes and embankments may be analysed by a number of techniques. The three most common are slip surface analysis, plasticity solutions and finite elements and finite difference techniques.*

*Slip surface analyses and plasticity theory cannot take into account the effects of embankment settlement on the expected properties of the reinforcement, nor on the performance of the embankment as a whole. The only methods that readily lend themselves to these situations are the finite element and finite difference techniques.*

*Recent advances in computer processing technology, coupled with advances in proprietary finite element software, has made this form of analysis a far more accessible technique over recent years for analysing geotechnical problems.*

For realistic results from the finite element and finite difference methods, both correct modelling procedures and close attention to the appropriate soil properties are essential. It should be noted however, that collapse analysis and large strain behaviour with associated non-linear geometry is very difficult to model with finite elements.

*NOTE Boutrup and Holtz [71], Rowe [72], Jones [73], Potts and Zdravkovic [74] provide further information on the use of finite elements to analyse and design reinforced soil structures and embankments.*

#### 8.3.2.6 Lateral sliding

The lateral sliding stability of the embankment fill [see Figure 69c] should give consideration to any preferential slip surface developed between the fill and the upper surface of the basal reinforcement. The basal reinforcement may be necessary to resist the horizontal outward thrust of the embankment fill.

The load in the reinforcement may be assumed to be a maximum at the edge of the crest of the embankment (see Figure 72). This reinforcement load  $T_{ds}$  may be determined from:

$$T_{ds} = 0.5K_a H(f_{fs}gH + 2f_q w_s)$$

where

- $T_{ds}$  is the tensile load in the reinforcement per metre "run" needed to resist the lateral thrust of the embankment fill;
- $K_a$  is the active earth pressure coefficient [=  $\tan^2(45^\circ - \phi'_{cv}/2)$ ];
- $H$  is the height of the embankment;
- $\gamma$  is the unit weight of the embankment fill;
- $w_s$  is the surcharge intensity on top of the embankment;
- $f_{fs}$  is the partial factor for soil unit weight (see Table 22);
- $f_q$  is the partial load factor for external applied loads (see Table 22).

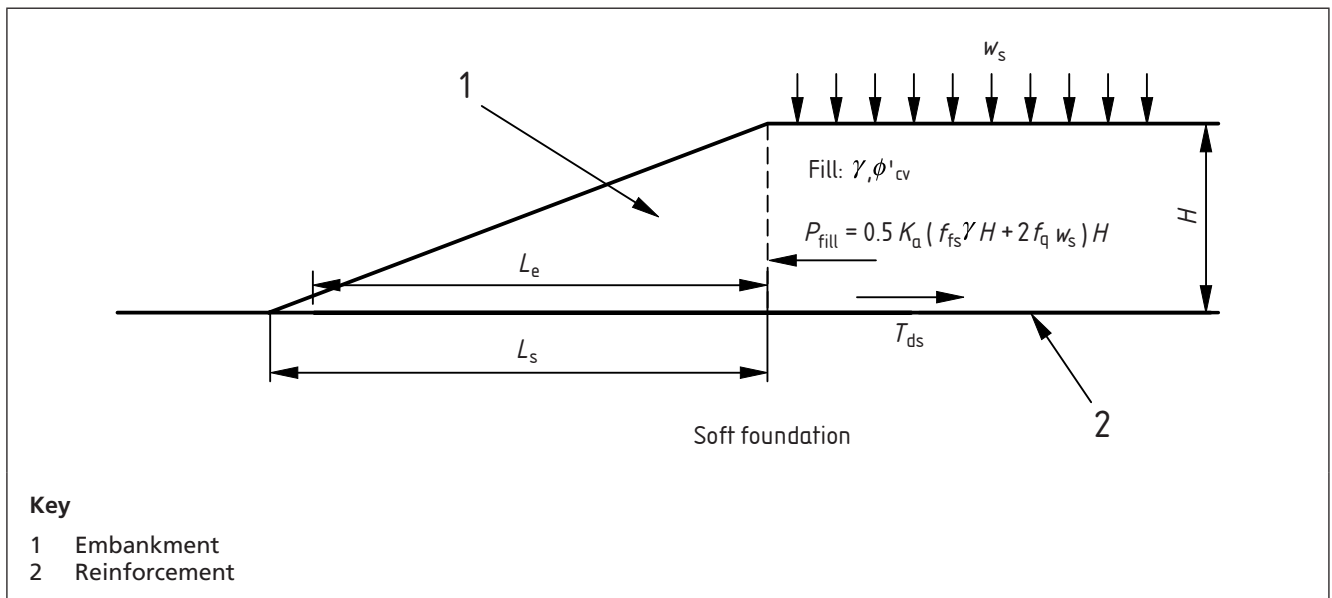
To generate the tensile load  $T_{ds}$  in the reinforcement, the embankment fill should not slide outwards over the reinforcement. To prevent this horizontal sliding the minimum reinforcement bond length  $L_e$  should be:

$$L_e \geq \frac{0.5K_a H (f_{fs} \gamma H + 2f_q w_s) f_s f_n}{\gamma h \frac{a' \tan \phi'_{cv}}{f_{ms}}}$$

where

- $f_s$  is the partial factor for reinforcement sliding resistance (see Table 22);
- $f_n$  is the partial factor governing the economic ramifications of failure (see Table 9);
- $h$  is the average height of embankment fill above the reinforcement length  $L_e$ ;
- $a'$  is the interaction coefficient relating the embankment fill/reinforcement bond angle to  $\tan \phi'_{cv}$ ;
- $\phi'_{cv}$  is the large strain angle of friction of the embankment fill under effective stress conditions;
- $f_{ms}$  is the partial material factor applied to  $\tan \phi'_{cv}$ , see Table 22.

Figure 72 Lateral sliding stability at fill/reinforcement interface



### 8.3.2.7 Foundation extrusion and bearing capacity

It should be noted that the geometry of the embankment induces outward shear stresses within the soft foundation soil, and where the foundation soil is soft and of limited depth the outward shear stresses can induce extrusion of the foundation [see Figure 69d)].

To prevent this extrusion the sideslope length of the embankment  $L_s$  should be great enough to prevent mobilization of these outward shear stresses.

The failure mechanism assumes the lateral extrusion of foundation soil from beneath the embankment (see Figure 73); to prevent this limit state from occurring the outward foundation movement should be limited by developing adequate lateral confinement over a sufficient surface area at the underside of the basal reinforcement. To achieve this two conditions should be satisfied.

- a) First, the overall shearing resistance on the underside of the reinforcement should be sufficient to resist the lateral loads developed in the foundation soil.
- b) Second, the basal reinforcement should have sufficient tensile strength to withstand the tensile loads induced by the shear stress transmitted from the foundation soil (see Jewell [70], Ingold [75]).

To prevent foundation extrusion the following relationship should apply [see Figure 73]:

$$R_{ha} \leq R_{hp} + R_s + R_R$$

where

- $R_{ha}$  is the factored horizontal force causing foundation extrusion;
- $R_{hp}$  is the factored horizontal force due to passive resistance of the foundation;
- $R_s$  is the factored horizontal force due to the shear resistance of the foundation soil at depth  $z_c$ ;
- $R_R$  is the factored horizontal force due to the shear resistance of the foundation soil at the underside of the reinforcement.

A sensitivity analysis using different values of  $z_c$  should be performed to determine the minimum sideslope length  $L_s$  needed to prevent foundation extrusion. The depth of  $z_c$  should be limited to a maximum of twice the height of the embankment.

It is common practice to carry out the analysis using undrained soil parameters; if the soft foundation is of limited depth, and has constant undrained shear strength with depth, then use of the relationships shown in Figure 73b) should be made to enable the minimum necessary sideslope length  $L_s$  to be determined as follows:

$$L_s \geq \frac{\left( f_{fs} \gamma_1 H + f_q w_s - \frac{4c_u}{f_{ms}} \right) z_c}{\frac{(1 + a'_{bc}) c_u}{f_{ms}}}$$

where

- $f_{fs}$  is the partial factor for soil unit weight (see Table 22);
- $f_q$  is the partial load factor for external applied loads (see Table 22);
- $\gamma_1$  is the unit weight of the embankment fill;
- $H$  is the maximum height of the embankment;
- $w_s$  is the surcharge intensity on top of the embankment;
- $c_u$  is the undrained shear strength of the soft foundation layer;
- $f_{ms}$  is the partial material factor applied to  $c_u$  (see Table 22);

- $z_c$  is the depth of the soft foundation layer when the foundation is of limited depth with constant undrained shear strength with depth;
- $a'_{bc}$  is the interaction coefficient relating the soil/reinforcement adherence to  $c_u$ .

Similarly an expression exists for the case of a foundation soil, with shear strength linearly increasing with depth, where the minimum factor of safety is given at a critical depth  $z_c$  below the ground surface, which may be determined by:

$$z_c = \sqrt{\frac{(1 + \alpha'_{bc})c_u n H}{2\rho}}$$

where

- $n$  is the side slope of the embankment;
- $\rho$  is the increase in shear strength per unit depth.

The minimum required sideslope length  $L_s$  may be determined as follows:

$$L_s = \frac{\left[ f_{fs} \gamma_1 H + f_q w_s - \frac{(2c_u + z_c \rho) 2}{f_{ms}} \right] z_c}{\frac{c_u (a_{bc} + 1) + \rho z_c}{f_{ms}}}$$

where

- $f_{fs}$  is the partial factor for soil unit weight, see Table 22;
- $f_q$  is the partial load factor for external applied loads, see Table 22;
- $\gamma_1$  is the unit weight of the embankment fill;
- $H$  is the maximum height of the embankment;
- $w_s$  is the surcharge intensity on top of the embankment;
- $c_u$  is the undrained shear strength of the soft foundation layer;
- $f_{ms}$  is the partial material factor applied to  $c_u$ , see Table 22;
- $z_c$  is the critical depth;
- $a'_{bc}$  is the interaction coefficient relating the soil/reinforcement adherence to  $c_u$ .

The tensile load generated in the basal reinforcement  $T_{rf}$  per metre run due to outward foundation shear stress may be taken from:

$$T_{rf} = \frac{a'_{bc} c_{u0} L_e}{f_{ms}}$$

where

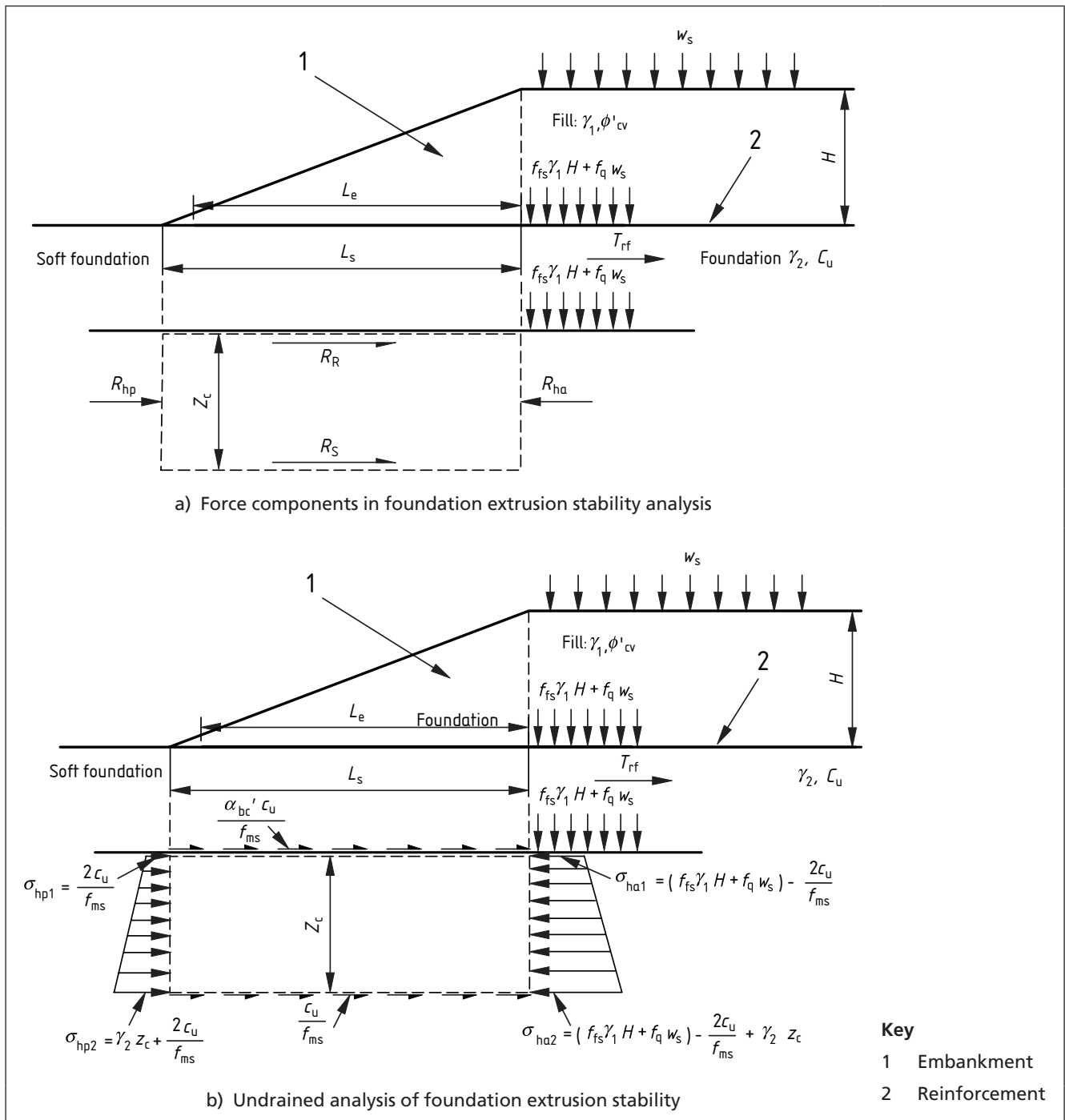
- $L_e$  is the length of the reinforcement required, see Figure 73;
- $c_{u0}$  is the undrained shear strength of the foundation soil at the underside of the reinforcement;
- $f_{ms}$  is the partial material factor applied to  $c_u$ , see Table 22.

Care should be taken in the choice of the value of  $a'_{bc}$  the adherence coefficient at the reinforcement/soft foundation soil interface; the magnitude of  $a'_{bc}$  is related not only to the surface characteristics of the

reinforcement but also to the strain in the reinforcement compared to the strain in the soft foundation.

Strain compatibility between the reinforcement and the soft foundation soil should be satisfied in order to achieve a maximum bond coefficient. This is particularly the case when dealing with sensitive foundation soils where the strain in the reinforcement (SLS case) should not exceed the strain at which the peak undrained shear strength is mobilized in the foundation soil.

Figure 73 Analysis of foundation extrusion stability



Additionally, it should be noted that where the foundation soil has previously experienced mass movement relic shear plains are likely to exist; the soil in this sheared zone is likely to have undergone large strains as a result of this movement and be at a condition close to the residual shear strength of the foundation soil.

In such soils, residual shear strengths should be used in the analysis and the critical depth is often fixed by the lowest level of the sheared zone.

#### 8.3.2.8 Overall stability

For embankments founded on deep deposits of very soft soil overall stability should be checked to ensure deep-seated rotational failures (see Figure 69e, Bjerrum [76]) or block sliding failures cannot occur. Conventional slip surface analyses, using the appropriate partial factors, may be used to examine this potential limit state.

#### 8.3.2.9 Basal mattress reinforced embankments

One form of reinforcement that may be used in embankments is a basal mattress, which is a three dimensional honeycombed structure formed from a series of interlocking cells.

These cells may be fabricated directly on the soft foundation soil from grid or mesh reinforcement and then filled with granular material resulting in a structure, usually one metre deep; the use of a mattress at the base of an embankment is shown schematically in Figure 67c).

A basal mattress reinforcement may be incorporated to interact with the embankment and produce:

- a) a good adhesive interface between the soft foundation and the contained granular fill of the mattress; and
- b) a relatively stiff platform to ensure both an even distribution of load onto the foundation and a more uniform stress field within the soft foundation.

These properties enable the basal mattress to influence the deformation of the soft foundation and hence may be used to mobilize its maximum shear strength and bearing capacity. While the basal mattress may be analysed using the procedure detailed in 8.3.2.3 to 8.3.2.7, a method based on slip line fields for the analysis of foundation stability should generally be used (see Figure 74, Bush et al [77]). The plastic deformation of the soft foundation soil should be examined using the slip line fields and the ultimate bearing capacity calculated. The overburden stresses and the available bearing capacity should then be compared to ensure that equilibrium conditions are satisfied (see Figure 74). The basal mattress may be checked to ensure it can support the tension generated by the outward thrust of the embankment fill in accordance with 8.3.2.6.

The basic assumption may be made that normal slip failure mechanism cannot form due to the strength and stiffness of the cellular mattress; when the thickness of the subsoil is relatively thin compared with the embankment base width, the Prandtl type punching failure cannot take place and plastic flow in the soft foundation layers becomes the critical mechanism.

*NOTE 1 The soft underlying soil is essentially sandwiched between two, rigid horizons giving a situation similar to that of the compression of a block between, rough, rigid parallel platens.*

It should be noted that the basal mattress technique can be particularly effective with relatively thin soft foundation layers where the ratio of embankment width to depth of soft soil is greater than four.

Once the bearing capacity conditions have been satisfied, the tensile loads in the reinforcement forming the basal mattress may be determined by using a method described in Jenner et al [78], which examines the stress condition at the underside of the mattress to calculate the lateral loads that need to be resisted by the reinforcement allowing for the resistance provided by the foundation soil.

*NOTE 2 The correct installation and construction sequence of the cellular mattress is paramount to its performance. The installation procedure for cellular mattresses is described in detail in 8.5.4, see also Cowland and Wong [79], and Payne [80].*

#### 8.3.2.10 Stability in the direction along the embankment

The differential fill height along the embankment should be limited to a minimum during construction of the embankment, but there will inevitably be a need for the basal reinforcement to provide some degree of stability in the longitudinal direction and at the ends of the embankment. The reinforcement force needed should be determined using the same techniques as described previously, taking account of the likely differential fill heights during construction.

#### 8.3.2.11 Allowable reinforcement strains

The strains developed in the reinforcement should not exceed predetermined values derived from serviceability limit state considerations, see Figure 70. Strains in the reinforcement should be determined from the applied loads.

As a general guide, the maximum strain  $\varepsilon_{\max}$  in the basal reinforcement should not exceed 5% for short term applications and 5% to 10% for long term conditions.

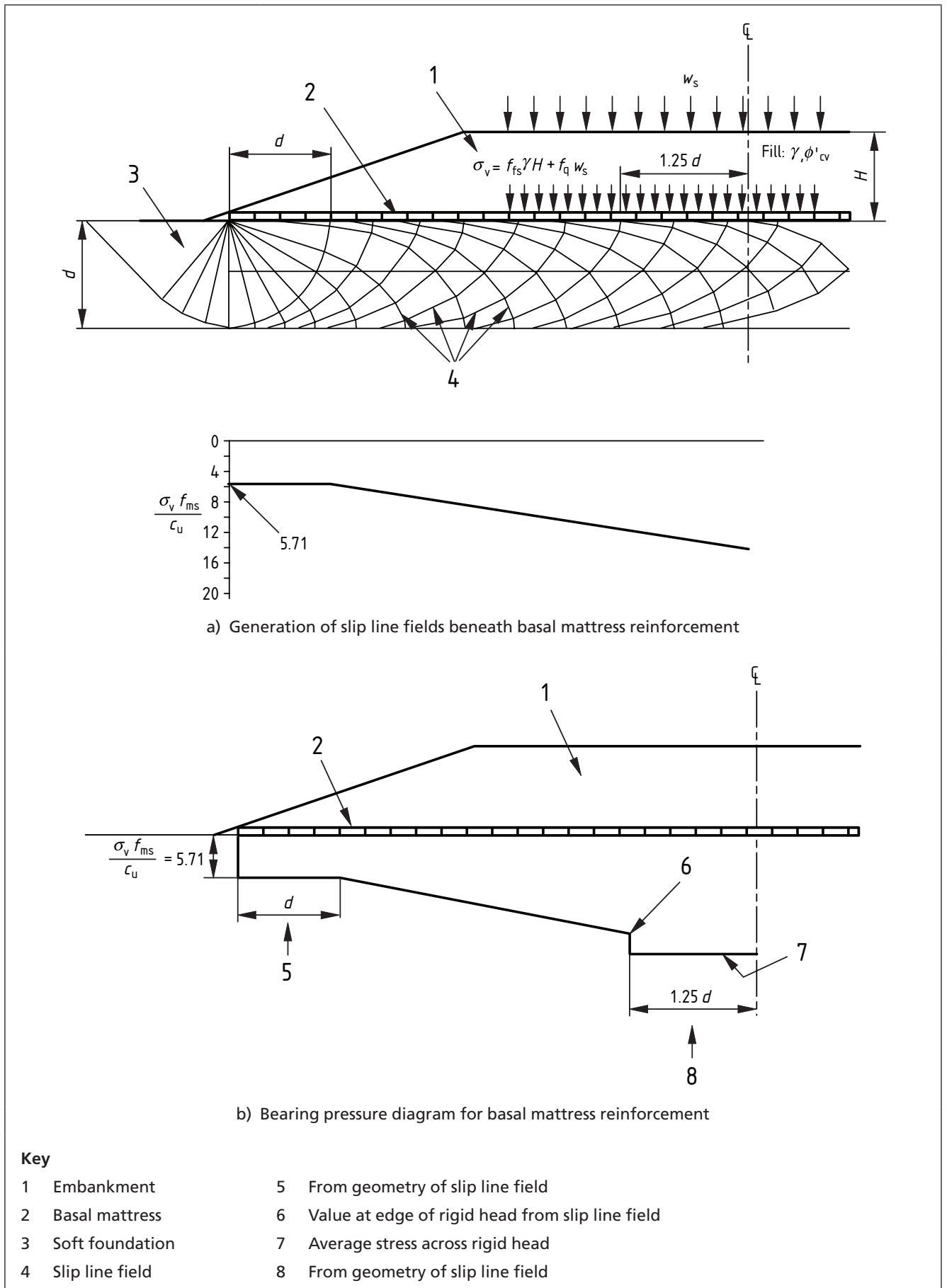
*NOTE Observations have shown that reinforced embankments have performed satisfactorily with these limiting values (see Ingold and Miller [81]). There are other mechanisms such as the construction process that can induce strain in the reinforcement; these strains are difficult to quantify.*

Where basal reinforced embankments are constructed over soft sensitive foundation soils the maximum allowable reinforcement strain may be reduced (typically < 3%) to ensure strain compatibility with the foundation soil, see 8.3.2.7.

The peak shear strength and associated mobilized soil strains of sensitive soils may be determined from standard triaxial test and shear box tests in accordance with 4.2.5.



Figure 74 Ultimate limit state stability analysis for basal mattress reinforcement



### 8.3.2.12 Multiple reinforcement basal layers

The maximum limit state tensile force  $T_r$  to be resisted by the basal reinforcement as outlined in 8.3.2.2 could theoretically be provided by two or more multiple reinforcement layers installed at the base of the proposed embankment.

However, observations of field trials (see Rowe and Li [82]) have indicated that where settlements are relative large ( $> H/25$ ) and varying strength reinforcement materials have been installed in basally reinforced embankments, the stronger reinforcement attracts a disproportionately higher level of the mobilized resisting force. Similarly, when two identical strength layers are incorporated as basal reinforcement, the lowest layer attracts a higher proportion of the resisting force (see Blume et al [83]).

The precise distribution of forces is not fully understood; it is therefore recommended that, where possible, the maximum limit state tensile force  $T_r$  should be provided in one reinforcement layer. Where this is not possible then consideration should be given to the use of multiple layers of equal strength and stiffness to provide design tensile reinforcement in each layer equivalent to:

$$\frac{T_D}{f_n} \geq \Omega T_1 + \Omega T_2 \dots + \Omega T_n$$

where

- $T_D$  is the design strength of the reinforcement calculated in accordance with 5.3.3;
- $f_n$  is the partial factor governing the economic ramifications of failure (see Table 9);
- $\Omega$  is a coefficient dependent on the sequence of the reinforcement layer, for the first/lowest layer  $\Omega = 1$ , second reinforcement layer  $\Omega \leq 1$ , and any subsequent reinforcement layers,  $\Omega \leq 0.5$ .

### 8.3.2.13 Foundation settlement

The presence of basal reinforcement alone does not influence the settlement characteristics of the embankment; thus settlement analyses may be performed using conventional procedures.

Foundation settlement may be assumed to increase the tensile strain and hence load, in the reinforcement. Intermediate and long term settlements may be expected to offset any reduction in reinforcement load due to an increase in embankment stability.

### 8.3.2.14 Basal reinforced embankments with vertical drains

Technical as well as economic benefits may be gained in accelerating the rate of consolidation (and hence the rate of shear strength increase) of soft foundation soils. For example, a higher load level in the reinforcement may be utilized if the time over which the reinforcement is needed is reduced.

A number of methods of accelerating consolidation may be used, including the use of lime columns, stone columns, surcharge, vacuum preloading and vertical drains. The technique using vertical drains is shown in Figure 67b). Ideally, the reinforcement should be placed after the vertical drains have been installed as damage to the reinforcement due to drain installation is avoided. In some instances

however, reinforcement placement prior to drain installation might be inevitable, in which case allowances should be made for loss of tensile strength in the reinforcement due to localized rupture during driving of the drains through the reinforcement. The function of the drainage blanket should not be impaired.

### 8.3.2.15 Temporary working platforms

For most embankments constructed over soft ground some form of temporary working platform may be required to allow access of construction plant, piling equipment, etc. The design of temporary working platforms may incorporate geosynthetic reinforcement, however this is outside the scope of this document and guidance is provided elsewhere (see BR 470 [84], CIRIA SP123 [85]).

## 8.3.3 Reinforcement used as a component to control embankment stability and settlement

### 8.3.3.1 General

Various techniques may be used to increase the effective shear strength of soft foundation soils and to control their post-construction consolidation, including drainage, grouting, piling, and complete soil replacement. The technique of piling enables embankments to be constructed to unrestricted heights at any construction rate (assuming the fill is suitably stable) with subsequent, controlled post-construction settlements. Basal reinforcement may be used to bridge across the tops of pile caps to distribute the load, and maximize the economic benefits of piles installed in soft foundations (see Figure 75).

### 8.3.3.2 Basis for design of piled embankments with basal reinforcement

A range of pile types may be used beneath embankments, including driven or cast in situ concrete piles, timber piles, stone or concrete columns, grout injected stone columns, lime columns, or sand compaction piles.

It may normally be assumed that all of the embankment loading will be transferred through the piles down to a firm stratum. Consequently, the performance of the embankment, and the characteristics of the soft foundation soil, should be considered only with regard to the type of piles used and their installation.

*NOTE Some piling techniques such as stone columns (see Priebe [86]) and geotextile encased granular columns (see Raithel and Kempfert [87]) rely on the soft soil between the columns to provide some lateral support of the column.*

The installation of most pile types should be done from a temporary working platform (see 8.3.2.15). The consolidation and associated effects on the piles, (such as negative skin friction) associated with the installation of a temporary working platform should be taken into account with respect to the allowable load carrying capacity of each pile in the pile group  $Q_p$  (see 8.3.3.4) and in relation to inducing consolidation of the existing ground between the piles below the reinforcement level and consideration of any temporary partial support.

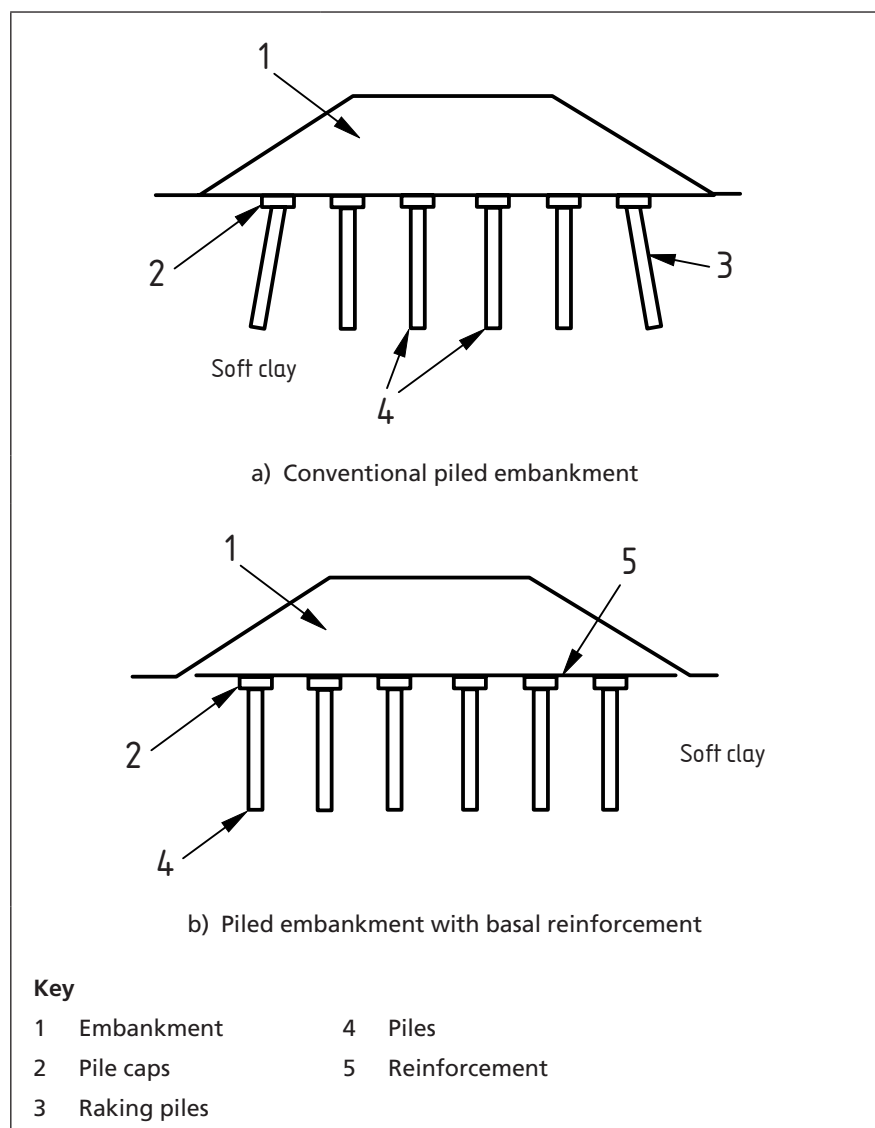
Basal reinforcement spanning across the pile caps may be used to transfer the embankment loading onto the piles.

Basal reinforcement may be used to permit the spacing of the piles to be increased and the size of the pile caps to be reduced. In addition the reinforcement may be used to counteract the horizontal thrust of the embankment fill and the need for raking piles along the extremities of the foundation can be eliminated (see Figure 75, Reid and Buchanan [88], Jones et al [89]).

The subsequent expressions presented in this section are based on the assumption of a square pile arrangement, which is the most commonly used arrangement in practice. The reinforcement should normally be provided by two separate layers generally orientated parallel and orthogonal to the centre line of the embankment or pile alignment.

The design of piles at the base of an embankment and the geosynthetic reinforcement spanning between the piles is often undertaken by separate specialists; however, greater collaboration should be encouraged between the two disciplines since changes in pile spacings, orientation and layout are likely to affect the geosynthetic reinforcement requirement.

Figure 75 Piled embankment configuration



### 8.3.3.3 Limit states

The ultimate limit states that should be considered are:

- pile group capacity [see Figure 76a)];
- pile group extent [see Figure 76b)];
- vertical load shedding onto the pile caps [see Figure 76c)];
- lateral sliding stability of the embankment fill [see Figure 76d)];
- overall stability of the piled embankment [see Figure 76e)].

The serviceability limit states that should be considered are:

- excessive strain in the reinforcement [see Figure 81a)];
- settlement of the piled foundation [see Figure 81b)].

The maximum ultimate limit state tensile load  $T_r$  per metre run, in the basal reinforcement should be as follows.

- a) In the direction along the length of the embankment the maximum tensile load should be the load needed to transfer the vertical embankment loading onto the pile caps  $T_{rp}$  (see 8.3.3.5 and 8.3.3.6).
- b) In the direction across the width of the embankment the maximum tensile load should be the sum of the load needed to transfer the vertical embankment loading onto the pile caps  $T_{rp}$  (see 8.3.3.5 and 8.3.3.6) and the load needed to resist lateral sliding  $T_{ds}$  (see 8.3.3.11).

To ensure the ultimate limit state governing reinforcement rupture is not attained over the design life of the reinforcement the following condition should be observed:

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

where

$T_D$  is the design strength of the reinforcement calculated in accordance with 5.3.3;

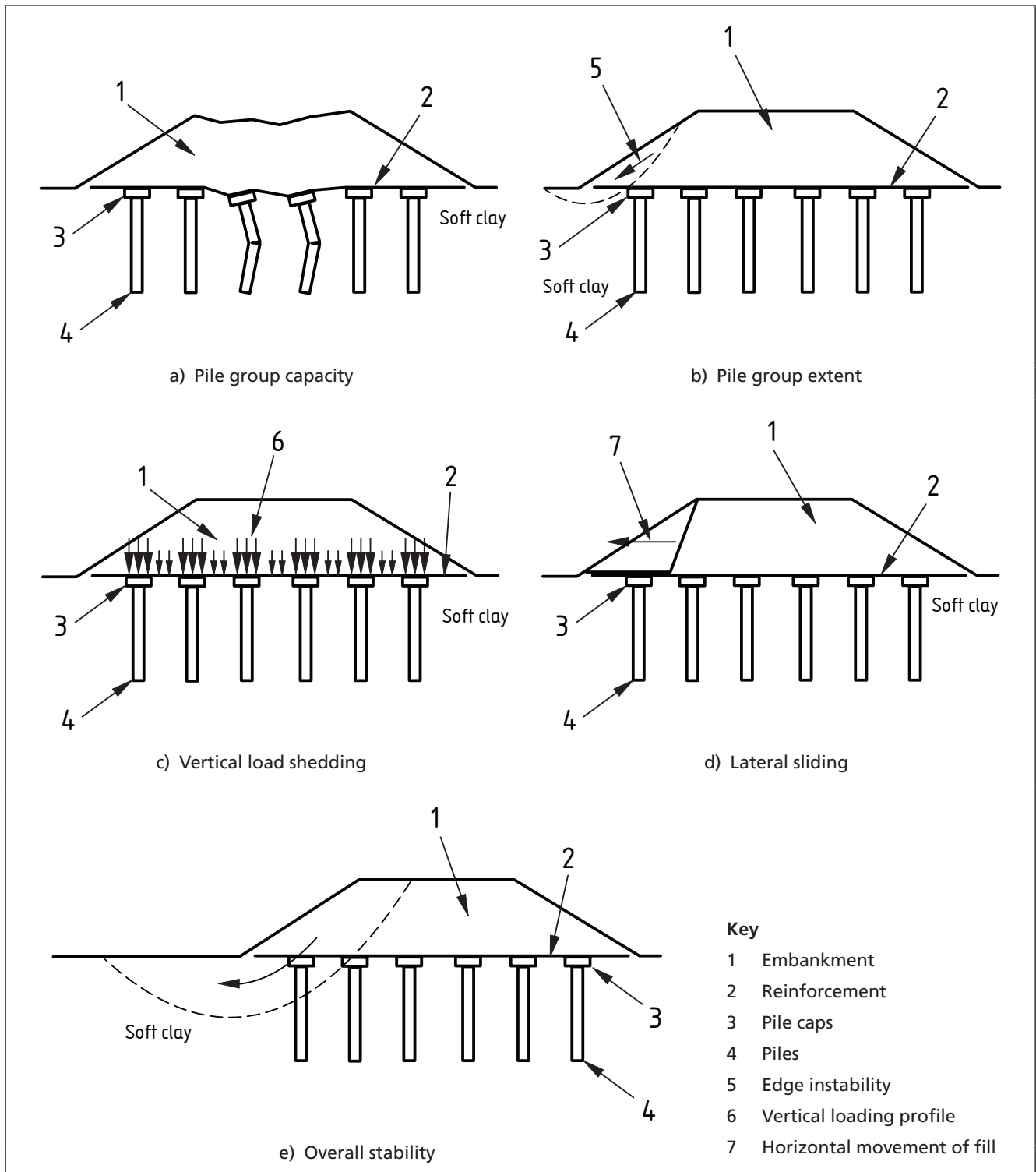
$f_n$  is the partial factor governing the economic ramifications of failure (see Table 9).

To ensure that the ultimate limit state tensile load can be developed in the basal reinforcement adequate bond should be provided between the reinforcement and the adjacent soil.

For each of the limit state tensile forces to be determined, the associated reinforcement bond should also be checked to ensure the limit state tensile load can be generated in the reinforcement, see 8.3.3.11.

The maximum allowable serviceability limit state strain in the reinforcement  $\varepsilon_{\max}$  should be as stated in 8.3.3.13.

Figure 76 Ultimate limit states for basal reinforced piled embankments



### 8.3.3.4 Pile group capacity

The total design resistance of the pile group should be designed according to BS EN 1997-1:2004, and should include any reduction in pile capacity due to group action [see Figure 76a)]. Piles at the base of an embankment may often be designed for a greater allowable settlement

than would otherwise be tolerated with structural piles. If piles are to be installed on a square grid the maximum pile spacing  $s$  should be:

$$s = \sqrt{\frac{Q_p}{(\gamma H + w_s)}}$$

where

- $Q_p$  is the total design resistance of each pile in the pile group;
- $\gamma$  is the unit weight of the embankment fill;
- $H$  is the height of the embankment;
- $w_s$  is the external surcharge loading.

The total design resistance of each pile in the pile group  $Q_p$  should make allowance for the effects of negative skin friction on the piles associated with the installation of a temporary working platform below the geosynthetic reinforcement (see 8.3.2.15). Additionally, the piles should be checked against other failure modes (i.e. transverse loading, buckling, shear, bending, etc.) according to BS EN 1997-1:2004.

### 8.3.3.5 Pile group extent

The piled area should extend to a distance beyond the edge of the shoulder of the embankment to ensure that any differential settlement or instability outside the piled area will not affect the embankment crest [see Figure 76b)]. The edge limit of the outer pile cap should be taken as:

$$L_p = H(n - \tan \theta_p)$$

where

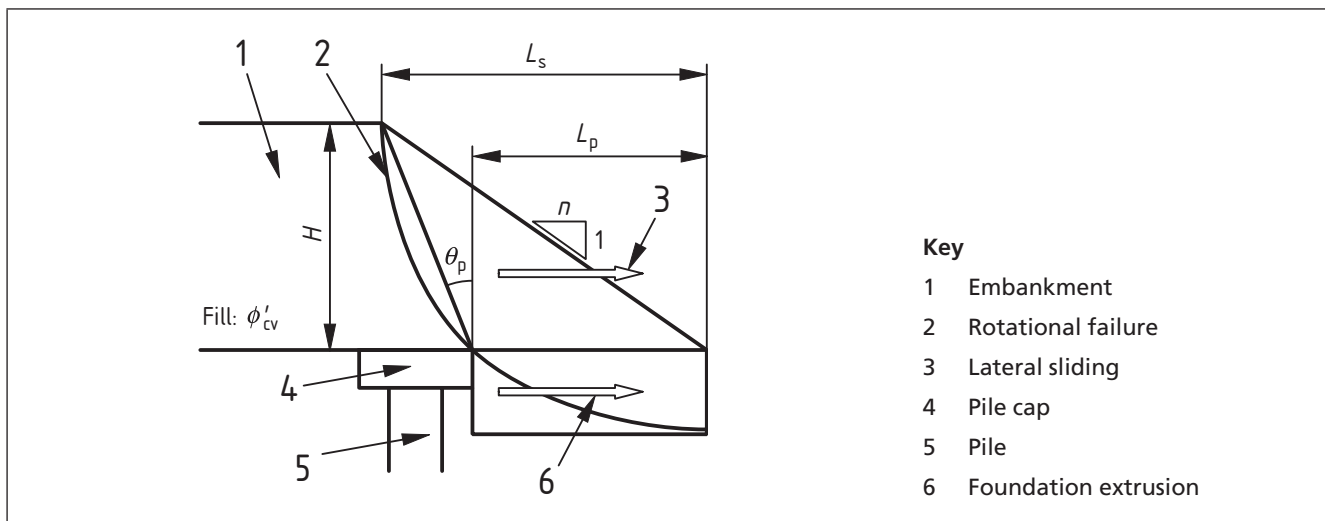
- $L_p$  is the horizontal distance between outer edge of pile cap and toe of embankment (see Figure 77);
- $H$  is the height of the embankment;
- $n$  is the sideslope of an embankment;
- $\theta_p$  is the angle to the vertical between the outer edge of the outside pile cap and the shoulder of the embankment (see Figure 77).

Where  $\phi'_{cv}$  describes the embankment fill,

$$\theta_p = (45^\circ - \phi'_{cv}/2)$$

The edge limit of the outer pile given by  $L_p$  should be considered the minimum distance across the embankment that the piles ought to extend. It may be desirable to extend the piled area to control settlements or instability of the embankment side slopes. The area beyond the edge pile should be checked against rotational failure (8.3.2.5) lateral sliding (8.3.2.6) and foundation extrusion (8.3.2.7) in accordance with the methods described previously. Additionally, consideration should be given to the effects of consolidation of the ground in the un-piled zone beyond the outer edge of the pile caps and to the effects of the earthworks programme, construction sequencing and temporary traffic and surcharges adjacent to the piled zone, and their potential influence on the outer row of piles.

Figure 77 Outer limit of pile caps



### 8.3.3.6 Vertical load shedding

The vertical embankment loads should be transferred onto the pile caps [see Figure 76c)].

To ensure localized differential deformations cannot occur at the surface of embankments (which can be a problem with shallow embankments) it is recommended that the relationship between embankment height and pile cap spacing be maintained to:

$$H \geq 0.7(s - a)$$

where

- $a$  is the size of the pile caps (assuming full support can be generated at the edges of the caps);
- $s$  is the spacing between adjacent piles;
- $H$  is the height of the embankment.

Where circular pile caps are to be used, the diameter of the pile cap should be reduced to produce an effective pile cap width  $a_{\text{equ}}$ , which should be used in the subsequent formulation:

$$a_{\text{equ}} = \sqrt{\frac{\pi D^2}{4}};$$

$$a_{\text{equ}} = 0.886D$$

Additionally, measures should be considered to guard against pile cap/reinforcement abrasion such as chamfering or rounding the edges of the pile caps or providing an additional sand, or regulating layer, or geosynthetic cushioning/protection layer at the pile cap/reinforcement interface. Raising the geosynthetic above the level of the pile cap will change the geometry, which might alter tensions in the reinforcement. Where a layer of sand is provided as a cushion a new, longer span has to be determined. The spacing of the piles will not change but the size of the pile cap should be reduced by the height of the sand cushion.

Because of the significant differences in deformation characteristics which exist between the piles and the surrounding soft foundation soil, the vertical stress distribution across the base of the embankment



should be assumed to be non-uniform. It should also be noted that soil arching between adjacent pile caps induces greater vertical stresses on the pile caps than on the surrounding foundation soil.

### 8.3.3.7 Calculation of distributed load on reinforcement

#### COMMENTARY ON 8.3.3.7

The arching coefficients in 8.3.3.7.1 are appropriate for the 3-dimensional analysis of pile embankments. However, the Marston formula was originally derived from plane strain tests on flexible culverts under high embankments. The phenomenon of soil arching has been studied using a variety of physical, analytical and numerical models to try to gain a better understanding and quantify the load acting across the reinforcement and distributing directly to the pile caps (see Stewart and Filz [90], Kempton et al [91], Rogbeck et al [92], Horgan and Sarsby [93], Alexiew [94], Love and Milligan [95], Britton and Naughton [96], Ellis and Aslam [97,98]). The most commonly used of these methods is based on the work of Hewlett and Randolph [99] and is described in 8.3.3.7.2. In some circumstances the use of finite element or finite difference methods may be warranted. Guidance in this connection is given in 8.3.2.5.4.

Determination of the distributed load is dependent on the clear span between the pile caps. The pile spacing and cap size used in determination of the distributed load has to take into consideration construction position tolerances.

#### 8.3.3.7.1 Distributed load on reinforcement calculated using Marston's formula

The ratio of the vertical stress exerted on top of the pile caps to the average vertical stress at the base of the embankment ( $p'_c / \sigma'_v$ ) may be estimated by use of Marston's formula for positive projecting subsurface conduits (see Spangler and Hardy [100], John [101]):

$$\frac{p'_c}{\sigma'_v} = \left[ \frac{C_c a}{H} \right]^2$$

where

- $p'_c$  is the vertical stress on the pile caps;
- $\sigma'_v = (f_{fs}\gamma H + f_q w_s)$  and is the factored average vertical stress at the base of the embankment;
- $\gamma$  is unit weight of the embankment fill;
- $H$  is the height of the embankment;
- $w_s$  is the uniformly distributed surcharge loading;
- $a$  is the width of the pile caps (or  $a_{\text{equ}}$  for circular the pile caps);
- $C_c$  is the arching coefficient.

Values for the arching coefficient  $C_c$  may be taken from Table 23.

Table 23 Arching coefficient  $C_c$  for basal reinforced piled embankments

Pile arrangement	Arching coefficient
End-bearing piles (unyielding) <sup>A)</sup>	$C_c = 1.95H/a - 0.18$
Friction and other piles (normal) <sup>A)</sup>	$C_c = 1.5H/a - 0.07$

<sup>A)</sup> See A Guide to design loadings for buried rigid pipes [102].

The distributed load  $W_T$  carried by the reinforcement between adjacent pile caps may be determined from:

For  $H > 1.4 (s - a)$ ;

$$W_T = \frac{1.4s f_{fs} \gamma (s - a)}{s^2 - a^2} \left[ s^2 - a^2 (p'_c / \sigma'_v) \right]$$

For  $0.7(s - a) \leq H \leq 1.4 (s - a)$ ;

$$W_T = \frac{s (f_{fs} \gamma H + f_q W_s)}{s^2 - a^2} \left[ s^2 - a^2 (p'_c / \sigma'_v) \right]$$

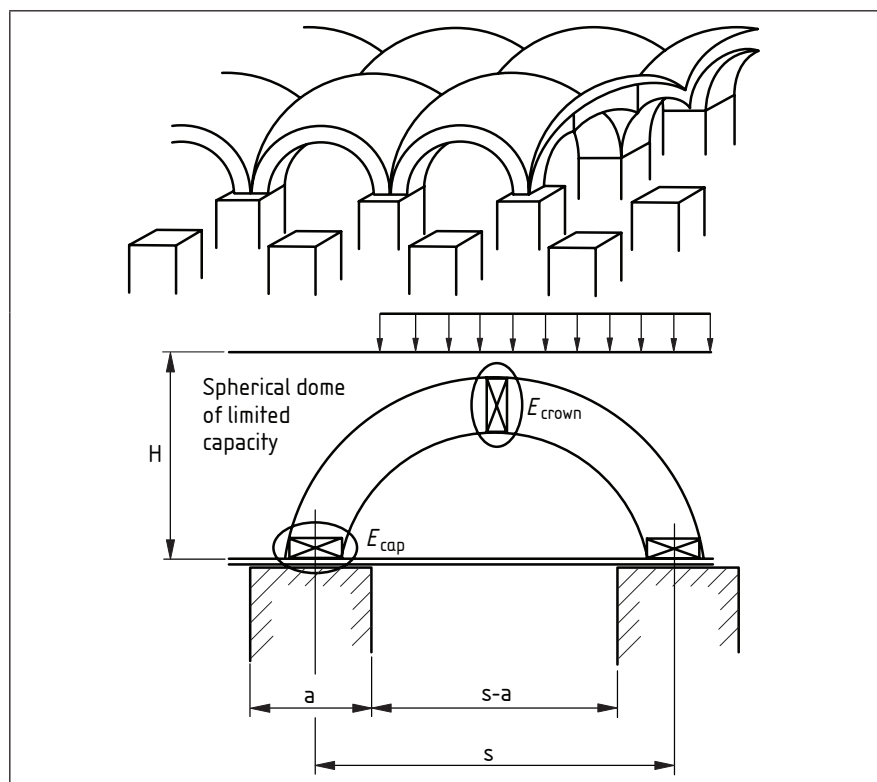
**NOTE** The distributed load, acting across the longitudinal and transverse reinforcement layers might not be equal and is dependant on several variables including the longitudinal and transverse pile spacings.

### 8.3.3.7.2 Distributed load on reinforcement calculated using the Hewlett and Randolph method

An alternative theoretical solution which may be used to determine the vertical load acting across the reinforcement was presented by Hewlett and Randolph [99]; this was based on the observed mechanism from model tests and considers a series of hemispherical domes.

The theory determines the arching efficiency  $E$  as the proportion of the embankment weight carried by the piles, hence the proportion of the embankment weight carried by the geosynthetic reinforcement may be determined  $(1 - E)$ . It should be noted that the system was shown to fail at one of two critical locations, either at the crown  $E_{\text{crown}}$  of the arch or the pile cap  $E_{\text{cap}}$  (see Figure 78). Generally for low embankment heights (relative to the pile spacing) arching efficiency may be assumed to govern the design, and as the embankment height increase pile cap arching efficiency may be assumed to govern.

Figure 78 Arching theory according to Hewlett and Randolph



The proportion of embankment weight supported directly by the piles may be determined from.

$$E_{\text{crown}} = 1 - \frac{(s^2 - a^2)}{s^2 \gamma H} \left[ \sigma_i + \gamma(s-a)/\sqrt{2} \right]$$

where

- $s$  is the pile spacing between adjacent piles;
- $\sigma_i$  is the pressure acting at the under surface of the hemispherical dome;
- $\gamma(s-a)/\sqrt{2}$  is the weight of the soil beneath the hemispherical dome;
- $\gamma$  is unit weight of the embankment fill;
- $H$  is the height of the embankment;
- $a$  is the size (or equivalent size) of the pile caps;

Hence  $E_{\text{crown}}$  may be determined from:

$$E_{\text{crown}} = 1 - \left[ 1 - \left( \frac{a}{s} \right)^2 \right] (A - AB + C)$$

where  $A$ ,  $B$  and  $C$  are calculation coefficients given by:

$$A = \left[ 1 - \left( \frac{a}{s} \right)^2 \right]^{2(K_p - 1)}, \quad B = \frac{s}{\sqrt{2}H} \left( \frac{2K_p - 2}{2K_p - 3} \right) \quad \text{and} \quad C = \frac{s-a}{\sqrt{2}H} \left( \frac{2K_p - 2}{2K_p - 3} \right)$$

where

$$K_p = \frac{1 + \sin(\phi'_p)}{1 - \sin(\phi'_p)}$$

Arching efficiency at the pile cap  $E_{\text{cap}}$  may be taken as:

$$E_{\text{cap}} = \frac{\beta}{1 + \beta}$$

where  $\beta$  is a coefficient given by:

$$\beta = \frac{2K_p}{(K_p + 1) \left( 1 + \frac{a}{s} \right)} \left[ \left( 1 - \frac{a}{s} \right)^{-K_p} - \left( 1 + K_p \frac{a}{s} \right) \right]$$

The minimum pile load arching efficiency  $E_{\text{min}}$ , the minimum proportion of embankment loading acting on the piles should be used in the subsequent formulation to determine the maximum distributed load  $W_T$  carried by the reinforcement between adjacent pile caps:

$$W_T = \frac{s(f_{fs}\gamma H + f_q W_s)}{(s^2 - a^2)} (1 - E_{\text{min}}) s^2$$

where

$E_{\text{min}}$  is the minimum of  $E_{\text{crown}}$  and  $E_{\text{cap}}$ .

### 8.3.3.8 Minimum distributed load acting across reinforcement

A precise theory that describes the phenomenon of soil arching, does not currently exist; dynamic and cyclic loading effects on the development of the soil arching/stress redistribution may obviously

warrant further work. The effects of creep strain and strain softening of granular soils subjected to cyclic loading is well documented (see Festag and Katzenbach [103]), and it may be envisaged that such cyclical loading conditions would cause further stress redistribution at the base of the embankment increasing the embankment load carried directly by the reinforcement and a reduction in the degree of vertical stress distributed directly to the pile caps. This change in stress concentrations due to cyclic loads has been observed in monitored shallow full scale test embankments (see van Eekelen et al [104]).

The minimum distributed load acting across the reinforcement may be assumed to be dependent on the area coverage ratio of the pile caps relative to the pile spacing ( $a^2/s^2$ ) and the relative height of the embankment relating to the clear span [ $H/(s - a)$ ] between the piles. Irrespective of the arching theory, pile layout and embankment geometry, the geosynthetic reinforcement should be designed to carry at least a practical minimum proportion of the embankment loading equivalent to 15%.

The minimum distributed load  $W_{Tmin}$  carried by the reinforcement between adjacent pile caps may be assumed to be equivalent to:

$$W_{Tmin} = s0.15(f_{fs}\gamma H + f_q w_s)$$

### 8.3.3.9 Tension in the reinforcement

Once the distributed load  $W_T$  acting across the reinforcement is determined, then for an extensible reinforcement the tensile load  $T_{rp}$  per metre run, generated in the reinforcement resulting from the distributed load  $W_T$  should be taken as (see also Leonard [105]):

$$T_{rp} = \frac{W_T(s-a)}{2a} \sqrt{1 + \frac{1}{6\varepsilon}}$$

where

$T_{rp}$  is the tensile load in the reinforcement, see Figure 79;

$\varepsilon$  is the strain in the reinforcement.

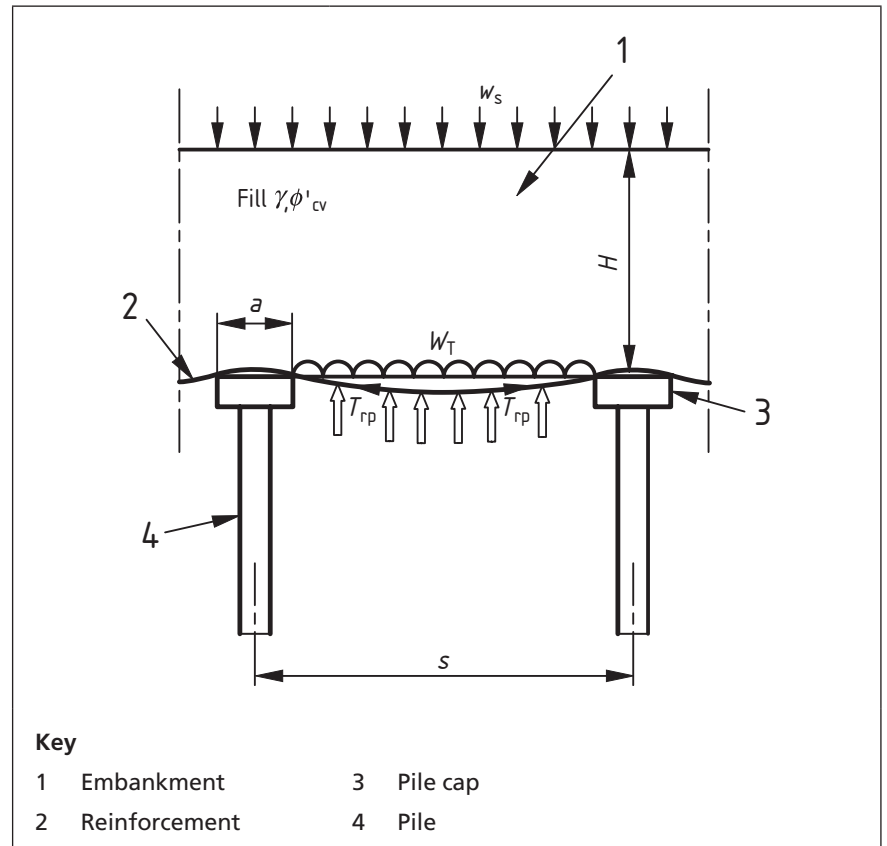
The above equation has two unknowns  $T_{rp}$  and  $\varepsilon$ , and may be solved for  $T_{rp}$  by taking into account the maximum allowable strain in the reinforcement (see 8.3.2.2) and by an understanding of the load/strain characteristics of the reinforcement at different load levels.

The tensile load  $T_{rp}$  may be assumed to develop as the reinforcement deforms under the weight of the embankment; this normally occurs during embankment construction but in situations where the reinforcement cannot deform during construction the reinforcement will not carry the applied loads until the foundation settles.

When dealing with relatively shallow embankments [ $h < 0.7(s - a)$ ], it may be prudent to limit the allowable design strain to provide a maximum mid span deflection of the supporting reinforcement (see 8.3.2.11). It should also be ensured that most of the design strain is mobilized during the construction period and that any post construction reinforcement strain and associated deflections at the surface of the embankment are minimized.

The above equation for  $T_{rp}$  may be used for those reinforcements that can undergo deformation during loading, i.e. extensible reinforcements (e.g. polymeric). For inextensible reinforcements (e.g. metallic) alternative relationships should be used to determine their recommended tensile strength.

Figure 79 Variables used in determination of  $T_{rp}$



### 8.3.3.10 Partial support

The basic assumption may be made that the entire load from the overlying embankment is either taken directly by the piles or distributed to the piles by the reinforcement spanning between them. In reality, the soil between the piles will need to undergo some initial degree of consolidation, either relating to an increase in stress from the installation of the temporary working platform (see 8.3.2.15) or as a result of initial fill placement (see Figure 79).

Some design approaches for reinforced embankments consider the contribution of the partial support offered from the soft ground between piles and granular columns (see Stewart and Filz [90], Kempfert et al [106], Russell et al [107], Jenner et al [108], Colin [109]) whereby a net distributed load is considered to act across the reinforcement  $W_{Tnet}$ . However the consideration of long term partial support offered by the ground between piles should not be used for normal design, since the effective stress conditions acting on the existing ground can alter irrespective of the embankment loading, e.g. seasonal fluctuations in the ground water level can increase the effective stress on the in-situ soil resulting in further consolidation and loss of contact with the underside of the reinforcement.

### 8.3.3.11 Lateral sliding

The reinforcement should resist the horizontal force due to lateral sliding [see Figure 76d)]. This reinforcement tensile load should be generated at a strain compatible with allowable lateral pile movements thereby eliminating the need for raking piles. The reinforcement tensile load  $T_{ds}$  needed to resist the outward thrust of the embankment is (in accordance with Figure 80).

Whilst greater collaboration with the pile designer should be encouraged, the design of laterally loaded piles is outside the scope of the document and reference should be made elsewhere (see Elson [110] and BS EN 1997-1:2004]). However, the piles should be designed to resist a minimum lateral load equivalent to 10% of  $T_{ds}$  multiplied by the longitudinal pile spacing, distributed proportionally between piles under the sloping edge of the embankment.

The reinforcement tensile load  $T_{ds}$  needed to resist the outward thrust of the embankment may be taken as:

$$T_{ds} = 0.5K_a(f_{fs}\gamma H + 2f_q w_s)H$$

where

- $T_{ds}$  is the tensile load in the reinforcement per metre run needed to resist the lateral thrust of the embankment fill;
- $K_a$  is the active earth pressure coefficient [ $= \tan^2(45^\circ - \phi_{cv}/2)$ ];
- $H$  is the height of the embankment;
- $\gamma$  is the unit weight of the embankment fill;
- $w_s$  is the surcharge intensity on top of the embankment;
- $f_{fs}$  is the partial load factor for soil unit weight (see Table 22);
- $f_q$  is the partial load factor for external applied loads (see Table 22).

To generate the tensile load  $T_{ds}$  in the reinforcement the embankment fill should not slide outwards over the reinforcement [see Figure 76d)]. To prevent this horizontal sliding, the minimum reinforcement bond length  $L_e$  should be:

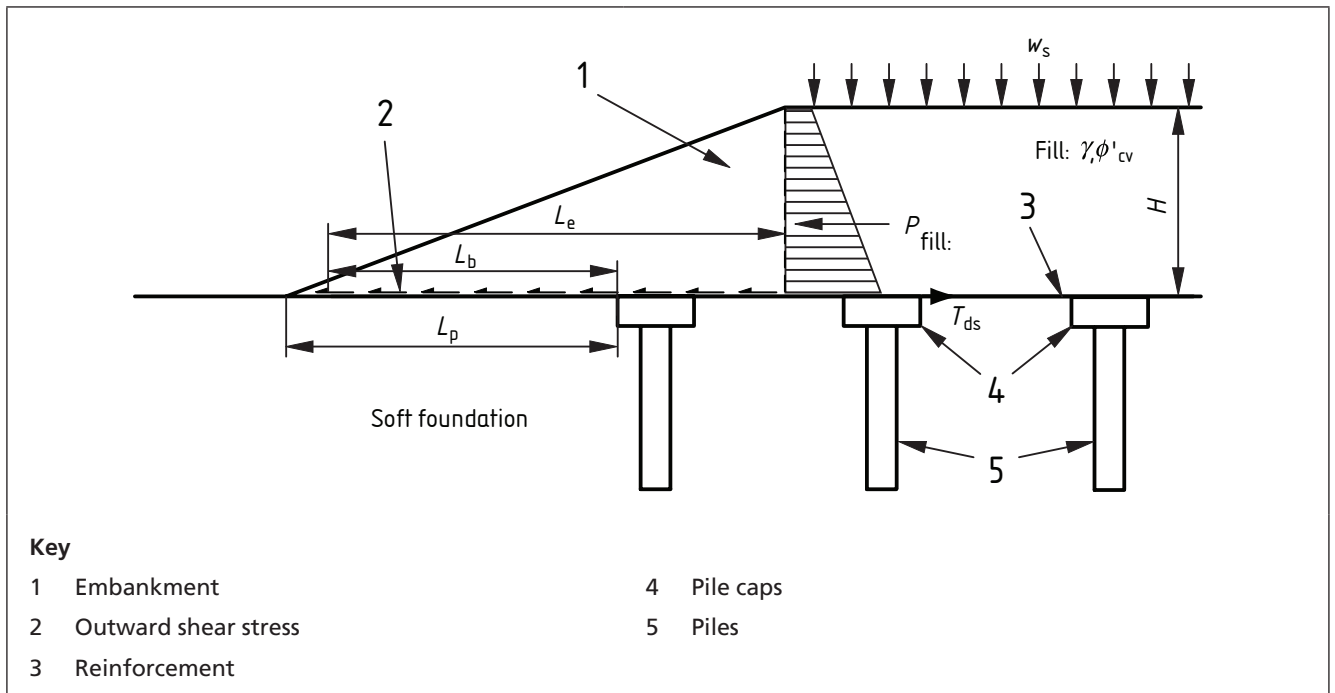
$$L_e \geq \frac{0.5K_a H (f_{fs}\gamma H + 2f_q w_s) f_s f_n}{\gamma h \frac{a' \tan \phi'_{cv}}{f_{ms}}}$$

where

- $f_s$  is the partial factor for reinforcement sliding resistance (see Table 22);
- $f_n$  is the partial factor governing the economic ramifications of failure (see Table 9);
- $h$  is the average height of embankment fill above the reinforcement length  $L_e$ ;
- $a'_1$  is the interaction coefficient relating the soil/reinforcement bond angle to  $\tan \phi'_{cv}$ ;
- $\phi'_{cv}$  is the large strain angle of friction of the embankment fill under effective stress conditions;
- $f_{ms}$  is the partial material factor applied to  $\tan \phi'_{cv}$  (see Table 22).

In addition the local stability of the embankment fill should be checked (see 8.3.2.4).

Figure 80 Lateral sliding at fill/reinforcement interface



### 8.3.3.12 Reinforcement bond

The reinforcement should achieve an adequate bond with the adjacent soil at the extremities of the piled area; this is to ensure that the maximum limit state tensile loads can be generated (across the width and along the length of the embankment) between the outer two rows of piles. Across the width of the embankment the reinforcement should extend a minimum distance beyond the outer row of piles given by:

$$L_b \geq \frac{f_n f_p (T_{rp} + T_{ds})}{\gamma h \left( \frac{a'_1 \tan \phi'_{cv1}}{f_{ms}} + \frac{a'_2 \tan \phi'_{cv2}}{f_{ms}} \right)}$$

where:

- $L_b$  is the reinforcement bond length needed beyond the outer row of piles across the width of the embankment (see Figure 80);
- $f_n$  is the partial factor governing the economic ramifications of failure (see Table 9);
- $f_p$  is the partial factor applied to the pull-out resistance of the reinforcement (see Table 22);
- $T_{rp}$  is the factored tensile load determined in accordance with 8.3.3.9;
- $T_{ds}$  is the factored tensile load determined in accordance with 8.3.3.11;
- $h$  is the average height of fill over the bond length of the reinforcement;
- $\gamma$  is the unit weight of the embankment fill;
- $a'_1$  is the interaction coefficient relating the soil/reinforcement bond angle to  $\tan \phi'_{cv1}$  on one side of the reinforcement;

$a'_2$  is the interaction coefficient relating the soil/reinforcement bond angle to  $\tan\phi'_{cv2}$  on the opposite side of the reinforcement;

$f_{ms}$  is the partial material factor applied to  $\tan\phi'_{cv}$  (see Table 22).

Along the length of the embankment, the reinforcement should extend a minimum distance beyond the outer row of piles given by:

$$L_b \geq \frac{f_n f_p T_{rp}}{\gamma h \left( \frac{a'_1 \tan\phi'_{cv1}}{f_{ms}} + \frac{a'_2 \tan\phi'_{cv2}}{f_{ms}} \right)}$$

Depending on the geometry of the embankment, it may be difficult to achieve an adequate bond length at the extremity of the piles by maintaining the reinforcement in a horizontal alignment as depicted in Figure 80. One solution that may be considered is to use a row of gabions [see Figure 81a)] as a thrust block along the top of the outer row of piles (Young and Ritty [111]. The reinforcement may be extended around the row of gabions and returned into the embankment fill to develop the necessary bond length. Another detail that may be considered [see Figure 81b)] is the inclusion of a small periphery trench just beyond the edge piles, running parallel to the centreline of the embankment; the trench is typically only as deep as the piling mat or pile cap depth. The reinforcement can be extended into the trench and when backfilled, will return into the embankment fill to develop the necessary bond.

### 8.3.3.13 Overall stability

The overall stability of the piled embankment structure should be analysed by conventional stability methods, e.g. BS EN 1997-1:2004, with modifications to take into account the presence of the piled foundation and the basal reinforcement [see Figure 76e)]. The analysis may be performed using effective stress parameters taking account of pore water pressures (see Figure 82), however an analysis of short term stability should assume undrained conditions.

To ensure stability, the following relationship should be satisfied for all locations along the base of the embankment:

$$M_D \leq M_{RS} + M_{RP} + M_{RR}$$

where

$M_D$  is factored disturbing moment at all locations along the base of the embankment (see Figure 82);

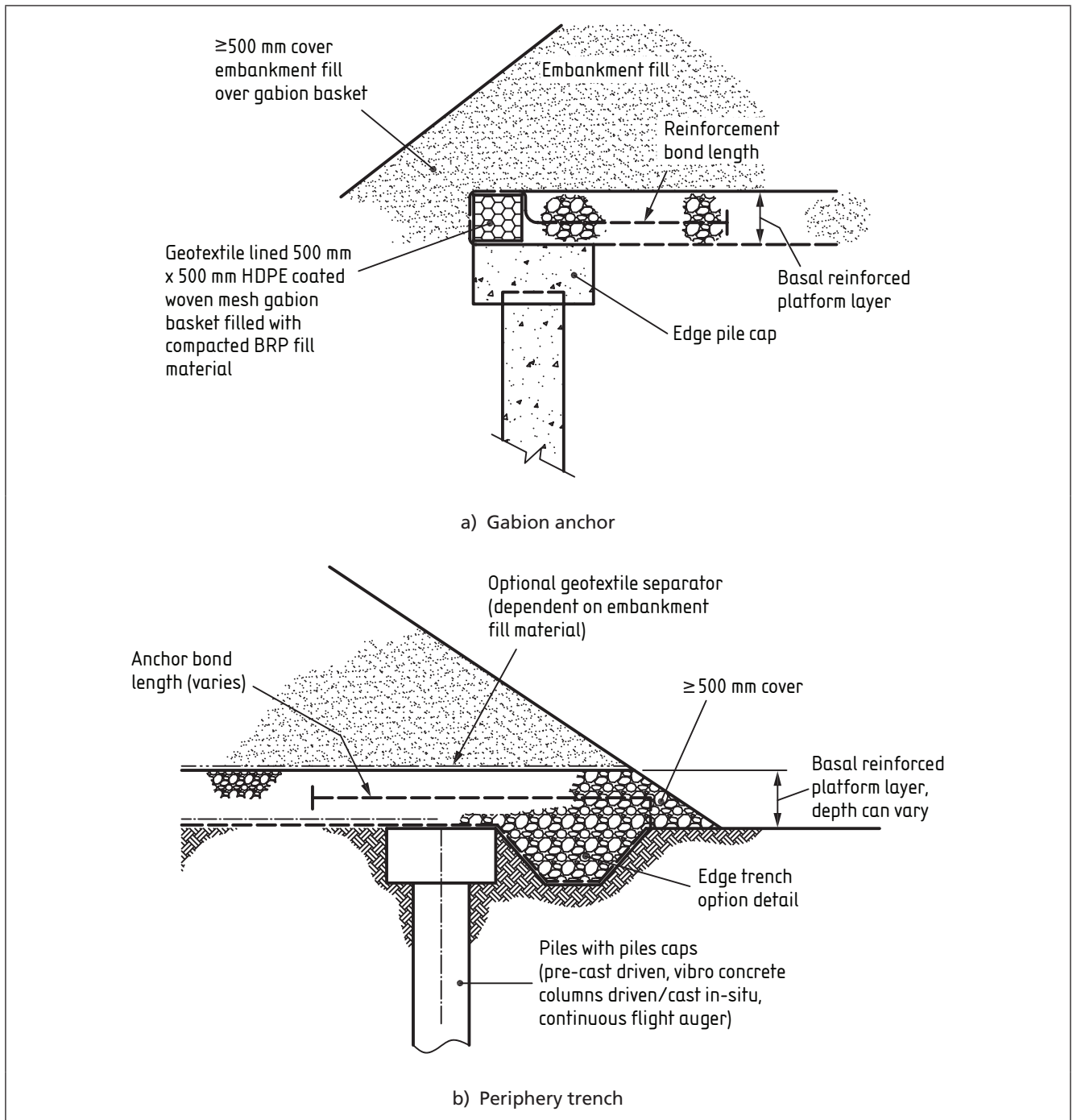
$M_{RS}$  is the factored restoring moment due to the soil at all locations along the base of the embankment (see Figure 82);

$M_{RP}$  is the restoring moment due to the piles along the base of the embankment (see Figure 82);

$M_{RR}$  is the restoring moment due to the reinforcement at all locations along the base of the embankment (see Figure 82).

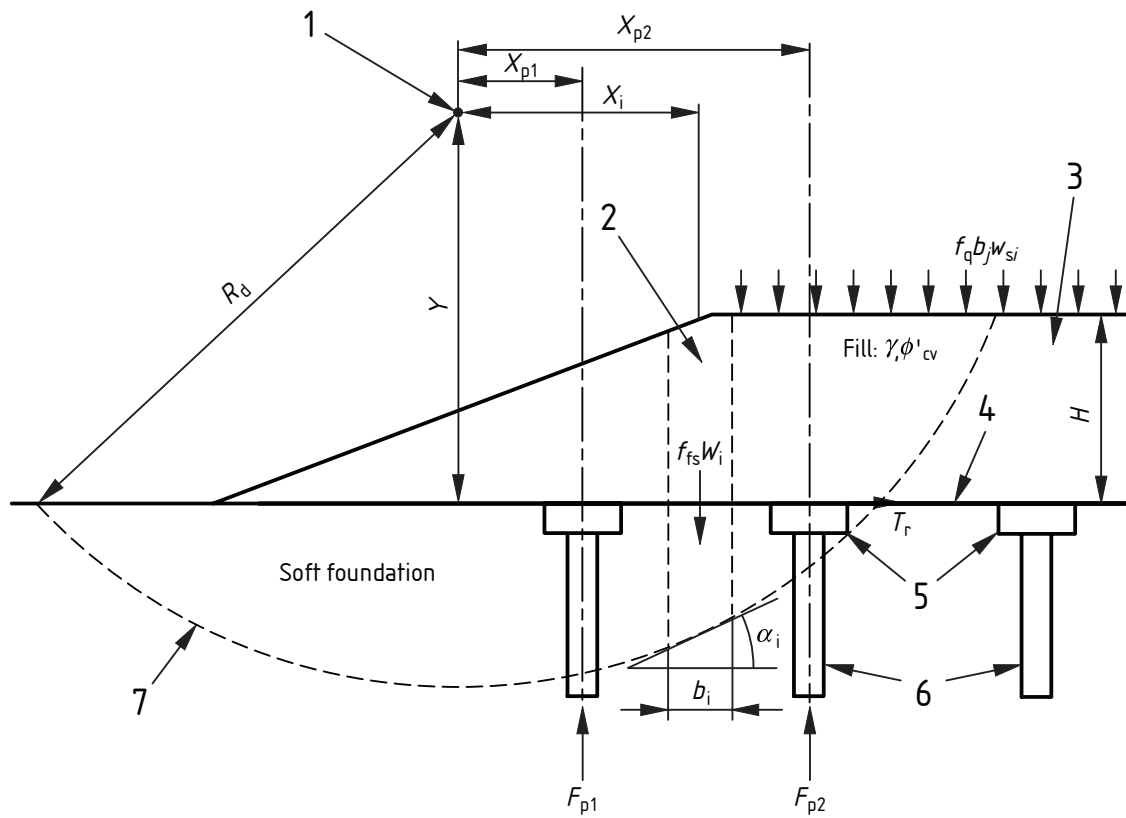


Figure 81 Typical anchorage options



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Figure 82 Variables used in analysis of overall stability of basal reinforced piled embankments



Disturbing moment due to soil and loading:

$$M_D = \left[ \sum (f_{fs}W_i + f_q b_j w_{si}) \sin \alpha_i \right] R_d$$

Restoring moment due to soil:

$$M_{RS} = \sum_{i=1}^n \frac{\left[ \frac{c' b_i}{f_{ms}} + (f_{fs}W_i + f_q b_j w_{si}) (1 - r_u) \frac{\tan \phi'_{cvi}}{f_{ms}} \right] \sec \alpha_i R_d}{1 + \frac{\tan \phi'_{cvi} \tan \alpha_i}{f_{ms}}}$$

Restoring moment due to piles:

$$M_{RP} = F_{p1} X_{p1} + F_{p2} X_{p2}$$

Restoring moment due to reinforcement:

$$M_{RR} = T_r Y$$

**Key**

- |                      |                              |
|----------------------|------------------------------|
| 1 Slip circle centre | 5 Pile caps                  |
| 2 Slice <i>i</i>     | 6 Piles                      |
| 3 Embankment         | 7 Most critical slip surface |
| 4 Reinforcement      |                              |

### 8.3.3.14 Reinforcement strain/stiffness

The maximum allowable strain in the reinforcement  $\varepsilon_{\max}$  should be limited to ensure differential settlements do not occur at the surface of the embankment [see Figure 83a)]; this can be a problem with shallow embankments where the soil arch cannot develop fully within the embankment fill.

The initial tensile strain in the reinforcement is needed to generate a tensile load; a practical upper limit of 6% strain should be imposed to ensure all embankment loads are transferred to the piles. With shallow embankments [for example where  $H < 0.7(s - a)$ ] where the arch cannot be fully supported on the piles, differential deformations can occur at the embankment surface [see Figure 83a)]. In this case, the choice of suitable reinforcement might be dominated by tensile stiffness considerations rather than tensile load. This upper strain limit may have to be reduced to prevent differential movements at the surface of the embankment. Typically a lower strain limit of  $\leq 3\%$  may be adopted during design, as reinforcement stiffness has been shown to be an important property in controlling surface deformations (Lawson [112]).

The maximum mid-span deflection  $y$  of extensible reinforcement, spanning between pile caps may be determined from the formulation below (see Giroud [113] and Giroud et al [114]) and should typically be limited to a practical maximum of 300 mm.

$$y = (s - a) \sqrt{\frac{3\varepsilon}{8}}$$

where

- $a$  is the size of the pile caps (assuming full support can be generated at the edges of the caps);
- $s$  is the spacing between adjacent piles;
- $\varepsilon$  is the strain in the reinforcement.

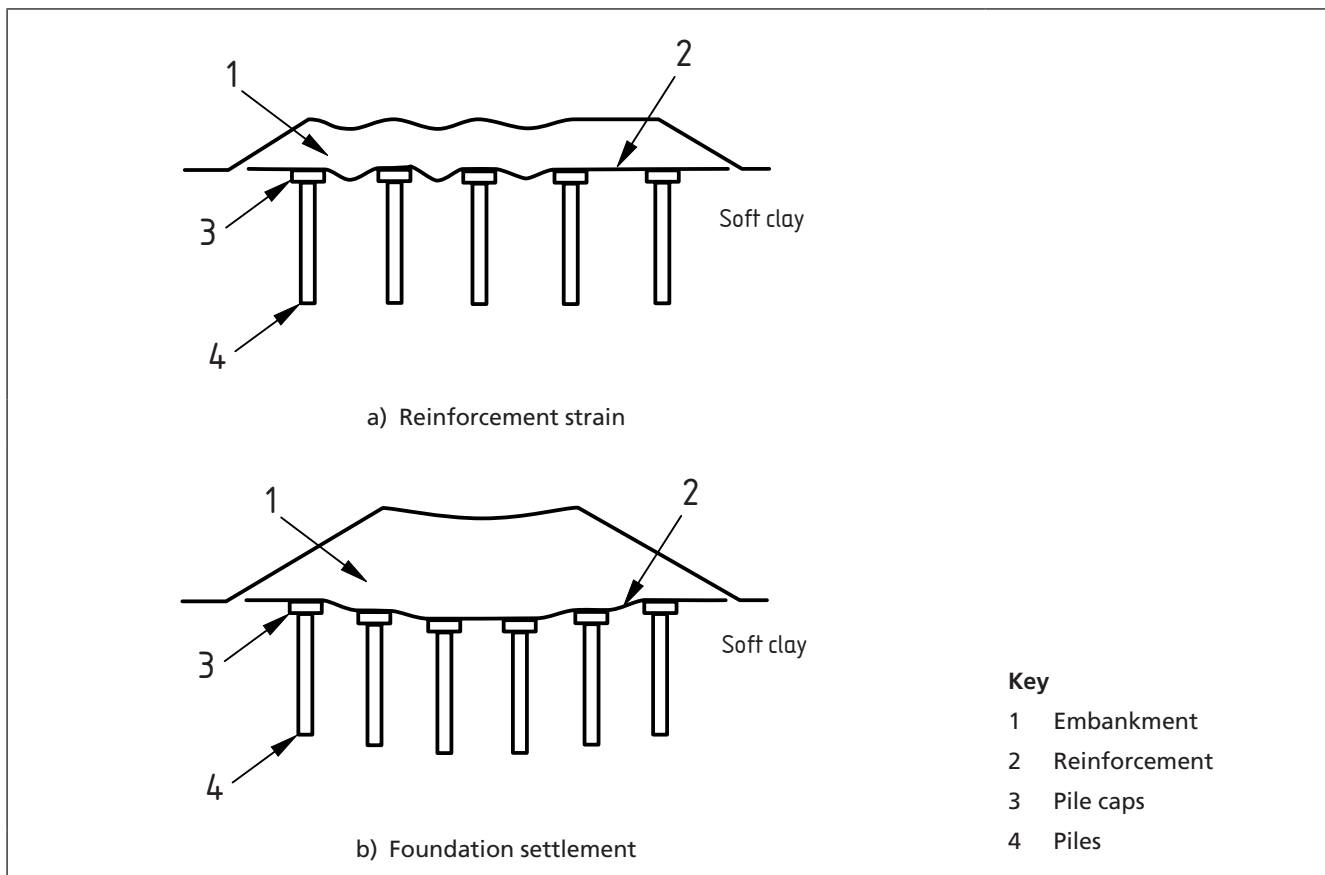
The actual maximum deflection of the reinforcement is shown to occur diagonally mid-span between four adjacent pile caps and may be assumed to be twice the magnitude calculated spanning orthogonally between two pile caps (see Almedia et al [115]).

Additionally, the mobilized reinforcement strain needs to be compatible with allowable movements of piles (see 8.3.3.11).

The long term strain (due to creep) of the reinforcement should be kept to a minimum to ensure that long term localized deformations do not occur at the surface of the embankment. A maximum creep strain of 2% over the design life of the reinforcement should be allowed. The post-construction creep strain in the reinforcement should be determined in accordance with the guidance in PD ISO/TR 20432.

Where reinforcement is designed for long-term use for the construction of rafts or tension membranes over weak soils, mine workings or areas of cavitations, consideration should be given to the use of a high visibility warning layer placed above the level of the reinforcement to guard against the future possibility of unplanned excavation extending down to the reinforcement level; this risk is increased with reduced installation depth.

Figure 83 Serviceability limit states for basal reinforced piled embankments



### 8.3.3.15 Foundation settlement

The design of the piled foundation should ensure that excess settlements do not occur [see Figure 83b)].

It should be noted that excess settlements can affect performance by:

- a) promoting differential settlements between the piled embankment and adjacent structures; and
- b) increasing the tensile loads in the basal reinforcement.

Good pile design practice is necessary to prevent excessive foundation settlements; it should be noted that often the piles at the base of an embankment may be designed for a greater allowable settlement than would otherwise be tolerated with structural piles.

## 8.4 Reinforced embankments over areas prone to subsidence

### 8.4.1 Areas of application

The following should be considered in dealing with areas prone to subsidence.

- a) Subsidence normally results from the collapse of a void below the ground surface. Subterranean voids can arise from natural processes (e.g. soil erosion in karstic areas) or from man-made processes (e.g. ground water pumping or underground mining).

- b) The consequences of subsidence occurring beneath structures can range from a loss of serviceability to total collapse (see Sims and Bridle [116]).
- c) Embankments, fills, and pavements are essentially flexible structures. Thus, the techniques used to minimize damage resulting from subsidence normally involve confining the vertical differential displacement of the structure to within predetermined tolerances. From this point of view either rigid foundation rafts, or reinforced soil techniques have proved effective (see Smith and Worrall [23], Kempton [117]).

#### 8.4.2 Basis for design

Reinforcement may be used to limit the amount of surface deformation caused by subsidence (see Figure 84). A void developing beneath a reinforced embankment some time after construction may be repaired by filling the void with grout in which case the reinforcement should act temporarily. If the void is left open the reinforcement should be specified to act for the remaining design life of the structure.

Voids under high cost structures (e.g. motorway embankments) should be filled, while for lower cost structures (e.g. low trafficked pavements) the cost of filling the voids may not normally be justified.

Reinforcement may be utilized in two different ways: internal reinforcement within the embankment structure and reinforcement at the base of the embankment. For internal reinforcement a number of layers of reinforcement may normally be included within the height of the embankment; the analysis of this technique is complex and is not covered further in this code (see Jones [25], Murray et al [26], Elias and McKitterick [118]).

The formulation contained within 8.4.4.4 are based on two principal assumptions:

- a) constant volume of soil in "zone of depression"; and
- b) no arching within the embankment fill.

Both of these assumptions can lead to conservatism in design. The assumption of no arching in the embankment fill may be taken as valid for low embankment height to void size ratios ( $H/D < 1$ ) and hence the "zone of deformation" depicted in Figure 84 of an inverted, truncated wedge or cone influencing the reinforcement is valid.

Strain limits in the reinforcement should be controlled, at the base of the zone of deformation, to limit depression at the embankment surface. By considering the geometry of the zone of influence, ignoring arching and assuming a constant volume and equating the volumetric movement of the geosynthetic and the volumetric movement of the soil, a relationship may be presented for the maximum allowable reinforcement strain in 8.4.4.4.

##### COMMENTARY ON 8.4.2

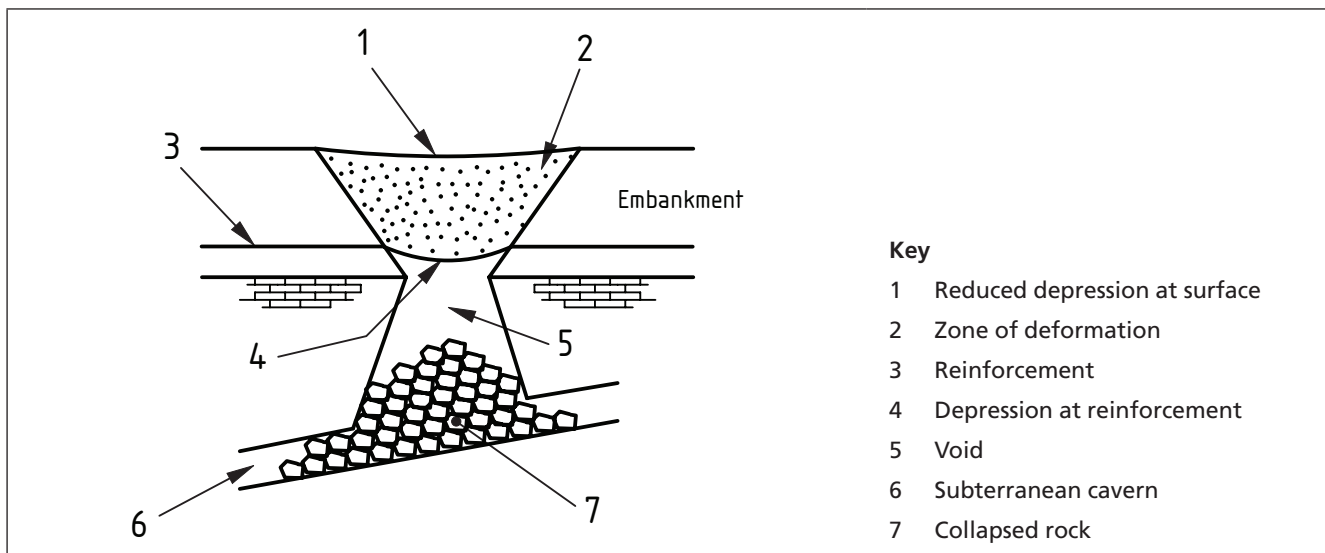
*Alternative approaches are available for predicting surface settlements based on an increase in the volume of soil in the "zone of depression" due to dilatancy (Bruhier and Sobolewski [119], and Alexiew et al [120]) By considering bulking of the fill in the collapse zone it is possible to effectively design a larger allowable reinforcement deflection or strain for a given or prescribed allowable surface deflection. However re-compaction of dilated fill in the "zone of depression" may also be likely due to repeated loading cycles from imposed surface loadings.*

As the relative height of the embankment in relation to the anticipated void diameter increases ( $H/d > 1$ ) then the possibility for arching increases. Terzaghi, suggests that once the ratio of the relative height of the embankment to void diameter is greater than 3 ( $H/d > 3$ ) then the pressure acting across the reinforcement is constant and almost independent of the state of stress above this level.

Consideration of arching within an embankment fill will effectively reduce the "zone" of soil to be supported by the geosynthetic and may result in minimal predicted deflections at the surface of the embankment. Arching over voids has been demonstrated both by numerical and physical models (see Potts and Zdravkovic [121]).

Consideration of different arching theories is discussed in 8.3.3.7 in relation to arching in pile supported embankments. UK guidance exists which does consider arching of engineered fills over potential voids (Jones and Pine [122], Jones and Dixon [123]). This guidance is in relation to new engineered landfills over areas prone to solution features or over areas of former waste disposal that might be subject to sudden collapse. Approaches that rely on arching or fill dilatancy are not considered further in this code.

Figure 84 Conceptual role of reinforcement in limiting surface deformations due to subsidence



### 8.4.3 Limit states

The reinforcement should be selected to ensure that serviceability is maintained and that the ultimate limit state (collapse) does not occur. Thus, the presence of the reinforcement should fulfil serviceability limit state criteria for the embankment structure in total.

The maximum limit state tensile load  $T_r$  to be resisted by the basal reinforcement should be the value  $T_{rs}$  determined by 8.4.4.5.

The maximum load carried by the reinforcement in considering the ultimate limit state condition should be:

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

where

$T_D$  is the design strength of the reinforcement calculated in accordance with 5.3.3;

$f_n$  is the partial factor governing the economic ramifications of failure (see Table 9).

The maximum allowable limit state strain in the reinforcement  $\epsilon_{\max}$  should be determined by 8.4.4.4 and 8.4.4.5.

## 8.4.4 Design procedure

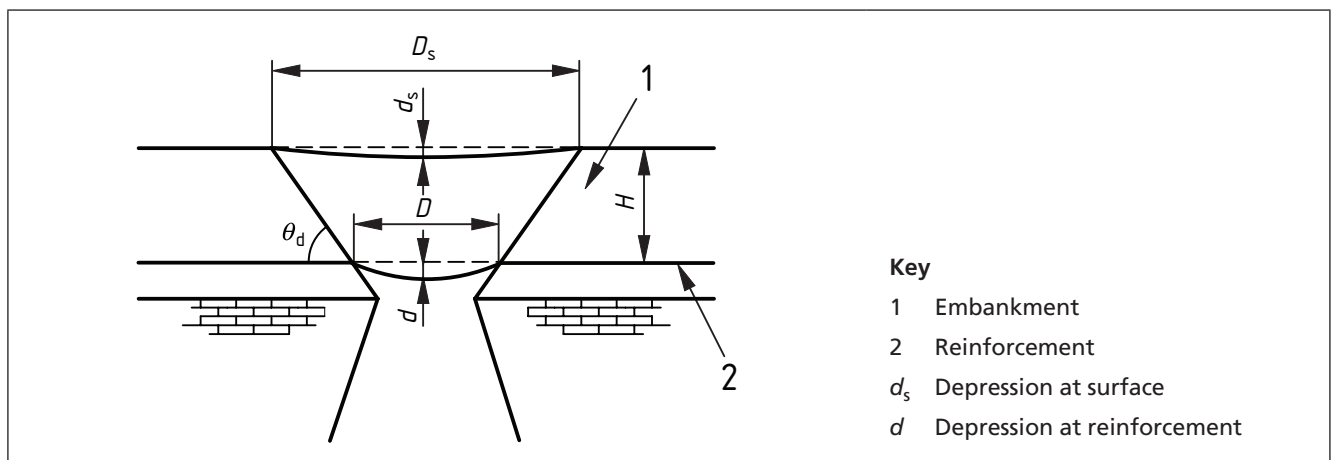
### 8.4.4.1 General

While the presence of the reinforcement protects against a serviceability limit state for the embankment as a whole, when designing the reinforcement both the ultimate limit state and the serviceability limit state for the reinforcement should be considered.

The general design procedure that should be followed to determine the characteristics needed by the reinforcement includes:

- determination of the maximum acceptable surface deformation limits for the pavement or embankment;
- determination of a suitable design value for the void diameter  $D$  (see Figure 85);
- determination of the maximum allowable strain in the reinforcement such that the criterion in a) is satisfied;
- determination of the tensile properties of the reinforcement needed for the design.

Figure 85 Parameters used to determine reinforcement



### 8.4.4.2 Acceptable surface deformations

The degree of acceptable surface deformation may be estimated dependent on the design philosophy for the supporting reinforcement.

The reinforcement may be designed to support the overlying embankment for the design life of the infrastructure and the surface settlement to remain within acceptable serviceability limits. Similarly the reinforcement may be designed to support the embankment for a shorter period of time allowing for some form of remedial measures to infill/reduce the void; the latter approach is likely to require some form of detection and remediation protocol to be established by the infrastructure owner/operator.

For principal roads, except Department of Transport trunk roads and motorways, the maximum differential surface deformation ( $d_s/D_s$ ) (see Figure 83) should be limited to 1%. For non-principal roads, differential

settlement should be limited to 2%. Other deformation limits for trunk roads and motorways may be needed.

For consideration beneath railway lines, more stringent allowable deflections should be considered; these may be derived from the maximum allowable cross-rail differential movements to prevent a twist fault or derailment (see Alexiew et al [126], Villard et al [127]).

#### 8.4.4.3 Design void diameter

The determination of a suitable design value for the void diameter should normally be based on experience of similar conditions, a subterranean survey, and/or a probabilistic approach. A conservative value should be assumed because of the uncertainties of future subsidence, and the consequent risks involved.

#### 8.4.4.4 Maximum allowable reinforcement strain

The deflected shape of the reinforcement spanning the void may be approximated to a parabola, where the maximum allowable strain in the reinforcement is:

For plane strain conditions (i.e. long voids);

$$\varepsilon_{\max} = \frac{8 \left( \frac{d_s}{D_s} \right)^2 \left( D + \frac{2H}{\tan \theta_d} \right)^4}{3 D^4}$$

For axisymmetric conditions (i.e. circular voids);

$$\varepsilon_{\max} = \frac{8 \left( \frac{d_s}{D_s} \right)^2 \left( D + \frac{2H}{\tan \theta_d} \right)^6}{3 D^6}$$

where

$\varepsilon_{\max}$  is the maximum allowable strain in the reinforcement;

$d_s/D_s$  is the maximum allowable differential deformation occurring at the surface of the embankment or pavement, see 8.4.4.2;

$D$  is the design diameter of the void, see 8.4.4.3;

$H$  is the height of the embankment;

$\theta_d$  is the angle of draw of the embankment fill, which is approximately equal to its peak friction angle, see Figure 85.

However, it should be borne in mind that observation of void development has suggested that a central zone of influence extending up from the void is formed prior to retrogressive slips developing along the sides of this central zone; additionally, if there is no volume loss from the fill in this central zone then retrogressive slips at the sides of the overlying fill will be prevented.

An additional criterion to satisfy the serviceability limit state should be to check for the surface serviceability requirements by considering a vertical zone of soil extending up from the edge of the void (ignoring shear along this vertical boundary) and design the supporting geosynthetic strain at the base of this central "zone of depression"



to have the same mid span deflection  $d$  equivalent to the allowable surface deflection  $d_s$  where:

$$d_s = D_s \sqrt{\frac{3\varepsilon}{8}} = d$$

where the maximum allowable strain in the reinforcement is:

$$\varepsilon_{\max} = \frac{8}{3} \left( \frac{d_s}{D_s} \right)^2$$

where

$\varepsilon_{\max}$  is the maximum allowable strain in the reinforcement;

$d_s/D_s$  is the maximum allowable differential deformation occurring at the surface of the embankment or pavement (see 8.4.4.2);

#### 8.4.4.5 Reinforcement tensile properties

For extensible reinforcements (e.g. polymeric) the tensile load  $T_{rs}$  in the deflected reinforcement should be taken to be:

$$T_{rs} = 0.5\lambda (f_{fs}\gamma H + f_q w_s) D \sqrt{1 + \frac{1}{6\varepsilon}}$$

where

- $T_{rs}$  is the tensile load in the reinforcement per metre "run";
- $\lambda$  is a coefficient dependent on whether the reinforcement support is to function as a one-way ( $\lambda = 1$ ) or two-way load shedding system ( $\lambda = 0.67$ );
- $\gamma$  is the unit weight of the embankment fill;
- $H$  is the height of the embankment;
- $w_s$  is the surcharge intensity on top of the embankment;
- $D$  is the design diameter of the void, see 8.4.4.3;
- $\varepsilon$  is the strain in the reinforcement which is less than or equal to  $\varepsilon_{\max}$ , see 8.4.4.4;
- $f_{fs}$  is the partial load factor for soil unit weight (see Table 22);
- $f_q$  is the partial load factor for external applied loads (see Table 22).

Spanning a void in two directions has been shown to be less efficient than providing uniaxial reinforcement in one principal support direction (see Villard et al [127]) hence for circular or rectangular voids (spanning two ways),  $\lambda = 0.67$ , while for longitudinal or circular voids (spanning one way),  $\lambda = 1.0$ .

The maximum tensile force  $T_{rs}$  to be resisted by the reinforcement could theoretically be provided by two or more multiple reinforcement layers installed in the principal support direction/s as described in 8.3.2.12.

It is recommended therefore, where possible, that the maximum limit state tensile force  $T_{rs}$  should be provided in one reinforcement layer in the principal support direction/s. Where this is not possible then consideration should be given to the use of multiple layers of reinforcement with equal strength and stiffness, provided the design tensile strength of the reinforcement  $T_n$  in each layer is reduced in accordance with 8.3.2.12.

The value of  $T_{rs}$  should be calculated using the value for reinforcement strain determined from 8.4.4.4 or from a knowledge of the strain of the reinforcement under consideration (provided the strain of this reinforcement satisfies the conditions given in 8.4.4.4). The strain value used in the above equation should be the initial strain of the reinforcement, i.e. before any allowances are made for creep.

The above equation is appropriate for extensible reinforcements; for inextensible reinforcements alternative methods should be used to determine the recommended reinforcement strength.

#### 8.4.4.6 Reinforcement bond

To generate the tensile load  $T_{rs}$  in the reinforcement adequate bond should exist between the reinforcement and the adjacent soil. The minimum reinforcement bond length  $L_b$  needed to carry  $T_{rs}$  should be taken as:

$$L_b \geq \frac{f_n f_p T_{rs}}{\gamma_h \left( \frac{a'_1 \tan \phi_{cv1}}{f_{ms}} + \frac{a'_2 \tan \phi_{cv2}}{f_{ms}} \right)}$$

where

$f_n$  is the partial factor governing the economic ramifications of failure (see Table 9);

$f_p$  is the partial factor applied to the pull-out resistance of the reinforcement (see Table 22);

$h$  is the average height of fill over the bond length of the reinforcement;

$\gamma$  is the unit weight of the embankment fill;

$a'_1$  is the interaction coefficient relating the soil/reinforcement bond angle to  $\tan \phi_{cv1}$  on one side of the reinforcement;

$a'_2$  is the interaction coefficient relating the soil/reinforcement bond angle to  $\tan \phi_{cv2}$  on the opposite side of the reinforcement;

$f_{ms}$  is the partial material factor applied to  $\tan \phi_{cv}$  (see Table 22).

## 8.5 Construction and maintenance

### 8.5.1 General

In planning the works, particular attention should be paid to difficulties that can arise from site access, site clearance and trafficability during construction. For construction over highly compressible or low strength soils, trafficability in particular can prove difficult and may require the use of special plant including low bearing pressure earth moving equipment.

*NOTE Further guidance is given in BS EN 14475:2006.*

## 8.5.2 Basal reinforcement

### 8.5.2.1 General

Construction methods for basal reinforcement are affected by the surface and near-surface site conditions; therefore, information on the engineering properties of the founding soil together with the presence and extent of any desiccated crust and the type and extent of vegetation should be established as part of the site investigation. Where the site is submerged a survey should be carried out to determine the depth of water over the area of the proposed construction.

### 8.5.2.2 Site access

If access to the site is impeded by poor trafficability, special provisions should be made for the construction of a temporary access road. At a convenient location close to the works a clean working platform and storage area should be constructed to accommodate jointing and storage of the basal reinforcement. Guidance for the design of temporary working platforms is given in **8.3.2.15**.

### 8.5.2.3 Reinforcement storage

Reinforcement in the form of rolls or folded sheets of geotextile or geogrid should be stored on dry ground and protected from exposure to sunlight. Where the reinforcement has been protected by a wrapping resistant to ultra-violet light attack no further protection against sunlight should be necessary.

### 8.5.2.4 Reinforcement jointing

It should be noted that the tensile strength, and other mechanical properties, of the reinforcement in the main load carrying direction perpendicular to the centre line will be governed by any joints. If possible, the reinforcement should extend across the width of the embankment in one continuous piece (i.e. no joints in this direction).

Joints along the length of the embankment are inevitable; this should be taken into account in assessing longitudinal stability of the embankment during and immediately after construction. The joints should conform to **3.2.3**.

### 8.5.2.5 Site preparation

On vegetated sites only substantial vegetation such as bushes or trees should be cut down flush to the natural ground level. Obstructions that can damage the reinforcement should be removed. Debris likely to cause puncturing or other mechanical damage to the reinforcement should be removed from the areas prepared to receive the reinforcement. Organic material will decay and consideration should be given to the long term effects of substantial deposits if they are to be left on the site.

Root systems of felled trees or bushes and vegetation giving ground cover should be left in place. On sites known to have a desiccated crust care should be taken not to rupture this crust during site preparation and initial filling.

Before placement of the reinforcement commences, all abrupt changes in ground profile should be evened out by placement of suitable fill. Where a regulating layer of fill is used to cover uneven ground including submerged ground and obstructions, care should be taken to ensure that the regulating layer as placed does not impair the vertical hydraulic conductivity of the natural ground. Where possible such fill should be granular, and a nonwoven geotextile separator should be placed between the soft ground and the fill to prevent contamination of the fill. In placing any regulating layer, care should be taken to ensure that this does not overstress or rupture any desiccated crust.

*NOTE Further guidance is given in BS EN 14475:2006.*

#### 8.5.2.6 Handling and placing of reinforcement

In favourable conditions, the basal reinforcement should be transported to site in rolls and rolled out into position with joints being made in situ. To facilitate jointing the laying and jointing sequence shown in Figure 86 may be used as a guide: step 1 involves rolling out the reinforcement across the width of the embankment; a second layer of reinforcement is rolled out on top of the first (step 2), and a joint is made along one edge, as shown in step 3; the top layer of reinforcement is then folded over onto the ground (step 4), and the process repeated by rolling out another sheet of reinforcement on top of this layer followed by jointing (steps 5 and 6).

For poorer site access conditions construction techniques should be modified accordingly. Where conditions make rolling out and in situ jointing difficult sheet basal reinforcement should be prefabricated on stable ground or a working platform and moved into final position manually. Where poor ground is of limited extent this process may be eased by mechanical rope hauling from distant stable ground. Such construction processes can prove very rigorous and consequently the reinforcement should be selected to have sufficient mechanical properties to fulfil its design function and survive the construction process. Further complications can arise where basal reinforcement is to be placed through shallow water on submerged sites such as swamps and marshes: where water is shallow the reinforcement may again be placed manually using pre-cut and jointed reinforcement. Flotation of reinforcement with a specific gravity less than unity should be prevented by local weighting. In deeper water or where the basal reinforcement cannot be man-handled, placing may be achieved by shallow draft vessels or craft using rope hauling techniques.

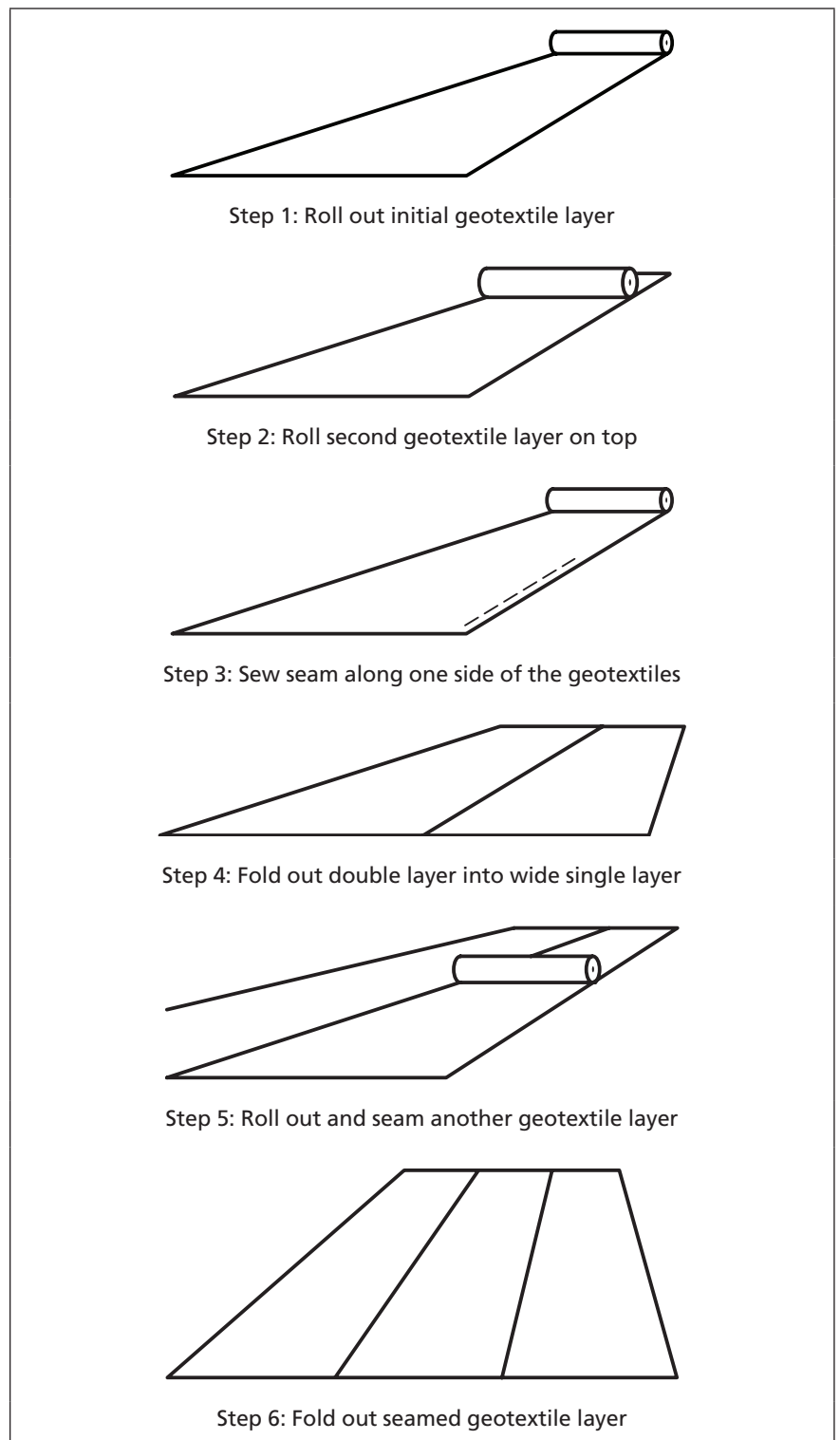
Where strong geotextiles or geogrids are used on weak, swampy or marshy ground they may be rolled out by workmen walking on the reinforcement itself.

*NOTE Further guidance is given in BS EN 14475:2006.*

#### 8.5.2.7 Exposure to sunlight

A maximum period should be specified for which polymeric reinforcement may be exposed to sunlight (or other sources of ultra-violet light) after removal from its protective wrapping and before burial. This should be as short as possible for the application. If necessary, detailed advice should be sought from the manufacturer.

Figure 86 Laying and jointing sequence for basal reinforcement



#### 8.5.2.8 Placement of fill

Fill should be compacted in accordance with the *Specification for Highway Works* [1].

The sequence of fill placement should be considered with care, particularly over very poor foundation soils where the bearing capacity can be exceeded with even the smallest loading. Fill may be

deposited by end tipping on site. The actual placement of the fill over the reinforcement should be achieved by a machine which can cascade fill onto the reinforcement.

Two placement techniques have been used successfully (see Holtz [128]): the first technique is to advance the fill, full width; extreme care should be taken in ultra-soft ground to control any bow wave of mud or ooze formed as the fill advances; once formed such bow waves can be difficult to disperse (see Figure 87).

When fill is advanced full width, the effects of any mud wave should be reduced by placing the central section of fill slightly ahead of the fill placed at the toes. In such a chevron pattern the mud wave should be rolled towards the toes of the embankment (see Figure 88).

To prevent the generation of a substantial wave, care should be taken to restrict the first lift of fill to the minimum needed for trafficability by lightweight plant.

The second technique that may be used, shown in Figure 89, involves the initial construction of end-dumped dykes along each toe of the embankment to serve as access roads. These should be subsequently widened to anchor the basal reinforcement at each toe before infilling the central portion of the embankment cross section. This method of construction may become less feasible for embankments with narrower base widths (see Haliburton et al [129], Christopher and Holtz [130]).

Before placement of the main reinforcement an initial trial should be undertaken to determine the extent of any mud wave generation. The development of high amplitude mud waves beneath reinforcement placed well ahead of filling can be detrimental since large mud waves can rupture or move the reinforcement during their forward advance; this may be avoided by placing the reinforcement in shorter lengths along the line of the embankment.

Where possible, the first lift of fill should be placed over the entire length of reinforcement before further lifts are placed. This technique may be used to give early protection to reinforcement against ultra-violet attack, allows some consolidation, provides a construction platform and enhances the resistance to the development of mud waves during subsequent filling.

On sites with a desiccated crust or vegetation cover there is little potential for the development of mud waves; care should be taken in establishing the initial layers of fill to avoid locally overstressing the foundation soil. This may exclude direct end tipping or tipping of stockpiles for later spreading.

Since the first lifts of fill are usually granular drainage material, adequate compaction should be achieved by trafficking of construction plant. The thickness of any granular drainage blanket should be greater than the anticipated differential settlement across the embankment.

Construction plant should be controlled to prevent access of inappropriate plant that might damage the basal reinforcement. This is important where consolidation of the foundation soil is to be accelerated by the installation of wick drains; if installation of such drains requires access by heavy plant, sufficient fill should be in place to prevent localized or overall failure during trafficking.

*NOTE Further guidance is given in BS EN 14475:2006, 8.3.2.15.*

Figure 87 Advancing mud wave

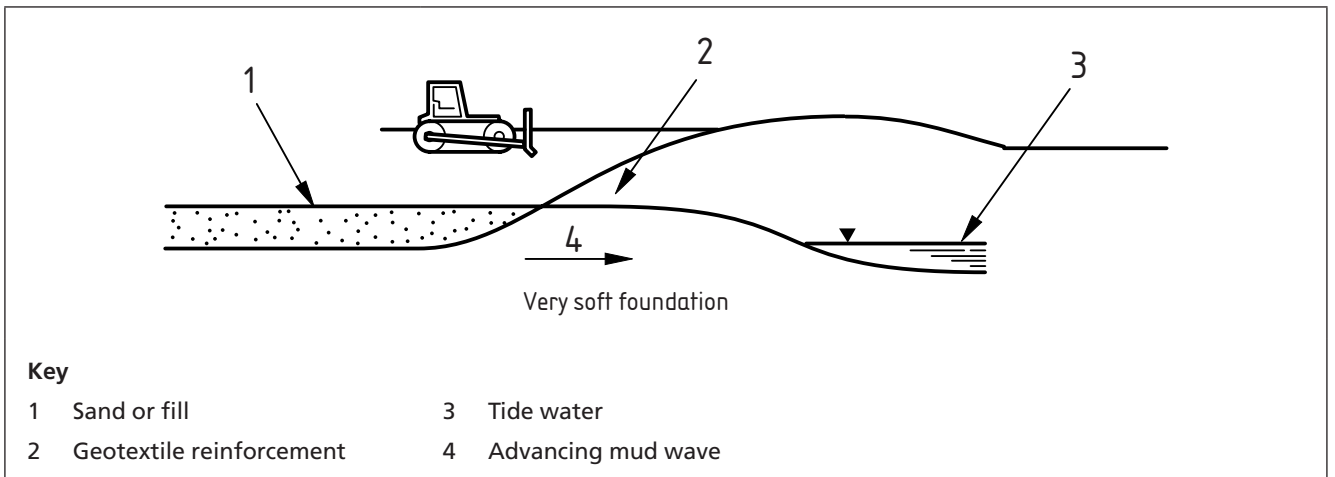


Figure 88 Inverted "U" construction

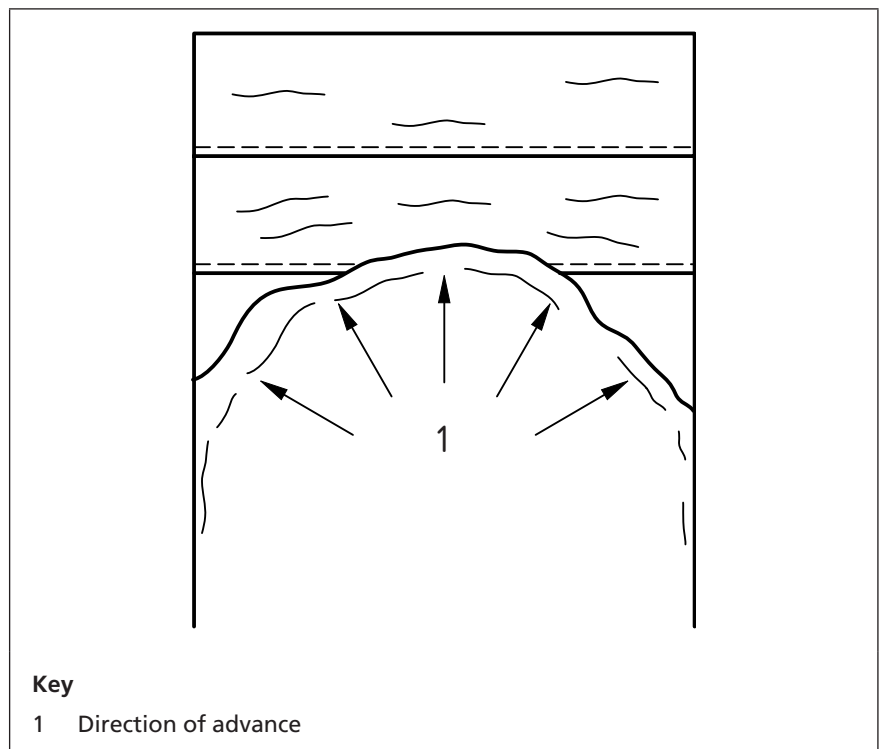
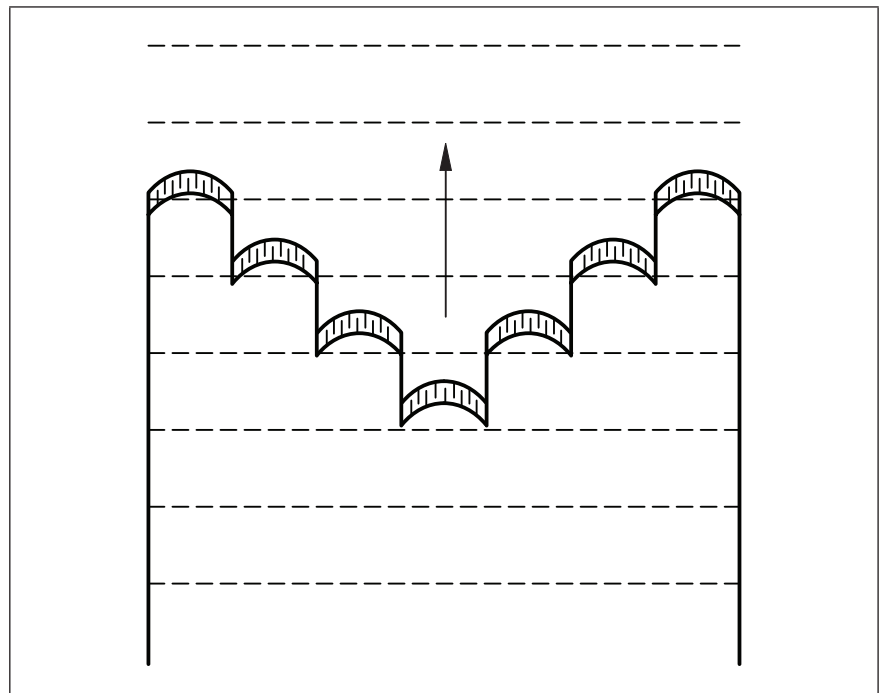


Figure 89 Construction of a "U" shaped leading edge



### 8.5.3 Rafts and tension membranes

*NOTE* Construction methods are affected by the surface and near surface site conditions.

Usually rafts or tension membranes are needed to span areas of potentially weak ground or areas of cavitation and the design will be often based on considerations of axial stiffness rather than strength (see Smith and Worrall [23], Jones et al [89], Leonard [105], Elias and McKitterick [118], Haliburton et al [129], Gilchrist [131], Kempton and Jones [132]). In these cases it is important that the reinforcement should be laid out reasonably flat to reduce the amount of take up due to undulations.

Level control of the layers of reinforcement should be carried out by checking the various levels of reinforcement using conventional survey techniques.

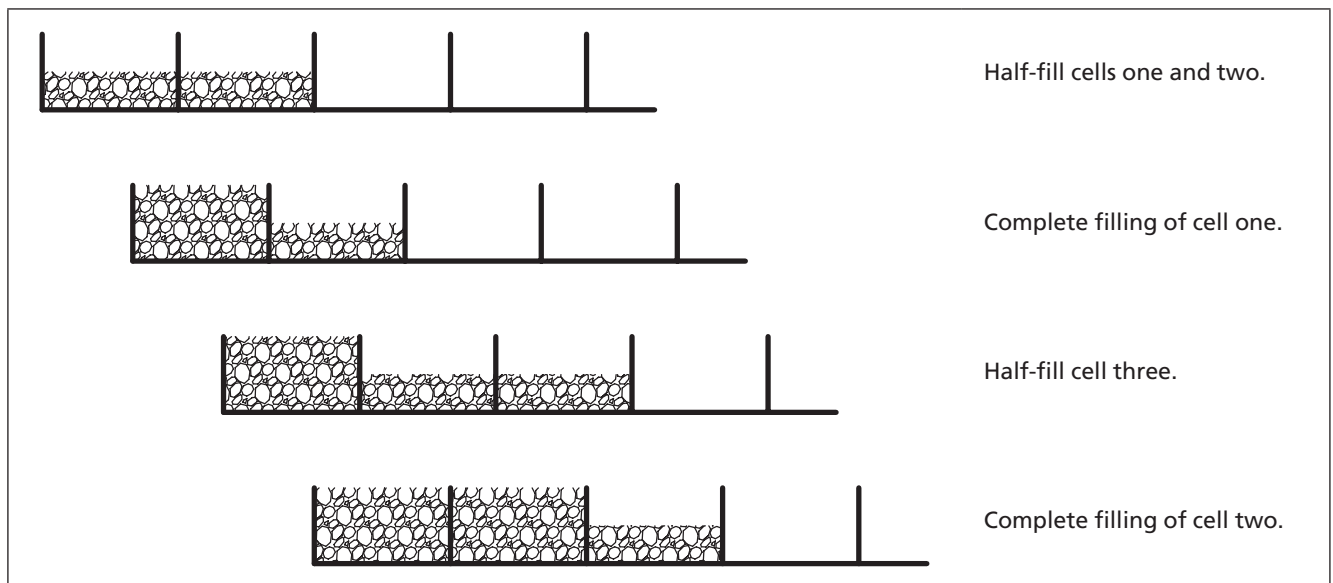
Where tension membranes are used adjacent to curing concrete (e.g. in piled embankments) care should be taken to ensure that the type of geosynthetic used is not adversely affected by the concrete curing process. The procedure for installing tension membranes should be similar to that for basal reinforcement, see 8.5.2.6.

### 8.5.4 Mattresses

A basal foundation mattress may be constructed in two phases. In the first phase, a honeycombed structure should be formed by constructing a series of interlocking cells using polymeric geogrids in a vertical orientation connected to a base geogrid. The mattress may be up to one metre deep. During the second phase, the cells should be filled sequentially with granular material (see Figure 90).



Figure 90 Fill sequence for basal foundation mattress



### 8.5.5 Maintenance and warning layer

Construction of reinforced foundations over weak highly compressible soils should be designed as a short term expedient with a design life equal to the period of time needed for the embankment foundation to consolidate. It should be borne in mind that once installed, the reinforcement is inaccessible and maintenance is not possible.

Rafts and tension membranes used for construction of embankments over weak soils, mine workings or areas of cavitation should be installed as a precaution against collapse or to reduce settlements. It should be borne in mind that these structures are also inaccessible and no maintenance is possible.

High visibility warning layers should be installed above the reinforcement level to guard against the risk of subsequent damage to the reinforcement layer from un/planned future excavation; this is particularly relevant where the reinforcement is installed at shallow depth beneath an embankment.

## Annex A (normative) Assessment of partial material factors for reinforcements

### A.1 Reinforcement design strength – General

In walls, slopes and certain foundation applications, embankments supported on piles (see Figure 68), the design load is assumed to remain constant over the reinforcement design life. Consequently the design strength for the reinforcement should be based on the strength assessed to be available at the end of the design life. For reinforcement subject to creep strain, the design strength may be governed by considerations of serviceability rather than tensile creep rupture. For embankments constructed over weak foundation soils the load taken by the reinforcement may decrease with time and consequently strength of the reinforcement during, or at the end of, construction may be the critical value. These strengths may be determined from isochronous load/strain (creep) curves.

PD ISO/TR 20432 provides reduction factors to determine the long-term strength of geosynthetic materials. Although PD ISO/TR 20432 quotes typical service lives of 50 years to 100 years, the methodology is equally valid for service lives of 120 years. The design strength  $T_D$  should be taken as the long term strength as determined using PD ISO/TR 20432.

Care should be exercised when using material performance factors derived using PD ISO/TR 20432 in the design methods contained in this standard to ensure that factors derived are appropriate to the service life required. Further information on the assessment of the durability of geosynthetics is provided in DD ISO/TS 13434.

The unfactored strength of the reinforcement is  $T_B$  and this is reduced by the reinforcement material factor  $f_m$  to define the reinforcement design strength  $T_D$  such that:

$$T_D = \frac{T_B}{f_m}$$

The design strength may be governed by the ultimate limit state of collapse or a serviceability limit state. A clear distinction is made between basal reinforcement of embankments over weak foundation soil and the reinforcement of walls, slopes and special embankment foundations. For embankments over weak foundations the maximum design load occurs at the end of construction provided it is assumed that there is no consolidation of the foundation soil during construction. As post-construction consolidation proceeds so the design load decreases with time. This means that at any instant in time, the factored design strength equals or exceeds the design load (see 8.3.2.1).

In the case of walls and slopes, the design load is assumed to remain constant over the selected design life. The design strength to be defined is that prevailing at the end of the selected design life of the wall or slope. This approach is conservative if applied to basal reinforced embankments over weak foundations where the design load decreases with time after the end of construction.

Where the design life exceeds the duration of tests to determine how  $T_B$  decreases with time it is necessary to define  $T_B$  by extrapolation of the test data. In the case of metallic reinforcement extrapolation involves an assessment of how the dimensions of the reinforcement are decreased with time by electrochemical corrosion. These reductions

have been determined for steel metallic reinforcements and prescribed allowances are set out in Section 3.

For polymeric reinforcement or metallic reinforcements which exhibit creep  $f_m$  is determined by extrapolation, as necessary, of product specific tensile creep strain, tensile creep rupture test data, with weathering and environmental data.

## A.2 Metallic reinforcement

### A.2.1 Metallic reinforcement – General

In general, the reinforcement base strength  $T_B$  should be the ultimate tensile strength based on net cross-sectional area for metallic reinforcements. However for those metallic reinforcements which exhibit creep the reinforcement base strength should be determined by A.3.3. The ultimate tensile strength of metallic reinforcements should be determined by a recognized test method, e.g. BS 1449-1.

### A.2.2 Partial material factor $f_m$

In 5.3.3.1 two components of  $f_m$  are listed. These are:

- $f_{m1}$ , which is a partial material factor related to the intrinsic properties of the material; and
- $f_{m2}$ , which is a partial material factor concerned with construction and environmental effects.

For metallic reinforcements, the relationship between the components is such that:

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

For plain or galvanized steel reinforcements subject to axial tensile loads only, the value of  $f_m$  should be 1.50, where  $T_B$  is based upon minimum manufactured dimensions and minimum tensile rupture stress and using a minimum thickness of 4 mm. Other reinforcements will have values of  $f_m$  depending upon the magnitude of the components which comprise  $f_m$  and the types of loads sustained (e.g. tensile, bending, shear, etc.).

### A.2.3 Partial material factor $f_{m1}$

#### A.2.3.1 Components

Table 11 lists components of  $f_{m1}$ . These are:

- $f_{m11}$ , which is partial material factor related to the consistency of manufacture of the reinforcement and how strength may be affected by this and possible inaccuracy in assessment; and
- $f_{m12}$ , which is partial material factor related to the extrapolation of test data dealing with base strength.

For metallic reinforcements the relationship between the components is such that:

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

For polymeric reinforcements, the relationship between all the components and sub-components of  $f_{m1}$  is as shown in A.2.

### A.2.3.2 Partial material factor $f_{m11}$

#### A.2.3.2.1 General

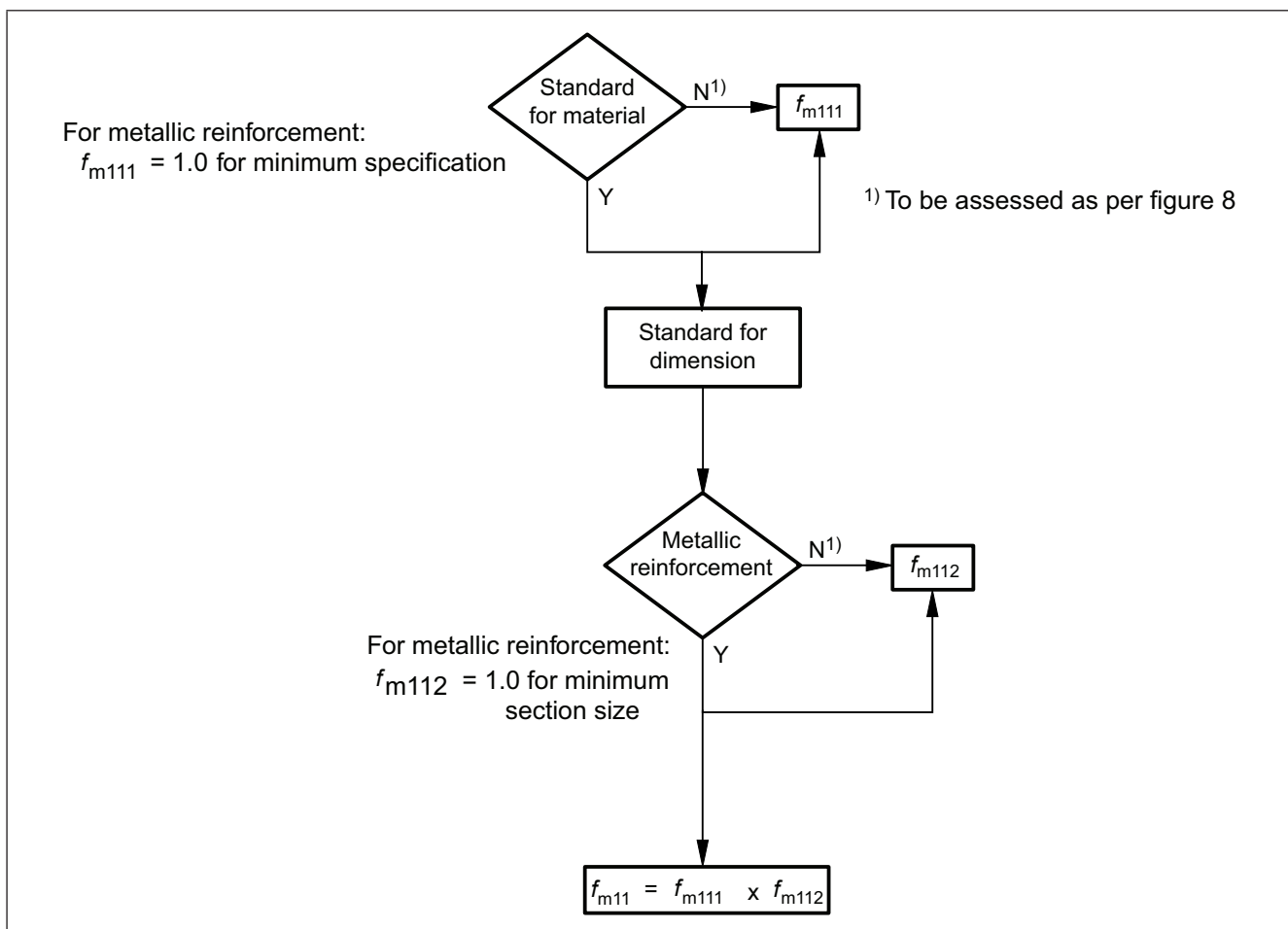
This partial factor deals with the consistency of manufacture and how variations in this may affect strength.

For metallic reinforcements of regular cross section and not subject to creep, the constituents of the base material should conform to prescribed formulations such as those set out for steel in Table 4. For such materials strength is defined as the product of cross-sectional area and rupture stress. Strength will be affected by permitted variations in rupture strength introduced by the manufacturing process. Consequently variations in strength may be modelled by considering variations in dimensions and variations in rupture stress.

This partial factor  $f_{m11}$ , which deals with material manufacture, should take into account the following (see Figure A.1):

- whether or not a standard for specification, manufacture and control testing of the reinforcement exist (related to a partial material factor  $f_{m111}$ ); and
- whether or not standards exist for the dimensions and tolerances of the particular product being manufactured (related to a partial material factor  $f_{m112}$ ).

Figure A.1 Assessment of  $f_{m11}$



**A.2.3.2.2 Partial material factor  $f_{m111}$** 

For metallic reinforcements quality should be specified on the basis of minimum base strength and then  $f_{m111}$  should be taken as 1.0. In this case appropriate quality control procedures should be employed, e.g. UK Certification Authority for Reinforcing Steel (UK CARES).

**A.2.3.2.3 Partial material factor  $f_{m112}$** 

For metallic reinforcements the dimensions should conform to well defined tolerances. The reinforcement base strength may then be based upon either the minimum permitted cross section in which case  $f_{m112}$  should be taken as 1.0 or upon the nominal cross section in which case a value of  $f_{m112}$  greater than unity should be used.

Figure A.1 outlines the general procedure to be adopted in determining  $f_{m111}$ ,  $f_{m112}$  and  $f_{m11}$ .

**A.2.3.3 Partial material factor  $f_{m12}$** **A.2.3.3.1 General**

This partial factor which deals with extrapolation of test data should take into account the following:

- a) the assessment of available data in order to derive a statistical envelope (related to a partial material factor  $f_{m121}$ ); and
- b) the extrapolation of this statistical envelope over the expected service life of the reinforcement (related to a partial material factor  $f_{m122}$ ).

**A.2.3.3.2 Partial material factor  $f_{m121}$** 

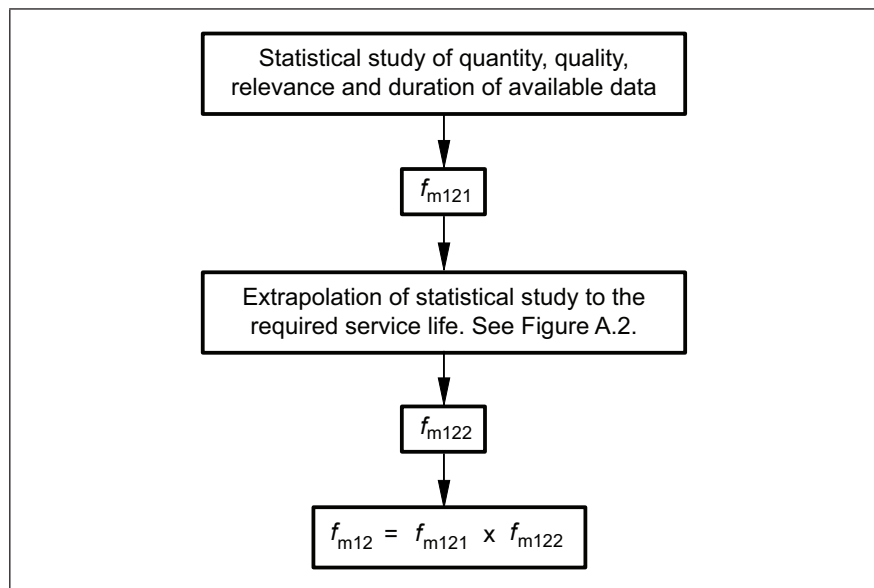
For metallic reinforcements statistical procedures according to the relevant standards should be applied to the available data in order to derive an envelope for future extrapolation,  $f_{m121}$  represents a measure of the confidence in the available data which is to be subsequently extrapolated. For large amounts of directly relevant data available over a long period of time the statistical analysis would permit a value of 1.0 for  $f_{m121}$ . If, for example, only very few test results were available then the statistical analysis would yield a value of  $f_{m121}$  of greater than unity.

**A.2.3.3.3 Partial material factor  $f_{m122}$** 

For metallic reinforcements this material factor relates to the confidence in extrapolation of data beyond the duration of test data to the selected design life of the reinforcements. It is recommended practice to only extrapolate by one log cycle of time in which case  $f_{m122}$  takes a value of 1.0. However, extrapolation of data by up to two log cycles of time is permissible provided the extrapolation is supported by test data derived from ongoing accepted real time and/or accelerated tests such as those carried out at temperatures exceeding the maximum operational temperature of the reinforcements.

For design lives in excess of 10 years data should be derived from creep tests with a minimum duration of  $10^4$  h to warn of deterioration. For design lives of 10 years or less the test duration should be at least 10% of the design life.

Figure A.2 outlines the general procedure to be adopted in determining  $f_{m121}$ ,  $f_{m122}$  and  $f_{m12}$ .

Figure A.2 Assessment of  $f_{m12}$ 

## A.2.4 Partial material factor $f_{m2}$

### A.2.4.1 General – Components

Table 11 lists two components of  $f_{m2}$ . These are:

- $f_{m21}$ , which is partial material factor related to the susceptibility of the reinforcement to damage during installation in the soil;
- $f_{m22}$ , which is partial material factor related to the environment in which the reinforcement is installed.

For metallic reinforcements, the relationship between the the components is such that:

$$f_{m2} = f_{m21} \times f_{m22}$$

### A.2.4.2 Partial material factor $f_{m21}$

This partial factor which deals with installation damage should take into account the following:

- the immediate or short term effects of damage prior to and during installation (related to a partial material factor  $f_{m211}$ ); and
- the long term effects of the short term damage (related to a partial material factor  $f_{m212}$ ).

For steel metallic reinforcements  $f_{m21}$  has a value of 1.0 when the minimum steel thickness is greater than or equal to 4 mm, and provided the fill material conforms to the recommendations of Section 3. For thinner reinforcements and fill material outside the limits listed in Section 3, a value of greater than unity may be necessary and should be supported by site damage trials and follow the procedure described in Annex D.

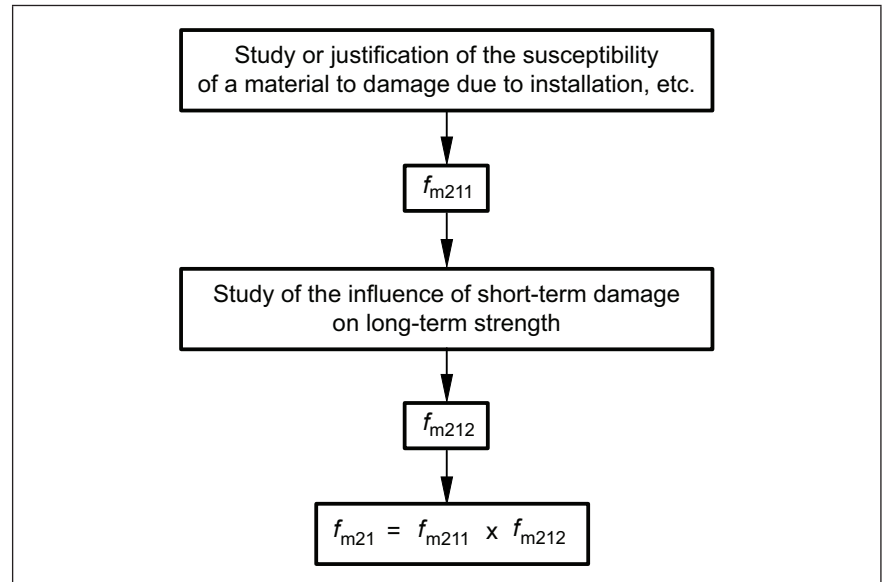
Corresponding criteria for other metals should be determined on the basis of experience or site trials.

Metallic reinforcements can be affected by installation damage. The degree to which this occurs depends on handling of reinforcements

prior to installation, the structure of the reinforcements, the nature of the soil in which the reinforcements are installed (primarily soil particle size) and the compaction forces applied.

Figure A.3 outlines the general procedure to be adopted in determining  $f_{m211}$ ,  $f_{m212}$  and  $f_{m21}$ .

Figure A.3 Assessment of  $f_{m21}$



### A.2.5 Partial material factor $f_{m22}$

This partial factor allows for any detrimental effects the soil environment can have on the reinforcements. These include any action or reaction which can raise the operational temperature above the maximum value assumed in design or, more commonly, the effects of chemicals. Where there is a probability of aggressive chemicals coming into contact with the reinforcements then this should be taken into account or prevented by the incorporation of suitable drainage or sealing.

In addition to the effects of the soil environment, the state of stress and the selected design life of the reinforcement should also be taken into account when determining  $f_{m22}$ . For reinforcements which utilize protective layers or coatings these can be more resistant to attack than the load carrying elements being protected. If the effect of installation damage is to expose the load carrying elements to the soil environment then the effects of this should be incorporated in determining  $f_{m22}$ . Similarly the combined effects of stress, short term damage and long term exposure to the soil environment can be synergistic and therefore amplify the effects of the soil environment alone.

Partial material factor  $f_{m22}$  is made up of:

- $f_{m221}$ , which is a factor which takes account of the reduction in strength caused by the detrimental effects of the soil environment in which the reinforcement is placed; and
- $f_{m222}$ , which takes account of any statistical uncertainty in the predicted loss of strength.

The general procedure to be adopted for determining  $f_{m221}$ ,  $f_{m222}$  and  $f_{m22}$  should be similar to that shown for the  $f_{m21}$  components in Figure A.3.

### A.3 Polymeric reinforcement

In accordance with 5.3.3.3 the reinforcement base strengths  $T_B$  should be either of the following.

- a) For the ultimate limit state the base strength ( $T_B$ ) is  $T_{CR}$ , the tensile creep rupture strength at the appropriate times and design temperature.

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

where, as defined in PD ISO/TR 20432, Table 3,

$T_{char}$  is the characteristic short term strength; and

$RF_{CR}$  is the reduction factor for creep.

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

where,

$f_m$  is the material safety factor; and, as defined in PD ISO/TR 20432, Table 3,

$RF_{ID}$  is the reduction factor for installation damage;

$RF_W$  is the reduction factor for weathering;

$RF_{CH}$  is the reduction factor for chemical / environmental effects; and

$f_s$  is the factor of safety for the extrapolation of data (PD ISO/TR 20432, 10.1).

The design strength for the ultimate limit is state (see 5.3.3.3) is to be calculated as

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

- b) For the serviceability limit state, the base strength ( $T_B$ ) is  $T_{CS}$ , which induces the prescribed limiting value of strain in the reinforcement as described in 5.3.3.3 for various structures.

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

where,

$f_m$  is the material safety factor, and, as defined in PD ISO/TR 20432, Table 3,

$RF_{ID}$  is the reduction factor for installation damage;

$RF_W$  is the reduction factor for weathering;

$RF_{CH}$  is the reduction factor for chemical/environmental effects; and

$f_s$  is the factor of safety for the extrapolation of data (PD ISO/TR 20432, 10.1).

The design strength for the serviceability limit is state (see 5.3.3.3) is to be calculated as

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

All reduction factors and the overall factor of safety are determined in accordance with the methods described in PD ISO/TR 20432.

*NOTE* Corresponding reduction factors for the same material for the ultimate and serviceability limit state can be different.



**Annex B (normative) Projects for the Highways Agency, Scottish Executive, The National Assembly for Wales and The Department for Regional Development Northern Ireland**

Design of permanent walls and bridge abutments for these projects should conform to the following:

- a) design life of 120 years;
- b) only hard facings shall be used and consist of one or more of the following:
  - 1) concrete (either in situ or precast units);
  - 2) galvanized carbon steel sheets or grids;
  - 3) proprietary material having a current third-party certificate<sup>4)</sup>.

Any restrictions or limitations on use of materials or techniques in this Standard should be superseded by the details in relevant third party certification.

This standard lists standard materials permitted in reinforced soil works (see also 3.5 and Figure 8); any non-standard material, i.e. proprietary material, should have third party certification for the particular product and the requirements laid down therein should be followed.

In places in this standard which refer to calibration checking, approval, etc., these activities should be as agreed with the appropriate authority above.

**Annex C (informative) Propping forces**

The deformation of full height facings is controlled through the temporary use of props to support the face during placement of the fill. The use of props is a simple construction technique and the horizontal load supported by the props  $P_L$  may be determined from:

$$P_L = \frac{K_a \gamma h_t^2}{6}$$

where  $h_t$  is the height of the fill above the toe of the facing.

Prop forces less than those developed by the above equation can be achieved by following a specific construction sequence for a wall height  $H$ . A proven sequence of releasing the props when the prop height  $h_p > H/2 < H$  and where the toe of the wall is wedged includes the following steps:

- a) fill to level  $h_t > H/2$ ;
- b) remove the wedge (prop) supporting the toe;
- c) remove the prop.

With this construction sequence the horizontal propping force  $P_L$  may be reduced to:

$$P_L = \frac{K_a \gamma h_t^3}{6h_p}$$

<sup>4)</sup> Third-party certification is accredited by UKAS ([www.ukas.com](http://www.ukas.com)) in the UK and members of the IAF ([www.iaf.nu](http://www.iaf.nu)) in the rest of the world. For example, BBA and BRE are UKAS accredited.

When the structure is built on a soft foundation, the apparent rotation of the face into the fill may further reduce the prop force  $P_L$  (see Kempton et al [33]).

## Annex D (normative) Site damage test

### D.1 General

The following procedure for site damage test should be followed for both metallic and polymeric reinforcements. However, the extent of the test may be curtailed at the discretion of the assessment authority. Other test layouts, configurations and procedures may be considered to those detailed in the following clauses.

The purpose of the site damage test should be:

- a) to place the reinforcement under a range of fills that conform to the grading limits of the *Specification for Highway Works* [1] and to compact those fills in accordance with and in excess of that specification;
- b) to recover the reinforcement and measure its tensile strength and stiffness and estimate the site damage;
- c) quantify any loss of strength of the reinforcement due to the construction process.

### D.2 Test site

A level site should be prepared and laid out in nine bays each 3.5 m × 3.5 m as shown in Figure D.1, leaving working space for construction plant to gain access around the test area without crossing the bays.

Where the reinforcement is planned to be used within a zone of compacted fill, e.g. a reinforced soil retaining wall or fill slope, a 150 mm thick layer of material (or 1.5 times the maximum fill size, whichever is the greater) should be placed and compacted in each bay prior to the installation of the reinforcement. This material should be the same as that to be placed in the layer above the reinforcement.

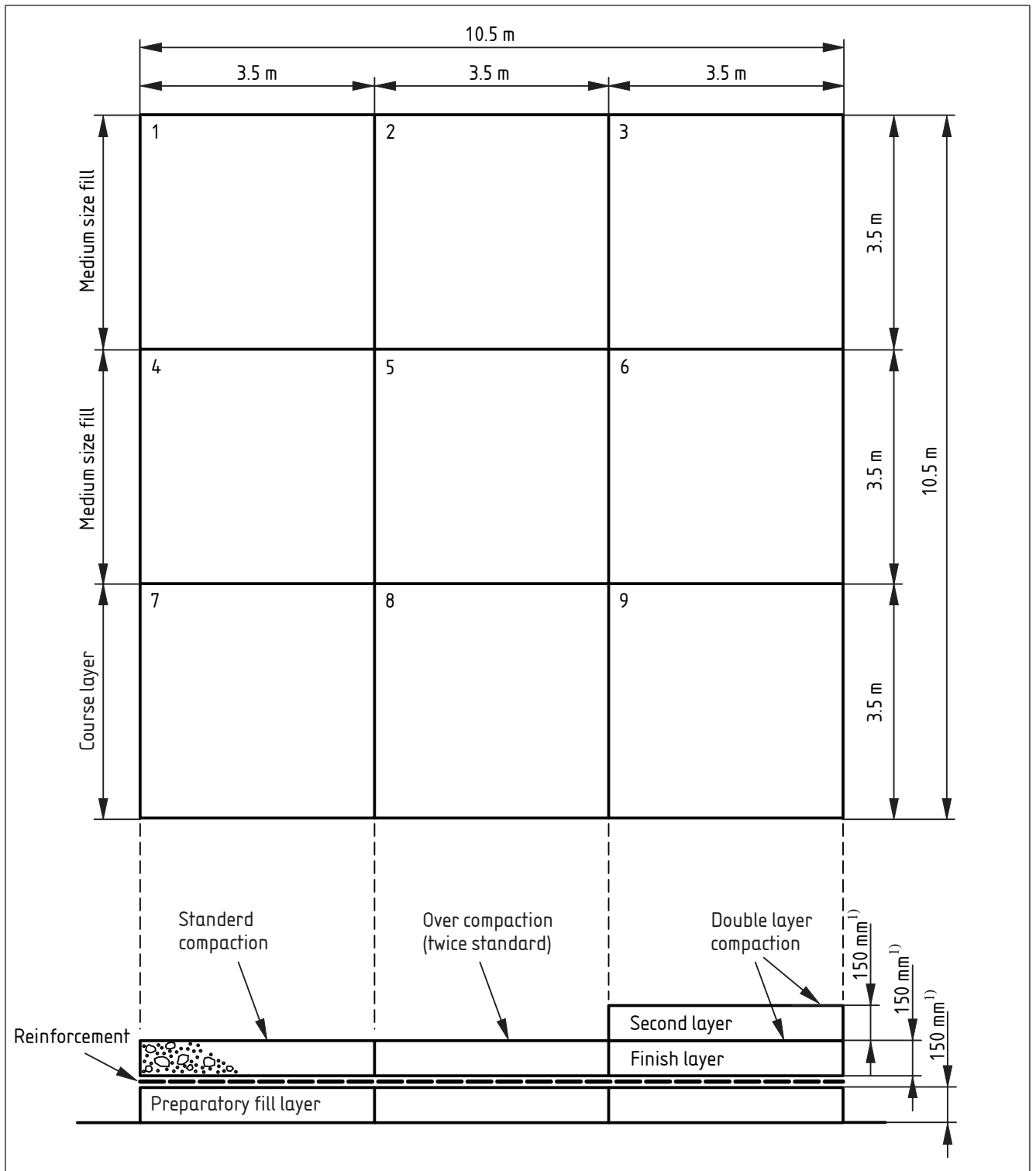
Where the reinforcement is planned to be used beneath a zone of compacted fill, e.g. a basal reinforced embankment on soft soil, a 150 mm thick layer of the typical foundation soil (or 1.5 times the maximum fill size, whichever is the greater) should be placed in each bay prior to the installation of the reinforcement.

### D.3 Arrangement of the reinforcement

Sufficient portions of reinforcement each 10.5 m long should be prepared. These should be placed across each bay as shown in Figure D.1. No tension should be applied to the reinforcement.

The QC roll number and/or batch number of the reinforcement should be recorded. Sufficient unused reinforcement from this batch should be retained and prepared for tensile testing as control samples.

Figure D.1 Schematic layout of test bays



#### D.4 Fill materials

Three different gradings of the fill material proposed for use should represent the coarse, middle and fine fill.

The fill material may be frictional fill, cohesive frictional fill or other materials as defined elsewhere in this code of practice.

Each layer of material should be compacted to a thickness of 150 mm or 1.5 times the maximum particle diameter, whichever is greater.

Particle size distribution for each type of fill should be determined by dry sieve analysis.

Fill should not be end tipped onto the reinforcement but should be spread over by bulldozer.

### D.5 Compaction plant

Compaction of the fill should be carried out in accordance with the *Specification for Highway Works* [1].

The roller mass per metre width and number of passes should be selected in accordance with 9.2.3.2 and Table 6/4 in the *Specification for Highway Works* [1].

The direction of traverse of the compaction equipment should be reported with respect to the orientation of the reinforcement.

### D.6 Compaction

A maximum of three levels of compaction should be used in the trial.

- a) *Standard compaction*. The number of passes of the roller to compact the fill to the selected layer thickness should be in accordance with 9.2.3.2 and Table 6/4 of the *Specification for Highway Works* [1].
- b) *Over-compaction*. Twice the number of passes as specified in a) should be used to compact the fill to the selected layer thickness.
- c) *Double-layer compaction*. The formation of two layers of selected compacted thickness. The number of passes of the roller as given in a) to compact the first layer of fill to the selected layer thickness. A second layer of fill should then be spread and compacted to the selected layer thickness by the number of passes of the roller as given in a).

*NOTE* A compaction trial may have to be carried out in advance of the site damage trial to determine the uncompacted fill thickness that will compact to the selected layer thickness under the three compactive efforts.

### D.7 Site testing

The reinforcement should be subjected to a variety of fill materials and compactive efforts as shown in Figure D.1.

Levels should be taken on a 1 m<sup>2</sup> grid within each bay after completion of each layer to determine the mean layer thickness.

Variations in surface levels of the compacted fill should not exceed  $\begin{matrix} +20\text{mm} \\ -30\text{mm} \end{matrix}$  of the true finished level as measured on a 1 m<sup>2</sup> grid.

### D.8 Recovery of the reinforcement

After completion of the site testing the compacted fill should be manually removed from all bays.

Those pieces of reinforcement that are accidentally damaged by spades should not be used in subsequent tests.

### D.9 Preparation of samples

Three specimens should be prepared from each length of the reinforcement for visual assessment of site damage and tensile testing.

### D.10 Visual assessment of site damage

A visual assessment of the site damage to each sample should be made and recorded.

Damage should be classified into four categories.

- a) *General abrasion*. This describes the condition of the reinforcement when damaged by contact with many small stones which leave the surface of the reinforcement covered in small scratches and abrasions.
- b) *Splits, cuts, bruises and coating removal*. This describes the damage caused by the action of larger particles.
  - 1) A split should describe the region of a strip or rib when locally split into a number of small strands so that light passes through.
  - 2) A cut should describe the rib or strip when a sharp indentation is made across or along the reinforcement, such that fibres are cut or the cross-section of the rib is reduced.
  - 3) A bruise should describe the rib or strip when flattened but no light passes through.
  - 4) Coating removal applies when the coating or sheath has been removed locally, leaving the reinforcing fibres visible.

Visual inspection of the coatings of metallic reinforcements should be the same as that made for polymeric reinforcements.

### D.11 Reinforcement test method

The reinforcement should be tested in accordance with a recognized test method. For metallic reinforcements this should be done according to BS 1449-1. For polymeric reinforcements this should be done according to BS EN ISO 10319. For reinforcements, it is sufficient to cut parallel specimens; a diagonal arrangement as specified in BS EN ISO 9862 is not necessary.

Both the control samples and damaged samples should be tested.

The tensile strength, peak extension and the classified damage should be reported for each site damaged specimen and comparisons made with the properties of the control sample.

## Annex E (normative)

### Design of base slabs supporting vehicle parapets for highways

#### COMMENTARY ON ANNEX E

*Guidance is given in this annex on the design of base slabs supporting vehicle and vehicle/pedestrian parapets founded on reinforced soil structures, and also on determining the load effects to be used in the design of reinforced soil structures which support such base slabs.*

*The design of reinforced concrete base slabs to resist vehicular collision loads on the metal, concrete, and combined metal and concrete vehicle and vehicle/pedestrian parapets which they support is given. Concrete*

components may be either pre-cast or cast in situ. The design of the reinforced soil structure on which such base slabs are founded is covered.

This Annex is not applicable to:

- a) parapet-supporting structures, which form an integral part of a bridge deck, bridge abutment or retaining wall - including the facing units to a reinforced soil structure;
- b) base slabs which are not founded directly on a strengthened/reinforced soil structure;
- c) base slabs which support parapets that are not intended to contain vehicles.

Further, this annex does not cover:

- 1) the design of parapets, their attachment systems and anchorage units (see BD 52/93 [133]);
- 2) the geometrical requirements of plinths to metal parapets and to those that form part of a combined metal and concrete parapet (see BD 52/93 [133]).

## **E.1 Definitions**

### **E.1.1 vehicle restraint system**

installation to provide a level of containment for an errant vehicle which may be used to limit damage or injury to users of the highway

### **E.1.2 highway parapet**

barrier at the edge of a bridge, or on top of a retaining wall or similar structure, associated with a highway

### **E.1.3 vehicle parapet**

highway parapet that acts as a vehicle restraint system

### **E.1.4 vehicle pedestrian parapet**

vehicle parapet with additional safety features for pedestrians and animals

### **E.1.5 attachment system**

system of attachment of the parapet to the anchorage, usually consisting of holding-down bolts

### **E.1.6 anchorage**

that part contained within the parapet-supporting base slab to which the parapet is directly fixed by means of the attachment system

## **E.2 Design of base slabs to resist vehicular collision effects**

### **E.2.1 General**

Base slabs supporting vehicle and vehicle/pedestrian parapets should be designed for vehicular collision effects on the supported parapets. The load effects that should be taken into account include those due to dead loads, vehicle collision or parapet failure, associated live loads, wind and earth pressures.

To facilitate the replacement or repair of a parapet following a vehicle collision, a progressive increase in resistance should be provided from the point of impact to the supporting base slab.

A base slab should be designed to resist all the loads which the parapet is capable of transmitting, up to and including failure in any mode that might be induced by a vehicle collision, without damage to the base slab or the structure upon which it is founded.

*NOTE* The loads transmitted by a vehicle collision generate:

- a) local effects, i.e. acting on the supporting elements in the vicinity of the impact; and
- b) global effects, i.e. acting on the structure as a whole.

The recommendations for considering a) and b) vary according to the relative masses of the containment system and supporting structure because these control the degree of attenuation of the collision forces.

With normal and higher levels of containment parapets the design loads due to vehicle collision could be exceeded and the parapet posts should be designed to achieve their full plastic moment before either the attachment system or anchorage fails, so design of the supporting base slab is based on the ultimate capacity of the parapet posts. Therefore the base slab for normal and higher levels of containment parapets should be designed to take account of loads due to local effects only. The global effects of vehicle collision need not be considered for normal and higher levels of containment parapets.

However, very high level of containment parapets resist much greater design loads due to vehicle collision. Therefore the base slab for very high level of containment parapets should be designed to take account of loads due to both local effects and global effects.

The local effects of the loads resulting from a vehicle collision with a normal, higher or very high level of containment parapets should be considered in the design of the elements supporting the parapet. The local effects of the loads should be considered for:

- a) sliding of the base slab;
- b) toppling of the base slab;
- c) rupture of the base slab.

For normal and higher levels of containment parapets, the failure of the supporting strengthened/reinforced soil structure, as given in E.3, should not be considered because only local effects need to be dealt with.

The global effects of the loads resulting from a vehicle collision with very high level of containment parapets should be considered in the design of the elements supporting the parapet, but such effects need not be considered for collisions with other types of parapet. The global effects of the loads should be considered for:

- 1) sliding of the base slab;
- 2) toppling of the base slab;
- 3) rupture of the base slab;
- 4) the failure of the supporting strengthened/reinforced soil structure (see E.3).

Global effect loads should be considered separately from local effect loads.

*NOTE* The recommended design sequence is summarized in Figure E.1.

### E.2.2 Design life

The design life of a base slab to a detachable parapet should be as in Table 7. The design life of a base slab constructed integrally with the parapet should be taken to be the same as for the parapet.

### E.2.3 Structural adequacy

A partial factor limit state approach to design should be adopted. With this the design value of a load  $Q^*$  is determined from its nominal value  $Q_k$  by:

$$Q^* = \gamma_{fL} \cdot Q_k$$

The design load effect  $S^*$  is obtained from the design load by:

$$S^* = \gamma_{f3} \cdot (\text{effects of } Q^*)$$

The design resistance  $R^*$  is defined as:

$$R^* = \text{function}(f_{des}) = \text{function}(f_k)$$

where

$f_{des}$  is the design strength;

$f_k$  is the characteristic strength or its equivalent nominal value.

And for all appropriate combinations of load effects, the following shall be satisfied:

$$\sum R^* \geq \sum S^*$$

where  $\sum R^*$  and  $\sum S^*$  are the summed design resistances and load effects respectively.

### E.2.4 Limit states

To ensure both an adequate degree of safety and serviceability, parapet-supporting base slabs should be designed for both the ultimate and serviceability limit states.

For the *ultimate limit state* (ULS), design should ensure that the structure is sufficiently strong and stable to withstand the design load effects, taking due account of the possibility of toppling, sliding and rupture.

For the serviceability limit state (SLS), design should ensure that under normal service conditions the structure will not suffer damage that would reduce its intended service life or incur excessive maintenance costs. Due account should be taken of the possibility of excessive movements induced by vehicular collision on the supported parapet.

### E.2.5 Partial factors

The following partial factors are used:

- a)  $\gamma_{fL}$  – the load factor whose value should take account of the possibility of an unfavourable deviation of a load from its nominal value, and of the reduced probability that various loads acting together will attain their nominal values simultaneously; values of  $\gamma_{fL}$  for various loads are given in BD 37/01 [134]: values applicable to the loads generated on a parapet through a vehicle collision are given in below for ULS and SLS;
- b)  $\gamma_{f3}$  – the load effect factor whose value should take account of the inaccurate assessment of the effects of loading, unforeseen



stress distribution within the structure, and variations in the dimensional accuracy achieved in construction;

values of  $\gamma_{f3}$  for various cases are given in BD 24/92 [135]: values applicable to the design of a parapet-supporting base slab are given in below;

- c)  $\gamma_m$  – the material factor whose value should cover for possible reductions in the strength of the materials in the structure as a whole, compared with the characteristic or nominal value deduced from control test specimens, and possible weaknesses in the structure due to, for example, manufacturing tolerances and compaction operations;

values of  $\gamma_m$  for concrete components are given in BD 24/92 [135].

### E.2.6 Loads

The nominal value of a load should be appropriate to a return period equal to the design life of the base slab: appropriate values for various loads for a return period of 120 years are given in BD 37/01 [134]. The nominal loads arising from a vehicle collision with a parapet are given below.

The combinations of load to be considered in design, and the values of  $\gamma_{fL}$  for each combination, should be in accordance with BD 37/01 [134] except where superseded by this Annex. Where the combined dead load of the self weight of the base slab and fill on top of the base slab have a disturbing effect, the value of  $\gamma_{fL}$  applied to the dead load shall be increased to 1.5 in accordance with the recommendations of Section 6.

### E.2.7 Loads due to vehicle collision with parapets

Clause 6.7 of BD 37/01 [134] should be superseded by the following.

Loads due to vehicle collision with normal, higher and very high levels of containment parapets for determining “local effects”.

Nominal collision loads: in the design of a parapet-supporting base slab and the strengthened/reinforced soil structure on which the base slab is founded, the following nominal collision loads resulting from a vehicle collision with a parapet should be considered.

For concrete parapets (normal, higher and very high levels of containment) – the calculated ultimate design moment of resistance and the calculated ultimate design shear resistance of any 4.5 m length of parapet applied uniformly over that length (i.e. with  $\gamma_m = 1$ ).

For metal parapets (normal, higher and very high levels of containment) – the nominal collision loads are the more critical of a) or b) below.

- a) the calculated ultimate design moment of resistance of a post applied at each base of up to three adjacent posts combined with a shear force which is the lesser of:
- 1) the calculated force from ultimate design moment of resistance of a parapet post, ie with  $\gamma_m = 1$ , divided by the height of the centroid of all the effective longitudinal members above the base of the parapet applied at each base of up to any three adjacent parapet posts, or,
  - 2) the calculated ultimate design shear resistance of a parapet post, ie with  $\gamma_m = 1$ , applied at each base of up to any three adjacent parapet posts.

- b) the calculated ultimate design moment of resistance of a post applied at the base of a single post combined with a shear force which is the lesser of:
- 1) the calculated ultimate design moment of resistance of the parapet post, ie with  $\gamma_m = 1$ , divided by the height of the centroid of the lowest effective longitudinal member above the base of the parapet applied at the base of the post, or
  - 2) the calculated ultimate design shear resistance of a parapet post, ie with  $\gamma_m = 1$ , applied at the base of the post.

In the case of very high level of containment parapets, an additional single vertical load of 175 kN should be applied uniformly over a length of 3 m at the top of the front face of the parapet. The loaded length should be in that position which will produce the most severe effect on the member under consideration.

Associated nominal primary live load: where it has an adverse effect, the nominal primary live load should be represented by the HA surcharge load of 10 kN/m<sup>2</sup> and should be considered to act in combination with the loads arising from a vehicle collision with a parapet. The associated nominal primary live load may be taken to act only on the plinth base slab and not on any surfaces adjacent to the plinth base slab. The associated nominal primary live load should be ignored where it has a relieving effect.

Load combination: the loads arising from a vehicle collision with a parapet should be considered in combination 4 only, as defined in Table 1 of BD 37/01 [134], and need not be taken to coexist with other secondary live loads.

Design load: the values of  $\gamma_{fL}$  to be applied to the nominal collision loads and the associated nominal primary live load should be as follows:

Loading	$\gamma_{fL}$ values for the ultimate limit state	
	Normal and higher levels of containment parapets	Very high level of containment parapet
Loads arising from a vehicle collision with a parapet	1.50	1.40
Adverse associated primary live load	1.30	1.30

Loads due to vehicle collision with very high level of containment parapets for determining "global effects".

Nominal collision loads: in the design of the base slab and the supporting strengthened/reinforced soil structure, the following nominal collision loads should be applied at the top of the traffic face of a very high level of containment parapet:

- 1) a horizontal transverse load of 500 kN;
- 2) a horizontal longitudinal load of 100 kN;
- 3) a vertical load of 175 kN.

The loads should be applied uniformly over a length of 3 m measured along the line of the parapet. The position of the loaded length

should be such that it produces the most severe effect on the part of the structure under consideration.

Associated nominal primary live load: where it has an adverse effect, the nominal primary live load should be represented by the HA surcharge load of  $10 \text{ kN/m}^2$  and should be considered to act in combination with the loads arising from a vehicle collision with a very high level of containment parapet. The associated nominal primary live load may be taken to act only on the plinth base slab and not on any surfaces adjacent to the plinth base slab. This load should be applied so that it will have the most severe effect on the element under consideration. However the nominal primary live load should be ignored where it has a relieving effect.

Load combination: the loads arising from a vehicle collision with a parapet should be considered in combination 4 only, as defined in Table 1 of BD 37/01 [134], and need not be taken to coexist with other secondary live loads.

Design loads: the values of  $\gamma_{fL}$  to be applied to the nominal collision loads and the associated nominal primary live load shall be as follows:

Loading	$\gamma_{fL}$ values for the ultimate limit state
Loads arising from a vehicle collision with a very high level of containment parapet	1.40
Adverse associated primary live load	1.25

### E.2.8 Earth pressures

The magnitude and distribution of earth pressures should be calculated in accordance with the principles of soil mechanics.

### E.2.9 Pore water pressures

Where necessary account should be taken of pore water pressures, but the stabilising effect of negative pore water pressures should be ignored.

### E.2.10 Dispersal of load

It may be assumed that the load effects are dispersed vertically through any upstand plinth to the base slab as well as longitudinally through the base slab at a length-to-depth ratio of one longitudinally to one vertically and a length-to-width ratio of one longitudinally to one transversely, as shown in Figure E.2. Longitudinal dispersal should not be assumed to occur across a transverse joint in the base slab unless adequate shear connection is provided.

### E.2.11 Ultimate limit states of base slab foundation

For the design of the foundation for those load arrangements associated with the collapse of a parapet, which may or may not include the associated primary live load,  $\gamma_{f3}$  should be taken as 1.0. For all other load combinations  $\gamma_{f3}$  should be taken as 1.1 for the ULS, and 1.0 for the SLS.

- a) Toppling of base slab: the potential for rotation about the toe of the base slab should be checked, with the following condition being satisfied for all appropriate combinations of load:

$$\Sigma M_R \geq \gamma_{f3} \times \Sigma M_D$$

where

$\Sigma M_R$  is the sum of the design restoring moments;

$\Sigma M_D$  is the sum of the design disturbing moments.

Note that sizing a base slab by considering toppling alone may lead to high bearing pressures at the toe of the base slab, and the dimensions of a base slab may be governed by the requirements to limit settlement of the base slab (see E.2.18) and the pressures acting on the supporting structure beneath it (see E.3).

- b) Sliding of base slab: the potential for sliding between the base slab and the founding strengthened/reinforced soil structure should be checked, with the following condition being satisfied for all appropriate combinations of load:

$$\Sigma R_S^* \geq \gamma_{f3} \times \Sigma P_T$$

where

$\Sigma R_S^*$  is the sum of the design sliding resistances generated along the potential failure plane;

$\Sigma P_T$  is the sum of the design load components acting parallel to the potential failure plane.

The design sliding resistance at the soil/concrete interface should be assessed, using effective stress parameters, through the relation:

$$R_S^* = P_N \alpha (\tan \phi'_p) / f_{ms}$$

where

$P_N$  is the design value of the normal component of the resultant force acting on the interface;

$\alpha$  may be assumed to have a value of 1.0 for cast in situ base slabs and 0.75 for pre-cast base slabs, but different values may be used where they are supported by measurements;

$\phi'_p$  is the peak angle of shearing resistance of the fill beneath the base slab;

$f_{ms}$  is a partial factor against sliding, which should be taken as 1.2.

The effects of effective cohesion should be ignored.

## E.2.12 Serviceability limit state of base slab foundation

### E.2.12.1 Introduction

Differential settlement of base slab under collision loading: excessive tilting of the base slab may lead to spalling of concrete around the junction of the base slab and uppermost facing panel. Permanent deflection may affect the appearance due to a poor line along the stringcourse, make it difficult to realign the replacement parapet, and perhaps lead to cracking of the road pavement at the end of the slab. The dimensions of a base slab may therefore be governed by the requirement to limit the settlement of the slab.

Transient, and perhaps also permanent, deflection of a base slab might be generated by concentrated traffic loads acting on the base slab and by a vehicle collision on the parapet. Thus differential settlement can be assessed by considering the effects of the design traffic load and of vehicle collision loading acting independently of the other loads.

#### E.2.12.2 Load model

For convenience, the primary traffic load can be modeled using the Type HA surcharge load, which is a uniformly distributed load of 10 kN/m<sup>2</sup>.

#### E.2.12.3 Limiting deflection

Whilst it is difficult to establish a limiting deflection for all situations, in most cases it would be appropriate to limit the effect of loads (other than dead loads) to the generation of a settlement of 5 mm at the toe of the slab.

#### E.2.12.4 Method of analysis

There are a number of problems in calculating the deflection of a base slab, not least the possible interaction of the slab and the upper part of the wall: deflection at the toe of the slab might well be increased by any outward movement of the wall. Furthermore, in general, a static analysis will be carried out to represent a dynamic event. A number of options are available including the use of complex methods of analysis, such as finite elements, and 'elastic' stress equations, as given in texts such as Poulos and Davis [136]. In most cases a complex method would not be justified for this limit mode.

When determining the bearing capacity of a base slab, it is essential that the inclination of the applied load be taken into account. A number of methods are available including the following:

- a) That due to Meyerhof [38] where the pressure distribution beneath a base slab is assumed to be uniform over an effective width  $b$  as shown in Figure E.3 and given by;

$$b = B - 2e$$

where

$B$  is the width of the base slab;

$e$  is the eccentricity of the resultant design load acting at the interface.

- b) From equations given in standard texts and standards, such as Annex D of BS EN 1997-1:2004.

#### E.2.12.5 Partial safety factors

There will be some uncertainty in the method of analysis used to estimate settlement. The reliance that can be placed on the results of any of the methods is best assessed by the designer: in this case it would be inappropriate to substitute engineering judgement by a fixed value of  $\gamma_{f3}$ .

Similarly, the appropriate value of  $\gamma_m$  to be applied to the measure of "stiffness" will vary widely according to its derivation. For the sake of simplicity and economy in design, a value of  $\gamma_m$  of unity should be used in combination with a reasonably cautious estimate of "mean stiffness" of the backfill over the appropriate stress range.

The partial factors for the loads to be considered shall be as follows:

Loading	$\gamma_{fl}$ values for the serviceability limit state	
	Normal and higher levels of containment parapet	Very high level of containment parapet
Dead load of base slab	1.0	1.0
Dead load of fill on top of the base slab	1.0	1.0
Loads arising from vehicle collision with a parapet	1.20	1.15
Associated primary type HA traffic loading	1.10	1.10

#### E.2.12.6 Input data

All the methods of analysis require some measure of the stiffness of the backfill. This could be assessed from the results of laboratory or field tests on the material, or be based on published data tempered by experience. It should be appreciated that the stiffness of a soil under a dynamic load can be substantially higher than under a static load.

Stiffness may be derived from the results of a one-dimensional compression test or a plate bearing test – and expressed for example as an  $E$  or  $M^*$  value (see, for example, BD 12/01 [137]). It is difficult to ensure that the in situ characteristics of the material are reproduced by the laboratory test specimens. The results of plate bearing tests would also have to be extrapolated to the dimensions of the base slab.

#### E.2.13 Structural design of base slab

Base slabs should be designed in accordance with BD 24/92 [135] except as noted below.

Only load combinations 1 and 4 at the ULS, as given in Table 1 of BD 37/01 [134], should be considered.

When a parapet is designed to be monolithic with its foundation slab, the parapet should be designed with a predetermined failure section, ideally within the stem of the parapet and which is clearly visible to allow identification after an impact. Failure within the base slab should not be an option. Also when parapets are constructed monolithic with their foundations, consideration must be made to the practicality of repair or replacement of the parapet.

For convenience, the live load surcharges should be in accordance with Clause 5.8.2.1 of BD 37/01 [134], ie Type HA load is a uniformly distributed load of 10 kN/m<sup>2</sup>; Type HB load is a uniformly distributed load where 45 units HB = 20 kN/m<sup>2</sup>, 30 units HB = 12 kN/m<sup>2</sup> and intermediate values are obtained by interpolation.

#### E.2.14 Durability

Concrete cover to reinforcement of the base slab should be in accordance with BD 57/01 [138].

To prevent excessive cracking in the concrete base slab due to shrinkage and thermal movements, reinforcement should be provided in accordance with BD 28/87 [139].

Figure E.1 Design sequence for parapet-supporting base slabs on strengthened/reinforced soil structures

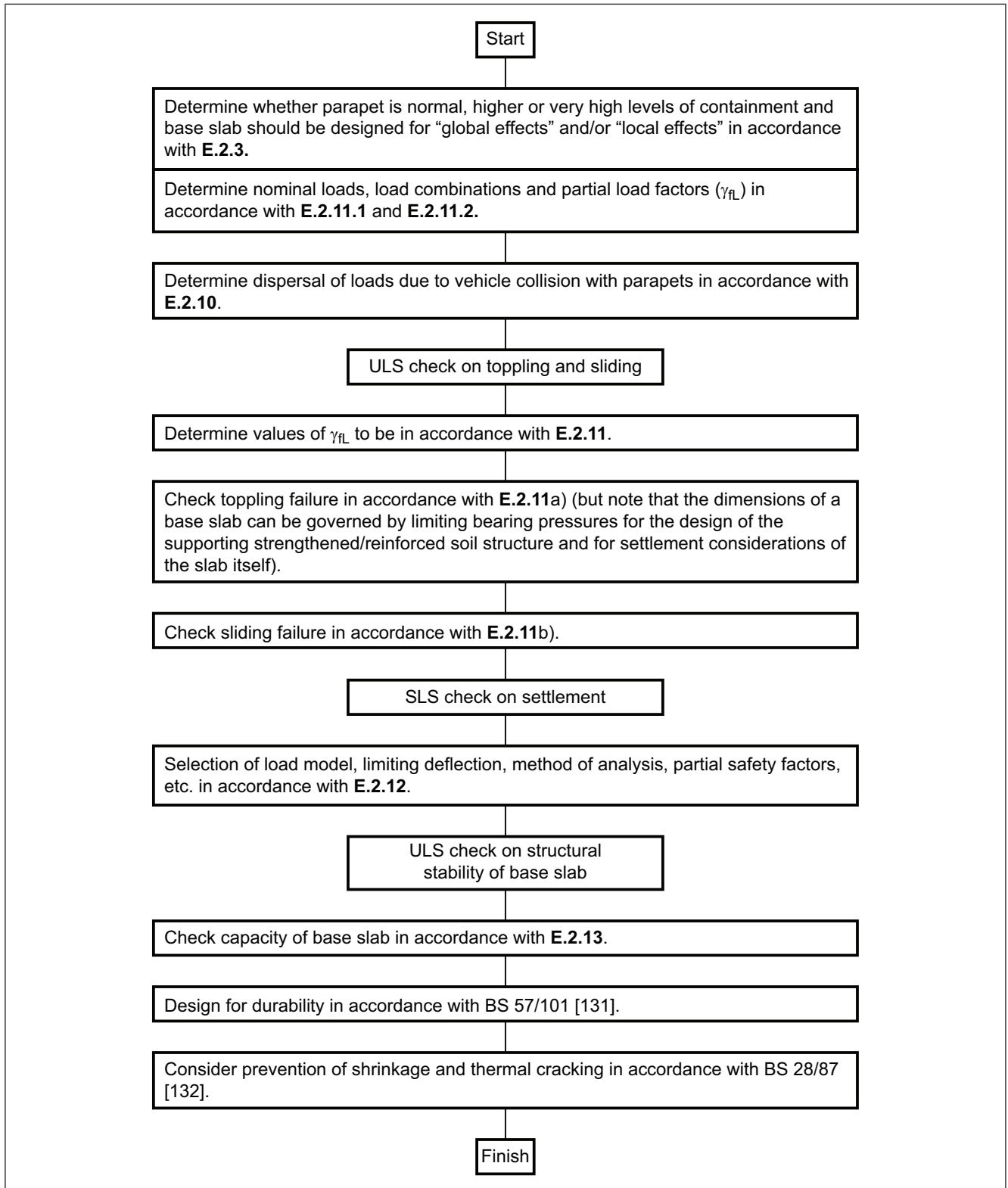


Figure E.2 Dispersal of load effects through a base slab

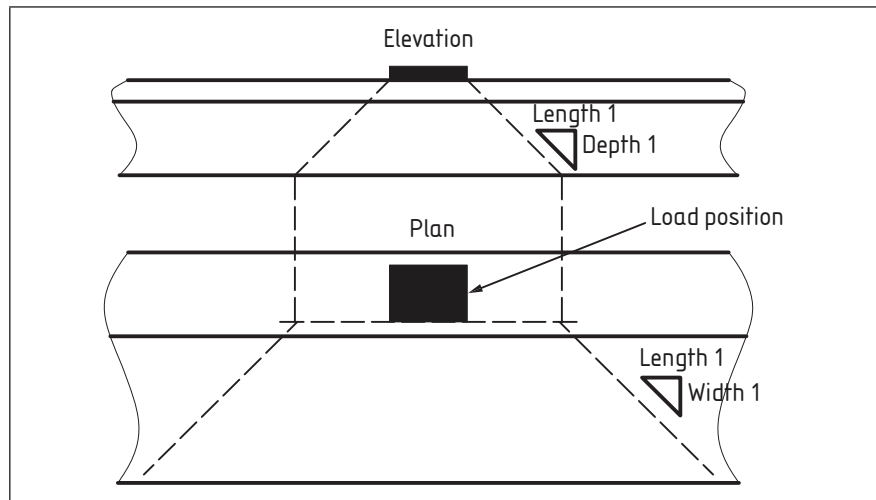
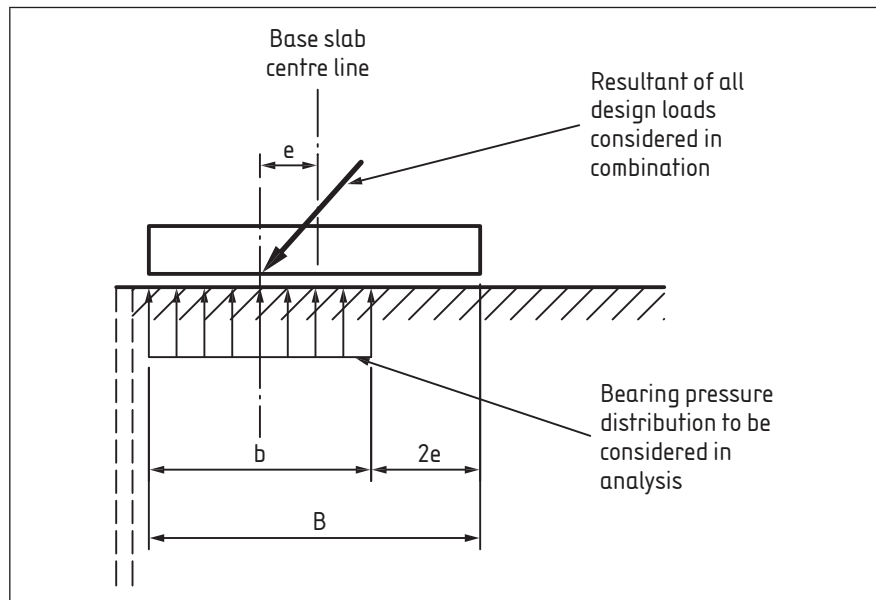


Figure E.3 Distribution of pressure beneath a base slab



### E.3 Design of strengthened/reinforced soil structures supporting base slabs to very high level of containment parapets

#### E.3.1 General

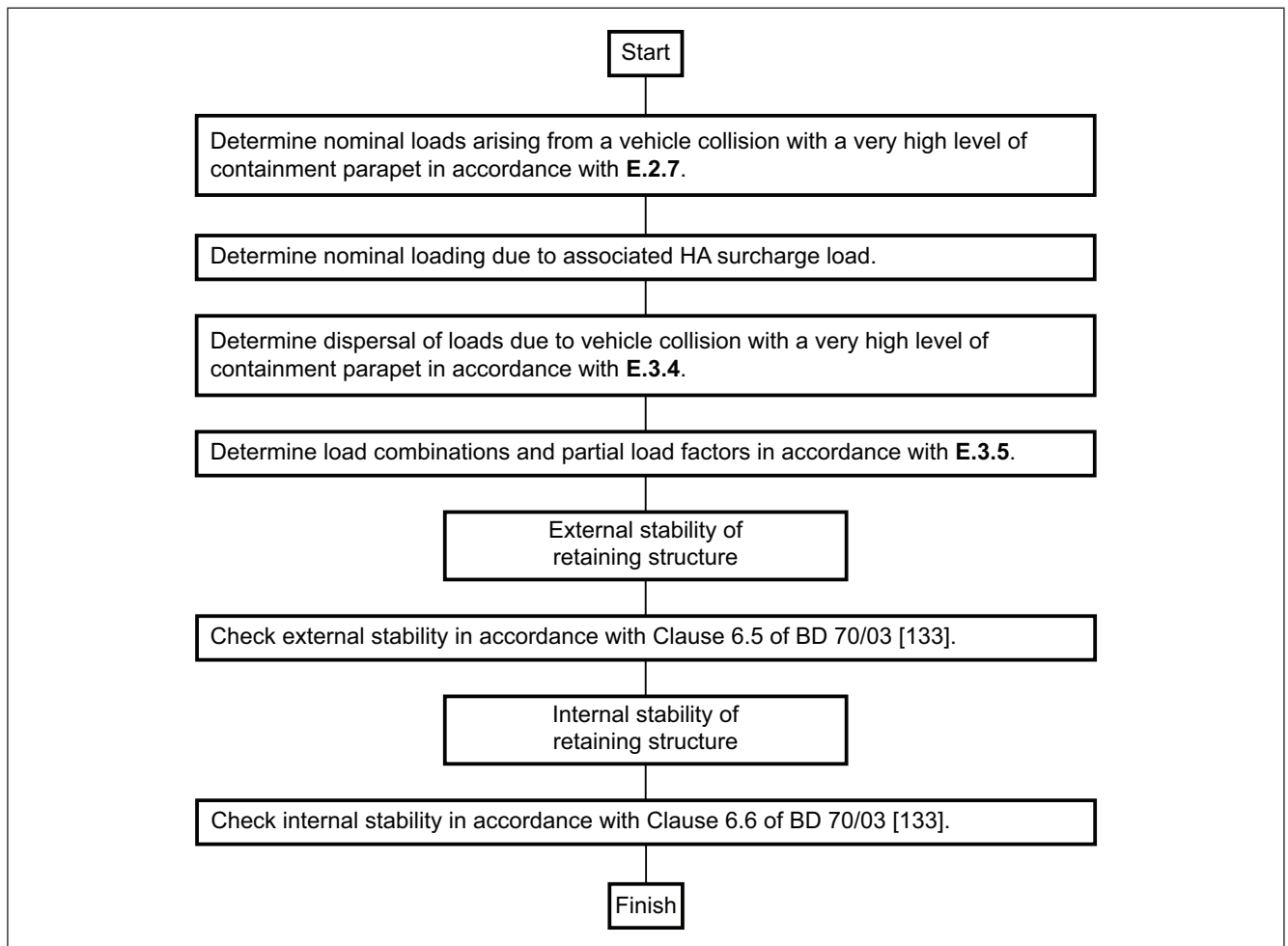
A strengthened/reinforced soil structure which supports a base slab to a very high level of containment parapet should be designed to resist the load effects resulting from a vehicle collision with the very high level of containment parapet and an associated HA load. The load effects resulting from vehicle collisions with very high level of containment parapets should be determined in accordance with E.3.

This section should not be applied to strengthened/reinforced soil structures that support base slabs to parapets of normal and higher levels of containment.

The recommended design sequence is summarized in Figure E.4.



Figure E.4 Design sequence for a strengthened/reinforced soil structure that supports the base slab to a high level of containment parapet



### E.3.2 Design approach

A limit state approach to design should be followed.

### E.3.3 Design life

The design life of a strengthened/reinforced soil structure supporting the base slab to a very high level of containment parapet should be in accordance with Table 7.

### E.3.4 Loads

The loads on the strengthened/reinforced soil structure on which the base slab to a very high level of containment parapet is founded should be determined in accordance with Section 6 and Section 7 except where superseded by this annex.

Loads arising from a vehicle collision with a very high level of containment parapet should be in accordance with E.2.1.1 and E.2.7.

The horizontal and vertical load effects on a strengthened/reinforced soil structure arising from a vehicle collision with a very high level of containment parapet should be uniformly applied to an effective width determined according to E.2.10 and as shown in Figure E.2. The

horizontal and vertical load effects should be dispersed within the reinforced soil structure in accordance with Section 6.

The pressure distribution beneath a base slab should be assumed to be uniform over an effective width  $b$  as given in E.2.12 and shown in Figure E.3.

### E.3.5 Load combinations and partial load factors

The combinations of loads and partial load factors for each combination should be in accordance with Section 6 and Section 7. In addition, application of the collision load effects on the strengthened/reinforced soil structure due to the base slab of a very high level of containment parapet should be in accordance with E.2.1.1 and E.2.7.

## E.4 Materials and construction

### E.4.1 General

Structural concrete should meet the requirements of the 1700 Series of Clauses of the *Specification for Highway Works* [1] and should be not less than Grade 30.

To prevent corrosion of the reinforcement, promoted for example by the action of de-icing salts, all exposed surfaces adjacent to the carriageway should be protected in accordance with BD 43/03 [140] and the 1700 Series of Clauses of the *Specification for Highway Works* [1].

The buried upper surface of a slab should be finished and waterproofed with a Permitted Waterproofing System in accordance with BD 47/99 [141] and the 2000 Series of Clauses of the *Specification for Highway Works* [1]. All other buried surfaces of the base slab should be waterproofed in accordance with the 2000 Series of Clauses of the *Specification for Highway Works* [1].

### E.4.2 Construction details

Typical construction details for parapet-supporting base slabs on strengthened/reinforced soil walls are presented in Figure E.5 and Figure E.6.

The minimum dimensions of the space between the parapet base slab and the facing unit shall be:

Vertical ( $d_v$  on Figure E.5): 20 mm

Horizontal ( $d_H$  on Figure E.6): 10 mm

### E.4.3 Compressible filler

The thickness and compressibility of the filler should be chosen to avoid loads being transferred from the parapet base to the facing units.

Figure E.5 Illustrative detail of a parapet-supporting base slab at the top of a strengthened/reinforced soil wall with in situ concrete coping

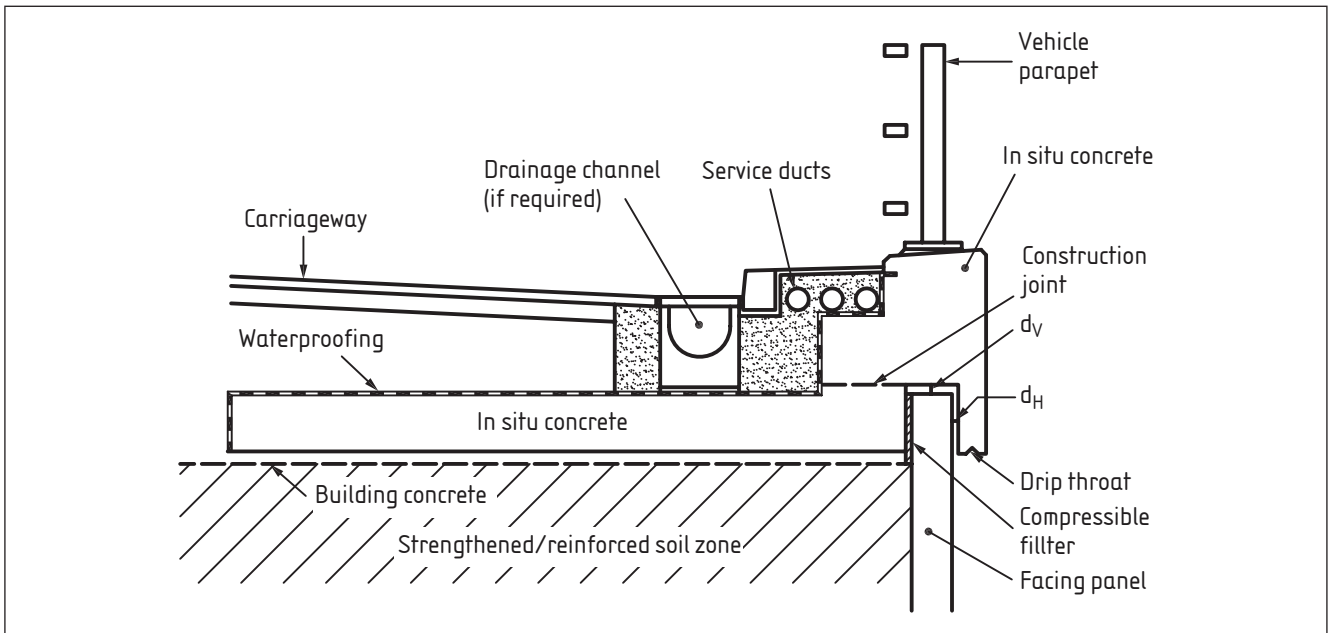
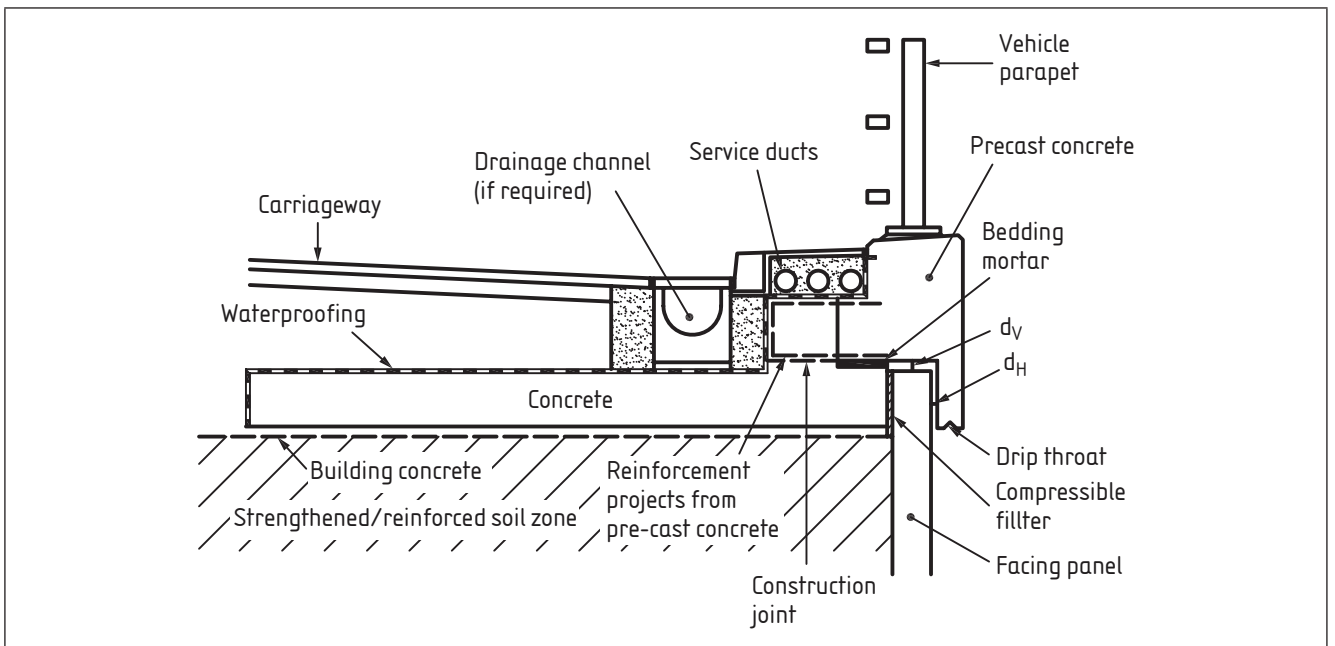


Figure E.6 Illustrative detail of a parapet-supporting base slab at the top of a strengthened/reinforced soil wall with pre-cast concrete coping



## Annex F (informative)

**Design of reinforced soil structures for earthquake resistance**

Reinforced soil structures have been extensively used in active seismic areas and have been found to perform well in many major earthquakes. As such they are a suitable option for designers to consider in such areas. This Annex is not intended to propose specific methods for designing reinforced soil structures for seismic events but is intended to inform designers of the experience in the behaviour of such structures which have been subject to seismic forces and to direct designers to known sources of design information.

While seismic events in the UK are relatively rare and generally of low intensity it is sometimes a requirement for structures to be designed for such events and this annex provides information in this regard.

Reinforced soil structures are known to respond well if subjected to significant foundation movements (such as large settlements on compressible soils or even large and more sudden movements as might occur in areas of mining subsidence). This inherent ability and flexibility also results in the structures offering designers advantages for retaining structures in areas of seismic activity. This ability of reinforced soil results in the structures having a natural resistance to seismic events even if they are not specifically designed to do so. There are several examples of structures in regions not known as seismically active, where seismic load cases were not part of the design procedure, but which have subsequently been subjected to earthquakes and have shown a good behaviour and remained undamaged.

There are several approaches for incorporating seismic forces into reinforced soil designs and these can be found in standards such as AFNOR and AASHTO. The design of earth retaining structures for earthquake loading conditions is dealt with generally in BS EN 1998-5. However the Eurocode does not deal specifically with the design of reinforced soil structures. The Eurocode and most Standards dealing with the seismic design of reinforced soil structures adopt pseudo-static methods for the design models. A force is applied to the structure to represent the effects of the earthquake and it is usual for this force to be expressed as a fraction of the force due to gravity. Standards of national origin set out the design acceleration to be used in the different regions of the world. If the acceleration is given as the maximum horizontal acceleration at ground level this may be converted to design acceleration as follows:

$$\frac{a}{g} = \frac{a_0}{g} \left( 1.45 - \frac{a_0}{g} \right), \quad \text{for } \frac{a_0}{g} \leq 0.45g \text{ and}$$

$$\frac{a}{g} = \frac{a_0}{g}, \quad \text{for } \frac{a_0}{g} > 0.45g$$

where:

- $a$  is the design horizontal acceleration;
- $a_0$  is the maximum acceleration at ground level;
- $g$  is the acceleration due to gravity;
- $a/g$  is the seismic coefficient.

Each method introduces vertical and horizontal forces into the structure to represent the effects of seismic activity, and at the same time reduces

the normal static case load and partial factors to reflect the extreme nature of the possible event. However, experience has shown that for seismic coefficients of around 0.2 or less, the load cases which determine the design are usually the static ones and that the normal static analysis is sufficient to complete a design.

Work by Seed and Whitman [142] and Segrestin and Bastick [143] on reinforced soil with metal reinforcements highlighted design methods appropriate for reinforced soil compared with traditional structures. A recent review of seismic design methods for geosynthetic reinforced soil walls and slopes has been carried out by Bathurst et al [144] which deals with such matters as the effect of the seismic event on the properties of the soil, reinforcing materials and soil/reinforcement interaction, rates of loading, vertical accelerations, failure surface orientations under seismic conditions and circumstances in which it is considered more appropriate to adopt other design approaches such as the Newmark-type sliding block analysis for dynamic loading or numerical modelling techniques.

Clearly a seismic event is unlikely to be accompanied by other maximum load conditions but the design should assume the worst combination of forces which are likely to occur. For these reasons it is normal to set load factors and combinations at levels which minimize vertical loads while maximizing horizontal loads.

Particular care should be taken in the design of reinforced soil structures which are likely to be subjected to seismic loads because of the dynamic nature of the loading event. Experience has shown that such loading conditions can be particularly onerous at points where reinforcing elements are connected to facing units in walls. Such connections should be positive and not rely on friction between facings alone and should be designed at an environmental temperature appropriate to the hottest months at the site of the structure.

The actual strength of polymeric materials at the time of the seismic event may be higher than the design creep rupture or creep strain value as a result of residual strength properties. This may enhance the structure's seismic performance.

For structures such as bridge abutments, often the detailing is more important than a particular seismic analysis, particularly for regions with small seismic coefficients. Invariably the mode of failure of conventional concrete stem bridge abutments under strong earthquake conditions is for the deck to become unseated from its abutment support and fall down on the ground below. In the case of reinforced soil, the deck has to be seated on a bankseat and to allow the deck loads to spread into the fill the bankseat will need to be of a certain width which will automatically mean that the seatings of the beams (therefore the ends of the beams) are set well back from the face of the abutment thus providing added security should the beams move on their seating – this is a key benefit of reinforced soil abutments.

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