AS 4678—2002
(Incorporating Amendment Nos 1 and 2)

AS 4678—2002

Australian Standard®

Earth-retaining structures

STANDARDS Australia

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- Association of Consulting Engineers, Australia
- Australian Industry Group
- Australian Geomechanics Society
- AUSTROADS
- Cement and Concrete Association of Australia
- Concrete Institute of Australia
- Concrete Masonry Association of Australia
- Construction Industry Advisory Council
- Institution of Engineers Australia
- Master Builders Australia
- University of New South Wales
- University of Technology, Sydney

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PREFACE

This Standard was prepared by the Standards Australia Committee, CE-032, Reinforced Soils and Retaining Structures, in response to a call from the building industry for the establishment of a Standard on earth-retaining systems, including reinforced soils.

This Standard incorporates Amendment No. 1 (July 2003) and Amendment No. 2 (August 2008). The changes required by the Amendment are indicated in the text by a marginal bar and amendment number against the clause, note, table, figure or part thereof affected.

The objective of this Standard is to provide designers of earth-retaining structures with design criteria and guidance for use in design applications.

The terms 'informative' has been used in this Standard to define the application of the appendix to which it applies. An 'informative' appendix is only for information and guidance.

As far as practicable, this Standard has been made consistent with the approach taken in the loading code for structures, AS 1170, *Minimum design loads on structures*. This enables the Standard to be used in combination with structure design Standards such as AS 1720, *Timber Structures*, AS 3600, *Concrete Structures*, AS 4100, *Steel Structures*, and AS 3700, *Masonry structures*. Some specific applications are covered by other Standards and documents. For example, HB 77, *Australian Bridge Design Code*, should be used to design earth-retaining structures associated with road bridges.

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STANDARDS AUSTRALIA

Australian Standard

Earth-retaining structures

SECTION 1 SCOPE AND GENERAL

1.1 SCOPE

This Standard sets out requirements and recommendations relating to the design and construction of structures required to retain soil, rock and other materials. It also includes requirements and recommendations for the reinforcement of soil and rock materials.

This Standard does not prescribe specific methods of analysis.

NOTE: Various organizations and authorities may develop detailed guides and specifications based on the principles set out in this Standard.

This Standard is in limit state format.

This Standard does not provide requirements and recommendations for 'revetment type' structures, which are sometimes used to retain soil, rock and other materials at slopes steeper than that which the soil, rock or other material would naturally assume.

The retaining structures encompassed by this Standard are indicated in Figure 1.1.

A1 Facings constructed up to 800 mm high in a Type 3 structure application are not covered by this Standard.

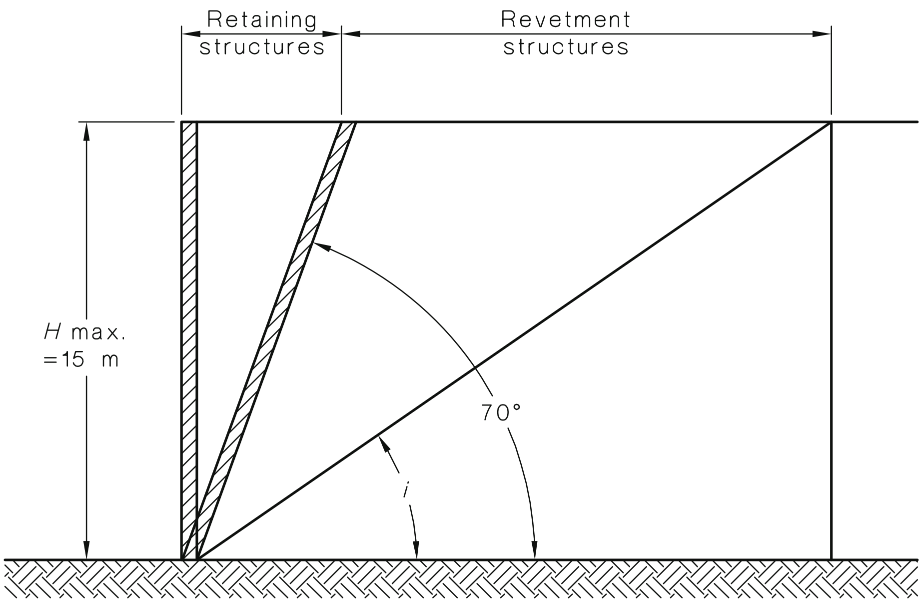


FIGURE 1.1 RETAINING AND REVETMENT STRUCTURES

A1

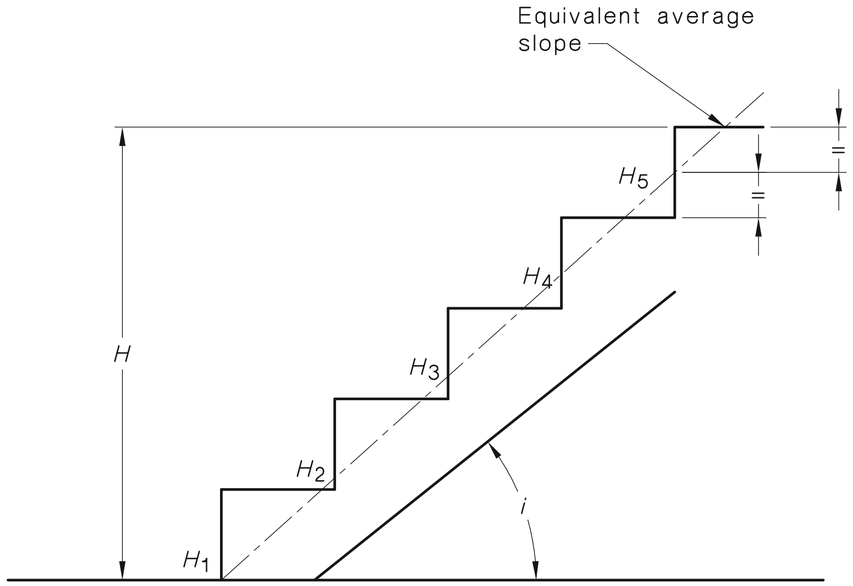


FIGURE 1.2 TERRACED STRUCTURES

1.2 APPLICATION

1.2.1 General

This Standard is applicable to retaining structures and reinforced soil structures that are commonly constructed for engineering works and infrastructure. Such structures are typically up to 15 m in height.

Structures of unusual shape, of large retained heights (in excess of 15 m) or founded in unusual ground conditions (such as soft ground, land slips, steep sides or deeply inclined gullies), together with structures subject to sustained cyclic loading, are outside the provisions of this Standard.

This Standard does not apply to the design and construction of water-retaining structures (such as dams and reservoirs) or bridge structures.

1.2.2 Structure classification

Structures shall be classified in accordance with Table 1.1.

A1 **NOTE: 'Not applicable'**

TABLE 1.1

STRUCTURE CLASSIFICATION

Classification	Examples of structures
C	Where failure would result in significant damage or risk to life
B	Where failure would result in moderate damage and loss of services
A	Where failure would result in minimal damage and loss of access

NOTES:

1 Classification B includes structures not covered by Classifications A or C

2 For more information on structure classification, see Appendix A.

Structures where failure would result in minimal damage and loss of access where the wall height (H) is greater than 1.5 m are deemed to be classification B structures.

1.3 REFERENCED DOCUMENTS

The following documents are referred to in this Standard:

AS	
1170	Minimum design loads on structures (known as SAA Loading Code)
1170.1	Part 1: Dead and live loads and load combinations
1170.2	Part 2: Wind loads
1170.4	Part 4: Earthquake loads
1289	Methods of testing soils for engineering purposes
1289.6.3	Method 3: Determination of the penetration resistance of a soil
1720	Timber Structures (all parts)
1726	Geotechnical site investigations
2159	Piling—Design and installation
2439	Perforated plastics drainage and effluent pipe and fittings (all parts)
2870	Residential slabs and footings
2870.1	Part 1: Construction
3500	National Plumbing and Drainage Code (set)
3600	Concrete structures
3700	Masonry structures
4100	Steel structures
SAI	
HB 77	Australian Bridge Design Code
BS	
8006	Code of practice for strengthened/reinforced soils and other fills

1.4 DEFINITIONS

For the purpose of this Standard, the definitions below apply.

1.4.1 General

1.4.1.1 *Action*

A cause of stress, dimensional change, or displacement in a structure or component of a structure.

1.4.1.2 *Action effect*

The internal force, moment, deformation, crack, or the like effect caused by one or more actions.

1.4.1.3 *Batter*

A slope of a cut or fill.

1.4.1.4 *Characteristic value*

Representative value of a soil, rock or a material property.

NOTE: For soil properties, the characteristic value is a cautious estimate, that is, close to but not greater than, of the mean value, (see Clause 5.2.3). The strengths of materials, such as concrete, steel, plastic and masonry, are contained in the appropriate Standards.

1.4.1.5 *Dead load*

The load determined in accordance with this Standard imposed by the components of the self-weight of the structure and retained soil or rock.

1.4.1.6 *Design life*

The intended period of time over which the structure is required to fulfil its function and remain in a serviceable state.

1.4.1.7 *Design load (action)*

The appropriate combination of loads (actions) and load factors as specified in this Standard.

1.4.1.8 *Ground anchor*

A tensile reinforcement, wire or bar, and its associated components that transmit force into soil or rock through bond over part of its length.

NOTE: Appendix B gives guidance on design of ground anchors.

1.4.1.9 *Limit state*

Any limiting condition for which structures are designed.

NOTE: The limit states design criteria considered in this Standard are strength limit state, stability limit state and serviceability limit state.

1.4.1.10 *Live load*

The load as defined in this Standard assumed to arise from the intended use of the structure including distributed, concentrated, impact and inertia loads, but excluding wind, snow, and earthquake loads.

1.4.1.11 *Load factor*

A factor specified in this Standard for structural design to be used with the loads (or actions) in deriving design loads (or design actions).

1.4.1.12 *Resistance effect*

The characteristic strength multiplied by the appropriate reduction factors for ultimate or serviceability limit state analysis.

1.4.1.13 *Rock bolt*

A tensile reinforcement, typically a bar of 15 to 50 mm in diameter, of steel or high strength polymer, which is inserted into a drill hole in rock and fixed in position by grout or mechanical anchorage.

NOTE: An initial tension is applied equal to or greater than 5%, and usually about 50% to 60%, of the ultimate tensile strength of the bar.

1.4.1.14 *Rock dowel*

As for rock bolt except that the applied tension is less than 5% of the ultimate tensile strength of the material used in the bar.

1.4.1.15 *Serviceability limit state*

A limit state for acceptable in-service conditions.

1.4.1.16 *Stressed ground anchor*

A tensile reinforcement, typically high tensile steel strand or wire or bar, and its associated components that transmit force into soil or rock through bond over part of its length. An initial tension is applied and the minimum capacity is usually checked by proof loading.

1.4.1.17 *Soil dowel*

A relatively stiff structural element used to take load in soils principally by shear action.

1.4.1.18 *Soil nail*

A tensile reinforcement, typically a steel bar or high strength polymer bar, placed in the ground and bonded for all or a majority of its full embedded length.

NOTE: Soil nails normally have no applied or nominal initial tension and mobilize axial force by relative movement with the soil. For more information on soil nailing for earth-retaining structures, see Appendix C.

1.4.1.19 *Strength limit state*

A limit state of collapse or loss of structural integrity.

1.4.1.20 *Stability limit state*

A limit state corresponding to the loss of static equilibrium of a structure or part of structure considered as a rigid body.

1.4.1.21 *Sprayed concrete*

A mixture of cement, aggregate and water (it may also include fibres and/or cement mixtures), conveyed through a hose, and projected at high velocity from a nozzle into place, to produce a dense homogeneous mass and may be applied by either a dry-mix, or a wet mix process.

A1 **1.4.1.22** *Facing*

Unit for erosion protection or aesthetic purposes constructed in front of an otherwise stable slope.

NOTE: The facing may or may not include drainage consideration.

1.4.1.23 *Stable slope angle (i)*

The angle that the soil, rock or other material would naturally assume in the long term under the expected variety of external (e.g., environmental), and internal (e.g., seepage) conditions.

1.4.1.24 *Wall height*

The overall height between the top and the toe of the wall where the line between the points is sloped equal to or greater than 70 degrees from the horizontal.

1.4.1.25 *Wall height for terraces*

The overall wall height for terraces is the height between the top of the top wall and the bottom of the bottom wall, except where the terraces are spaced sufficiently far apart horizontally so that the equivalent average slope of the terraced wall thus formed is below the stable slope angle, i (see Figure 1.2). In such cases the walls are considered separately.

1.4.2 Geosynthetic materials

1.4.2.1 *Geocomposite*

A composite geosynthetic material made up from geotextiles, geogrid and related products, as appropriate, to form a composite geosynthetic material.

1.4.2.2 *Geogrid*

A planar structure consisting of elements in an open grid pattern, used for ground reinforcement.

1.4.2.3 *Geosynthetic reinforcement*

A generic term that encompasses flexible synthetic materials used in geotechnical engineering, such as geogrids and geotextiles.

1.4.2.4 *Geosynthetic subsurface drainage system*

A pervious subsurface drainage system prefabricated from synthetic materials.

1.4.2.5 *Geotextile*

A permeable, polymeric material which may be woven, non-woven or knitted.

1.4.3 Soil and rock materials

1.4.3.1 *Cohesive fill*

Naturally occurring or processed materials with greater than 50% passing the 75 µm AS sieve and a plasticity index of less than 30% and liquid limit of less than 45%.

1.4.3.2 *Controlled fill—Class I*

Soil rock or other inert material that has been placed at a site in a controlled fashion and under appropriate supervision to ensure the resultant material is consistent in character, placed and compacted to an average density equivalent to 98% (and no test result below 95%) of the maximum dry density (standard compactive effort) for the material when tested in accordance with AS 1289.5.1.1.

NOTE: For cohesionless soils, material compacted to at least 75% Density Index is satisfactory.

1.4.3.3 *Controlled fill—Class II*

Soil rock or other inert material that has been placed at a site in specified layers in controlled fashion to ensure that the resultant material is consistent in character placed and compacted to an average density equivalent to 95% (and no test result below 92%) of the maximum dry density (standard compactive effort) for the material when tested in accordance with AS 1289.5.1.1.

NOTE: For cohesionless soils, material compacted to at least 65% Density Index is satisfactory. Generally the layer thickness is specified as a maximum of 300 mm.

1.4.3.4 *In situ material*

Natural soil, weathered rock and rock materials.

1.4.3.5 *Other fill*

Materials that are chemically unstable or physically decomposable, which may include blast furnace slag, fly ash and topsoil.

1.4.3.6 *Reinforced soil*

The material in a reinforced soil structure in which reinforcing elements are installed to act through interface friction, or other means, to improve the stability of the soil or rock materials.

1.4.3.7 *Reinforced soil structure (RSS)*

A structure formed by reinforced soil, facing materials, drainage and associated foundation systems.

1.4.3.8 *Retained backfill*

The fill material located behind a retaining structure.

1.4.3.9 *Select fill*

A soil, rock or other inert material that is selected from various materials, that may be available on a site, and that has particular properties specified by a designer.

NOTE: Select fill will also generally be a soil or rock material that does not include highly plastic clays, topsoil, large boulders or waste products.

1.4.3.10 *Uncontrolled fill*

Soil, rock or other inert material that has been placed at a site and that does not fall under the definitions for materials in Clauses 1.4.3.2 and 1.4.3.3.

1.4.4 Drainage

1.4.4.1 *Piped drains*

A system of impervious pipes designed to lead drainage away from the retaining structure.

1.4.4.2 *Subsoil drain*

A system of pervious subsurface pipes, which is used in soil materials to collect ground water.

1.4.4.3 *Subsurface drainage system*

A system of drainage measures designed to remove water and seepage from the ground, which may include all, or a combination of, pervious granular materials, geocomposite, subsoil drains and piped drains.

1.4.4.4 *Vertical structure drain*

A vertical drainage system located behind and adjacent to a retaining wall, reinforced soil structure or its facing, comprising a pervious medium for the dewatering of the retained soil mass.

1.5 NOTATION

The following symbols are used in this Standard:

A_c = corrosion loss at t = 1 year

A_o = original cross-section area

a = acceleration coefficient derived in accordance with AS 1170.4

a_h = horizontal coefficient of acceleration (pseudo-static)

A1 **'Not applicable'**

a_m = maximum wall acceleration coefficient at centroid

a_{mh} = average amplified horizontal acceleration coefficient

a_{mv} = average amplified vertical acceleration coefficient

a_v = vertical coefficient of acceleration (pseudo-static)

B = base width

c = characteristic cohesion of a soil, which is a parameter in the Mohr-Coulomb failure law, also referred to as the cohesion intercept

c' = characteristic effective cohesion of a soil

c_u = characteristic undrained cohesion of a soil

c^* = design cohesion value

E_{ac} = external dynamic earth pressure load

A1 E_i = inertia force

'Not applicable'

F_{eq} = earthquake loads calculated in accordance with AS 1170.4

A1 **'Not applicable'**

G = dead load

G^R = part of the dead load tending to resist stability

G_s = specific gravity of the soil

G_1 = dead load of structure

G_2 = dead load of fill on structure

G_3 = earth pressure due to fill behind structure

G_4 = dead load of fill in front of structure

A2 g = gravitational constant (9.81 m/s^2)

H = height of wall

= overall height of wall from top to toe of wall at 70 degrees or greater (see 'Wall height' and 'Wall height for terraces')

H_w = height of water in front of wall

A1 i = slope angle; or

= stable slope angle

'Not applicable'

I_p = plasticity index

k_A = parameter relating to angularity

k_B = parameter relating to grading

k_C = parameter relating to density

K_a = static active earth pressure coefficient

K_{ac} = total active earth pressure coefficient (including seismic effects)

k_h = horizontal seismic coefficient

k_v = vertical seismic coefficient

n = constant representing rate of change of corrosion rate

N' = design number of blows from the standard penetration test

A1 **'Not applicable'**

P_p = proofload during anchor load

P_w = calculated working load of the anchors (after all losses)

p_{wd} = resultant dynamic water pressure

Q = live loads (including impact, if any)

A1 **'Not applicable'**

Q_1 = traffic or other live load on structure

Q_2 = traffic or other live load behind structure

Q_3 = traffic or other live load in front of structure

A1 **'Not applicable'**

R^* = design resistance effect

S = site factor for structure in accordance with AS 1170.4

S^* = design action effect

A1 **'Not applicable'**

T_c = connection strength determined by test

A1 T^*_{cd} = design connection strength

A1 T^*_{cj} = design connection force at the j^{th} layer

T^*_d = design tensile strength of reinforcement

A1 **'Not applicable'**

T^*_{di} = design soil/reinforcement interaction strength

T_{ij} = soil/reinforcement interaction capacity of the reinforcement

A1 T^*_j = design maximum tensile force resisted by the j^{th} layer of reinforcement

T^*_{ij} = design soil interaction force at the j^{th} layer

T_u = short-term ultimate tensile strength

T_o = original tensile strength

t = time in years

A1 **'Not applicable'**

W = weight of the structure and its superstructure

W_s = wind load for strength limit state

W_u = wind load for serviceability limit state

ΔK_{ac} = the incremental increase in active earth pressure coefficient

ΔT = loss of tensile strength

A1 ΔA = loss of section area

θ = arc tan $[k_h/(1 \pm k_v)]$

A1 δ = external friction angle between soil and structure

ϕ = characteristic internal friction angle of a soil, which is a parameter in the Mohr-Coumlomb failure criteria, also referred to as the internal friction angle, or simply friction angle

ϕ' = effective internal friction angle of a soil

ϕ_{cv} = constant volume internal friction angle of a soil

ϕ_r = residual internal friction angle of a soil

ϕ_u = undrained internal friction angle of a soil

ϕ^* = design internal friction angle

Φ_b = partial material reduction factor for anchor bond

Φ_k = partial importance category factor for anchor pull out

Φ_t = partial material reduction factor for anchor tendon strength

Φ_n = structure classification factor

Φ_r = reduction factor

Φ_{rc} = partial material creep reduction factor

Φ_{ri} = partial material reduction factor due to construction or installation

Φ_{rs} = partial material strength reduction factor

Φ_{rt} = partial material thickness reduction factor

Φ_{rst} = partial material temperature reduction factor

Φ_{u} = uncertainty factor for soil/reinforcement interaction

Φ_{ua} = partial material uncertainty factor for soil/reinforcement interaction

Φ_{uc} = partial design uncertainty factor for cohesion of the soil and backfill materials

Φ_{ud} = partial material uncertainty factor for overall degradation

Φ_{ue} = partial material extrapolation uncertainty factor

Φ_{up} = partial material product uncertainty factor

Φ_{ucon} = uncertainty factor for connection strength

$\Phi_{u\phi}$ = partial design uncertainty factor for friction of the soils and backfill materials

ΦR = design capacity of the structural component

γ_1 = characteristic value for weight per unit volume of structure

γ_2 = characteristic value for weight per unit volume of fill behind structure

γ_3 = characteristic value for weight per unit volume of fill above structure

γ_4 = characteristic value for weight per unit volume of fill in front of structure

γ_b = bulk unit weight of the fill behind the wall

γ_{g1} = load factor for γ_1

γ_{g2} = load factor for γ_2

γ_{g3} = load factor for γ_3

γ_{g4} = load factor for γ_4

γ_{gw} = load factor for γ_w

γ_{q1} = load factor for Q_1

γ_{q2} = load factor for Q_2

γ_{q3} = load factor for Q_3

γ_w = density of water

γ_m = moist bulk weight in kilonewtons per cubic metre

γ_s = saturated bulk weight in kilonewtons per cubic metre

Ψ_c = live load combination factor used in assessing the design load for strength limit state

A1 * = superscript indicates design value (e.g., factored capacity (resistance))

SECTION 2 INVESTIGATION AND TESTING

2.1 INVESTIGATIONS

2.1.1 Site and geotechnical data

The design of earth-retaining structures requires adequate information on the physical conditions in the vicinity of the structure including the topography, site layout, nature of soil, foundation bearing capacity and the ground water conditions (including tidal conditions when appropriate), together with information on any adjacent structures and services that may be affected. The investigation shall include an assessment of the regional geotechnical conditions (e.g. the potential for slope instability) where they may impact on the proposed development.

All site investigations shall be carried out on the basis of a suitable scope of work, taking into account the classification of the structure (see Table 1.1 and Table 2.1).

2.1.2 Extent of investigation

The extent and complexity of the site investigations shall be related to the size, complexity and form of the structure to be designed. Attention shall also be paid to the properties of the soils in the immediate vicinity of the structure as well as underneath the structure.

The depth of the investigation shall be related to the structure type and size as well as the site geology. Where appropriate, careful attention shall be paid to both the overall geology and local geology, including geological defects and discontinuities, bedding planes, joints and faults. Account shall be taken of conditions such as reactive soils and uncontrolled fill. Special consideration shall be given to areas of landslip, any tendency of the site to shift, creep or settle, and areas of possible mine subsidence.

Where a site has been filled, it shall be ascertained whether or not the fill has been placed as a Class I or Class II 'controlled fill' (see Clauses 1.4.3.2 or 1.4.3.3), uncontrolled fill (see Clause 1.4.3.10) or 'other fill' (see Clause 1.4.3.5).

Due to the possibility that even with a careful site investigation unexpected conditions may be revealed during construction, appropriate inspections shall be carried out during the course of the construction process to ensure that the conditions revealed during the construction are in accordance with the assumption made during the design. If the conditions are not in accordance with these assumptions, then necessary alterations shall be made to the design to take account of the observed new conditions.

Of special importance to all types of retaining structures is the determination of ground water, site seepage and surface run off. The investigation of all types of retaining structures shall include—

(a) a study of the ground water regime in the vicinity of the structure, including sources, directions of lateral flows, and seasonal or tidal variations;

(b) chemical composition of the ground water; and

(c) the effect of the proposed retaining structure construction on the ground water regime.

The effect of water discharge from any drains, pipes, and the like on the site conditions shall also be considered. An assessment should be made of the effects of ground water and site seepage over the design life of the structure.

2.1.3 General investigation requirements

The extent and amount of detailed information obtained in a particular investigation shall include the considerations given in Table 2.1.

TABLE 2.1

INVESTIGATION CONSIDERATIONS

A1

Investigation considerations	Structure classification		
	C	B	A
Substrata type	●	●	●
Effect of drainage discharge onto surrounding site	●	●	●
Nature of retained material	●	●	●
Site topography	●	●	●
Foundation and embankment strength parameters	●	●	❍
Existing ground water levels and seepage	●	●	❍
Effect of excavations or filling	●	●	❍
Location of existing or proposed adjacent structures	●	●	❍
Effect of modified water table on surrounding site	●	❍	❍
Global stability	●	❍	❍
Impact of structure 'zone of influence'	●	❍	❍
Ground movement	●	❍	❍

● denotes detailed site-specific information required.

❍ denotes only general consideration of the item required.

2.1.4 Investigations for reinforced soil structures

2.1.4.1 *General*

In addition to the general investigation requirements (see Clause 2.1.3), as the soil material used in a reinforced soil structure acts as part of the structure itself, consideration shall be given to the properties of the material that forms the reinforced soil mass.

2.1.4.2 *Fill materials*

Fill materials for reinforced soil structures shall be selected fill materials, placed and compacted as a controlled fill (see Clauses 1.4.3.2 and 1.4.3.3).

NOTE: Guidance on the selection of fill material for reinforced soil structures is given in Appendix D.

2.1.4.3 *Fill properties*

Where appropriate, testing shall be undertaken to confirm the following properties and characteristics of the proposed fill:

(a) Physical characteristics (size, grading and the like).

(b) Strength parameters.

(c) Chemical properties.

(d) Electrochemical properties (appropriate when metallic soil reinforcement is proposed).

(e) Compaction characteristics.

The testing of representative samples shall be carried out in accordance with the relevant test method of AS 1289 and with any special test procedures defined in this Standard.

2.1.4.4 *Investigations for in situ reinforced soil structures*

Additional investigations are required for in situ reinforced soil structures. Where appropriate, investigations shall include the following:

(a) Identification of the site geology, joints, relic joints, defects, bedding planes, seepage and water paths, in situ stresses, watertable and the like.

(b) Determination of structural discontinuities and orientation of defects.

(c) Identification of all active and inactive services (e.g. water, sewer, power and communications) and structures within the 'zone of influence' of the structure.

(d) Assessment of the effect of any movement, or deformation, of the reinforced soil structure on nearby developments or buildings.

2.2 TESTING

2.2.1 Fill materials

Sampling of proposed fill materials for an earth-retaining structure shall be undertaken to ensure that the samples are truly representative of the materials to be used in the structure and that their condition duplicates, as far as is practicable, the design conditions in the actual structure. In particular, materials that are susceptible to crushing under compaction (e.g. crushed sandstone) shall be pretreated prior to testing to simulate their in-service condition.

NOTE: Some types of walls need careful attention paid to the type and compaction of the fill in the wall structure (e.g. crib walls).

2.2.2 In situ materials

Where the properties of in situ materials and existing fill forming the foundations, as well as the in situ reinforced areas, are critical to the performance or stability of the structure, then sampling and testing shall be carried out by the relevant undisturbed sampling and testing procedures set out in AS 1289.

Where the in situ materials or existing fill are suspected to have significant shrink/swell characteristics (i.e. the site, as defined in AS 2870.1, is a Class M or higher), then the impact of shrink/swell characteristics of the in situ and fill materials shall be considered.

2.2.3 Laboratory testing

Unless otherwise specified, the laboratory tests shall be undertaken in accordance with the relevant test method of AS 1289.

SECTION 3 DESIGN REQUIREMENTS

3.1 GENERAL

3.1.1 Introduction

This Section sets out the requirements for the design of earth-retaining structures to ensure that strength, stability and serviceability are achieved over the intended life of such structures. Drainage aspects and the influence of construction on the adjacent ground shall be considered. In the initial design process, the selection of an appropriate retaining structure shall consider anticipated site conditions, the constructability of the structure and the need for temporary support measures.

3.1.2 Design principles

For the purpose of reinforced soils and retaining structures design, a limit state is deemed to be reached when one of the following occurs:

(a) *Ultimate limit state (strength and stability)*—collapse or major damage as a result of rupture or instability of the structure, ground, reinforced soils, reinforcement or other components.

(b) *Serviceability limit state*—movement or damage in excess of stated design limits.

For both limit states, the design resistance effect (R*) shall be greater than or equal to the design action effect (S*) for the most critical loads and combinations, i.e.,

$$R^* \geq S^*$$

where R* and S* are appropriately factored.

3.1.3 Partial factor method

The requirements of Clauses 3.1, 3.2 and 3.3 are deemed to be met if the requirements of Sections 4 and 5 are complied with.

3.1.4 Design using other methods

The requirements of Clause 3.1, 3.2 and 3.3 may be met by the adoption of engineering principles and appropriate loads, resistances and factors.

NOTE: For information on design models and methods see Appendix E.

3.2 DESIGN CONSIDERATIONS FOR ULTIMATE LIMIT STATE

Structure classification (see Table 1.1), design life, durability and maintenance of the structure shall be considered as part of the design process.

The design for ultimate limit state shall take into consideration, but not necessarily be limited to, the following six ultimate limit state modes:

(a) *Limit Mode U1* Sliding within or at the base of the retaining structure (see Figure 3.1(A).

(b) *Limit Mode U2* Rotation of the structure (see Figure 3.1(B)).

(c) *Limit Mode U3* Rupture of components and connections (see Figure 3.1(C)).

(d) *Limit Mode U4* Pull-out of reinforcing elements or anchors (see Figure 3.1(D)).

(e) *Limit Mode U5* Global failure mechanisms (see Figure 3(E)).

(f) *Limit Mode U6* Bearing failure (see Figure 3.1(F)).

3.3 DESIGN CONSIDERATIONS FOR SERVICEABILITY LIMIT STATE

The design for serviceability limit state shall be adequate in that movement of the foundation, retaining wall, or reinforced soil structure does not cause local damage to the structure, which could shorten the structure's intended life or incur excessive maintenance costs. Possible deformation modes involving rotation, translation or bulging and settlement are shown, but not necessarily limited to, the following three serviceability limit state modes:

(a) *Limit Mode S1* Rotation of the structure (see Figure 3.2(A)).

(b) *Limit Mode S2* Translation or bulging of the retaining structure (see Figure 3.2(B)).

(c) *Limit Mode S3* Settlement of the structure (see Figure 3.2(C)).

The concept of serviceability depends very much on the end use requirements of the structure. Normally, serviceability requirements for retaining structures are prescribed in terms of acceptable deformations. These limits will vary with the type of structure. In addition, the limits required will come into operation at different stages according to the type of structure.

NOTE: 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 caused by, but not limited to, one or a combination of the following:

(i) External ground movements (e.g. landslip).

(ii) Foundation settlement.

(iii) Creep extension of synthetic soil reinforcement.

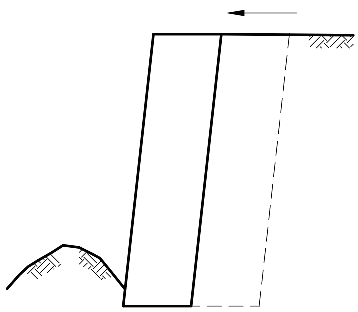
(iv) Creep movement of soil/reinforcement interface.

(v) Consolidation of poorly compacted backfill.

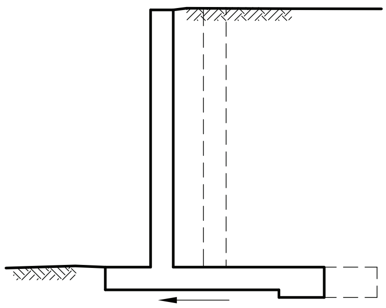
(vi) Displacement and deformation of the facing caused by the likes of construction equipment loading.

(vii) Deterioration of the reinforcement due to metal corrosion or polymer degradation.

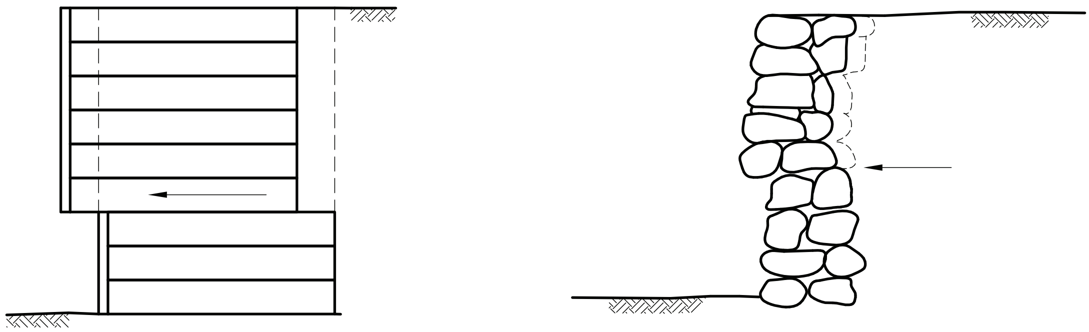
Where soils having significant shrink/swell characteristics are encountered at a site, then consideration of the effects of the shrink/swell movements shall be taken into account in the design of the structure.



(a) Sliding at the base



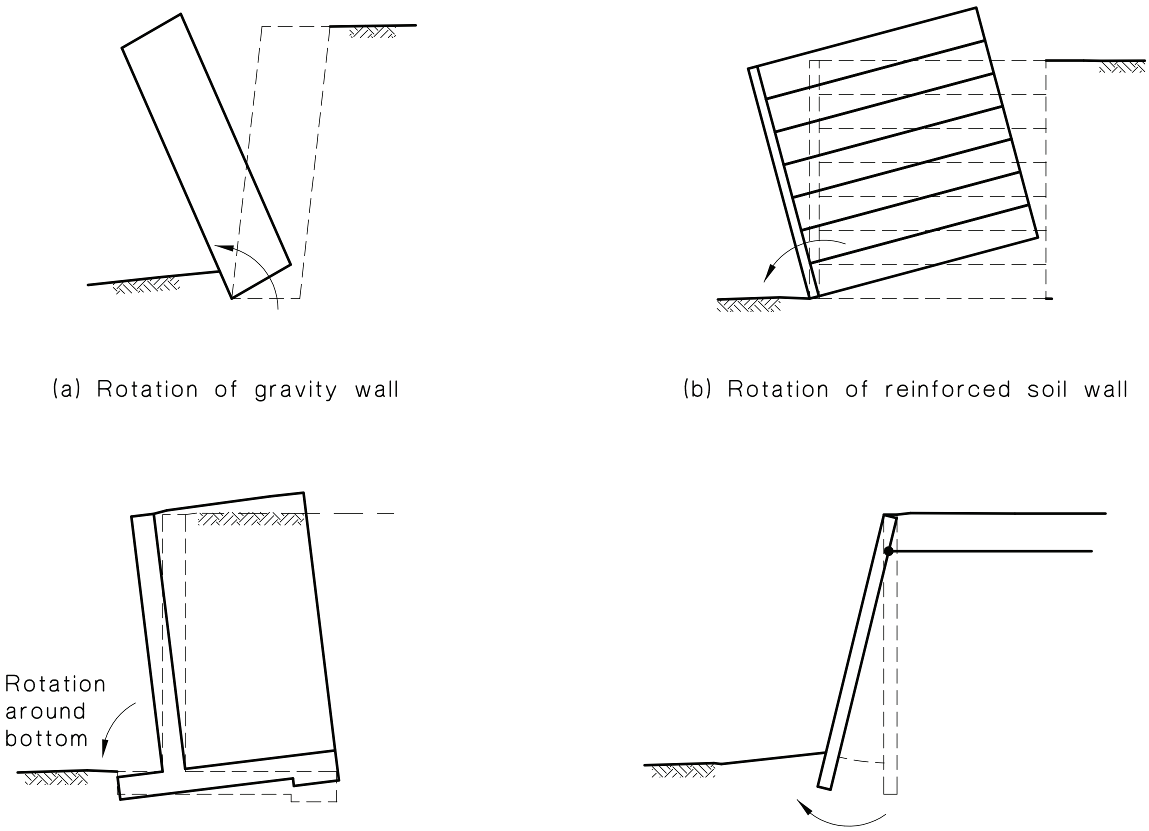
(b) Sliding of cantilevered wall



(c) Sliding within a wall

NOTE: These diagrams are illustrative of the failure mode.

FIGURE 3.1(A) LIMIT MODE U1: SLIDING—ULTIMATE LIMIT STATE



(a) Rotation of gravity wall

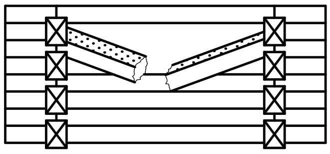
(b) Rotation of reinforced soil wall

(c) Rotation of cantilevered wall about base

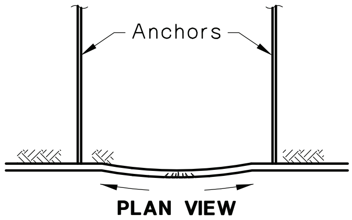
(d) Rotation of toe of wall about top

NOTE: These diagrams are illustrative of the failure mode.

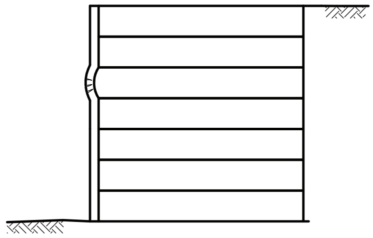
FIGURE 3.1(B) LIMIT MODE U2: ROTATION—ULTIMATE LIMIT STATE



(i) Rupture of crib wall stretcher

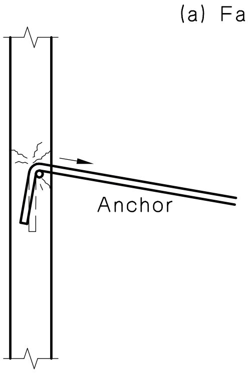


(ii) Rupture of facing

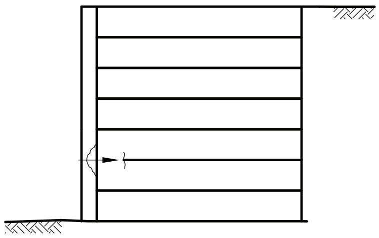


(iii) Rupture of reinforced soil wall facing

(a) Facing element rupture

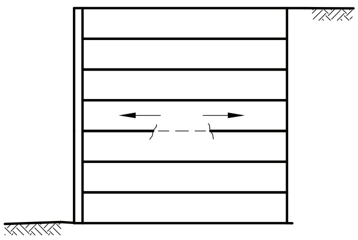


(i) Rupture of anchor connection

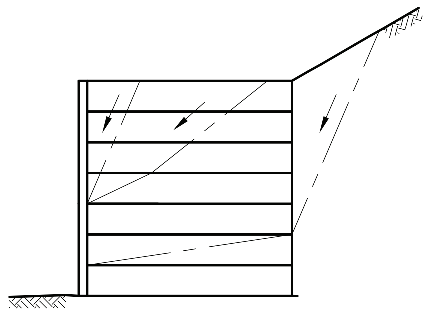


(ii) Rupture of reinforcement connection

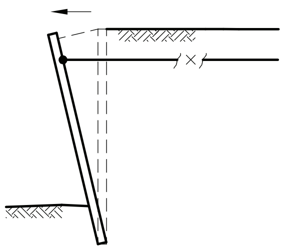
(b) Connection rupture



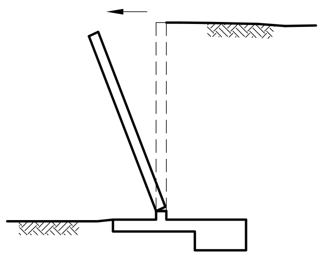
(i) Rupture of reinforcement layer



(ii) Rupture of elements due to wedge failure



(iii) Anchor failure

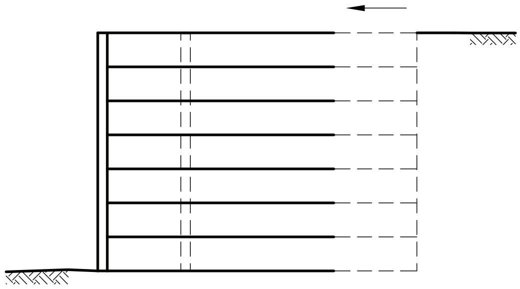


(iv) Cantilever failure

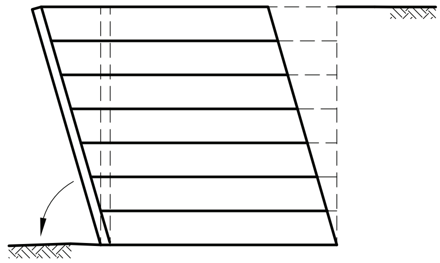
(c) Component rupture

NOTE: These diagrams are illustrative of the failure mode.

FIGURE 3.1(C) LIMIT MODE U3: RUPTURE OF COMPONENTS AND CONNECTIONS—ULTIMATE LIMIT STATE

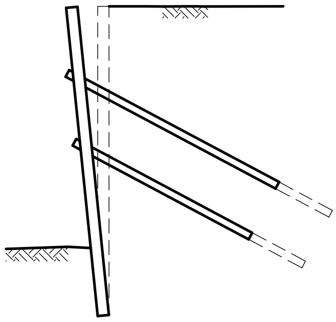


(i) Elements I

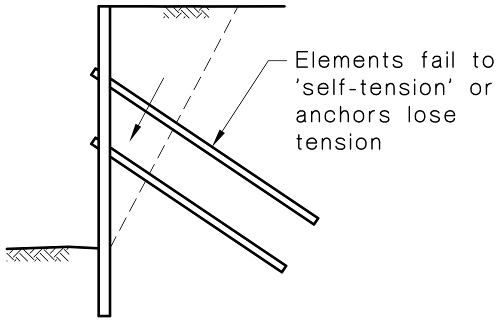


(ii) Elements II

(a) Pull-out of reinforcing elements



(i) Anchors I

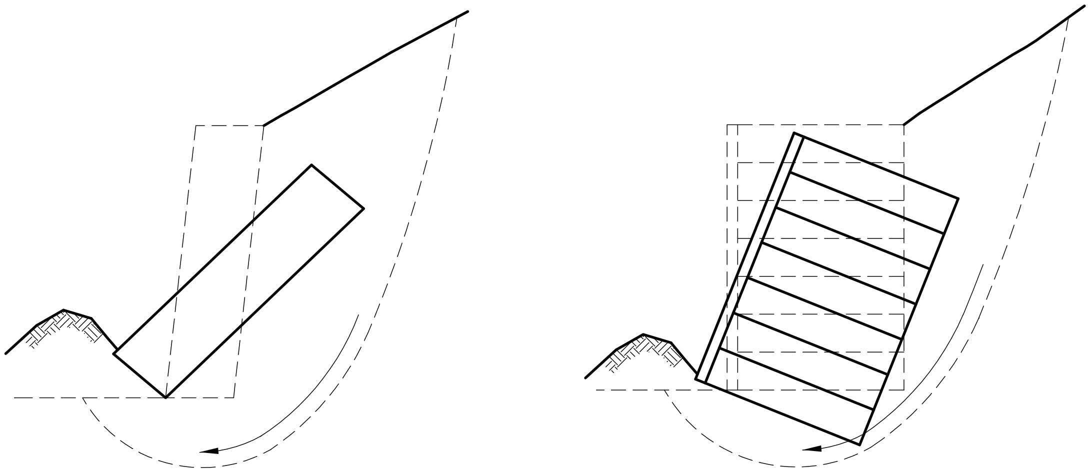


(ii) Anchors II

(b) Pull-out of anchors

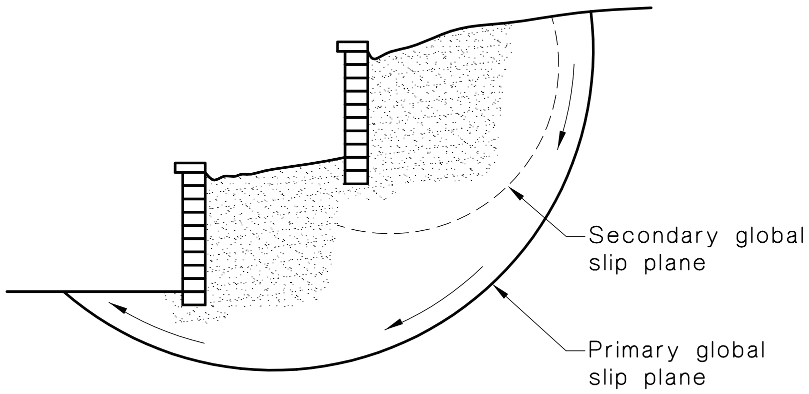
NOTE: These diagrams are illustrative of the failure mode.

FIGURE 3.1(D) LIMIT MODE U4: PULL-OUT OF ANCHORS OR REINFORCING ELEMENTS—ULTIMATE LIMIT STATE



(a) Global failure of gravity wall (b) Global failure of reinforced soil wall

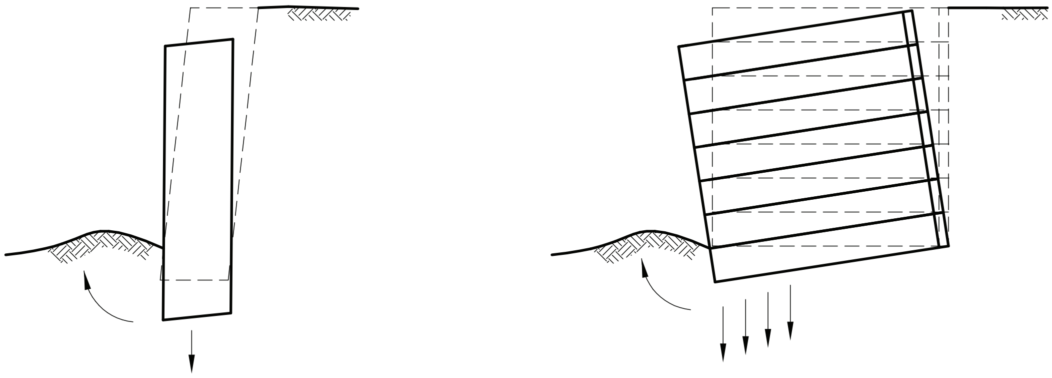
A1



(c) Global failure of gravity block wall

NOTE: These diagrams are illustrative of the failure mode.

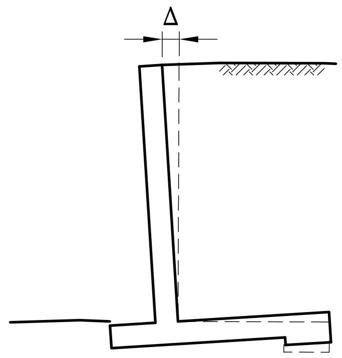
FIGURE 3.1(E) LIMIT MODE U5: GLOBAL FAILURE (WEDGE OR SLIP SURFACE) ULTIMATE LIMIT STATE



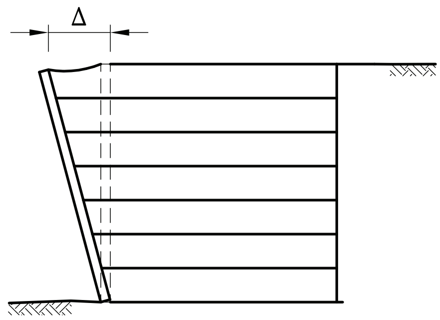
(a) Bearing failure of gravity wall (b) Bearing failure of reinforced soil wall

NOTE: These diagrams are illustrative of the failure mode.

FIGURE 3.1(F) LIMIT MODE U5: BEARING FAILURE ULTIMATE LIMIT STATE



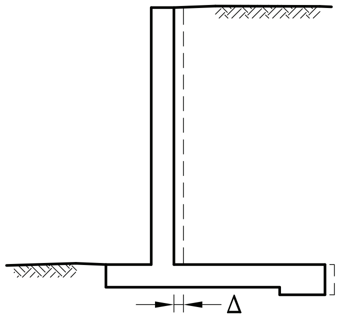
(a) Rotation of cantilevered wall



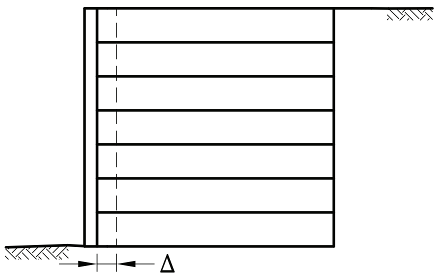
(b) Rotation of reinforced soil wall

NOTE: These diagrams are illustrative of the failure mode.

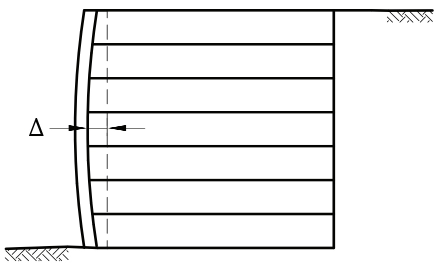
FIGURE 3.2(A) LIMIT MODE S1: ROTATION—SERVICEABILITY LIMIT STATE



(a) Translation of cantilevered wall



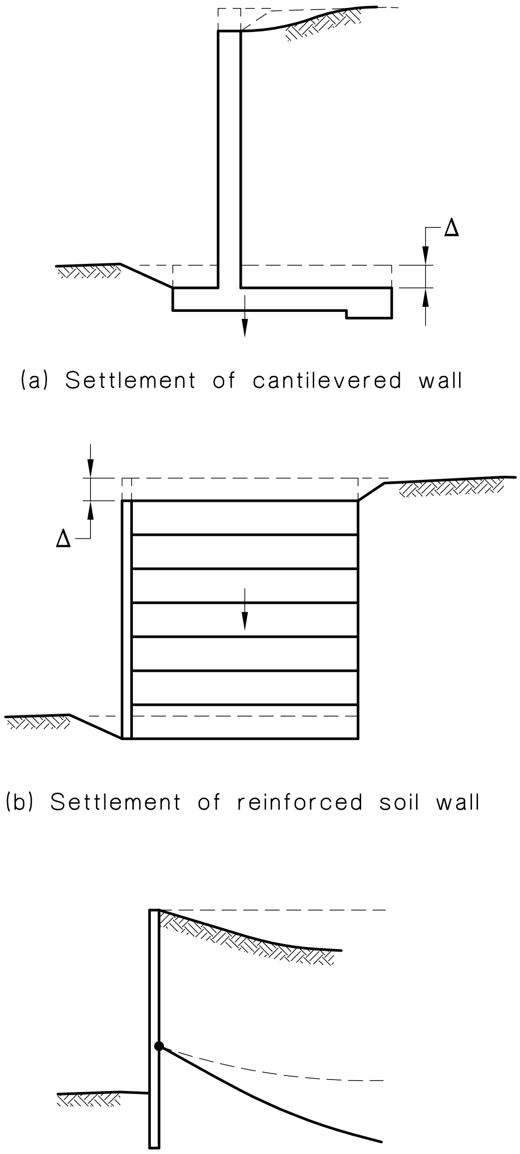
(b) Translation of reinforced soil wall



(c) Bulging of reinforced soil wall

NOTE: These diagrams are illustrative of the failure mode.

FIGURE 3.2(B) LIMIT MODE S2: TRANSLATION OR BULGING—SERVICEABILITY LIMIT STATE



(a) Settlement of cantilevered wall

(b) Settlement of reinforced soil wall

(c) Settlement of anchored structure

NOTE: These diagrams are illustrative of the failure mode.

FIGURE 3.2(C) LIMIT MODE S3: SETTLEMENT—SERVICEABILITY LIMIT STATE

3.4 DESIGN LIFE

3.4.1 General

Design life shall be considered in the design process.

Each component of an earth-retaining structure shall be durable enough to provide satisfactory performance over the design life of the structure.

In reinforced soil structures, the life of the reinforcing elements, associated facings and reinforcement connections shall have compatible durability requirements with the remainder of the structure, unless the reinforcement is required to function only for a shorter time while the surrounding ground gains strength.

Typical design life for various applications is given in Table 3.1.

3.4.2 Design life considerations

For increasing design life, consideration should be given to possible changes over time in the following:

(a) Loads and load factors.

(b) Material reduction factors appropriate to the structure classification.

(c) Environmental considerations.

(d) Durability and corrosion.

TABLE 3.1

DESIGN LIFE

Design life (years)		Typical application
Short	5	Temporary site works
Medium	10 30	Mine structures Industrial structures
Long	60 90 120	River and marine structures, residential dwellings Minor public works Major public works

3.5 DURABILITY AND PERFORMANCE OVER TIME

Durability is the ability of the structure to resist loss of properties or wear such as erosion, corrosion, creep, chemical, biological and installation damage so that the required strength is maintained over the design life of the structure. The design shall take into account changes in relevant properties over time.

NOTES:

1 A check list for investigation of reinforcement products for reinforced soil structures is given in Table F1, Appendix F.

2 Factors affecting the durability and performance of earth-retaining structure materials are presented in Table F2, Appendix F.

3.6 DRAINAGE

3.6.1 General

Design of retaining structures shall consider the drainage aspects of the site, including the short- and long-term subsurface hydrological conditions of the site.

Drainage and the possible development of pore water pressure within a soil are amongst important considerations in the design of retaining structures because the presence of water behind an earth-retaining structure has a significant effect on the pressures applied to the structure. Even when there is no water in direct contact with the structure, increased pressures can occur on a retaining structure due to an elevated phreatic surface developed from water seepage into the failure wedge behind the structure.

NOTE: Additional information on drainage measures is given in Appendix G.

3.6.2 Design considerations

The design of an earth-retaining structure shall carefully consider the effects of water, water seepage and potential hydrostatic pressures in the design, as well as the long-term performance of any drainage system installed. In particular, the designer shall consider and allow for at least the following items in the design:

(a) Existing site ground water conditions and how such conditions are likely to be affected by the construction of the structure.

(b) Necessity for and specific details of any drainage systems to be provided behind the structure. Such details would normally include—

  (i) the subsurface drainage system, including pipe drains and subsoil drains;

  (ii) weep holes;

  (iii) filters and geotextiles;

  (iv) fill materials;

  (v) other drainage details; and

  (vi) any associated pipework, clean-out systems and method of drainage efflux.

(c) The 'long-term' performance of the drainage system and the necessity for maintenance.

The design shall be based on the predicted 'long-term' condition of the drainage system rather than the expected performance of the system immediately after installation, including, if necessary, an assumption of failure or partial failure of the drainage system.

Where clean gravel screenings, pervious materials, geocomposites and the like are specified by the designer for installation behind the structure, then adequate provisions shall be made to ensure that surface and stormwater run-off does not overload, foul or clog the subsurface drainage system.

Where possible, surface water shall be directed away from the top of earth-retaining structures and not be allowed to pond behind the top of the wall or enter the subsurface drainage system.

3.7 INFLUENCE OF CONSTRUCTION ON ADJACENT GROUND AND STRUCTURES

Consideration shall be given to the possible adverse effect on the adjacent land or structures of the construction of an earth-retaining structure. This is particularly important on sloping ground where any excavation associated with the construction of the retaining wall may induce or reactivate slope instability.

3.8 SUBMERGED STRUCTURES

Structures, temporarily or permanently submerged, shall be designed with due consideration of the effect of static and transient pore water pressures both within and external to the structure. Other effects such as wave action, water flow and velocity (scour potential), piping and liquefaction require particular consideration. Typical situations include the following:

(a) Sea walls.

(b) Reclamation.

(c) Earth bunds.

(d) Abutments for river and creek crossings.

(e) Flood-prone areas.

For temporarily or permanently submerged or tidally affected structures, consideration shall be given to the following where appropriate:

(i) Draw-down effects, differential and residual watertables, and tidal lag.

(ii) Piping or material loss from within the structure, the structure backfill or the foundations.

(iii) Scour or erosion in front of the structure.

(iv) Wave action, including overtopping.

(v) Loss of backfill through the wall due to wave and tidal action.

(vi) Durability of materials.

NOTE: The above factors require particular consideration on a case-by-case basis. The relevant methodologies are not covered in this Standard and appropriate specialist advice should be sought.

3.9 CONNECTIONS

The connection of the different elements of an earth-retaining structure shall be designed to take into account the load-carrying capacity of the connection system. Connections shall be designed to the same durability criteria as the overall structure and shall be compatible with the connected elements.

NOTES:

1 The facing of a reinforced soil structure is supported by the reinforced soil mass usually by connecting directly to the soil reinforcement. The overall stability of the reinforced soil structure is not necessarily reliant on the stability of the facing system; however, the stability of the facing system itself relies directly on the connection of the facing elements to the soil reinforcement elements. The effect of structure movement and load redistribution shall be considered in the design of the connection.

2 Guidelines for connection design of facings are given in Appendix H.

SECTION 4 DESIGN LOADS

4.1 LOADS

Earth-retaining structures shall be designed for the following loadings and other actions, during construction and in service:

(a) Dead loads and live loads, including earth pressure and hydrostatic pressure, as specified in AS 1170.1 except that—

A1 (i) all structures shall be designed for the live loads resulting from the intended use of the structure but not less than the values given in Table 4.1;

(ii) for the strength and stability limit states, a load factor of 1.0 shall be applied to water pressure in lieu of the requirement in AS 1170.1 for a load factor of 1.25. The estimate of the height of the water table shall represent the worst credible location during the life of the structure; and

(iii) for the strength and stability limit states, the requirement in AS 1170.1 for a load factor of 1.5 to be applied to earth pressures is deemed to be met by applying a load factor of 1.25 on the dead loads (including soil weight) and 1.5 on the live loads in combination with the material factors specified in Clause 5.2 of this Standard.

NOTE: Typically, earth pressure will be calculated by determining the total stresses deducting the pore water pressure and multiplying the residual by the appropriate load factor of 1.25.

(iv) Lower partial load factors shall be used where the lower factors lead to a more critical limit state.

(b) Wind loads as specified in AS 1170.2.

(c) Earthquake loads calculated in accordance with AS 1170.4 and Appendix I.

(d) Other loads that may act on the structure (e.g. snow and ice).

A1

TABLE 4.1

LIVE LOAD kPa

Classification	Backfill slope (horizontal/vertical)	
	Steeper than 4:1	4:1 or flatter
B, C	2.5	5
A	1.5	2.5

4.2 LOAD COMBINATIONS

The load combinations for strength, stability and serviceability shall be as specified in AS 1170.1. Where a reaction is generated by the application of a factored load, no further factor shall be applied.

A1

NOTES:

1 For specific applications (such as structures associated with roads and highways), loads, load factors and loading combinations other than those set out in AS 1170 (all parts) may be appropriate. If other loads, load factors or loading combinations are used, it may also be appropriate to vary the associated material factors given in Section 5.

2 Appendix J gives guidance for determination of the appropriate loading combinations in greater detail than that given in AS 1170.1. The designer should determine whether such loads, load factors and loading combinations are still valid at the time the design is carried out.

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3 The load factors specified in AS 1170.1 should be applied to the live loads specified in Clause 4.1.

4 The dead loads should include—

(a) self weight of the structure;

(b) backfill weight on top of the structure;

(c) imposed earth pressures acting behind the structure;

(d) imposed water pressures acting behind the structure; and

(e) static sill beam loadings (both vertical and horizontal) on earth-retaining structures designed as bridge abutments.

5 The live loads should include—

(a) traffic surcharge;

(b) bridge vertical live loads acting on the sill beam;

(c) transient loads acting on bridge sill beam, which might include vehicle braking and thermal effects; and

(d) impact loading.

6 The determination of earthquake design loads should require—

(a) assessment of the seismicity of the site and the potential for liquefaction; and

(b) determination of the response characteristics of the structure.

(c) AS 1170.4 may be used to determine earthquake-induced loadings on structures, taking into account—

(i) structure classification and related importance factor;

(ii) acceleration coefficient (a); and

(iii) site factor (S).

For the purpose of global stability analysis, the horizontal and vertical distribution of earthquake-induced forces on the structure may be calculated in accordance with AS 1170.4 and Appendix I, using a static analysis approach, or using a dynamic analysis.

For the purpose of internal stability analysis, the designer should determine the load distribution and structure resistance behaviour from properly identified dynamic response characteristics of similar structural systems. Characteristics of structures that need to be taken into account to differentiate behaviour include facing systems (flexible or stiff) and type of soil reinforcement (extensible or inextensible).

7 The loads applied by water forces should be taken into account in the design of the structure. Allowance for the effects of water pressure and buoyancy effects should be made where the structure is subject to immersion.

8 The design of the structure should allow for the proper drainage of seepage from behind the structure.

A1

9 This Standard has been drafted with reference to AS 1170.1—1989 for combinations. The intention is to revise this Standard to be fully consistent with AS/NZS 1170.0.

SECTION 5 MATERIAL DESIGN FACTORS

5.1 GENERAL

This Section sets out the determination and combination of partial material strength design factors for earth-retaining structures.

Partial design factors for earth-retaining structures are grouped by subscripts as follows:

(a) $_{\mathrm{r}}$—the reduction group of factors (Φ_{r}) that cover the effect of identified causes of decrease in strength and which are drawn from test data.

(b) $_{\mathrm{u}}$—the uncertainty group of factors (Φ_{u}) that cover unknowns and uncertainties.

(c) $_{\mathrm{n}}$—the structure classification group of factors (Φ_{n}) that cover the consequences of failure of the particular structure.

For each distinct soil stratum, backfill material or structural component, the design resistance effect (R*) shall be determined by calculating the material strengths and resistances based on the characteristic strengths for the material multiplied by Φ_{r}, Φ_{u} and Φ_{n}.

Soil strengths shall be factored in accordance with Clause 5.2, except where unfactored values lead to greater critical limit states.

Material strength of structural components shall be factored in accordance with Clause 5.3.

Tensile strength of soil reinforcement shall be factored in accordance with Clause 5.4.

Partial material strength factors shall account for the following:

(i) Variability in the manufactured strengths of the materials.

(ii) Time-dependent stress/strain behaviour.

(iii) Uncertainty of extrapolation of test data.

(iv) Effect of construction-induced damage.

(v) Uncertainties of environmental degradation.

(vi) Time dependent material and strength loss.

An overall structure classification factor shall be applied in accordance with Clause 5.5.

5.2 MATERIAL STRENGTH FACTORS FOR SOIL SHEAR STRENGTH

5.2.1 Cohesion

The partial design factor for the cohesion of the soils and other backfill materials is Φ_{uc} (see Table 5.1(A) and Table 5.1(B)). It shall be applied to c, so that the design cohesion value c* is given by the following equation:

$$c^{*} = \Phi_{\mathrm{uc}} c \qquad \text{. . . 5.2(1)}$$

5.2.2 Internal friction angle

The partial design factor for the internal friction angle of the soils and other backfill materials is $\Phi_{\mathrm{u}\phi}$ (see Table 5.1(A) and Table 5.1(B)). It shall be applied to (tan ϕ), so that the design internal friction angle ϕ^{*} is given by the following equation:

$$\phi^{*} = \tan^{-1}\left(\Phi_{\mathrm{u}\phi}(\tan\phi)\right) \qquad \text{. . . 5.2(2)}$$

TABLE 5.1(A)

MATERIAL STRENGTH AND SERVICEABILITY UNCERTAINTY FACTORS FOR SOIL (based on peak values of c' and ϕ')

A1

Soil or material uncertainty factor		Soil or fill conditions			
		Controlled fill		Uncontrolled fill	In situ material
		Class I	Class II		
Strength	$\Phi_{u\phi}$	0.95	0.90	0.75	0.85
	Φ_{uc}	0.90	0.75	0.50	0.70
Serviceability	$\Phi_{u\phi}$	1.0	0.95	0.90	1.00
	Φ_{uc}	1.0	0.85	0.65	0.85

NOTES:

1 Where fill materials defined by the term ‘other fill’ are used, then the partial factors to be used will depend on the type and quality of the fill materials. In no case shall the partial factor be greater than the partial factors for Class 1 ‘controlled fill’.

2 The soil strength parameters c' and ϕ' are effective strength parameters. Appendix D gives information on constant volume and residual strength parameters.

TABLE 5.1(B)

MATERIAL STRENGTH AND SERVICEABILITY UNCERTAINTY FACTORS FOR SOILS (based on c_u and Φ_u)

A1

Soil or material uncertainty factor		Soil or fill conditions			
		Controlled fill		Uncontrolled fill	In situ material
		Class I	Class II		
Strength	$\Phi_{u\phi}$	0.0	0.0	0.0	0.0
	Φ_{uc}	0.6	0.5	0.3	0.5
Serviceability	$\Phi_{u\phi}$	0.0	0.0	0.0	0.0
	Φ_{uc}	0.9	0.8	0.5	0.75

5.2.3 Determination of c and ϕ

The characteristic values of c and ϕ shall be determined by a combination of the following:

(a) Laboratory or field testing or both.

(b) Knowledge of the geological conditions of the site, its stress history and the likely soil behaviour.

(c) Engineering judgement.

5.3 MATERIAL FACTORS FOR STRUCTURAL COMPONENTS

The structural components of retaining walls, including their connections, shall be designed in accordance with AS 3600, AS 3700, AS 4100 or AS 1720, as appropriate.

5.4 STRUCTURE CLASSIFICATION FACTOR

The partial design factor to account for structure classification, as defined in Clause 1.2, Φ_n, shall be as defined in Table 5.2.

TABLE 5.2

STRUCTURE CLASSIFICATION DESIGN FACTOR (ϕ_n)

Structure classification	Design factor, (ϕ_n)	
	Ultimate limit state	Serviceability limit state
C	0.9	1.0
B	1.0	1.0
A	1.1	1.0

A1

NOTE: Depending on the structure classification (see Table 1.1), the consequences of the failure and the cost of stabilization, it may be necessary to adopt different partial factors to arrive at a practical and economic solution. Each case has to be assessed on its merits and the assignment of reduced partial factors, whilst likely to be necessary in some cases, should be carefully judged with respect to the type of structure and expected performance.

5.5 STRENGTH FACTORS FOR SOIL REINFORCEMENT

5.5.1 Tensile strength of soil reinforcement

5.5.1.1 *General*

Material strength factors shall be applied to the short-term ultimate tensile capacity of all soil reinforcement. These factors are related to the properties of the material itself as well as to construction and environmental effects and are either reduction or uncertainty factors.

Material property factors take into account manufacturing variations and time-dependent stress/strain behaviour (creep, rupture and extrapolation).

Construction and environmental factors take into account construction damage and environmental degradation (material loss and strength loss).

The partial material reduction and uncertainty factors are classified in Table 5.3 and described in this Section.

NOTE: Recommendations for the determination of partial material reduction and uncertainty factors are set out in Appendix K. The designer should nominate the appropriate factors and their basis for selection. The determination of reduction factors should be based on adequate test results. In the absence of test results, the values for reduction factors given in Appendix J should be used.

TABLE 5.3

MATERIAL REDUCTION AND UNCERTAINTY FACTORS

Factors	Manufacturing	Time-dependent stress/strain behaviour	Installation (construction)	Environmental
Material reduction factors	—	Creep rupture (Φ_{rc})	Installation damage (Φ_{ri})	Thickness (Φ_{rt}) Strength (Φ_{rs}) Temperature (Φ_{rst})
Material uncertainty factors	Product manufacture (Φ_{up})	Extrapolation (Φ_{ue})	—	Degradation (Φ_{ud})

A1

5.5.1.2 *Manufacturing variation factor*

The short-term ultimate tensile strength (T_u) shall be multiplied by an uncertainty factor
A1 (product) (Φ_{up}) which depends on the degree of quality control and whether it is a minimum or some other characteristic value.

5.5.1.3 *Time-dependent stress/strain behaviour factors*

The creep rupture strength is derived from the time at which samples of the reinforcing product are expected to fail when constantly loaded at a certain percentage of their initial resistance. The creep rupture strength corresponding to the required minimum service life shall be the appropriate reference as far as tensile strength is concerned.

For a required service life in excess of the limit of test data, it shall be necessary to carry out an extrapolation, which is valid only if there is no risk of a transition or 'knee' in the rupture mechanism, knowing that in a log-normal scale diagram the decrease of the creep rupture strength is represented by a straight line.

The short-term ultimate strength (T_u), shall be multiplied by the following factors:

(a) A partial material creep reduction factor (creep) (Φ_{rc}), which is based on test data conducted at the average estimated in-service temperature.

(b) A partial design uncertainty factor for cohesion of the soil and back fill material (extrapolation) (Φ_{ue}), which covers the uncertainties of the extrapolation.

A1

5.5.1.4 *Construction damage factor*

From manufacture to installation, the strength of the reinforcing product may be reduced due to—

(a) weathering during the storage period; and

(b) damage due to backfilling and compaction.

The amount of damage depends on the material and structure of the product as well as the nature of backfill, assuming reasonable compactive effort.

A1 The short-term ultimate tensile strength (T_u) shall be multiplied by a reduction factor (construction or installation) (Φ_{ri}), which is based on test data.

5.5.1.5 *Environmental degradation factor*

All types of reinforcing materials are potentially affected by chemical, biological or physical actions due to the soil, its constituents, the air and water.

The short-term ultimate tensile strength (T_u) shall be multiplied by the following factors:

(a) A thickness reduction factor (Φ_{rt}), which accounts for the diminishing average thickness or section area of the material and which depends on the required service life. It typically concerns metallic corrosion, but may also be applied to the superficial erosion of polymers.

(b) A strength reduction factor (Φ_{rs}), which accounts for the loss of strength and which depends on the required service life. It covers the internal alterations of the material itself as well as the superficial unevenness, which makes strength decrease more than the average section area of the material, typically in the case of metals.

(c) A temperature reduction factor (Φ_{rst}), which accounts for the loss of strength and which depends on the temperature of the material during the life of the structure.

(d) An uncertainty factor (overall degradation) (Φ_{ud}), which depends on the limitation of knowledge or experience and which covers the unknowns regarding long-term behaviour.

For strength calculations, a combined reduction factor (Φ_{rt} Φ_{rs}) may be calculated from a nominal structural corrosion allowance, normally specified for metals.

5.5.1.6 *Design tensile strength of the soil reinforcement*

The design tensile strength of the soil reinforcements shall be determined from the following equation:

A2 $$T_d^* = T_u(\Phi_{up})\ (\Phi_{rc}\ \Phi_{ue})\ (\Phi_{ri})\ (\Phi_{rt}\ \Phi_{rs}\ \Phi_{rst}\ \Phi_{ud})\ (\Phi_n) \qquad \ldots 5.5(1)$$

where

T_d = design tensile strength

T_u = short-term tensile strength

Φ_{up} = uncertainty factor (product)

Φ_{rc} = reduction factor (creep)

Φ_{ue} = uncertainty factor (creeptime extrapolation)

Φ_{ri} = reduction factor (installation)

Φ_{rt} = reduction factor (thickness)

Φ_{rs} = reduction factor (strength)

Φ_{rst} = reduction factor (temperature)

Φ_{ud} = uncertainty factor (degradation)

Φ_n = structure classification factor (see Table 5.2)

A1 The maximum tensile force to be carried by the j^{th} layer of reinforcement shall not exceed the design strength of the reinforcement, i.e.

$$T_j^* \leq T_d^*$$

where

A1 T_j^* = design maximum tensile force in the j^{th} layer

5.5.2 Soil/reinforcement interaction strength

5.5.2.1 *General*

A material strength factor shall be applied to the soil/reinforcement interaction strength. This factor shall be assessed by the designer for relevance to the subject soil characteristics and reinforcement configuration and is an uncertainty factor.

The uncertainty factor for soil/reinforcement interaction shall be Φ_u.

The appropriate range of values is given in Table 5.4.

TABLE 5.4

MATERIAL STRENGTH FACTORS FOR SOIL/REINFORCEMENT INTERACTION

Soil reinforcement interaction factors	Soil or fill conditions	Strength and stability load cases	Serviceability load case
Φ_u (Sliding resistance across reinforcement surface)	Controlled fill (Class I or II)	0.8	1
Φ_u (Pull out resistance)	Controlled fill (Class I or II)	0.8	1
	Natural or in situ soil	0.75	1

5.5.2.2 *Design soil/reinforcement interaction strength*

The design soil/reinforcement interaction strength of the soil reinforcements shall be determined using the following equation:

$$T_{di}{}^{*} = T_{ij} \Phi_{u}\, \Phi_{n} \qquad \ldots\ 5.5(2)$$

where

$T_{di}{}^{*}$ = design soil/reinforcement interaction strength

T_{ij} = soil/reinforcement interaction capacity of the reinforcement

Φ_{u} = uncertainty factor for soil/reinforcement interaction (see Table 5.4)

Φ_{n} = structure classification factor (see Table 5.2)

A1 The maximum interaction force (sliding or pull out) shall not exceed the design soil interaction strength of the soil reinforcement, that is,

$$T_{ij}{}^{*} < T_{di}{}^{*}$$

where

$T_{ij}{}^{*}$ = design soil interaction force at the j^{th} layer

5.5.3 Connection strength

The design connection strength shall be determined using the following equation:

$$T_{cd}{}^{*} = T_{c} \Phi_{ucon}\, \Phi_{n} \qquad \ldots\ 5.5(3)$$

where

$T_{cd}{}^{*}$ = design connection strength

T_{c} = connection strength determined by test

Φ_{ucon} = uncertainty factor for connection strength = 0.75

Φ_{n} = structure classification factor (see Table 5.2)

The maximum force in the connection shall not exceed the design strength of the connection, i.e.

$$T_{cj}{}^{*} < T_{cd}{}^{*}$$

where

$T_{cj}{}^{*}$ = connection force at the j^{th} layer

SECTION 6 CONSTRUCTION

6.1 GENERAL

All earth-retaining structures covered by this Standard shall be detailed on drawings or other relevant documents that provide sufficient information to enable them to be constructed in accordance with designs based on the requirements of Sections 3, 4 and 5.

All construction shall be—

(a) in accordance with the drawings or other relevant documents; and

(b) in compliance with Clauses 6.2 and 6.3.

6.2 CONSTRUCTION TOLERANCES

The designer shall specify construction tolerances that take into account movement during construction and ensure the intent of the design is not compromised. The requirement is deemed to be satisfied if the tolerances defined in Table 6.1 and illustrated in Figure 6.1 are met. If other structures in the vicinity of the wall require closer tolerances, the designer shall specify appropriate tolerances for the items in Table 6.1 accordingly.

TABLE 6.1

CONSTRUCTION TOLERANCES

Element	Vertical position	Horizontal position	Vertical alignment	Horizontal alignment
Soil surface	±100 mm	*	*	*
Facings and wall structures	±50 mm	±50 mm	±20 mm in 3.0 m	±20 mm in 3.0 m
Footings or supports	±50 mm	±50 mm	±20 mm in 3.0 m	±20 mm in 3.0 m

* Not applicable

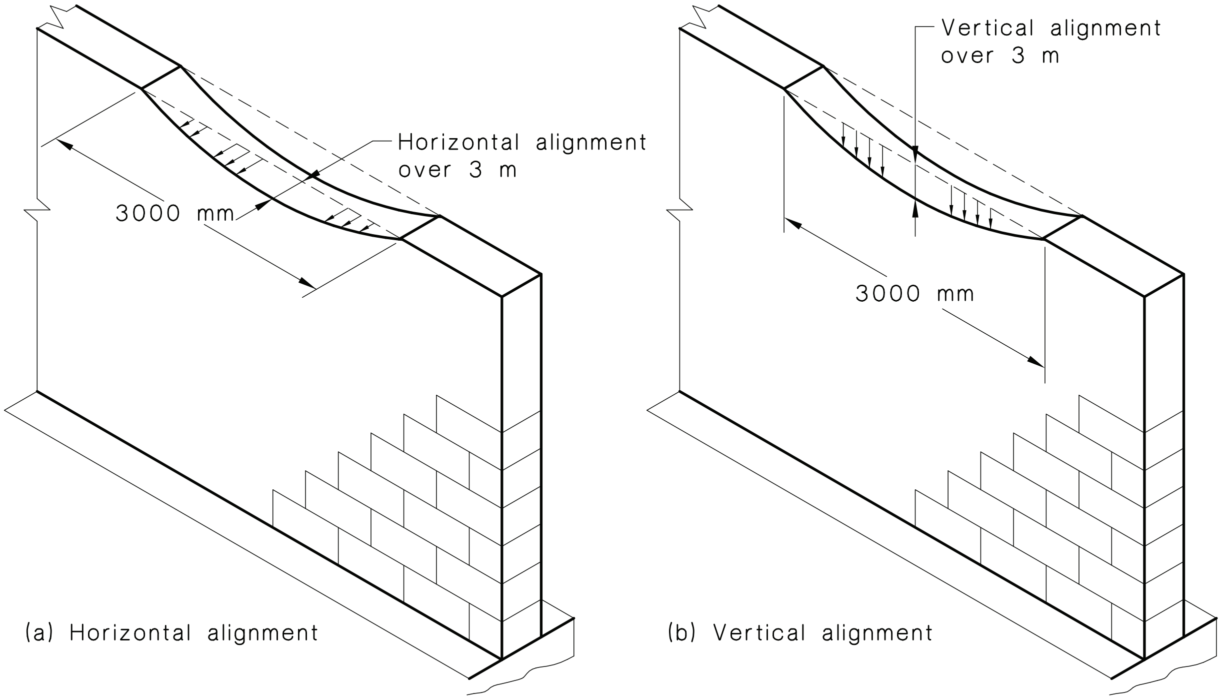


FIGURE 6.1 TOLERANCES

6.3 SPECIFIC REQUIREMENTS

6.3.1 General

In addition to the construction requirements detailed in the drawings and other relevant documents, the specific requirements given in Clauses 6.3.2 to 6.3.10 shall apply.

6.3.2 Site preparation/excavation

Excavation shall include the removal of all materials necessary for the construction of the earth-retaining structure.

6.3.3 Foundation soils

The foundation shall be compacted to provide approximately uniform subgrade stiffness and required bearing capacity. Soft spots shall be removed and replaced with suitable fill.

6.3.4 Footings

When required, a plain concrete levelling pad or footing shall provide a firm, level surface to facilitate erection of retaining wall units or facing panels.

6.3.5 Fill material

Fill for earth-retaining structures shall comply with the design.

NOTE: Guidelines for fill materials are given in Appendix D.

6.3.6 Placement of fill/compaction

Where fill is to be compacted, the work shall be performed such that the compaction can be achieved without damaging or dislodging adjacent structures or reinforcement.

6.3.7 Soil reinforcement

Where more than one strength grade of soil reinforcement is to be used, the reinforcement or ‘bundles’ of reinforcement shall be clearly marked to indicate the strength/grade.

Soil reinforcement shall be laid and tensioned (if necessary) to avoid excessive deformation of facing panels. Where layers of soil reinforcement overlap, fill may be required between these layers.

6.3.8 Facings

Facing panels shall be stored and handled to prevent damage to units. Alignment of units shall be checked after placement and compaction of fill to ensure units have not been displaced. Provision shall be made for compressible joint filler where appropriate.

6.3.9 Drainage

Where required by the design, a drainage system shall be provided such that it—

(a) is capable of relieving hydrostatic head behind the structure or within the reinforced soil mass;

(b) drains freely to a location removed from the structure or reinforced soil mass;

(c) incorporates a means of preventing siltation; and

(d) does not lead to scouring or erosion.

6.3.10 Connections

When required, connections shall ensure a positive and integral connection between facing elements and soil reinforcement or other anchorages.

SECTION 7 PERFORMANCE MONITORING

A1 The appropriate level of performance monitoring is indicated in Table 7.1. A program for regular monitoring of retaining structures and stabilized slopes that have been classified in structure classification C or B shall be recorded and implemented.

TABLE 7.1

MONITORING LEVELS FOR EARTH-RETAINING STRUCTURES

Structure classification (see Table 1.1)	Recommended monitoring level
C	Regular visual inspection and inspections after events such as floods or earthquakes. Monitoring of vertical and horizontal deformations, flows from drainage system, possibly pore pressures, earth pressures and stresses in selected soil-reinforcing elements and anchors. Check on corrosion or degradation of soil reinforcing materials (e.g. exhuming of dummy reinforcing elements embedded in the backfill during construction)
B	Regular visual inspection and inspections after events such as floods or earthquakes. Includes check of effectiveness of the drainage system. Basic monitoring of lateral deformation
A	No monitoring required

A1

NOTES:

1 Basic indicators of the performance of earth structures are—
   (a) lateral deformation; and
   (b) settlement.

2 A visual inspection should detect excessive movement, cracks or ruptures, lack of drainage capacity, physical changes and changes in the environmental conditions.

3 Where a wall is evidencing structural distress, to assess the degree of instability and to assist in the selection of remedial measures, the following could also be considered:
   (a) Seepage flows (including flows from drainage pipes).
   (b) Pore pressures.
   (c) Vertical earth pressures.
   (d) Lateral earth pressures.
   (e) Vibrations.
   (f) Stresses in structural components (including reinforcing elements).
   (g) Rate of corrosion or chemical degradation of reinforcing strips and meshes.
   (h) Temperature.
   (i) Any other factor considered likely to affect performance.

4 Inspections carried out once per year could be considered to be ‘regular’ for purposes of this table.

5 The purpose of monitoring is to ensure one or both of the following objectives:
   (a) Ensuring the safety of the structure.
   (b) Checking assumptions and predictions of the design.

6 Contributing to data, which in the longer term will lead to an improvement in design procedures.

APPENDIX A

STRUCTURE CLASSIFICATION

(Informative)

A1 GENERAL

Clause 1.2 requires earth-retaining structures to be classified according to Table 1.1. For such structures, the classification should be assessed with respect to likely consequences of structure failure.

A2 STRUCTURE FAILURE CONSEQUENCE

The consequence of structure failure is related to the loading type and its location on the structure, as well as the relationship between the wall slope and geometry and associated structures and facilities.

The structure should be classified according to the consequences of structure failure by considering the following questions (see Table 1.1):

(a) What is the likely loss or damage to associated structures or facilities?

(b) What is the likely loss of use of function or facility?

(c) What is the possibility of injury to people?

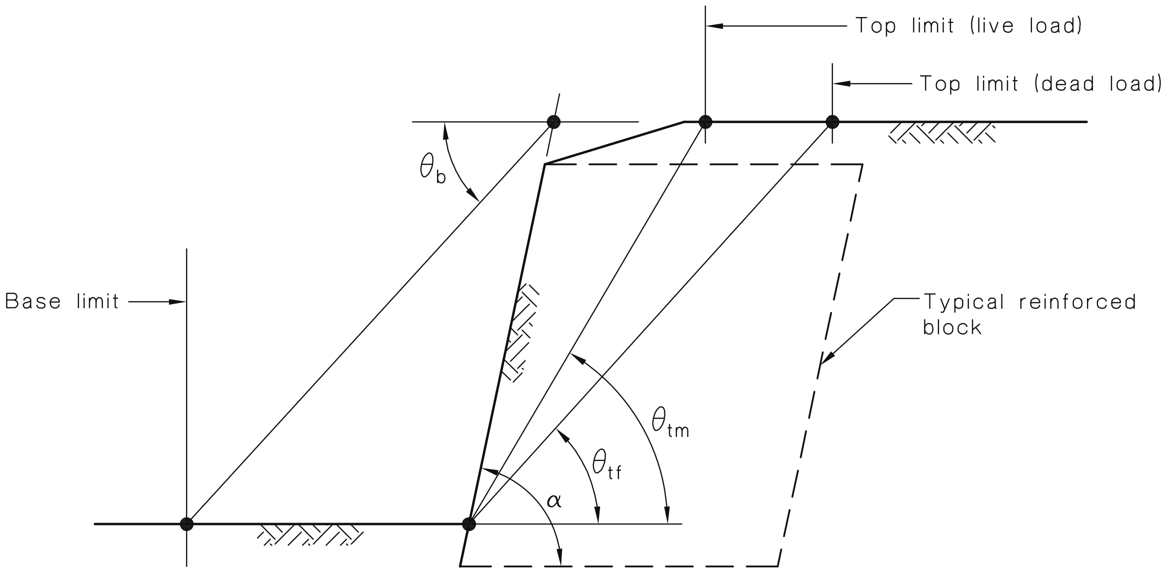
(d) What is the likely size and weight of facing elements?

(e) What is the height of wall?

(f) What is the proximity of nearby structures?

For a reinforced earth- be classified according to the consequences of structure failure by considering the following questions (see Table 1.1):

What is the likely loss retaining structure, the classification should be assessed with respect to the consequences of failure with reference to the location of facilities on top, or at the base, and the structure geometry as shown in Figure A1.



$$\theta_{tm} = \frac{2\alpha + \phi}{3}$$

$$\theta_b = \theta_{tf} = \frac{2\alpha + \phi}{5}$$

A1 NOTE: Structures beyond the base limit or beyond the top limit are unlikely to be affected by, or have an effect on, the structure classification.

FIGURE A1 WALL SLOPE AND GEOMETRY

A1
A3 GUIDANCE ON CLASSIFICATION OF STRUCTURES

Table A1 gives examples of the classification of earth-retaining structures.

Table A2 gives a general method for classifying an earth-retaining structure according to the human life and other consequences arising from the failure of the structure. For example, a low wall supporting a road that is the only access to a major hospital might have high social consequences in the event of a failure, even though it is inexpensive to repair, and its failure would not in itself constitute a high risk to life. The consequences of failure during events such as earthquakes or cyclones might be relevant in this case.

A1

TABLE A1

EXAMPLES OF STRUCTURE CLASSIFICATION

Classification	Description	Examples
A	**Low** consequence for loss of human life, *or* **small or moderate** economic, social or environmental consequences	Where failure would result in minimal damage and loss of access: Walls in areas rarely visited by people Walls on private property supporting gardens, fences, etc.
B	**Medium** consequence for loss of human life, *or* **considerable** economic, social or environmental consequences	Where failure would result in moderate damage and loss of services: Walls not included in classifications A or C Walls supporting or above normal structures Walls supporting minor roads Walls above public spaces Walls with height ≥1.5 m
C	**High** consequence for loss of human life, or **very great** economic, social or environmental consequences	Where failure would result in significant damage or risk to life: Walls supporting structures identified for post-disaster recovery Walls supporting or above access or services to structures identified for post-disaster recovery

TABLE A2

GENERAL METHOD FOR STRUCTURE CLASSIFICATION

Public impact (economic, social or environmental)	Risk to life		
	Low	Medium	High
Low	A	B	B
Medium	B	B	C
High	B	C	C

APPENDIX B

GROUND ANCHORS

(Informative)

B1 GENERAL

The term ground anchor refers to a tensile reinforcement, which may typically be a high tensile steel strand, wire, bar or high-strength polymer, placed in a hole drilled at an aspect into a foundation material. Whilst this Appendix sets out guidelines for the classification and design of ground anchors, the notes provided in this Appendix should not prevent the use of alternative designs and materials in accordance with engineering principles.

The relevant sections of these guidelines are also applicable to the design and installation of rock bolts, soil nails and dowels.

B2 ANCHOR COMPONENTS

Anchors generally have the parts as illustrated in Figures B1 and B2. The parts are defined as follows:

(a) *Anchor head*—a device, usually consisting of a bearing plate and head or nut assembly which permits the stressing and lock-off of the anchor tendon or bar.

(b) *Tendon*—a tensile reinforcement used to transfer load from the anchor head to the anchorage zone, which may consist of high-tensile steel tendon, wire strand or bar. The anchor may also be of polymer materials.

(c) *Anchorage*—that section of the tendon which is bonded, or otherwise secured, in the anchorage zone.

(d) *Bond length (grouted anchors only)*—that portion of the anchor fixed in the grout bulb through which load is transferred to the surrounding soil or rock.

(e) *Bond zone (mechanically anchored anchors only)*—that portion of the anchorage zone in which the load is transferred from the plate or expansion anchor into the surrounding soil or rock.

(f) *Free length*—the unbonded length of the anchor, which is free to elongate elastically and transmit the resisting force from the anchorage zone to the structural element via the anchor head.

(g) *Grout*—either a cement-based mixture, which may include additives, or a specially formulated chemical compound (e.g. epoxy resin).

NOTES:

1 In grouted anchors, the grout also transfers load from the tendon to the soil or rock.

2 In some temporary anchor installations, the anchor-free length may not be grouted.

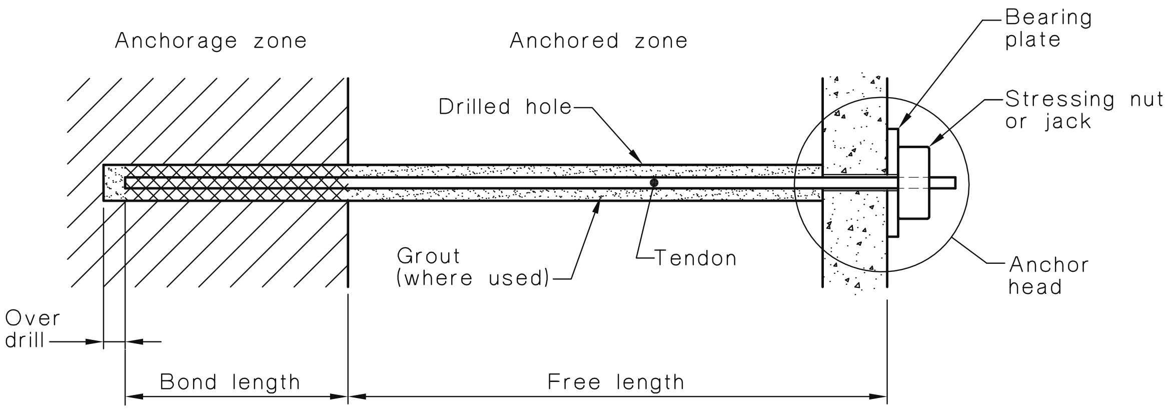


FIGURE B1 GROUND ANCHOR TERMINOLOGY (GROUTED ANCHORAGE)

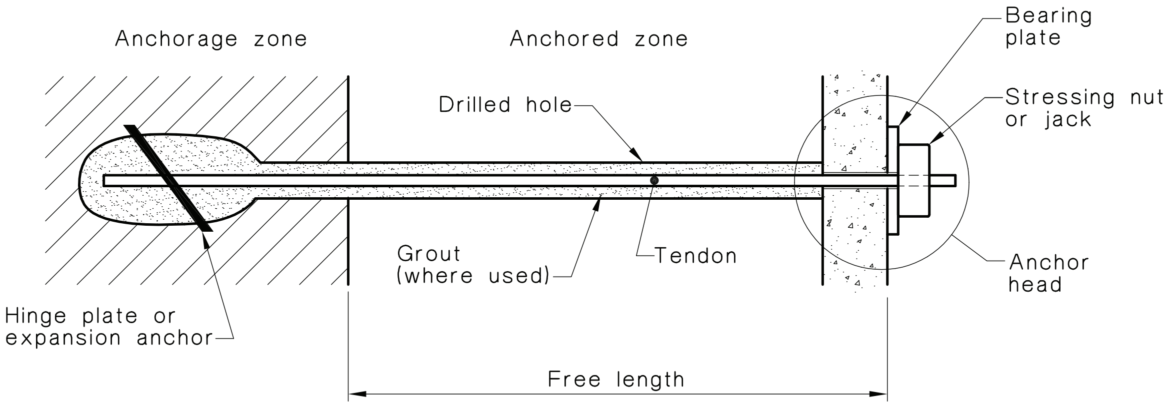


FIGURE B2 GROUND ANCHOR TERMINOLOGY (MECHANICAL ANCHORAGE)

B3 TYPE OF ANCHORS

B3.1 Permanent anchors

An anchor installation that is required to ensure the stability and satisfactory performance of the permanent structure being supported.

B3.2 Temporary anchors

An anchor installation designed for a short project life, usually 5 years (see Table 3.1).

B4 DESIGN

B4.1 General

The design of the anchors should take into account—

(a) the movement possibilities of the structure;

(b) the rigidity of the structure;

(c) the compatibility of the movements of the anchors and the anchored zone;

(d) the behaviour of the ground under stressing loads;

(e) anisotropy, inhomogeneity and discountinuties of the rock;

(f) overall stability and failure mechanisms; and

(g) their design life (see Table 3.1).

NOTES:

1 These design considerations are also applicable to rock bolts, soil nails and dowels.

2 The state of the art for ground anchors has not yet progressed to the point where the application of formulae can necessarily ensure adequate capacity. As such, whilst anchor capacities can be estimated from bond lengths and load transfer to the anchorage zone, the service and design loads should be verified by testing each production anchor to assure load capacity.

3 Where appropriate regular checks of anchor loads should be made.

B4.2 Design of anchors with a rock anchorage zone

B4.2.1 *General*

The determination of the details of the anchorage, bond and anchor free length requires consideration of—

(a) uplift resistance of rock mass;

(b) tendon design;

(c) rock to grout bond and grout to tendon bond (grouted anchors);

(d) soil to tendon bond for soil nails or soil reinforcement; and

(e) strength of mechanical anchorage for mechanically secured anchors.

B4.2.2 *Uplift resistance of rock mass*

For an isolated vertical anchor, the uplift resistance may be equated to the weight of an inverted cone of rock having either a base angle of 60° or 90° when measured from the middle of the bond length or of a cone having a base angle of 60° or 90° and measured from the bottom of the anchor grout as shown in Figure B3. The 60° angle is to be used when the rock mass is soft, heavily fissured or weathered. Where the anchor is below the water table, the buoyant weight of the rock shall be used. In either case, the mass of the cone has to be sufficient to balance a test force of 85% of the characteristic ultimate tensile resistance of the tendon. With the inclusion of the rock shear resistance, the anchorage should be able to develop the characteristic ultimate tensile resistance of the tendon, but care has to be exercised in highly fissured rock, especially when high pore water pressures occur.

For multiple anchors at close centres, the rock cones may intersect and a wedge-shaped rock section should be considered for resistance calculations.

For a concentrated anchor layout where the fixed ground anchors are spaced at less than 0.5 times the fixed anchor length, it is recommended that a vertical stagger of 0.5 times the fixed anchor length be used to prevent possible laminar separation of the rock.

The resistance of the rock to anchorage loads should in all cases be evaluated in conjunction with adequate field and laboratory investigation and testing. Lesser anchorage depths may be used where it can be demonstrated that the required forces can be achieved.

In the case of horizontal anchors, the anchorage should be located so that the tendon crosses any known plane of failure and the pull-out capacity should be calculated from the principles of rock mechanics.

A minimum fixed anchor length of 3 m is recommended to guard against variable rock quality and constructional imperfections.

The design resistance effect should be greater than the design action effect. For the case involving dead and live loadings, the requirements may be expressed by the following:

$$S^* \leq 0.8\Phi_{\mathrm{n}}\Phi_{\mathrm{k}}\, G^{\mathrm{R}}$$

where

G^R = part of the dead loading tending to resist instability

Φ_k = see Table B1

Φ_n = see Table 5.2

Further details on the application of load combinations are given in Appendix I.

TABLE B1

MINIMUM PROOF LOAD

Anchor category		Importance category reduction factor Φ_k	Minimum proof load/working load $\frac{P_P}{P_W}$
A1 1	Temporary anchors where the service life is less than six months of a structure classification A structure, that is, where failure would have few serious consequences and would not endanger public safety, e.g. short-term pile test loading using anchors as a reaction system	0.9	1.1
2	Temporary anchors with a service life of up to five years of a structure classification B structure, that is, where, although the consequences of load failure are quite serious, there is no danger to public safety without adequate warning, e.g. retaining wall tie backs	0.85	1.25
3	Any permanent anchors and also temporary anchors of a structure classification C structure, that is, where the consequences of failure are serious, e.g. temporary anchors for main cables of a suspension bridge, or as a reaction for lifting heavy structural members	0.8	1.5

NOTE: For class of structure refer to Table 1.1.

TABLE B2

MINIMUM MATERIAL REDUCTION FACTORS

Φ_k	*Tendons*	0.9
Φ_b	*Bond*	0.7

B4.2.3 *Rock to grout bond*

Bond at the rock to grout interface may be considered as uniformly distributed along the bond length for design purposes. In reality, however, the load is carried non-uniformly with a maximum transfer stress at the junction of the free and bond lengths, diminishing quickly towards the distant end of the bond length.

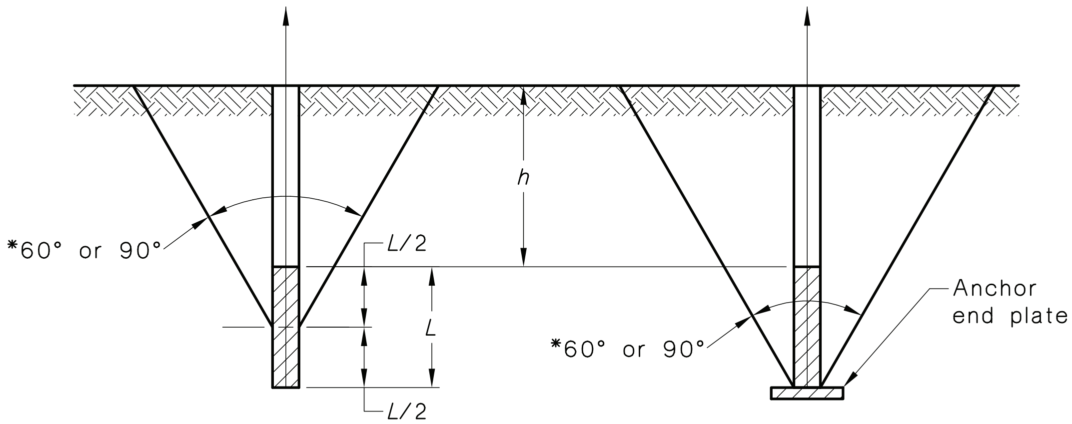
When shear strength tests are carried out on representative samples of the rock mass, due allowance should be made for in-place joints or other defects, the average effective design bond stress not exceeding the minimum shear strength of the rock multiplied by the relevant material factors and importance factors specified in Tables B1 and B2.

In the absence of shear strength data or pull-out tests refer to Table B3 for typical values of characteristic ultimate rock-to-grout bond stresses.

A minimum fixed anchor length of 3 m is recommended to guard against variable rock quality and constructional imperfections.

Experience has shown that increasing bond length beyond 10 to 12 m does not result in increased load capacity.

Weathering degree, degree of mineral bond and distance of disjunction surfaces are three conditions with the most unfavourable condition being decisive.

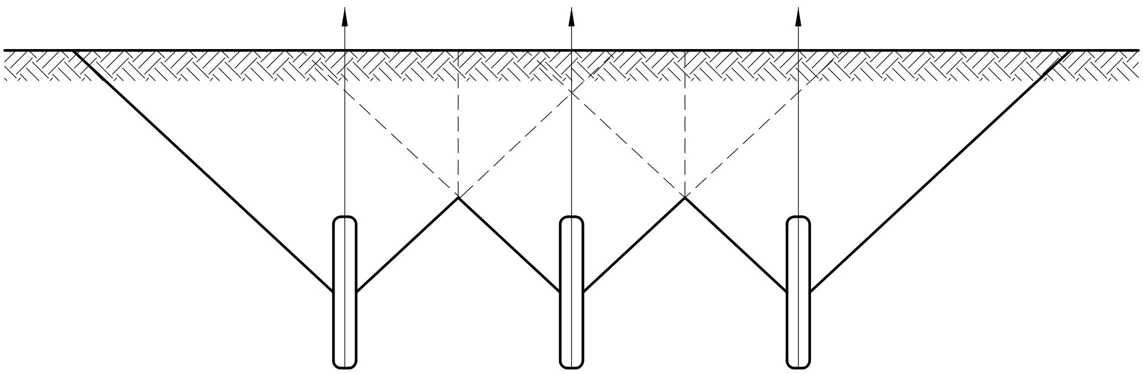


(1) Load transferred by bond (2) Load transferred by end plate

*60° used when rock mass is soft, heavily fissured or weathered

*90° used in all other rock conditions

(a) Geometry of cone (after Littlejohn and Bruce 1977)



(b) Interaction of cones for overall stability analysis (after Littlejohn and Bruce 1977)

FIGURE B3 TRANSIENT LOADING IN ROCK

TABLE B3

ULTIMATE BOND STRESSES FOR ROCK ANCHORS (MPa) (OSTERMAYER)

Condition	Rock type		
	Igneous and metamorphic	**Conglomerate and breccia**	**Argillaceous sediments**
Weathering degree (see AS 1726) Mineral bond Distance of disjunction surfaces	Granite, gabbro, basalt, tuff, diorite, gneiss, schist, slate, quartzite	Sandstone, limestone, chalk, dolomite, stone	Marl, shale, claystone, mudstone, siltstone
Unweathered (FR) Very good mineral bond More than 0.3 m	4.0	2.7	1.7
Weathered (SW) Good mineral bond Between 0.1 and 0.3 m	2.5	1.9	1.0
Distinctly weathered (DW) Poor mineral bond Less than 0.1 m	1.3	0.8	0.4

NOTE: For weathered rock description, see AS 1726.

B4.2.4 *Grout to tendon bond*

The characteristic ultimate value of bond resistance may be taken as—

A1 (a) 1.0 MPa for clean wire tendons; and

(b) 2.0 MPa for clean strand tendons.

For strand tendons, the effective area for bond should be taken as the surface area of a smoother cylinder having a diameter equal to the nominal diameter of the strand.

For bar tendons, the bond may be improved by using the nuts and washers in the bond length, with anchorage lengths determined by actual test.

For strand tendons, a weave pattern in the bond should increase bond resistance.

The relevant materials factor specified in Table B2 should be applied to the above values.

B4.2.5 *Tendon design*

Tendons for rock anchors should be designed such that—

$$S^* < \Phi_k \Phi_n \Phi_t f_p A_p \quad \text{. . . B4(1)}$$

where

f_p = tensile strength of the tendon

A_p = cross-sectional area of tendon

B4.3 Design of soil anchors (including soil nails)

Anchorage resistance should be determined generally as for rock anchors except that soil grout bond length may be estimated as follows:

(a) For bulb type anchors (see Figure B4) formed by high pressure grouting, the design ultimate pull-out resistance may be determined from the following equation.

$$T_1^* = \Phi_n \Phi_b N_1 L_f \tan\phi^* \quad \text{. . . B4(2)}$$

Values of N_1 are 130 to 160 kN/m of fixed anchor length.

(b) For multi-under-reamed anchorage (see Paragraph B3) in stiff cohesive soils with bells at 3D centres, design ultimate pull-out resistance may be determined as—

$$T_1^* = \Phi_b D L_f c^* \qquad \ldots \text{B4(3)}$$

where

L_f = fixed anchor length

D = diameter of bell of under-reamed ground anchor

Both these equations are simple but crude. A more accurate estimate can be obtained by taking into account factors such as contact pressure at the fixed anchor/soil interface, effective overburden pressure and end-bearing capacity.

Where inclined or horizontal plate anchors are used, the evaluation of the capacity of the anchor is more complex and will depend on the shape of the anchor, as well as its depth and the material properties.

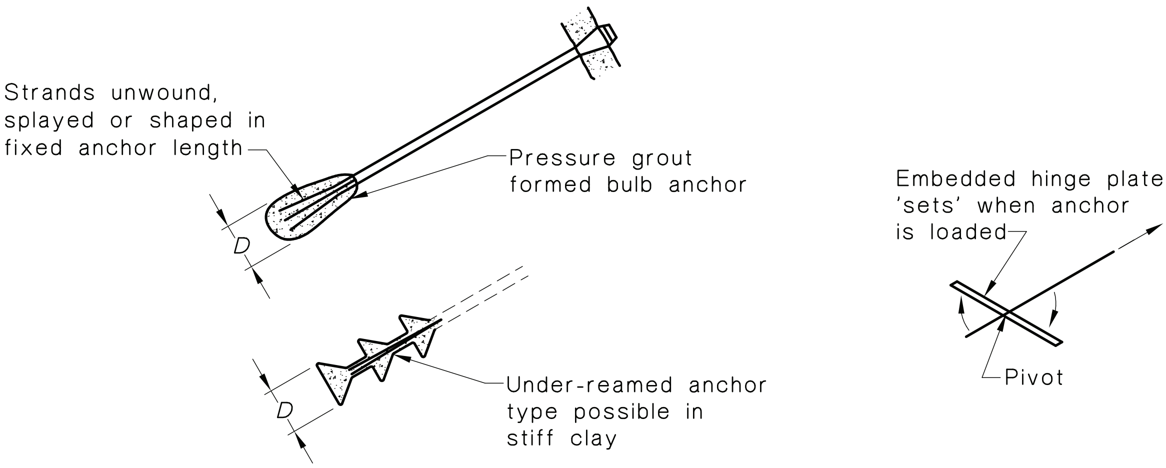


FIGURE B4 TYPES OF SOIL ANCHOR

B4.4 Anchorage loads

For a ground anchor to function effectively, it is important that the designer specify the following:

(a) P_W—the required anchor working load.

(b) P_P—the anchor proof (or test) load.

(c) P_{LO}—the anchor 'lock-off' load.

(d) P_{LT}—the long-term design load in the ground anchor.

These are more fully described as follows:

(i) P_W—the working load necessary to ensure that the structure behaves in a satisfactory manner. Under normal circumstances working loads should not exceed 60% of the characteristic yield strength of the tendon. In some situations (e.g. temporary anchors) it may be acceptable to load the tendon to 65% of the characteristic strength of the tendon. The above values apply to anchors manufactured of steel. For anchors manufactured of polymers, the designer should select appropriate values.

(ii) P_P—proof load to which the ground anchor is to be loaded to check the adequacy (see Table B1 for minimum proof load to working load factors).

(iii) P_{LO}—the lock-off load, which will be left in the ground anchor when stressed.

NOTES:

1 The lock-off load should not exceed 75% of the characteristic yield strength of the tendon.

2 In some applications the lock-off load will be very much less than P_W.

(iv) P_{LT}—the long-term load in the anchorage, which will exist in the anchorage as a result of its construction, prestressing, creep and subsequent ground movements.

NOTE: P_{LT} should be greater than or equal to P_W.

B4.5 Testing of anchors

All anchors should be ‘proof load’ tested during installation. The minimum proof load test should consist of—

(a) stressing of the anchor to the required ‘proof load’ and holding at that load for at least 15 min; and

(b) relaxing of the anchor to the specified ‘lock off’ load.

If an anchor fails to hold the proof load for the required time, then the anchor should be deemed to have failed, or the test result referred to the designer for comment and verification of capacity.

B4.6 Anchor technical description

The designer of a ground anchor should nominate the following as a minimum:

(a) Type of anchor (i.e. temporary or permanent) and the design life.

(b) Expected anchorage zone material for the anchor.

(c) Minimum free length.

(d) Ultimate design load.

(e) Test proof load and loading duration.

(f) Lock-off load.

B5 CORROSION PROTECTION

The tendon anchorage and anchor head arrangement has to be given adequate corrosion protection, which has to remain effective throughout the design service life of the anchorage. The effectiveness of the protection should not be impaired during storage, transport, installation and stressing of the anchorage. Proof of the suitability of the corrosion protection system should be supplied.

For permanent grouted anchors, the corrosion protection system should be fully encapsulated in polyethylene sheathing or better. The only exception, in special circumstances, is the use of prestressing bars coated with epoxy coating. Whilst corrosion protection by cement grout is usually sufficient for temporary anchors, in aggressive ground conditions it may be necessary to fully encapsulate the anchor. A film of rust on the tendon is not necessarily harmful and may improve the grout to tendon bond but any form of pitting is unacceptable.

APPENDIX C

SOIL NAILING FOR EARTH-RETAINING STRUCTURES

(Informative)

C1 GENERAL

Soil nailing may be used for the construction of either temporary or permanent walls or for stabilizing walls formed in cut.

Constructing a soil-nailed wall involves reinforcing an exposed excavated face as work proceeds. The reinforcing consists of introducing a pattern of passive bars that essentially work in tension; these are usually installed normal to the face or inclined slightly downward.

Using this method and working from the top down, a mass of reinforced soil is gradually created. In order to avoid the soil between the points of reinforcement becoming unstable, some form of facing needs to be installed; this is generally achieved with reinforced shotcrete. The facing can be vertical or battered and can include benches (see Figure C1).

Once constructed, a soil-nailed wall shows a certain similarity with a reinforced-soil wall. The fact that a soil-nailed wall will have been built downward by progressive excavation and nailing, with the soil being reinforced in situ, and a reinforced soil wall is constructed by placing successive layers of fill and reinforcement, constitutes an essential difference.

Nails can be installed in two principle ways—

(a) either by drilling and then grouting with cement grout or mortar in a pre-drilled hole; or

(b) by either percussive methods, vibro-drilling or ballistic techniques.

Soil nailing techniques are also well-suited to retaining-wall strengthening and repair.

Nails are generally made of steel, although other materials have been used.

The facing wall is generally constructed of sprayed concrete, although some installations have used soft (e.g. chain wire mesh) facings.

In the case of a facing wall of reinforced sprayed concrete, the thickness depends mainly on the grid layout of the nails, pull out forces at the nail head and corrosion considerations. Unlike other reinforced soil techniques, the building of a soil nailed wall has several critical phases, viz.

(i) local excavation stability during each earthworks excavation stage prior to the installation of nails;

(ii) overall excavation/partially completed wall stability during the excavation for a particular stage; and

(iii) the completed wall.

Each of these stages requires careful analysis by a designer. Also, the installation of drainage within and behind the facing of a soil nail wall requires careful detailing.

State-of-the-art techniques and design methods are reviewed elsewhere, (see Refs 1, 2 and 3). Details of field performance are given in the French national research project ‘Clouterre’, (Ref. 4).

C2 BASIS FOR DESIGN

Soil nailing has much in common with conventional reinforced soil in that the reinforcing members are generally used to provide tensile resistance. The principal applications of soil nailing have been the construction of new slopes in cuttings, or as remedial works to unstable or potentially unstable slopes, (Refs 2, 4, 5, 6, 8 and 9) and structures (Ref. 12).

As soil nailing is a process of installing dowel type soil anchors that are not tensioned, it is important that the following be observed:

(a) The dowel type anchors 'self-tension' under load. Consequently, the nails should be installed at an angle to the predicted active wedge shear plane which allows this 'self-tensioning' to occur. This normally requires the nails to be installed at angles of less than 15° to the horizontal.

(b) The nails are provided with a secure connection to the facing material; the strength of this connection should be as set out in Table G2 of Appendix G.

(c) The nails are in line with the suggestions on ground anchors (rock bolts and dowels) set out in Appendix B.

In its final configuration, a soil-nailed structure is similar to a conventional fill-reinforced soil structure. However, there are important differences in construction, particularly as the nails are inserted directly into an existing mass of earth rather than installed with the fill, as is the case for a reinforced soil structure (see Figure C2).

Furthermore, the construction process for excavated faces is such that the upper nails are loaded first, in contrast to a reinforced soil wall in which the lower reinforcements are loaded first. These different construction procedures thus influence the transient distribution of loads, and thus the required strength of the various reinforcing elements.

C3 LIMIT STATES

The ultimate limit states for soil-nailed walls are similar to those listed for reinforced fill walls. However, as a critical condition for a soil-nailed structure can occur during the staged excavation/nailed installation process this condition is an important 'Limit State'. The serviceability limit states for soil-nailed walls are similar to those for reinforced soil slopes and walls. In soil nailing, however, there will be some movement of the nailed mass of earth in order to generate the tensile and shear stresses required for stability.

C4 EXTERNAL STABILITY

External stability considerations for nailed walls are similar to those for reinforced fill slopes and walls.

C5 INTERNAL STABILITY

C5.1 General

All soil reinforcement systems, including soil nailing, have considerable scope for redistribution of load between elements. Thus, if an element is overloaded, by attaining limiting friction, it can slip and redistribute the excess load to others. A further safeguard with a soil-nailing system is that additional nails can be provided if movement of this structure is greater than anticipated.

Internal failure of a soil-nailed structure is assumed to result from quasi-rigid body rotation and two zones can be distinguished, an active zone, and a resistant zone, which are separated by a slip surface along which movement takes place. The surface has to be kinematically admissible, and the relative geometrical movements have to be feasible.

Several forms of failure surface have been proposed including wedge, two-part wedge, circular slip and log-spiral. Loads are induced in the nails, which extend into the resisting zone providing support for the active zone. The tensile and shearing forces in the nails should provide the restoring component to the out-of-balance force or moment between the disturbing forces and the available internal friction angle of the soil.

Although the model of deformation is usually taken to be two-dimensional, in some cases it may be three dimensional; for example, with skew nailing or where the nailed structure changes direction in plan.

The stages of a design usually involve the following:

(a) The determination of the position of the critical slip surface, and the resisting force or moment required to maintain the equilibrium of the active zone.

(b) The determination of the tensile and shear loads for an initial constant spacing and inclination of nails of constant stiffness and length.

(c) A check for each level, allowing for the stages of construction, against failure due to each of the following listed limit states:

  (i) Tension in the nail at the slip surface.

  (ii) Pull-out of the length of nail in the resistant zone.

  (iii) Bending and shear in the nail near the slip surface.

  (iv) Bearing failure of soil against the nail.

(d) The selection of a new and improved pattern and disposition of nails and re-analysis.

It is necessary, particularly for the construction of new slopes, to ensure that unsupported heights can withstand temporary forces.

C5.2 Methods of assessing internal stability

A procedure should be selected, which should ensure that the most critical failure surfaces are determined. Methods that are commonly used correspond to those generally applied for stability analyses of slopes, although other failure surfaces may be employed.

Where soil nailing is being used as a remedial measure in an unstable slope, attention should be paid to any existing failure surfaces. The geometry of the failure surface should be determined and relevant shear strength parameters used, including residual values where appropriate, to assess stability in these circumstances.

In the assessment of internal stability, the shear resistance of the nails may be significant (about 5%), in contrast to the usual reinforced soil structure. This is because in a reinforced soil structure built of fill materials, it is usual to use flexible reinforcements, which can only carry tension, and thus shear is neglected.

A number of techniques for analysing the stability of nailed slopes have been proposed, the most commonly used being the two-part wedge and log-spiral methods.

C5.3 Checks for nail loading

Where the nails are designed to act in tension only, a check against rupture, bond failure in the resistant zone, and bond failure in the active zone should be undertaken. Where the nails are subjected to the combined effect of tension and shear, reference may be made to RDGC (Ref. 4), Bridle and Bar (Ref. 13) or Jewell (Ref. 9).

C6 SERVICEABILITY

The serviceability limits governing the internal stability of a soil-nailed structure are similar to those for a reinforced soil structure. Where special concrete facings are used, any excess deformation of the facing may constitute a serviceability limit state.

NOTE: An advantage of soil nailing is that extra nails may be inserted into the structure to control observed deformations (at any time following construction) that are in excess of those specified.

C7 FACINGS

The most common form of facing for a soil-nailed structure is sprayed concrete applied progressively as the soil nail wall is constructed. In permanent structures, the sprayed concrete is often applied in a two-stage process, to ensure adequate concrete cover and corrosion protection to reinforcement. The two-stage process also greatly simplifies anchor head design and tensioning. The purpose of the sprayed concrete facing is primarily to distribute the soil wall anchor head loads over the exposed face and support the soil near the face between the anchor.

A variety of precast concrete facings have also been used as wall facing.

Flexible or soft facings (e.g. geogrids, catenary cables, etc.) have also been used for soil-nailed structures where they have the advantage of allowing the slope to be vegetated. These systems are however normally only effective in ‘weathered rock’ soil-nailed structures.

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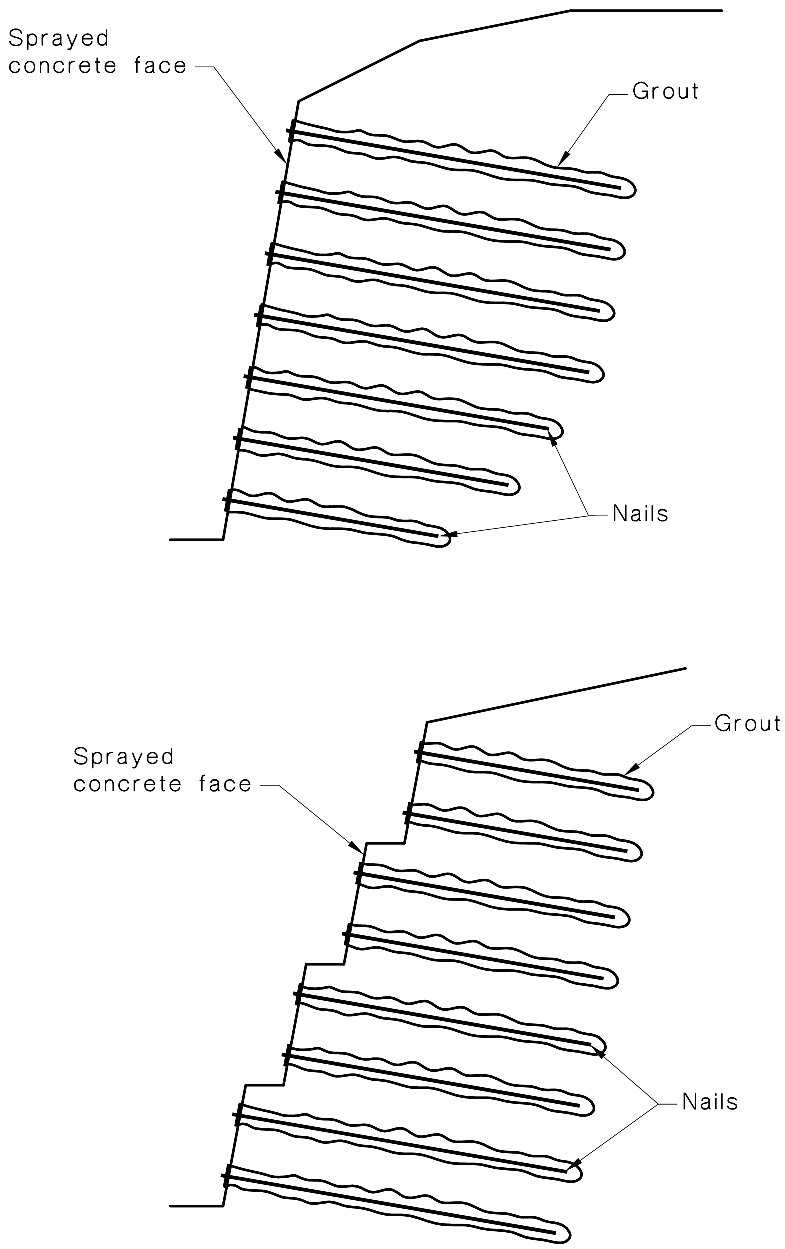
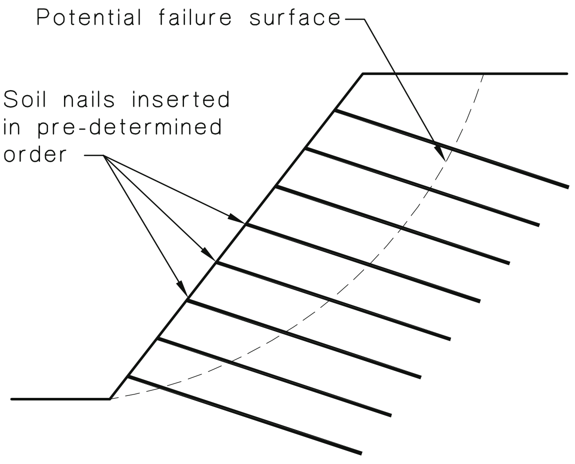
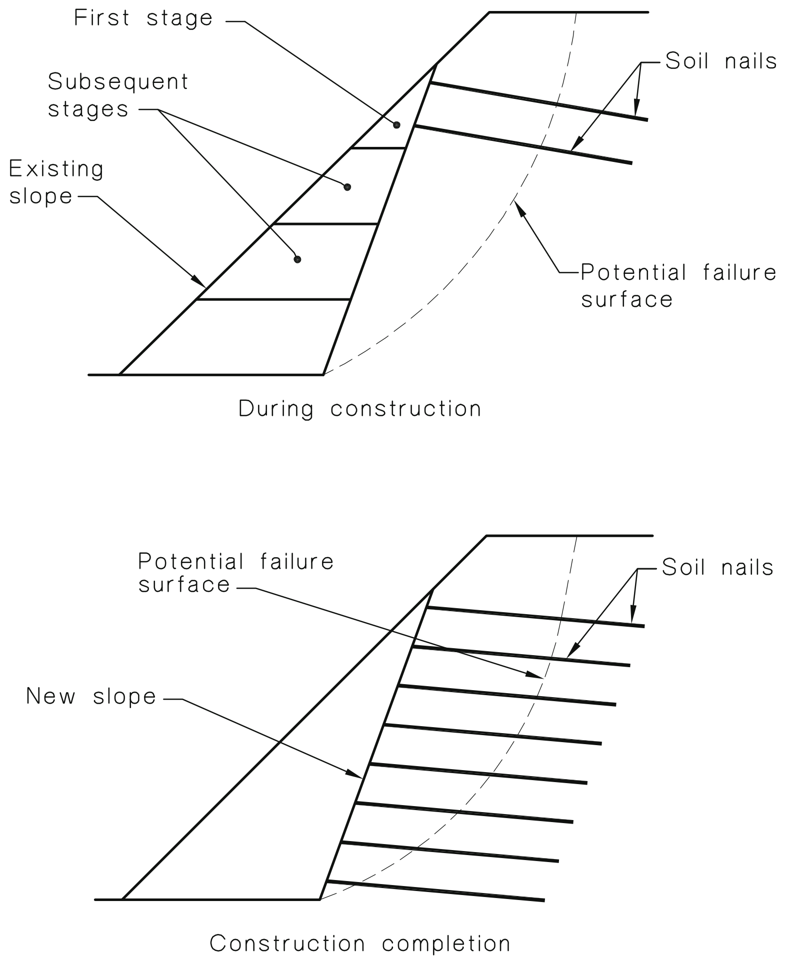


FIGURE C1 SOIL-NAILED WALLS



(a) Stabilization of existing slope



(b) New construction

FIGURE C2 APPLICATIONS OF SOIL-NAILING

APPENDIX D

SOIL AND MATERIAL PROPERTIES

(Informative)

D1 GENERAL

The design of earth-retaining structures is usually based on effective strength parameters. The material reduction factors recommended in this Standard are applied to both the effective strength parameters c' and ϕ' and, if used, the undrained shear strength c_u ($\phi_u = 0$). If constant volume or residual values (ϕ_{cv} and ϕ_r respectively) are used, a different reduction factor would be appropriate.

The selection of a representative value of a soil parameter should take into account the various matters referred to in Section 2 and determined as part of the site investigation process.

The assessment of the relevant soil parameter is often dependent upon the mechanism, or mode of deformation (or limit state) being considered for the retaining structure. For example, different representative strengths will be required for a shear failure in a fissured material, depending upon whether the shear surface is free to follow the fissures, or is constrained to intersect intact material. A range of values should also be considered if the soil parameters are likely to change during the lifetime of the retaining structure.

The soil properties may also be assessed or amplified by data from 'back analysis' of comparable retaining structures or experience with similar ground conditions.

For anchored walls and in situ reinforced soil structures, the shear resistance between the anchor/soil nail/dowel and the in situ material is a critical design parameter. Assessment of the interface shear strength depends on and has to take into account the following:

(a) Installation method.

(b) Type of grout, mix design and injection technique.

(c) In situ material properties.

NOTE: In some published literature the soil/reinforcement interface shear strength is also termed 'bond strength'.

For reinforced soil structures, tests may be required to determine the following:

(i) Soil/reinforcement interface shear strength.

(ii) Soil/reinforcement pull-out strength.

(iii) Shear strength of fill.

Particular care should be taken in the selection of a test method to correlate the sample size to the parameter being tested, and that the testing apparatus is of sufficient size to properly represent the field service condition.

D2 SOIL PARAMETERS

D2.1 Soil unit weights

Typical unit weights for a variety of soil materials are set out in Table D1 and may be used in the absence of reliable test results.

D2.2 Soil friction (ϕ) and cohesion (c)

D2.2.1 *General*

Figure D1, Tables D2, D3 and D4 provide guidance on various empirical relationships between classification and index tests for representative values of ϕ and c for both cohesive and cohesionless materials.

D2.2.2 *Cohesive soils*

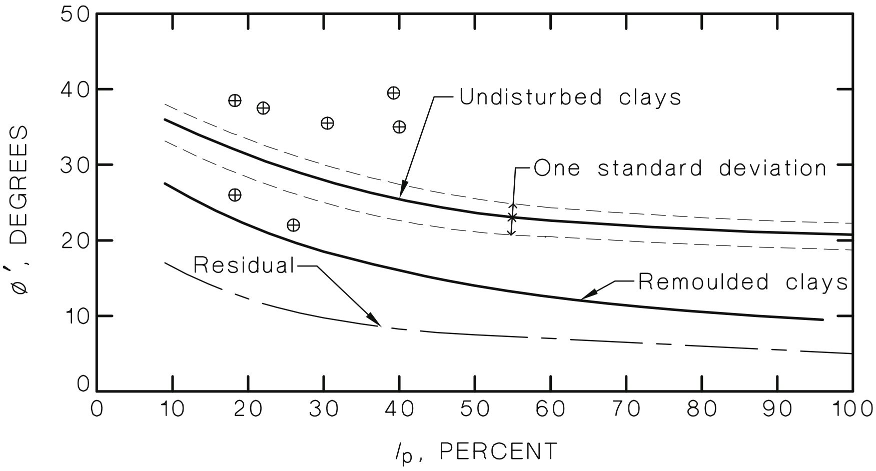
The undrained shear strength of a clay soil is not a reliable soil parameter. Different values may be recorded in triaxial compression and extension, direct shear and in pressure meter tests in situ. Further, the undrained strength of a soft clay with a small over-consolidation ratio (say, less than 3) increases when the positive pore water pressure dissipates.

In assessing the strength of soil, it is important to consider the likely moisture environment of the earth-retaining structure, as well as questions such as those dealing with clay sensitivity, strength reduction on remoulding, failure envelope and whether large strains can occur, which could result in the development of either the residual or critical state angle of shear resistance.

TABLE D1

UNIT WEIGHTS OF SOILS (AND SIMILAR MATERIALS)

Material	γ_m: moist bulk weight (kN/m^3)		γ_s: saturated bulk weight (kN/m^3)	
	Loose	Dense	Loose	Dense
A—Granular				
Gravel	16.0	18.0	20.0	21.0
Well-graded sand and gravel	19.0	21.0	21.5	23.0
Coarse or medium sand	16.5	18.5	20.0	21.5
Well-graded sand	18.0	21.0	20.5	22.5
Fine or silty sand	17.0	19.0	20.0	21.5
Rock fill	15.0	17.5	19.5	21.0
Brick hardcore	13.0	17.5	16.5	19.0
Slag fill	12.0	15.0	18.0	20.0
Ash fill	6.5	10.0	13.0	15.0
B—Cohesive				
Peat (very variable)	12.0		12.0	
Organic clay	15.0		15.0	
Soft clay	17.0		17.0	
Firm clay	18.0		18.0	
Stiff clay	19.0		19.0	
Hard clay	20.0		20.0	
Stiff or hard glacial clay	21.0		21.0	



NOTE: Approximately 80% of data falls within one standard deviation. Only a few extreme scatter values are shown (Data from several sources: Ladd et al. (1977), Bjerrum and Simons (1960), Kanja and Wolle (1977), Olsen et al. (1986).)

FIGURE D1 CORRELATION BETWEEN ϕ' AND PLASTICITY INDEX I_p FOR NORMALLY CONSOLIDATED (INCLUDING MARINE) CLAYS

D2.2.3 *Cohesionless soils*

The strength and stiffness of cohesionless soils vary with respect to density, angularity and grading of particles. An estimation for characteristic peak effective internal friction angle ϕ' can be given by—

$$\phi' = 30 + k_A + k_B + k_C \quad \text{. . . D1}$$

where the parameters k_A, k_B and k_C relate to the angularity, grading and density of the particles. Some conservative values of these parameters are set out in Table D2.

TABLE D2

ϕ' FOR SILICEOUS SANDS AND GRAVELS

Angularity (see Note 1)		(k_A) (degrees)
	Rounded	0
	Sub-angular	2
	Angular	4
Grading of soil (see Note 2 and Note 3		**(k_B) (degrees)**
	Uniform	0
	Moderate grading	2
	Well graded	4
N' (below 300 mm) (see Note 4)		**(k_C) (degrees)**
	<10	0
	20	2
	40	6
	60	9

NOTES:

1 Angularity is estimated from visual description of soil.

2 Grading may be determined from grading curve by the use of—

coefficient of uniformity = D_{60}/D_{10}

where D_{10} and D_{60} and 60% are particle sizes such that, in the sample, 10% of the material is finer

than D_{10} and D_{60} and 60% is finer than D_{60}.

Grading	*Uniformity*
Uniform	< 2
Moderate grading	2 to 6
Well graded	6

3 A step-graded soil should be treated as uniform or moderately graded soil according to the grading of the finer fraction.

4 N' from results of standard penetration test modified where necessary.

5 Intermediate values of k_A, k_B and k_C are given by interpolation.

D2.2.4 *Rock materials*

The engineering properties of rock, relative to the design of an earth-retaining structure, is usually controlled by the extent and orientation of bedding planes and joints within the rock mass, together with any water pressures on discontinuity planes.

Whilst site investigation processes should normally determine the appropriate friction angle to be adopted, some conservative values of ϕ' are set out in Table D3.

D2.2.5 *Fill materials*

Fills have been classified in this Standard as being Class I or Class II controlled fill, uncontrolled fill and other fill. Whilst a wide range of fills may be used as backfill behind retaining walls, selected cohesionless granular fill placed in a controlled manner behind the wall is usually the most desirable.

‘Other fill’ should not be used as fill under, within or behind retaining structures unless specific investigations show that it is suitable; as such, no recommendations for ‘other fill’ are made in the Tables in this Standard.

Where plastic cohesive fills are used behind walls, the same can cause problems during both the design and the construction phases; this is because of aspects such as shrink/swell of the clays, softening of the clays with saturation and so on. Consequently, where clays of higher than low-to-moderate plasticity (viz. a PI of greater than 20) are used, then special attention should be paid to items such as shrink/swell and softening.

In the case of reinforced soil structures, it is usually necessary to be very specific about the fill material to be used in the reinforced soil block. Some suggestions on the various types of fill for reinforced soil structures are provided in subsequent Paragraphs of this Appendix.

TABLE D3

ϕ' FOR ROCK

Stratum	ϕ' (degrees)
Chalk	35
Weathered granite	33
Fresh basalt	37
Weak sandstone	42
Weak siltstone	35
Weak mudstone	28

NOTES:

1 The presence of a preferred orientation of joints, bedding or cleavage in a direction near that of a possible failure plane may require a reduction in the above values, especially if the discontinuities are filled with weaker materials.

2 Chalk is defined here as unweathered medium to hard, rubbly to blocky chalk.

3 Weathered basalt may have very low values of ϕ'.

D3 TYPICAL SOILS

Whilst the variety of soils encountered in practice is very large, the usual range of soils can be classified as set out in Table D4.

TABLE D4

SOIL CLASSIFICATION

Soil group	Typical soils in group	Soil parameters	
		c' (kPa)	ϕ' (degrees)
Poor	Soft and firm clay of medium to high plasticity, silty clays, loose variable clayey fill, loose sandy silts	0 to 5	17 to 25
Average	Stiff sandy clays, gravelly clays, compact clayey sands and sandy silts, compacted clay fill (Class II)	0 to 10	26 to 32
Good	Gravelly sands, compacted sands, controlled crushed sandstone and gravel fills (Class I), dense well-graded sands	0 to 5	32 to 37
Very good	Weak weathered rock, controlled fills (Class I) of roadbase, gravel and recycled concrete	0 to 25	36 to 43

D4 FILL MATERIALS FOR REINFORCED SOIL STRUCTURES

D4.1 General

Fill materials for reinforced soil structures should be free from any organic, plastic, metal, rubber or any other synthetic material, inorganic contaminants, dangerous or toxic material or material susceptible to combustion. Fill materials should consist of naturally occurring or processed materials that are capable of being compacted in accordance with the specified requirements, to form a stable mass of fill.

Where reinforced soil structures are used to retain highly plastic or reactive soil materials, then the impact of the shrink/swell movements of such soils on the reinforced soil structures, embedded services and associated structures should be considered in the design.

Fill materials should have values of shear strength and soil/reinforcement friction consistent with the design parameters. The reinforced soils design should specify physical, chemical and electrochemical properties.

Select or other fill materials may be used for reinforced soil structures as defined in Paragraphs D4.2.2 and D4.2.3.

D4.2 Select fill for reinforced soil structures

D4.2.1 *General*

Select fill material for reinforced soil structures should be a frictional, non-aggressive material of either natural or industrial origin, free of organic material, meeting the physical, chemical and electrochemical criteria defined in Paragraph D4.2.

The use of select fill material as defined in Paragraphs D4.2.2 and D4.2.3 will allow appropriate design parameters to be adopted as defined in Paragraphs D4.2.4 and D4.2.5, provided that it is constructed as defined in Paragraph D4.2.6. This material will result in a sound, durable structure whose design and construction performance will be predictable over a wide range of construction and service conditions.

D4.2.2 *Physical properties*

Select frictional fill material should be defined based on the physical (size) properties of the fill material in place (after compaction) as follows:

(a) Grading as determined by AS 1289.3.6.3 should be within the limits defined in Table D5 below:

(b) If more than 15% of the material passes the 75 μm sieve, then not more than 10% of the material should have a diameter less than 20 μm.

(c) Coefficient of uniformity should be greater than 2.

(d) Plasticity index should be less than 12.

D4.2.3 *Chemical and electrochemical properties*

Select non-aggressive fill material may be defined based on the chemical and electrochemical properties described in Table D6.

TABLE D5

PARTICLE GRADING

A2

Particle size	Percent passing
150 mm	100
9.5 mm	25–100
2.36 mm	15–100
600 μm	10–100
75 μm	0–15

NOTE: For geosynthetic reinforcement, use of larger particle sizes may require a decrease in the damage factor.

TABLE D6

BASIC SOIL PROPERTIES FOR NON-AGGRESSIVE SELECTED BACKFILL

Classification	S1	S2
Resistivity (ohm, cm)	>5000	>1000
pH (min.)	>5	>5
pH (polyester, galvanized steel only)	<10	<10
Chlorides (mg/kg)	—	<200
Sulfates (mg/kg)	—	<1000

D4.2.4 *Design parameters, physical*

The soil shear strength parameters (required by Section 5), for select fill material as defined in this Appendix, should be taken as follows:

(a) Characteristic friction angle .... $\phi' = 36°$.

(b) Characteristic drained cohesion .... $c' = 0$.

D4.2.5 *Design parameters, chemical and electrochemical*

The corrosion allowances defined in Table D7 should be adopted to assess the combined reduction factors for strength and thickness (required by Section 5), for steel reinforcements buried in selected fill material.

TABLE D7

CORROSION ALLOWANCES (mm)

Design life (years)		5	30	100
Land based	Plain	0.5	1.5	4.0
	Galvanized	0	0.5	1.5
Freshwater	Plain	0.5	2.0	5.0
	Galvanized	0	1.0	2.0
Marine	Plain	1.0	3.0	7.0

NOTE: The corrosion allowance is the total effective loss of thickness in a section from which is determined the residual tensile strength.

D4.2.6 *Construction*

The select fill in the reinforced soil structure should be placed and compacted in layers of thickness appropriate to the compaction methods to be used and so that each layer of fill is completed at the connection to the facing panel.

The select fill placement should closely follow the erection of each course of facing panels and should be placed and spread in a direction parallel to the face of the structure. Plant with an equivalent static weight of more than 10 kN should be excluded from a zone extending to 1.5 m from the facing panels of the structure. Compaction of the select fill in this zone should be carried out using hand-operated equipment with an equivalent static weight of less than 10 kN, to achieve the equivalent density to that achieved in the main body of the reinforced soil structure.

The select fill should meet the requirements of a controlled fill (Class I) and compacted to provide a uniform density over the full width of the reinforced fill structure.

A minimum of 1 in situ density test to verify compaction should be carried out per layer, as follows, except in the case of structure classification 3:

(a) For plan areas of filling less than 1000 m^2, one test every 200 m^3.

(b) For plan areas of filling greater than 1000 m^2, one test every 500 m^3.

D4.3 Other fill for reinforced soil structures

D4.3.1 *General*

Fill for reinforced soil structures may be outside the criteria for select fill defined in Paragraph D4.2 provided that the design and construction of the structure takes into account the performance characteristics of the material, both in the short and long term, and appropriate controls are provided.

The use of highly plastic or expansive clays is not recommended for reinforced soil structures, because of the potential for movement due to changes in moisture content.

D4.3.2 *Physical properties*

The maximum particle size should be limited by the needs of the earthworks placement and compaction. As a guide, the maximum particle size should be limited to two-thirds the compacted layer thickness and should be consistent with the design criteria used to take into account construction damage.

The minimum particle size should be limited by the need to control plasticity and to achieve consistent shear strength and frictional resistance with the reinforcement under construction and in-service conditions. A minimum particle size limit of not more than 20% smaller than 20 μm is recommended, with up to 40% smaller than 20 μm being possible, subject to detailed testing and performance evaluation.

D4.3.3 *Chemical and electrochemical properties*

The chemical and electrochemical properties of the fill should be appropriate to the material and the service life of the structure, based on relevant test data.

D4.3.4 *Design parameters, physical*

The design soil shear strength and the friction coefficient should be assessed based on the soil fill and the reinforcement material to be used. Fill that exhibits a friction angle (ϕ) less than 24° should not be used, and a maximum cohesion value (c) of 5 kPa should be used in the calculation.

D4.3.5 *Design parameters, chemical and electrochemical*

Material reduction factors should be based on appropriate test data for the soil and environmental conditions expected in the design.

D4.3.6 *Construction*

The placement and compaction of fill in a reinforced soil structure should meet the recommendations of Paragraph D4.2.6.

The fill should be compacted to provide a uniform density over the full width of the reinforced soil structure. The minimum density should be consistent with the design of the soil and the reinforcement, and strength and performance criteria of the structure.

APPENDIX E

DESIGN MODELS AND METHODS

(Informative)

E1 DESIGN MODELS

There are various methods of calculation of earth pressures on retaining walls and some methods of calculation are only appropriate in very particular circumstances (for example, the Rankine theory is only applicable to vertical walls). Further, the effect of the 'restraint' conditions on the wall can vary enormously the distribution of the earth pressures behind a wall.

Therefore, it is very important to select—

(a) the appropriate method analysis; and

(b) the wall restraint conditions.

A guide as to the applicable earth pressure theories is given in Table E1, whilst the various restraint conditions are illustrated in Figures E1 to E5.

In addition, as an individual earth pressure theory may only inadequately describe the likely forces on the wall, the designer should consider the degree to which the design model is applicable to a particular site and include an appropriate 'partial factor', if required.

As the analysis of structures that slope backwards at an angle that approximates the typical 'Coulomb Wedge' (see Figures E3 and E4) requires a consideration of the overall stability of the slope using slope stability analysis methods, this Standard does not provide the recommendations or requirements for the design of this form of structure, which is usually termed a 'revetment structure' (see Figure 1.1).

Where retaining walls using 'embedded piles' are adopted at a site, the design of such structures should be carried out as suggested in AS 2159 for laterally loaded piles.

E2 DESIGN METHODS

E2.1 Safety in design

According to this Standard, safety is incorporated in the design process by the following:

(a) Using conservative soil properties in the analysis of stability, deformation, seepage or other ground engineering problems. This Standard prescribes or recommends factors by which characteristic material properties are multiplied in order to lead to a safe design (Section 5).

(b) Factoring up loads where they contribute to ground failure or excessive deformation, and factoring down loads that resist failure or reduce deformations (Section 4).

(c) The designer should be aware that additional safety in design may result from—

   (i) performing laboratory or field tests that tend to underestimate strength or overestimate deformation of soils;

   (ii) sampling and testing soils with a bias towards finding the most unfavourable result;

   (iii) using methods of analysis that are known to give conservative results; and

   (iv) using empirical correlations that tend to err on the safe side.

Conversely, safety may be reduced where the above techniques are deemed to yield non-conservative results.

NOTE: The following alternative design approaches to retaining walls may be used, provided the same design considerations and performance criteria as outlined in this Standard are satisfied:

(a) For walls other than reinforced soil walls, a global (lumped) geotechnical resistance factor may be used, rather than partial material design factors. No guidance is given in this Standard for the choice of global factors.

(b) A safe design of conventional retaining structures can also be achieved by analysing limit equilibrium conditions using the worst credible soil parameters. A factor of safety just exceeding 1 would be sufficient to prevent failure. However, if the chosen safety factor is also intended to limit displacements to a tolerable maximum, the lowest credible soil strength will need to be further reduced by dividing it by a partial factor. This approach is referred to as the Direct Assessment (Worst Credible Scenario) method. No guidance for this approach is given in this Standard.

E2.2 Representative material properties

E2.2.1 *General*

In this Standard, representative material properties are called characteristic values. The meaning of characteristic value may vary depending on the particular material involved and conventions in the relevant industry.

Material design factors (Section 5) are thus applied to characteristic values (refer Clause 1.4.1.4).

E2.2.2 *Soil shear strength parameters*

The Mohr-Coulomb failure criteria contains two parameters, c' and ϕ' commonly referred to as the cohesion and the friction angle respectively, regardless of their true physical interpretation. Different sets of strength parameters are defined depending on loading and drainage conditions or the stress-strain characteristics as follows:

(a) Peak values are strength parameters determined from the highest strength value recorded during the test. Peak values are traditionally used in the analysis of bearing capacity and the determination of earth pressures.

(b) Effective strength parameters c' and ϕ' as obtained from a drained shear test or an undrained shear test with pore pressure measurements. These parameters are used for the analysis of free-draining granular soils and the long-term stability of clays.

(c) Undrained strength parameters c_u and ϕ_u as obtained from an undrained shear test. These parameters are used for the analysis of short-term stability, or stability under sudden loading of clays.

(d) Ultimate, constant volume or critical values (ϕ_{cr} or ϕ_{cv}) are derived from measurements where the sheared specimen has reached constant volume conditions (usually at a strain of say 10%). These values are used in analyses based on the concept of critical state soil mechanics.

(e) Residual values (ϕ_r) are determined at very large strains (say 100%). They may be relevant for the analysis of the global stability of a wall in a slope with a history of instability.

For any analysis involving soil shear strength, an appropriate set of shear strength parameters has to be chosen.

For retaining structures with controlled backfill, analysis with effective strength parameters is usually critical. Partial factors given in Section 5 are applied to strength components based on effective strength parameters.

For retaining structures supporting excavations in saturated clay and possible for other soil or loading conditions, undrained analysis may be more critical than an analysis using effective strength parameters. For some problems, it may be appropriate to analyse both, undrained and drained (effective stress) conditions. Undrained strength of clays is referred to in Paragraph D2.2.2 of Appendix D.

E2.3 Notation

For the purpose of this Appendix, the following notation applies:

c = characteristic cohesion of a soil, which is a parameter in the Mohr-Coulomb failure criteria, also referred to as the cohesion intercept

c' = characteristic effective cohesion of a soil

c_u = characteristic undrained cohesion of a soil

A1 c^* = design cohesion of a soil

ϕ = characteristic angle of shearing resistance of a soil, which is a parameter in the Mohr-Coulomb failure criteria, also referred to as the internal friction angle or simply friction angle

ϕ' = characteristic effective internal friction angle of a soil

ϕ_u = characteristic undrained internal friction angle resistance of a soil

ϕ_{cv} = critical or constant-volume internal friction angle of a soil under effective stress conditions

ϕ_r = residual internal friction angle of a soil under effective stress conditions

ϕ^* = design internal friction angle of a soil

TABLE E1

DESIGN MODEL AND APPLICABILITY OF EARTH PRESSURE THEORIES

A1

Method of solution	Geometric complications			Basic problem							Loading complications			
	Irregular ground	Sloping ground	Sloping wall	Vertical wall	Horizontal ground	ϕ	δ	c	c–ϕ	c_W	Surcharge	Static water table	Flowing water	Trial and error
Rankine (see Figure E1)	—	*	—	Y	Y	Y	*	—	—	—	†	Y	—	—
Rankine-Bell (see Figure E2)	—	—	—	Y	Y	Y	—	Y	Y	—	†	Y	—	—
‘Coulomb’ analytical (see Figure E3)	—	Y	Y	Y	Y	Y	Y		—	—	—	‡	—	—
General wedge analysis (see Figure E4)	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y

* O.K. for special case of $\delta = \beta$

† Can handle (uniform areal) surcharge

‡ O.K. for horizontal ground and static water at ground surface

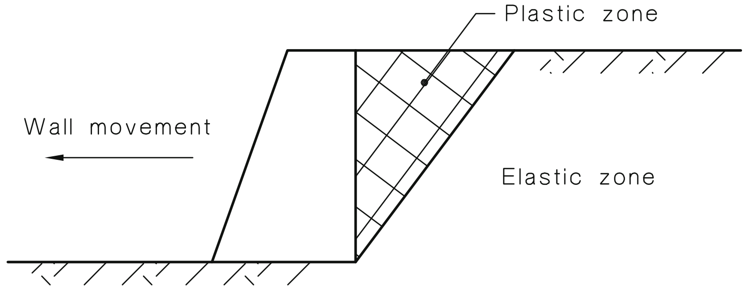
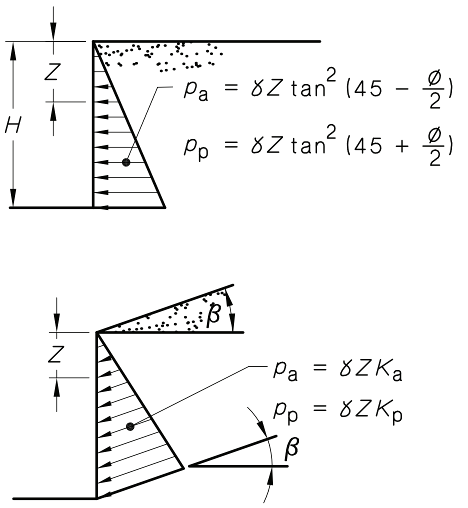


FIGURE E1 RANKINE DESIGN MODEL

A1

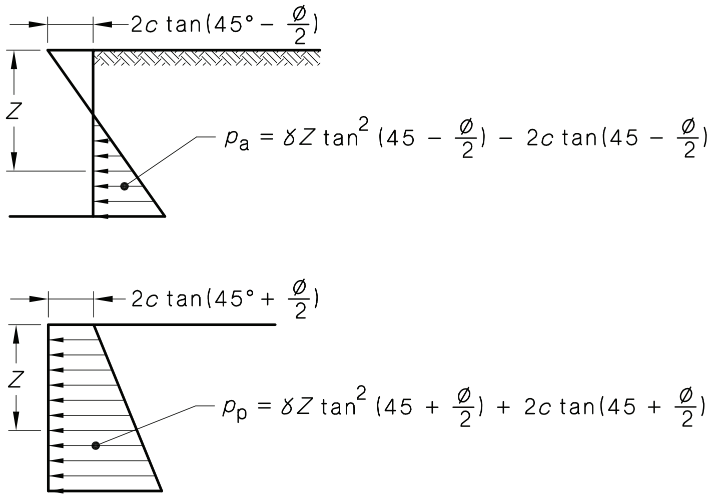


$$K_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$K_p = \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$K_a = \frac{1}{K_p}$$

(a) Granular soil



(b) Cohesive soil

FIGURE E2 RANKINE-BELL DESIGN MODEL

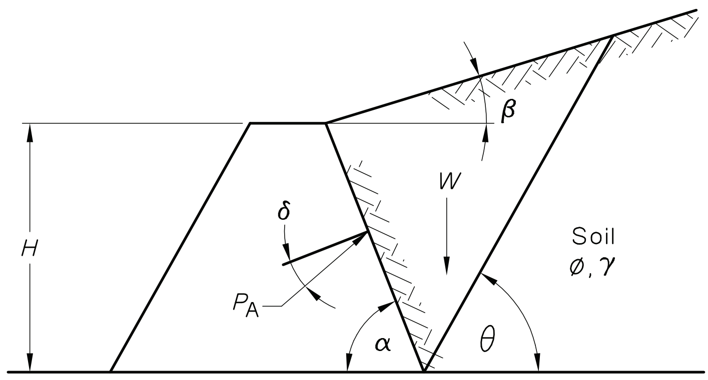
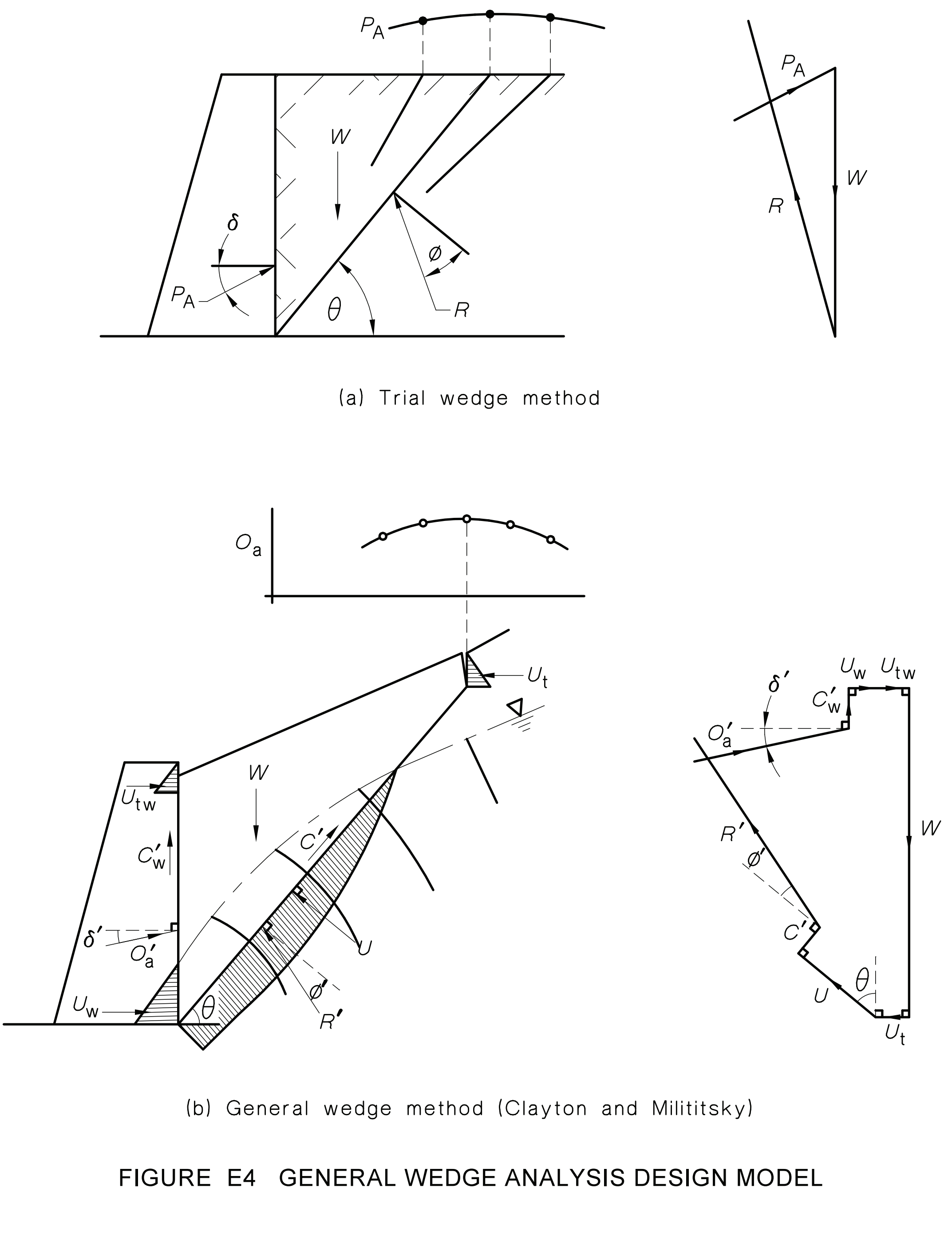


FIGURE E3 COULOMB ANALYTICAL DESIGN MODEL



(b) General wedge method (Clayton and Milititsky)

FIGURE E4 GENERAL WEDGE ANALYSIS DESIGN MODEL

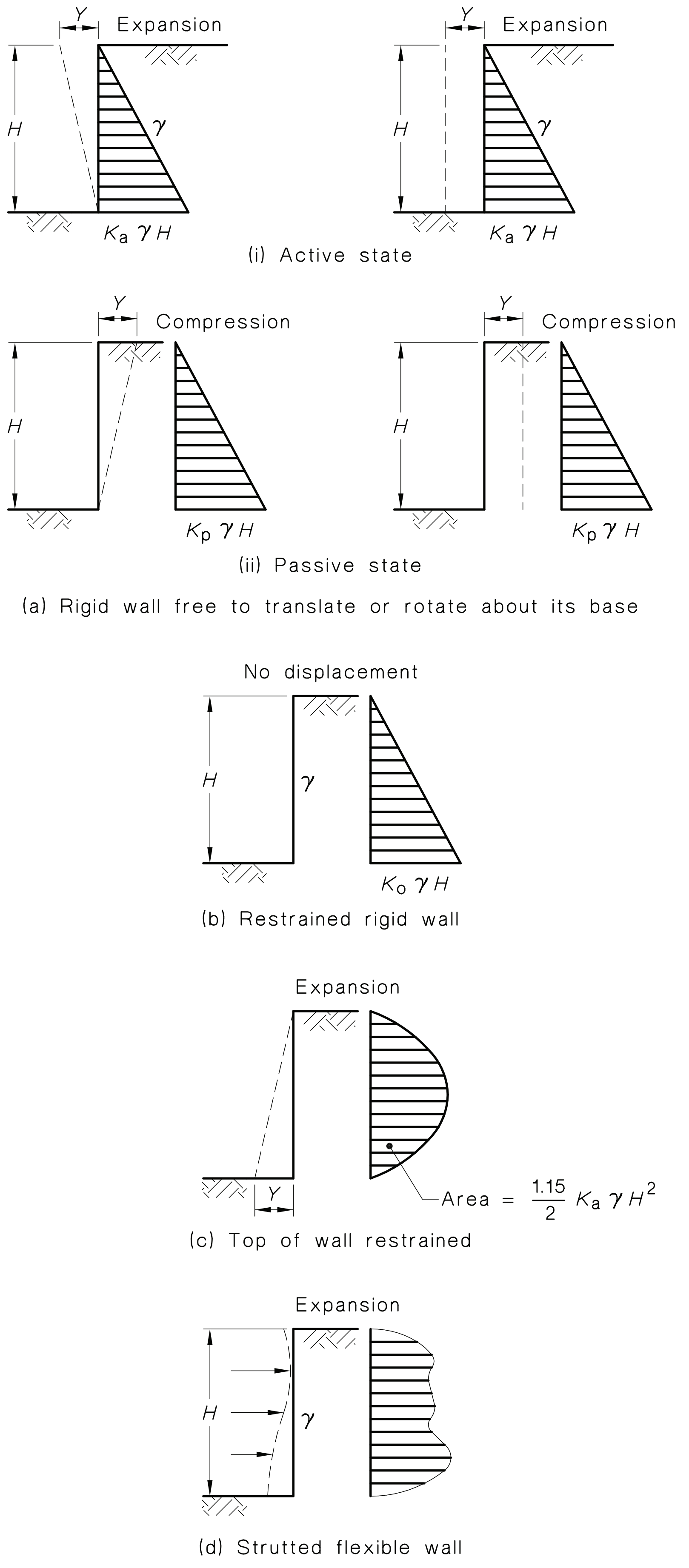


FIGURE E5 EFFECT OF RESTRAINT CONDITIONS ON EARTH PRESSURES

APPENDIX F
MATERIAL SELECTION AND DURABILITY
(Informative)

F1 GENERAL

The selection of materials for earth-retaining structures should take into account the characteristics of the material, its durability in the environment and its performance with time, as defined in Section 3.

F2 REINFORCED SOIL STRUCTURES

F2.1 General

The selection of soil-reinforcing products should take into account the following factors:

(a) Strength characteristics.

(b) Strain behaviour, at ultimate and serviceability limits states.

(c) Soil/structure/reinforcement interaction behaviour.

(d) Handling and construction behaviour.

(e) Durability.

F2.2 Durability

The principal factors that are likely to affect the durability of soil reinforcement are as follows:

(a) *General factors*:

  (i) Loading (metallic and polymeric).

  (ii) Water (metallic and polymeric).

  (iii) Damage (metallic and polymeric).

  (iv) UV exposure (polymeric).

  (v) Temperature (polymeric).

(b) *Specific factors*:

  (i) Bacterial (metallic and polymeric).

  (ii) Chemical (metallic and polymeric).

  (iii) Electrochemical (metallic).

  (iv) Corrosive fluids (metallic).

  (v) Aggressive fluids (polymeric).

A check list of factors that should be considered for the selection of soil-reinforcing products is given in Table F1.

F3 REINFORCED SOIL AND RETAINING STRUCTURES

The selection of materials for use in all earth-retaining structures should take into account the factors identified in Table F2.

TABLE F1

CHECK LIST FOR INVESTIGATIONS OF SOIL-REINFORCEMENT PRODUCTS

(a) *Physical and mechanical properties*:

(i) Full description of the material and relevant technical literature.

(ii) Short- and long-term data: ultimate and permissible values of tensile strength; stress strain curves; value of Young's modulus; creep; pliability, ductability; fatigue (dynamic and static) however induced, e.g. structural, thermal or tidal loading; coefficient of friction with fill (with no protective coating)

(b) *Durability*:

The effects of the following agencies on physical and mechanical properties to be investigated:

Extreme pH values; chlorides; sulfates; ozone; hydrocarbons and other chemicals common on transport highways; water; ultraviolet and infra-red rays (including short-term exposure prior to installation); bacteria and other animal and vegetable life forms; fire; vandalism

(c) *Performance during installation and use*:

(i)	Handling, storage	during deposition, spreading and
(ii)	Damage	compacting of fill in structure and
(iii)	Extension and movement	under test loading
(iv)	Effects on facing	

These topics to be checked by site trial(s)

(d) The effect of the combination of load, chemical environment and construction damage, i.e. combination of Items (a), (b) and (c)

(e) *Constructional details*

Connections: effect on Items (a)(ii) and (b)

(f) Quality control procedures and problems associated with production processes

(g) Preconstruction exposure to an aggressive environment, e.g. long-term effect of short-term UV exposure on synthetics, or ozone and marine spray on metallics

(h) The effects of static fatigue (creep) or dynamic fatigue however induced; e.g. structural loading, thermal loading, creep ratcheting

(i) The effects of blasts, fires or vandalism on the behaviour of the structure

TABLE F2

FACTORS AFFECTING THE DURABILITY AND PERFORMANCE OF EARTH-RETAINING STRUCTURES

Retaining wall elements	Factors affecting durability
Concrete/masonry	Loading Weathering Temperature (including freeze/thaw) Permeability to air/moisture Chloride attachment (marine environment) Composition of soil (acids/sulfates) Concrete quality (mix design, compaction and the like) Concrete strength (cement content) Corrosive liquids and gases Sulfate attack (seawater, ground water, industrial effluent)
Steel reinforcement (in concrete structures)	Loading Concrete quality (mix design, compaction and the like) Concrete permeability Aggressive salts (chlorides and sulfates) Concrete cover to steel Climate (temperature, humidity, rainfall and the like)
Timber	Loading Termites (white ant) Borers Moisture (causing swelling/splitting) Weathering (sunlight and rain causing 'silvering') Decay or fungal deterioration (also called 'rot') Fire Composition of timber (namely, the presence of resins that affect durability)
Metallic soil reinforcement	Loading Water Damage (especially to galvanized/sacrificial coatings) Temperature Electrochemical properties of soil Composition of soil (acids, chlorides, sulfates, sulfides, organic content) Chloride attack (marine environment) Sulfate attack (seawater, ground water, industrial effluent) Corrosive liquids and gases
High density polyethylene soil reinforcement	Loading Damage UV exposure (pre-use) Temperature Aromatic/aliphatic hydrocarbons Halogenated hydrocarbons
Polyester soil reinforcement	Loading Temperature Water (hydrolysis) Water absorption properties of the polyester Permeability and durability of type of outer protective coating (usually polyethylene) Damage (especially of outer protective coating) UV exposure (pre-use)
Polypropylene soil reinforcement	Loading Damage UV exposure (pre-use) Temperature Aromatic/aliphatic hydrocarbons Halogenated hydrocarbons

APPENDIX G

DRAINAGE OF EARTH-RETAINING STRUCTURES

(Informative)

G1 GENERAL

As drainage is perhaps the most important consideration in the design of earth-retaining structures, it is very important that the drainage systems are carefully designed and specified with sufficient detail to ensure that the system can be correctly constructed.

Water may affect a structure in the following ways:

(a) Percolation from the upper surface.

(b) Groundwater or seepage flow from the retained ground.

The drainage system needs to cater for both situations. It is also important to separate the surface drainage system from the subsoil drainage system to ensure the long life of the subsoil system. This is usually achieved by a 300 to 500 mm thick layer of compacted clay or sandy clay material placed over the subsoil system to act as a seal. Also, prevention of ponding of surface water behind the top of the wall is important.

Whilst this Appendix includes details of various methods of providing subsoil drainage relief behind retaining structures, it is very important that the designer of the wall consider carefully the operation of the subsoil and surface drainage system and produce a 'site specific' design.

G2 DRAINAGE MATERIALS

G2.1 Backfill materials

The backfill material for an earth-retaining structure needs to be carefully considered and specified. In this regard, careful attention should be paid to the permeability of the material, ease of compaction and availability. Further, 'soil filter' requirements to prevent the migration of fine particles into the more open-graded filter material should be considered.

Because of the difficulty in matching available filter materials with permeability and particle migration requirements, it is usual practice to incorporate a geotextile to separate the filter material from the in situ materials. Such geotextiles need, however, to be selected with care and specified in terms of material type, branch, permeability and function.

Subsoil drainage and filters must also be properly designed in accordance with recognized filter design criteria, to prevent piping as well as satisfying permeability requirements.

In timber or concrete crib walls, it is sometimes assumed that the infill material within the crib units acts as a sufficient drainage medium; however, experience has indicated that the usual crib infill material often lacks sufficient permeability when compacted. Further, if a sufficiently permeable crib infill is used, then there are usually problems with the material behind the crib wall; thus, both the backfill and infill material need to be carefully specified for a crib-type wall.

G2.2 Geosynthetic materials

Where geosynthetic materials are used (e.g. geotextiles and geocomposite drains) careful consideration should be given to the permeability and surface tension properties of the geotextile. This is because some materials are in fact water repellent until they are fully saturated and the surface tension effect removed. Should a geotextile material with a high surface tension (i.e. the geotextile is 'hydrophobic') be used, then low seepage flows will seek to move through the drainage system in a manner not intended by the designer. Such a situation can have very serious effects on the foundation materials on which the structure is built, as well as causing subsoil water effluxes in inappropriate locations.

Further, where the subsoil system is subject to high lateral pressures (e.g. wall in excess of 4 m), crushing of geocomposites and subsoil drainage pipes can occur if they are not designed to resist these pressures.

G3 PIPEWORK AND MAINTENANCE

G3.1 Pipework

The design drawings should clearly indicate the specific location of the various pipes required (including size, class, type and joining method) and all pipework should be provided with adequate falls to ensure free flow of the drainage pipes. In this regard attention is drawn to AS 3500 and AS 2439.

In addition, it is desirable that the design provides for—

(a) testing and cleansing of the installation (e.g. flushing points): and

(b) appropriate tolerance on the laying of pipes.

Where feasible, it is also preferable to provide weepholes in addition to any subsoil drainage system.

G3.2 Maintenance and clogging of drainage systems

In addition to the durability of the drainage material used in a particular application, the potential for clogging of the drainage system due to migration of fines, or the formation of iron oxide from the soil/ground water system, should be investigated. Experience has also indicated that some geotextile materials 'clog up' with time if a suitable sand filter (approximately 50 mm thick) is not placed between the geotextile and the in situ material. Thus, it is important to either provide for this extra filter layer, or design the geotextile on the basis that some long-term clogging takes place.

Where it is considered inevitable that some long-term clogging will take place, then the reduced effectiveness of the subsoil drainage should be included in the design calculations.

In the case of potential formation of iron oxide, special measures may also be incorporated in the design to allow for clearing of iron oxide blockage by flushing with suitable organic acids.

Whilst subsoil drainage systems are normally designed so that maintenance is not required, there may be some instance where maintenance is part of the design specification. In such cases, the requirements of Section 8 need to be carefully considered. It should be recognized, however, that in the majority of cases, the specified maintenance may not be done and, therefore, it is usually prudent to adopt a more conservative design (that is, failure or partial failure of drainage system) rather than to assume the maintenance has been carried out.

G4 TYPICAL DRAINAGE DETAILS

Typical details of reinforced soil structures (RSS) are shown in Figure G1.

The design requirements of an RSS are such that the reinforced soil zone is usually more permeable than the material behind the RSS structure and, thus, the reinforced soil mass can be effective as a drain without additional requirements, in some cases. However, before adopting this method of drainage, consideration should be given to—

(a) leaky services within the RSS structure; and

(b) ground water rise after RSS construction.

Where there is a requirement to divert seepage and run-off from the side (excavated) slope behind the RSS, the details shown in Figure G2 are more appropriate. Several methods are used to intercept and divert the seepage water away, as follows:

(i) A longitudinal subsoil pipe installed in a trench at the toe of the RSS either in front of the wall facing or immediately behind the facing.

NOTES:

1 Where the drain is laid in front of the wall facing, weepholes are normally required in selected panels.

2 Subsoil pipes should be sufficiently rigid and durable. The collected water should also be connected into the stormwater drainage system with a suitable 'non-return' protection system.

(ii) Blanket drains may be laid either under or behind the RSS in cases where medium to high water seepage conditions are anticipated. Alternatively, trench drains at regular intervals may be used along the length of the structures.

Where blanket drains are used, they are typically 200 to 300 mm thick. Such drains use a free-draining gravel material, encapsulated in a suitable geotextile.

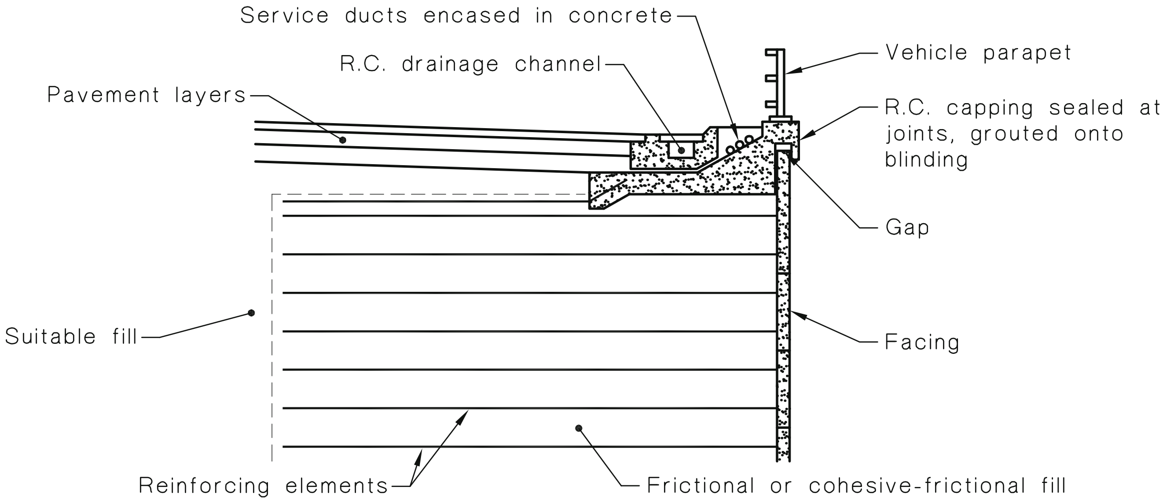
G5 TYPICAL DRAINAGE DETAILS—EARTH-RETAINING STRUCTURES

Unless the retaining wall is designed to resist both hydrostatic and earth pressures, a suitable subsoil drainage system should be incorporated into the structure design.

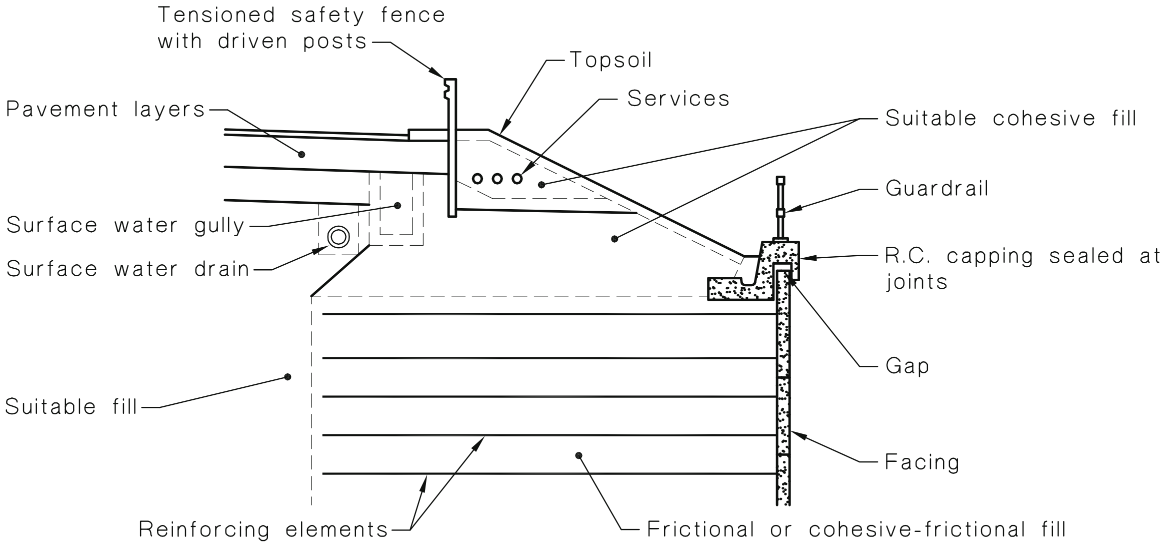
Where required, blanket drains may be laid either immediately behind the structure or at the interface between the natural ground and the retained backfill. Alternatively, geocomposite drainage products may be used in cases where the imposed overburden stresses do not damage or crush the geocomposite materials. It is also usual to adopt a combination of natural and geosynthetic materials, depending on the site conditions and construction limitations.

Figures G3 to G8 show a number of typical details for various types of wall structures; however, these details will need to be specifically modified for a particular situation.

The details do not provide the overall drainage plan required as part of the subsoil drain design. In this regard, in addition to the typical sections shown, it is necessary to provide a plan of the drainage works illustrating both the efflux points for the drainage and access and maintenance points (e.g. flushing points).



(a) Typical detail at top of full-height wall



(b) Typical detail at top of part-height wall

FIGURE G1 TYPICAL SECTION FOR REINFORCED SOIL STRUCTURES—DRAINAGE

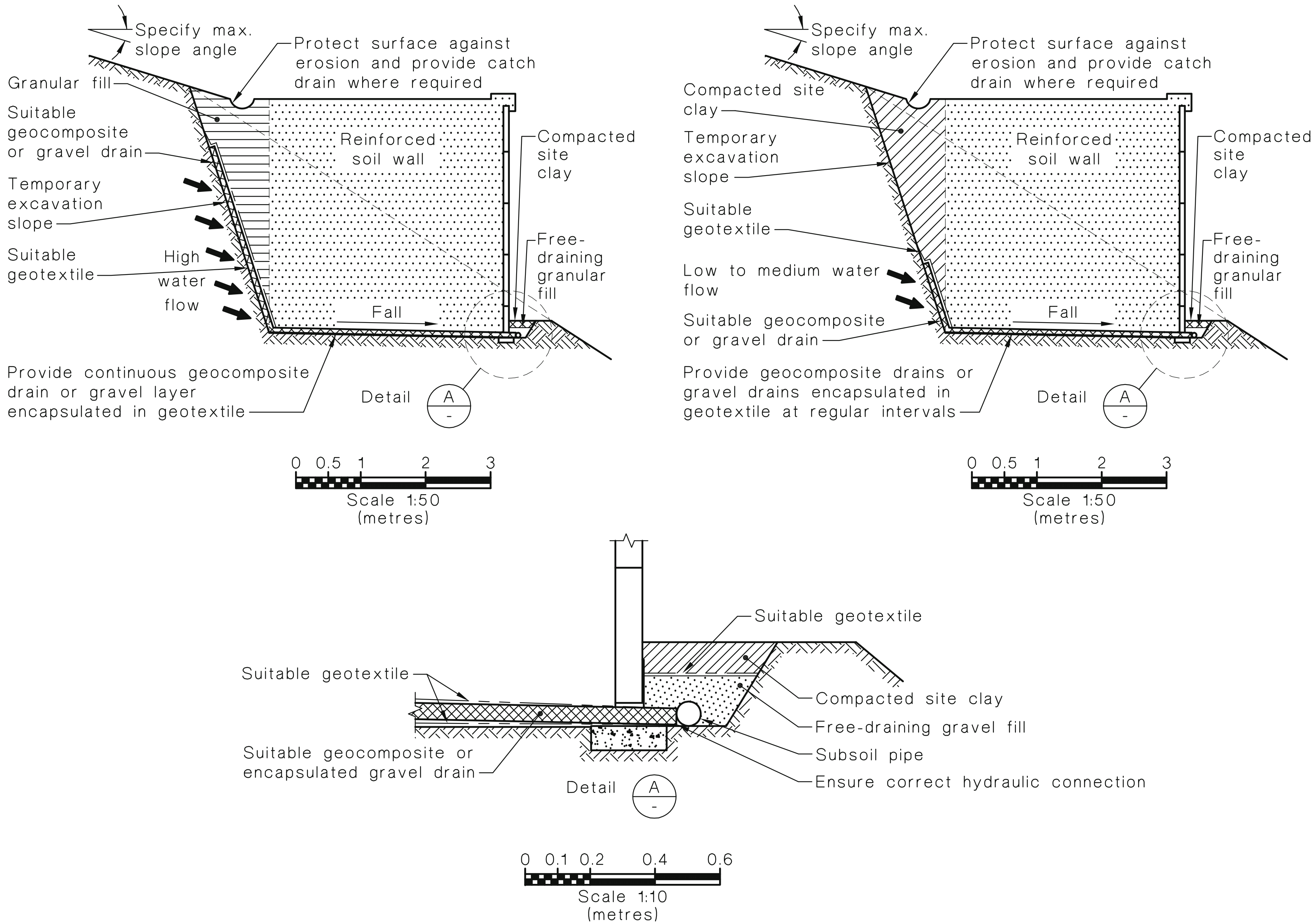


FIGURE G2 TYPICAL SUBSOIL DRAINAGE FOR REINFORED SOIL WALLS

G6 SOIL NAIL STRUCTURES

The drainage of soil nail structures is extremely important to their long-term successful performance. As such structures are usually formed by the installation of tensile elements into natural soil or weathered rock materials, drainage techniques typically involve the drilling of a number of subhorizontal boreholes into the in situ material and the providing of relief drainage at the surface of the soil nail structure.

Additionally, as there is sprayed concrete facing applied to the surface of the in situ material, regular weepholes in the sprayed concrete will be required, or a geocomposite drainage system will need to be installed between the excavated face of the in situ material and the sprayed concrete. Should a geocomposite system be used, then it is very important to ensure a proper hydraulic connection between each section of the geocomposite during the installation process.

Whilst weepholes at regular vertical and horizontal spacings are often used in conjunction with a short length of perforated pipe wrapped in geotextile through the sprayed concrete layer, such a method has been shown by experience to be aesthetically poor and, thus, the alternative of a geocomposite drain behind the sprayed concrete is usually preferred.

In addition, where significant seepage occurs within and behind the reinforced soil zone, deeper horizontal drainage systems should be used. In such cases, it is very important to ensure that the deeper drainage pipes do not act as a ‘focus’ for localized soil piping and thus possibly contribute to a long-term failure. In this regard, it is usual to wrap the deep subsoil pipe in a suitable geotextile prior to installation.

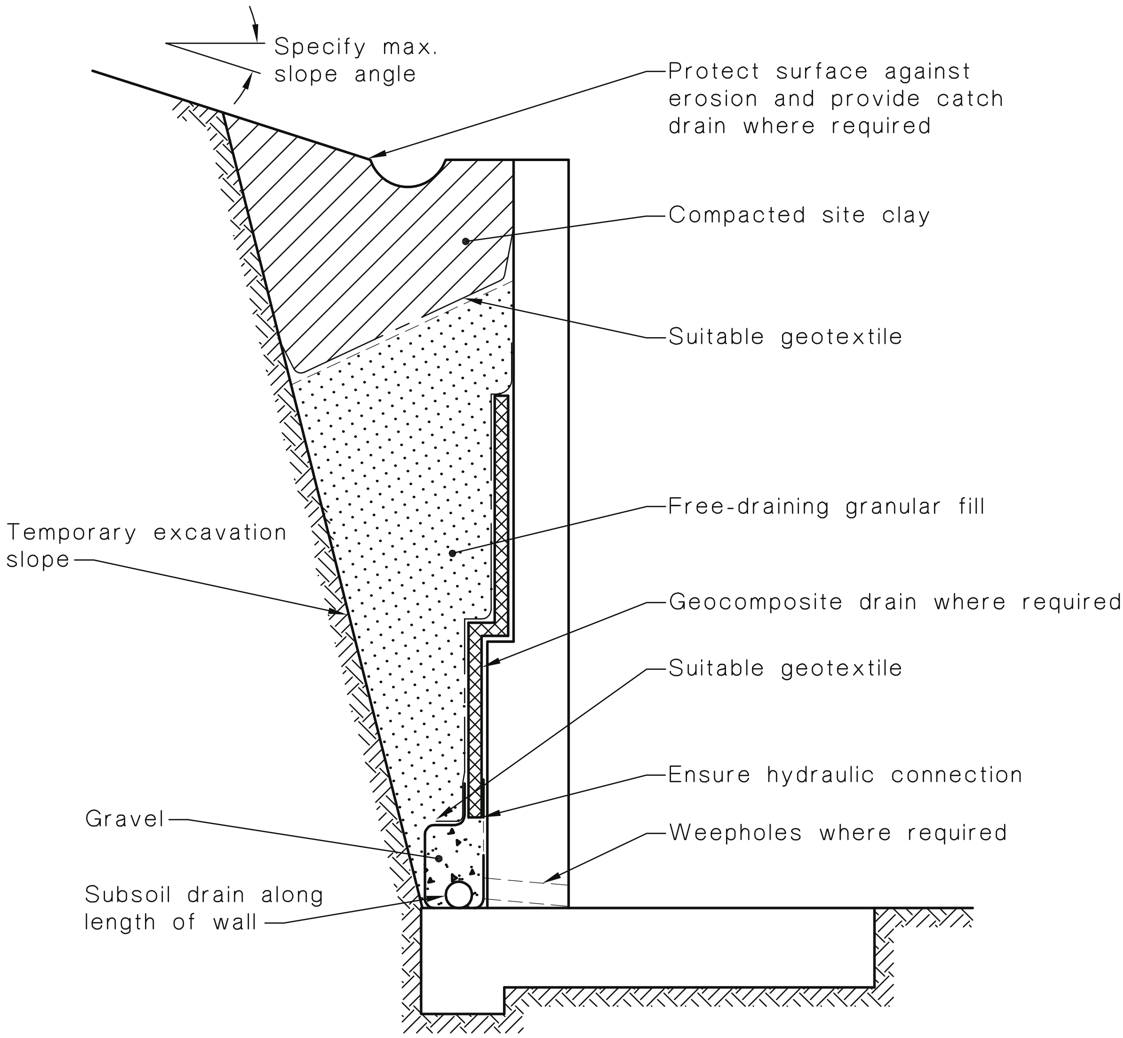


FIGURE G3 TYPICAL SECTION—CANTILEVERED WALL DRAINAGE, TYPE 1

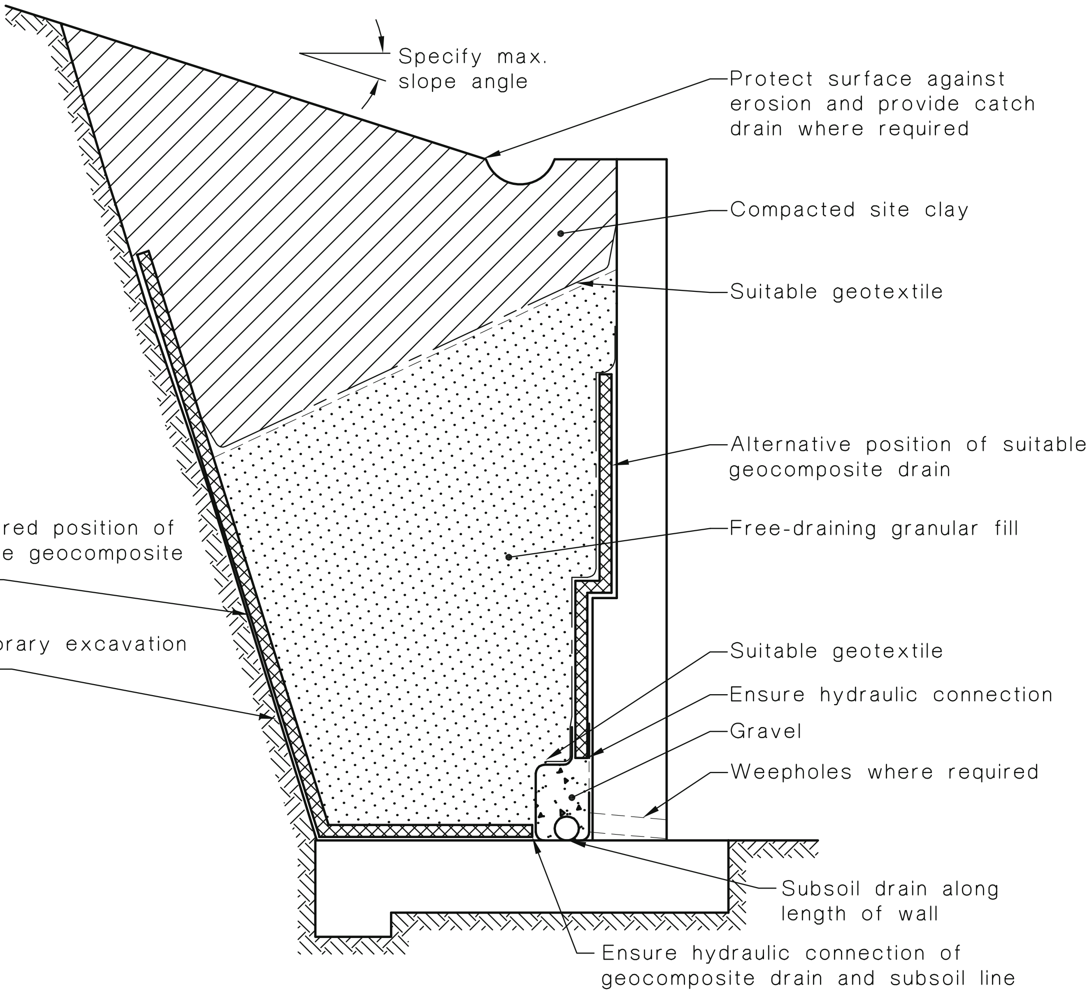


FIGURE G4 TYPICAL SECTION—CANTILEVERED WALL DRAINAGE, TYPE 2

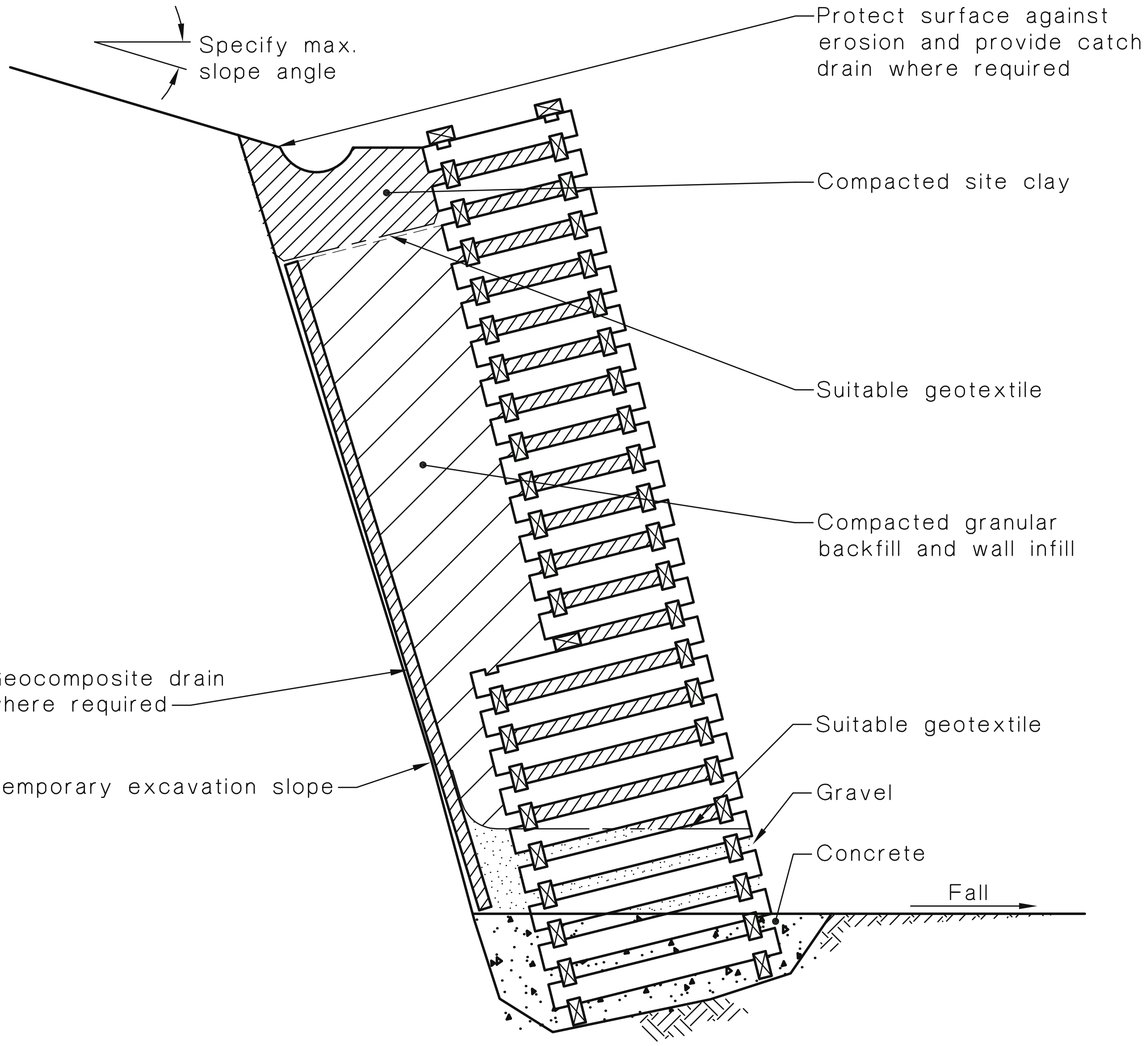


FIGURE G5 TYPICAL SECTION—CRIB WALL DRAINAGE

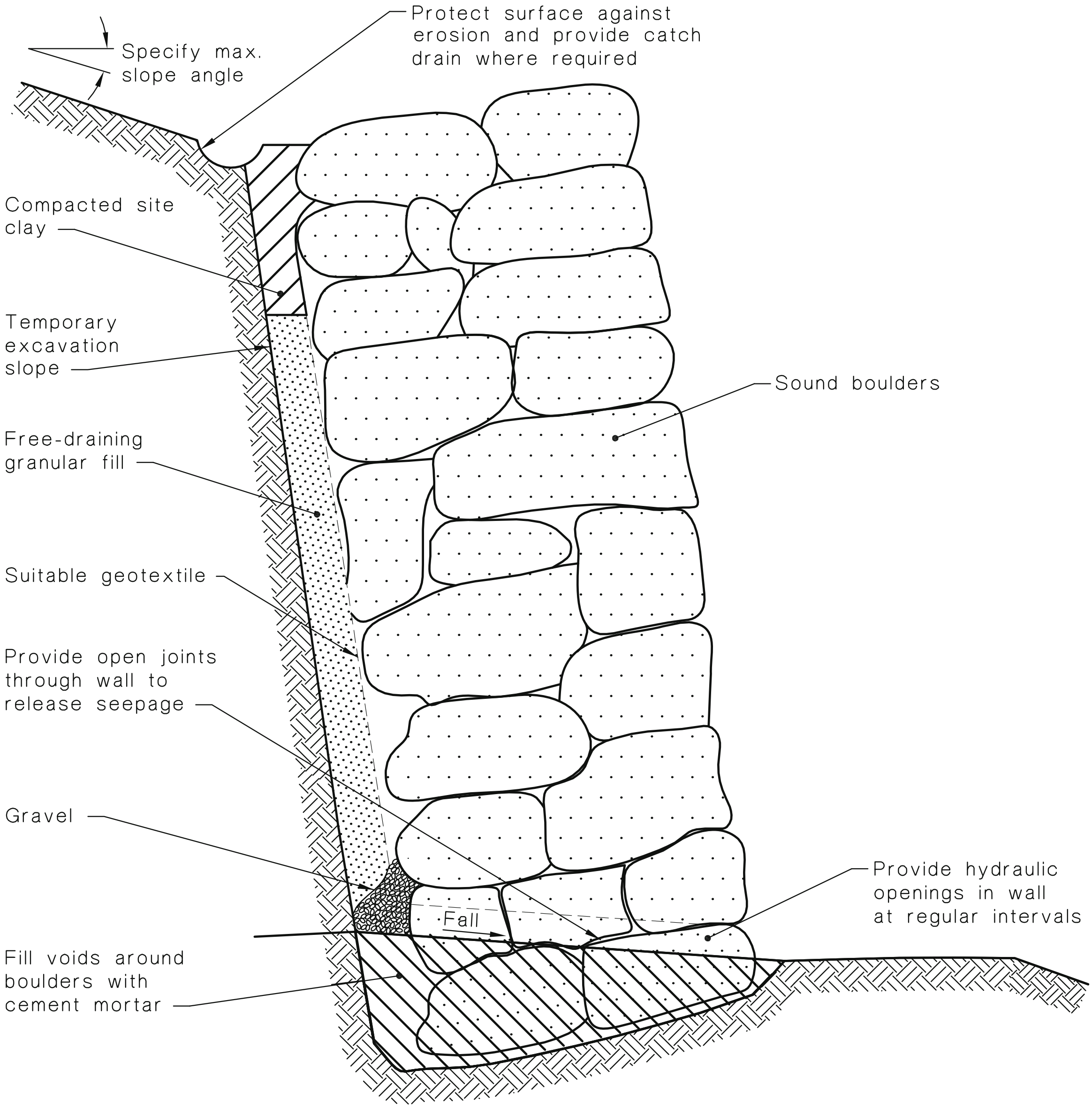


FIGURE G6 TYPICAL SECTION—GRAVITY BOULDER WALL DRAINAGE

A2

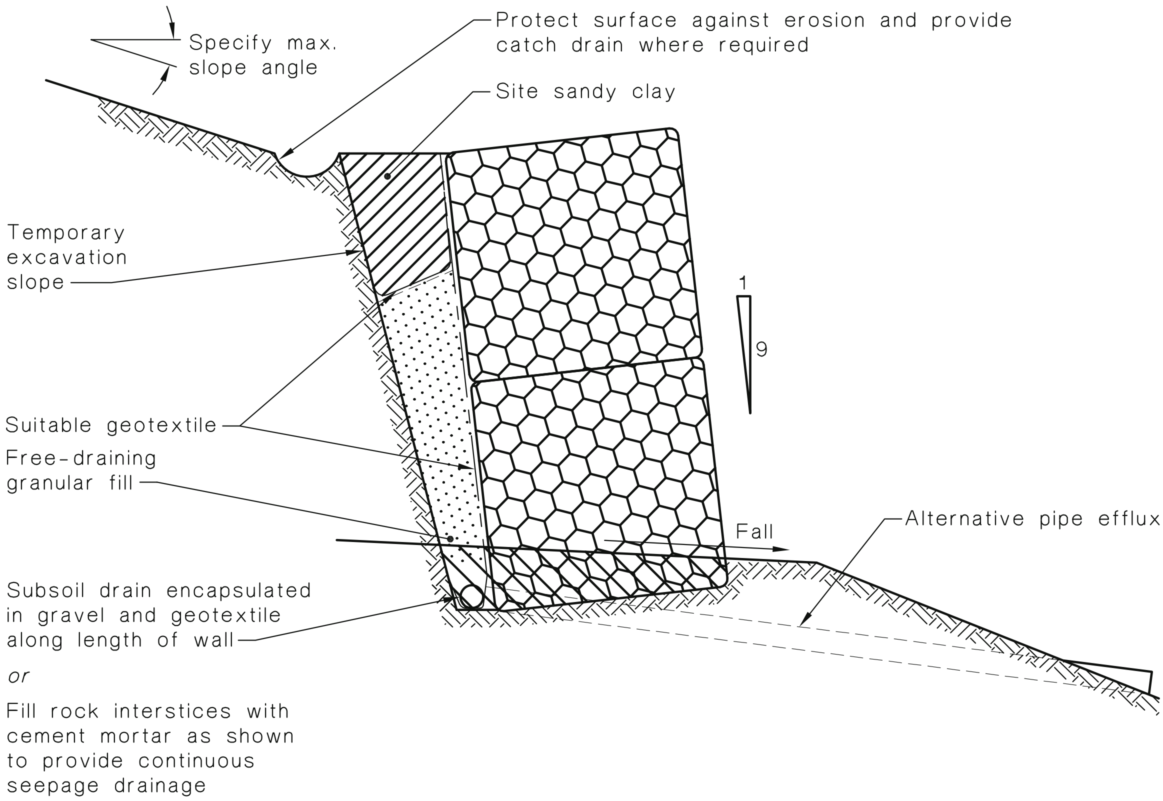


FIGURE G7 TYPICAL SECTION—GABION WALL DRAINAGE

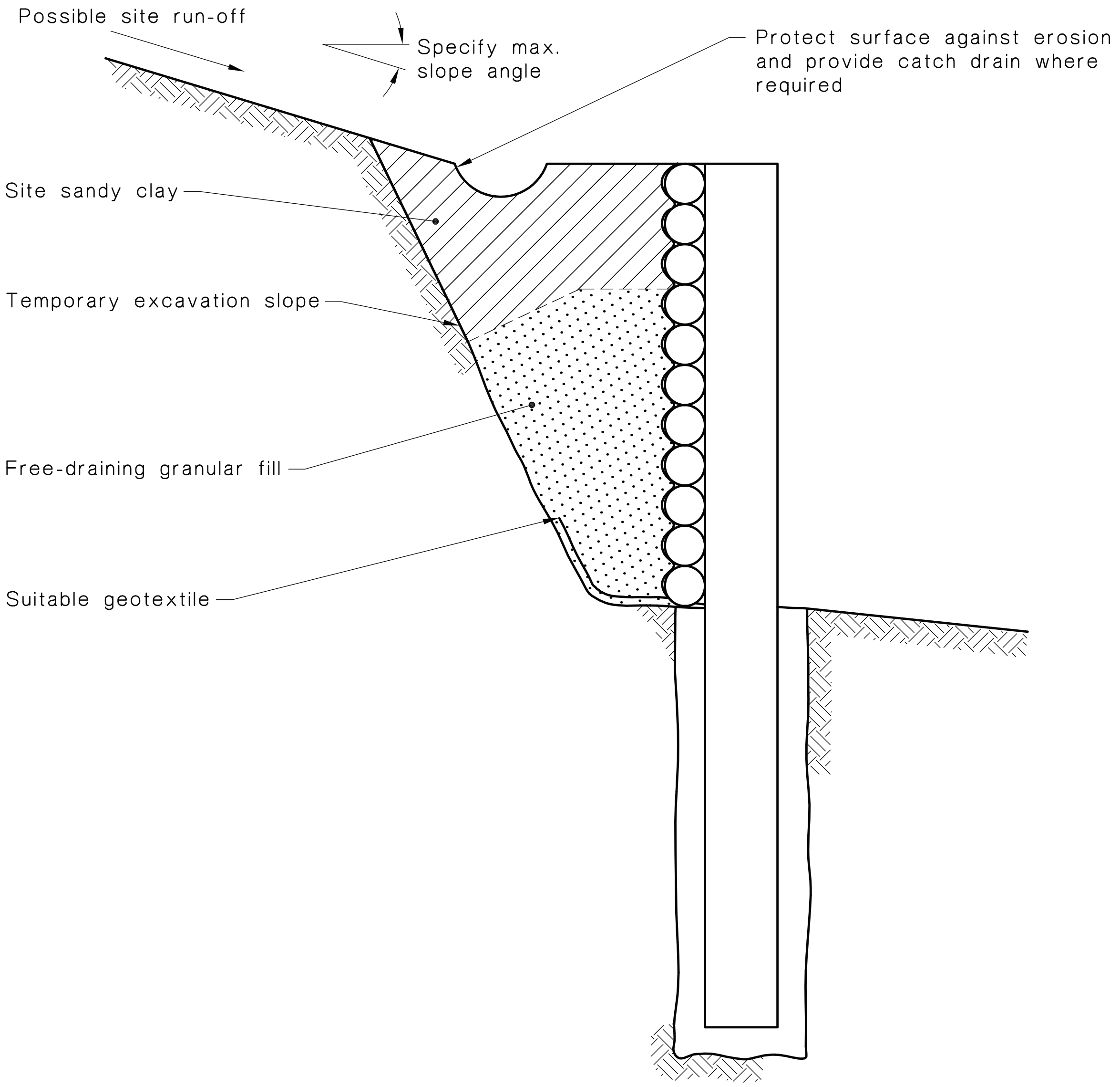


FIGURE G8 TYPICAL SECTION—POST-AND-LOG WALL DRAINAGE

APPENDIX H

REINFORCED SOIL FACING SYSTEM CONNECTION LOADS

(Informative)

H1 DESIGN LOAD

The design load at the connection between the facing elements and the soil reinforcement of a reinforced soil structure should be between 75% and 100% of the maximum load determined for the soil reinforcement in the structure, depending on the design model appropriate to the soil reinforcement stiffness and the facing system flexibility.

H2 COHERENT GRAVITY METHOD

For inextensible reinforced structures designed using the 'coherent gravity' method, the
A1 ratio of the design load at the reinforcement facing connection T^*_{cj} to the design maximum tensile force in the j^{th} layer T^*_j may be taken as indicated in Table H1.

TABLE H1
CONNECTION LOADS—COHERENT GRAVITY METHOD

Facing system	Location	
	From the top of structure to a depth of 0.6*H*, percent	At base of structure, percent
Flexible face (e.g. curved metal sheets)	75	100
Articulated panels with flexible joints (e.g. precast concrete panels with >1% joint compression capacity)	85	100
Vertically stiff walls or walls with no vertical movement capacity (e.g. full height panels, modular block walls)	100	100

NOTE: Values may be linearly interpolated at other locations.

H3 TIE-BACK WEDGE METHOD

For extensible reinforced structures designed using the 'tie back wedge' method, the ratio
A1 of the design load at the reinforcement facing connection T^*_{cj} to the design maximum tensile force in the j^{th} layer T^*_j may be taken as indicated in Table H2.

TABLE H2
CONNECTION LOADS—TIE-BACK WEDGE METHOD

Facing system	Location	
	At top of structure, percent	At base of structure, percent
Flexible face (e.g. curved metal sheets, geotextile wrap around)	75	100
Articulated panels with flexible joints (e.g. precast concrete panels with >1% joint compression capacity)	75	100
Vertically stiff walls or walls with no vertical movement capacity (e.g. full height panels, modular block walls)	100	100

NOTE: Values may be linearly interpolated at other locations.

APPENDIX I

EARTHQUAKE DESIGN

(Normative)

I1 GENERAL

This Appendix provides design requirements for the following:

(a) Situations where earthquake loading requirements are deemed to be met by static design for factored dead and live loads (including soil pressure and surcharges).

(b) The calculation of earthquake loads.

NOTE: The approach is generally consistent with that of AS 1170.4 (which is applicable to building design), except where modifications have been made to account for the differences between the behaviour of buildings and earth-retaining structures.

I2 PROCEDURE

The following steps shall be carried out:

(a) Determine the structure classification.

(b) Determine the acceleration coefficient for the particular location.

(c) Determine the soil profile and site factor relating the soil profile to the acceleration of the structure and retained soil.

(d) Determine the earthquake design category.

(e) Execute the required design method.

Where detailed analysis and design for earthquake loads are required, these shall be carried out in accordance with Paragraph I8.

I3 STRUCTURE CLASSIFICATION

The structure classification shall be determined in accordance with Clause 1.2.2 and Table 1.1.

NOTE: Clause 1.2.2 and Table 1.1 are based on assessment of the risk to life, and Appendix A gives further guidance on the determination of that risk, taking into account height of the structure, proximity to other structures and type of facing and cladding.

I4 ACCELERATION COEFFICIENT

The acceleration coefficient (a) shall be determined in accordance with AS 1170.4.

NOTE: Acceleration coefficient is related to the expected severity of earthquake ground motion before taking account of local site conditions. AS 1170.4 sets out maps showing the distribution
A2 of acceleration coefficients throughout Australia. Table I1, reproduced from AS 1170.4, provides the acceleration coefficient at the principal cities.

TABLE I1

ACCELERATION COEFFICIENT FOR MAJOR CENTRES

Major centres	Acceleration coefficient (*a*)
Adelaide	0.10
Albury/Wodonga	0.08
Ballarat	0.08
Bendigo	0.09
Brisbane	0.06
Cairns	0.06
Canberra	0.08
Darwin	0.08
Geelong	0.10
Gold Coast/Tweed Heads	0.06
Hobart	0.05
Latrobe Valley	0.10
Launceston	0.06
Melbourne	0.08
Newcastle	0.11
Perth	0.09
Rockhampton	0.08
Sydney	0.08
Toowoomba	0.06
Townsville	0.07
Wollongong	0.08

I5 SOIL PROFILE AND SITE FACTOR

The site factor (*S*) shall be determined in accordance with Table I2 for the appropriate soil profile. Interpolation for soil profiles in between those given in Table I2 is permitted. For A1 structures classification C in accordance with Table 1.1, the soil profile should be established from substantiated geotechnical data and classified in accordance with AS 1726.

TABLE I2

SITE FACTORS FOR GENERAL STRUCTURES

Soil profile below the structure	Site factor (*S*)
A profile of rock materials with rock strength Class L (low) or better	0.67
A soil profile with either— (a) rock materials Class EL (extreme low) or VL (very low) characterized by shear wave velocities greater than 760 m/s; or (b) not more than 30 m of— (i) medium dense to very dense coarse sands and gravels; (ii) firm, stiff or hard clays; or (iii) controlled fill.	1.0
A soil profile with more than 30 m of— (a) medium dense to very dense coarse sands and gravels; (b) firm, stiff or hard clays; or (c) controlled fill.	1.25
A soil profile with a total depth of 20 m or more and containing 6 to 12 m of— (a) very soft to soft clays; (b) very loose or loose sand; (c) silts; or (d) uncontrolled fill.	1.5
A soil profile with more than 12 m of— (a) very soft to soft clays; (b) very loose or loose sands; (c) silts; or (d) uncontrolled fill, characterized by shear wave velocities less than 150 m/s	2.0

NOTES:

1 For the purposes of this Standard, it is not intended that a detailed site investigation be carried out for any structure to determine a soil profile over large depths. Most major centres and regional areas have basic information available on the likely strata that should be used to assess the site factor (*S*).

2 Small variations such as isolated layers within the overall profile not exceeding 2 m average depth or changes at or near the surface not exceeding 3 m average depth need not be considered.

I6 EARTHQUAKE DESIGN CATEGORY

The earthquake design category shall be determined from Table I3 for the appropriate value of the product of acceleration coefficient and site factor (aS) and the structure classification.

TABLE I3

EARTHQUAKE DESIGN CATEGORY

A1

Product of acceleration coefficient and site factor (*aS*)	Design category		
	Structure classification		
	C	B	A
$aS \geq 0.2$	E_{er}	D_{er}	C_{er}
$0.1 \leq aS < 0.2$	D_{er}	C_{er}	B_{er}
$aS < 0.1$	C_{er}	B_{er}	A_{er}

I7 DESIGN REQUIREMENTS

Earth-retaining structures shall be designed as set out in Table I4, depending on the earthquake design category.

TABLE I4

DESIGN REQUIREMENTS

Earthquake design category	Design requirements
A_{er}	Design for static loads, without further specific analysis for earthquake, is deemed to meet the requirements of this Standard for earthquake design
B_{er}	Design for static loads, without further specific analysis for earthquake, is deemed to meet the requirements of this Standard for earthquake design
C_{er}	Design for static loads, with a dead load factor of 1.5 (in lieu of 1.25), is deemed to meet the requirements of this Standard for earthquake design
D_{er}	Analysis and design for earthquake loading, as set out below, is required in addition to design for static loads
E_{er}	Analysis and design for earthquake loading, as set out below, is required in addition to design for static loads

I8 SPECIFIC EARTHQUAKE ANALYSIS AND DESIGN REQUIREMENTS

Where detailed analysis and design for earthquake loads are required, these shall be carried out.

Factors that need to be considered in the design of retaining structures for earthquake loading include the following:

(a) Ground acceleration.

(b) Acceleration of the structure and retained soil.

(c) External dynamic earth pressure load on the structures.

(d) Dynamic load of the structure itself.

(e) Internal dynamic load (active zone mobilization in reinforced soil).

(f) Internal and external stability criteria.

(g) Safety factors.

(h) Load combinations.

(i) Deformations.

I9 GROUND ACCELERATION

The ground motion experienced at a particular site due to an earthquake is dependent on many factors including the following:

(a) Earthquake magnitude.

(b) Source mechanism.

(c) Geological conditions and characteristics of the wave transmission from the source to the site.

(d) Epicentral distance.

(e) The geology underlying the site soils.

(f) Local characteristics of site soils.

The ground acceleration adopted for design is based on a consideration of the above factors and would ideally include an assessment of seismicity at the particular site together with determination of the accepted probability of occurrence (or recurrence interval). For the determination of ground motion. The following definitions are common:

(i) Maximum design earthquake (MDE).

(ii) Operating basis earthquake (OBE).

I10 MAXIMUM DESIGN EARTHQUAKE (MDE)

The MDE is usually determined from a statistical assessment, which defines the ground acceleration likely to occur at a long recurrence interval of typically 100 000 years or longer.

The MDE may be used where major loss of life is possible and, although significant damage to the structure may be acceptable, complete failure should not occur. That is, a check for the MDE should ensure that catastrophic collapse does not occur.

I11 OPERATING BASIS EARTHQUAKE

The OBE is considered a normal design case, which should result in minimal damage occurring to the structure, which should remain in a serviceable condition.

The OBE is based on recurrence interval typical of the design life of the structure (e.g. 50 to 100 years).

For the design of routine structures, it would be usual to adopt the OBE acceleration for pseudo-static analysis. For most retaining walls, the pseudo-static acceleration is typically 0.5 times the peak ground acceleration (Elms and Martin, Ref. 1) and, although based on several simplified examples, would be adequate for most design purposes, provided that allowance is made for an outward displacement of the wall (in millimetres) of the order of 250 times the peak ground acceleration (units of g). In the absence of site-specific data or particular assessment of seismicity, the peak horizontal ground acceleration can be taken as the acceleration coefficient (a) from AS 1170.4 times g, recognizing that this has a 10% chance of exceedence in 50 years, or a recurrence interval of 475 years. This acceleration is considered representative of motions in rock at depth.

I12 SOIL PROFILE AND AMPLIFICATION

Amplification of seismic activity due to specific site conditions is an area of continuing research. The following provides some information on potential effects, but each site should be carefully assessed if site amplification is suspected.

The peak ground acceleration at a site is dependent on the nature of the subsoils, the characteristic site period and the period of the base rock excitation. Where soft clay deposits overlie stiffer materials, significant amplification may occur. In the case of soft to firm clay sites or for cohesionless soil sites at shallower depth, lower ground accelerations (say $<0.1\ g$) may be amplified with respect to the peak ground acceleration for rock sites, whereas for very high ground accelerations (say $>0.3\ g$) soft clays may attenuate maximum ground accelerations as shear stresses approach the material shear strength and high energy dissipation occurs (Martin, Ref. 2). For rock, stiff soil and deep cohesionless soil sites there is unlikely to be any significant influence of soil conditions on peak acceleration values.

The site factor (S) in Table I2 (which is consistent with AS 1170.4) may not be appropriate to determine the maximum ground acceleration, as this factor applies to the amplification experienced by framed buildings as a function of foundation conditions, with those structures whose natural period of vibration closely matches the characteristic period of the site being most affected.

A specific site assessment may be necessary for soft clay sites. Martin, (Ref. 2) has cited measured accelerations at the top of 35 to 40 m of soft clay lake-bed deposits of up to 6 times the peak horizontal accelerations of incoming rock motions at depth due to amplification.

Whilst it would be ideal for a site-specific seismicity assessment to be carried out for individual sites, this will rarely be warranted and probably only for significant, sensitive or high hazard structures. Care should be taken with soft soil foundations, where it may be necessary to determine amplification of rock mass motions relevant for design. Designers should refer to the current literature or seek specialist advice for such cases.

I13 ACCELERATION OF THE STRUCTURE AND ITS RETAINED SOIL

Traditionally, retaining wall design has been based on pseudo-static methods with zero vertical acceleration and zero amplification within the structure. In the design of freestanding walls, it is usual to assume that the wall/soil system has a short fundamental period of vibration and that the inertia loads correspond to the peak ground accelerations. Recent work has indicated that, in some cases, amplification of motion in the backfill can play an important role in displacement estimates. In addition the simultaneous application of vertical acceleration in the critical direction can increase the earthquake forces. For pseudo-static analysis such as the Mononobe-Okabe method (Ref. 3) discussed in Paragraph I14, acceleration coefficients are defined as follows:

(a) *Horizontal coefficient of acceleration*:

$$a_h = 0.5a$$

where

a = peak ground acceleration coefficient for deep rock motions from AS 1170.4

(b) *Vertical coefficient of acceleration*:

(i) a_v may vary from 0.3 to >1.0 times a_h. Due to the uncertainty of phase and direction of a_v with respect to a_h, a_v is often adopted as zero.

(ii) a_h and a_v may require amplification in soft clays and shallow granular layers as discussed in Paragraph I12. Where adopted, the coefficient a_v should be applied in the most adverse direction in determining applied stresses.

(iii) a_h is taken as positive acting outwards from the wall.

(iv) a_v is taken as positive acting upwards.

Dynamic numerical analysis for reinforced soil walls with metallic reinforcement and granular backfill have indicated amplification of motions within both the structure and the retained soil. For these structures, amplification may be assessed as follows:

(i) $a_{mh} = (1.45 - a_h)\,a_h$ for $a_h < 0.45$ . . . I13(1)

$= a_h$ for $a_h \geq 0.45$

(ii) $a_{mv} = (1.45 - a_v)a_v$ for $a_v < 0.45$ . . . I13(2)

$= a_v$ for $a_v < 0.45$

where a_{mh} and a_{mv} are the average amplified horizontal and vertical acceleration within the structure/retained soil.

Design values of acceleration coefficients (defined as k_h and k_v) have to be taken as a_h, a_{mh} and a_v, a_{mv} as appropriate. Although the above amplification has been developed for a particular wall type, it may be useful as a first approximation for other forms of reinforcement and other retaining structures in the absence of other information. Dynamic finite element analyses (Nadim and Whitman, Ref. 4) have also indicated that amplification of movements in backfill to conventional gravity walls can increase the design acceleration by 25–50% if motions occur at or near the fundamental frequency of the backfill.

I14 EXTERNAL DYNAMIC EARTH PRESSURE LOAD ON STRUCTURE

Seismic coefficients, k_h and k_v, are defined in order to calculate both external dynamic earth pressure and internal dynamic structure load. The seismic coefficients are related to the average maximum acceleration in the structure as follows:

A2

$k_h = a_{mh}$ . . . I14(1)

$k_v = a_{mv}$ . . . I14(2)

where amplification in the structure is not considered relevant.

A2

$k_h = a_h$ . . . I14(3)

$k_v = a_v$ . . . I14(4)

The k_h and k_v values may be halved in accordance with Paragraph I2, provided wall movement is estimated and checked against acceptable movements for the structures.

For structures in which the use of the coefficient of active earth pressure is appropriate, the external dynamic earth pressure load (E_{ac}) may be calculated according to the classic formula of Mononobe-Okabe (Ref. 3) as follows:

A2

$E_{ac} = (1/2)\,\gamma H^2\,\Delta K_{ac}$ . . . I14(5)

where $\Delta K_{ac} = [(1 - k_v)\,K_{ac}] - K_a$ . . . I14(6)

and K_{ac} is the total active earth pressure coefficient (including seismic effects), K_a is the static active earth pressure coefficient and ΔK_{ac} is the incremental increase in active earth pressure coefficient.

The calculation of the total active earth pressure coefficient, (K_{ac}) for cohesionless soils and a vertical wall (interior face), as proposed by Mononobe-Okabe (Ref. 3), is as follows:

$$K_{ac} = \frac{\cos^2(\phi - \theta)}{\cos\theta\cos(\delta + \theta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \theta)\cos i}}\right]^2} \quad \text{. . . I14(6)}$$

where

ϕ = angle of internal friction of the soil

θ = arc tan$[k_h /(1 - k_v)]$

δ = angle of friction between soil and structure

A1 i = backfill slope angle

Limitations on the magnitude of the above parameters apply as documented in the reference.

The point of application of the total active force is dependent on a number of factors and can typically be applied at 0.4 H to 0.55 H from the base of the wall (where H is the height of retained fill) and at the angle δ to the horizontal.

I15 DYNAMIC LOAD OF STRUCTURE ITSELF

A1 The inertia force (E_i) should be calculated from the total weight of the structure, and superstructure, if appropriate, (W), and the seismic coefficient, as follows:

$$E_{iv} = k_v W \quad \text{. . . I15(1)}$$

and

A2

$$E_{ih} = k_h W \quad \text{. . . I15(2)}$$

The inertia forces for the structure should be applied through the centre of gravity of the structure.

For larger structures such as reinforced earth walls, it is common to take 0.5 of the wall inertia effects in recognition of the likelihood that acceleration of the backfill and the wall may not be exactly in phase.

I16 DYNAMIC WATER PRESSURE

The increase in pore water pressures from earthquake inertia effects should be considered when soils are below the watertable or structures are inundated (e.g. wharfs or quays). It should also be noted that by far the most significant source of earthquake-induced damage to port and harbour facilities has been the pore pressure build up in loose to medium dense saturated cohesionless backfills. Damage results as a consequence of excessive backfill pressures and foundation liquefaction.

For the design of permanently inundated structures and backfill several effects or components of loading (in excess of static conditions) have to be assessed, as follows:

(a) Potential for liquefaction of backfill.

(b) Decreased water pressures outside the structure (if inundated).

(c) Dynamic water pressures in the backfill.

Liquefaction of the backfill is generally not critical for well compacted clay fills or granular fills at about 65% relative density or higher. Wall foundations may be assessed for liquefaction using conventional methods based on a standard penetration test (SPT) or cone penetrometer testing (CPT) data. This will be of greatest significance for structures founded on loose saturated sands where foundation deformation and instability may control the design.

For inundated structures, the reduced restoration effect from the outward displacement of the impounded water can be of significance. Hydrodynamic pressures from the free water in front of a wall may be calculated using Westergard's approximate solution for a vertical wall and a semi-infinite water reservoir (Ref. 5). The resultant dynamic water pressure tending to reduce the static water pressure is approximately—

$$P_{\mathrm{wd}} = \frac{7}{12} k_{\mathrm{h}} \gamma_{\mathrm{w}} H_{\mathrm{w}}^{2} \qquad \text{. . . I16(1)}$$

applied at $0.4H_{\mathrm{w}}$ above the base of the wall,

where

γ_{w} = density of water

H_{w} = height of water in front of wall

To determine the effect of variations such as an inclined wall surface and reservoir geometry, Matsuzawa et al (Ref. 7) provides relevant information. It should be noted that walls that are normally dry but are subject to flooding are highly unlikely to experience a simultaneous seismic event and, therefore, this design case is not considered relevant. For permanently submerged structures, the critical case for overall stability will occur when the hydrodynamic water pressure reduces the static water pressure in front of the wall and is in phase with the earthquake earth pressure increment on the wall.

Hydrodynamic water pressures can also develop within saturated backfilled materials. In the case of highly permeable granular backfills, it can be assumed that the pore water can move freely in the voids without any restriction from the soil particles. For free-draining saturated backfills, the calculation of pressures applied to the rear of the wall are determined in accordance with the recommendations of Matsuzawa et al (Ref. 6), as follows:

(a) A modified apparent angle of seismic coefficient for use in the Mononobe-Okabe formula θ' is calculated as—

$$\tan\theta' = \frac{G_{\mathrm{s}}}{G_{\mathrm{s}} - 1} \tan\theta \qquad \text{. . . I16(2)}$$

where G_{s} is the specific gravity of the soil—

θ = $\tan^{-1}[k_{\mathrm{h}}/(1 \pm k_{\mathrm{v}})]$

For $G_{\mathrm{s}} = 2.65$ (typically), $\tan\theta' \approx 6\tan\theta$.

θ' is used in the lieu of θ in the equation for K_{ac}.

(b) Static earth pressures are calculated using the buoyant density for zones below the water table.

(c) Dynamic earth pressure increment is calculated using the buoyant density and θ' as in Paragraph I4.

(d) Static water pressures are calculated as normal.

(e) Dynamic water pressures are calculated using Westergard's method (Ref. 6) and are additional to the static water pressures.

(f) The thrust on the wall is the summation of each component.

Matsuzawa et al (Ref. 7) provide more detailed information on assessing the effect of ground motion on a variety of backfill materials. Whilst the topic is a continuing subject of research, the approach illustrated here provides a basis for assessing the critical effect of pore water on seismic response, and may be used in the absence of site-specific measurements, previous data, or known site performance.

I17 TOTAL LOAD UNDER EARTHQUAKE

The total load is calculated by adding the static loads and the external dynamic earth pressure loads, the dynamic water pressure loads, plus internal dynamic loads of the structure itself.

Normally the loads should be taken in full; however, where appropriate, the use of a reduced seismic coefficient (as defined in Paragraph I12) or the use of a reduced external dynamic earth pressure load (usually 50%) may be adopted in accordance with commonly adopted practice for reinforced soil structures.

I18 INTERNAL DYNAMIC LOAD

The internal stability of a retaining structure should take into account the dynamic load mobilized by the active zone where it is appropriate for the design of the particular structural system. This may be a Coulomb wedge on the stem of a cantilever retaining wall or the active zone of a reinforced soil system.

For a reinforced soil structure, the internal dynamic load from the mobilized active zone is resisted by the reinforcement layers in proportion to their available frictional resistance.

The internal dynamic load is not considered to act concurrently with the external dynamic load.

I19 STABILITY AND STRENGTH CRITERIA

The stability criteria will depend on the design approach to be adopted (see Paragraph I12).

In traditional working stress design methods using acceleration coefficients for OBE analysis, there is no allowance for reduced safety factors and, thus, in the limit state method all partial factors and load factors would apply. Reduced accelerations may be used in accordance with Paragraph I2, provided that structure deformations are acceptable.

For MDE analysis, it is usual in a working stress approach to accept an overall factor of safety of greater than 1 for such a rare event. This may be checked in the limit state approach by ensuring that the force inequality is satisfied for the serviceability limit state. It would be acceptable to allow structural strength to approach yield conditions, provided that brittle or progressive failure is avoided.

I20 APPLIED LOADS

I20.1 General

The load combination for earthquake design ultimate limit state condition should follow the requirements given in Table J1 of Appendix J, using load case C.

Where traffic load or other live load is carried directly by the retaining structure, the partial factors for live load (i.e. γ_{q1}, γ_{q2}) should be taken as 0.5.

I20.2 Effect of superstructure loads

Where retaining structures support each fill (e.g. sloping batter above the structure) the inertial earthquake force generated by the overlying fill should be calculated in accordance with Paragraphs I15 and I17.

For reinforced soil structures supporting bridge abutments, the following is recommended.

(a) The reinforced soil structure should be designed for the factored design earthquake forces (including factored dead loads) transmitted from the bridge superstructure. Estimates of the dynamic load are to be provided by the bridge designer. These dynamic loads should be treated in conjunction with load combinations given in Paragraph I20.1; however, only 50% of the factored live loads (both vertical and horizontal) from the bridge superstructure transmitted through the still beam is required to be included in the analysis.

(b) The additional inertial earthquake force of the sill beam should be included. A reduced dynamic earth pressure of 50% is permitted for fill retained by the sill beam.

I21 DEFORMATIONS

Relatively large earthquakes (i.e. peak ground acceleration greater than 0.3*g*) could result in significant permanent deformation. The likelihood of such movement and its potential magnitude may be determined by reference to procedures presented by Wood and Elms (Ref. 8).

I22 REFERENCES

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APPENDIX J
LOAD COMBINATIONS
(Informative)

J1 GENERAL

This Appendix is aimed at assisting designers to determine the appropriate loading combinations by reproducing those set out in AS 1170.1, AS 1170.2, and AS 1170.4, and any other existing relevant loading specifications at the time of publication of this Standard Italicized words reflect the requirement of AS 1170, not the requirements of this Standard. The designer should determine whether such combinations are still valid at the time when the design is to be carried out.

Other loads to be included with the basic combination given in AS 1170.1 for dead load (including earth load), live load and earthquake load are hydrostatic pressure and loading due to temperature movement.

NOTE: For the strength and stability limit state, this Standard requires the application of a load factor of 1.25 on soil load and 1.0 on liquid pressure together with a reduction factor of between 0.95 and 0.75 on tan(ϕ) which is used to calculate the soil pressures. The combined effect of these two factors is to give an effective load factor on active soil pressure in the range of 1.3 to 1.7 for most common cases. This is consistent with AS 1170.1, which specifies load factors of 1.5 for earth pressure and 1.25 on liquid pressure behind a structure.

J2 LOAD COMBINATIONS FOR STRENGTH LIMIT STATES

The loads for strength design have to be the most adverse combination of factored loads that can reasonably coexist, factored from, but not limited to, the following combinations as applicable:

(a) $1.25\,G + 1.5\,Q$

(b) $1.25\,G + W_u + \psi_c Q$

(c) $1.25\,G + 1.0\,F_{eq} + \psi_c Q$

(d) $0.8\,G + 1.5\,Q$

(e) $0.8\,G + W_u$

A1 (f) $0.8\,(G + \psi_c Q) + 1.0\,F_{eq}$

Where G includes dead load of retained soil and ψ_c may be taken from AS 1170.1.

J3 LOAD COMBINATIONS FOR STABILITY LIMIT STATES

The structure has to be designed to prevent instability due to overturning, uplift and sliding, as follows:

(a) The loads determined in accordance with Clause 4.1 *have to* be subdivided into components tending to cause instability and components tending to resist stability.

(b) The design action effect (S*) *has to* be calculated from the components of the loads tending to cause instability factored and combined in accordance with Clause 4.2.

(c) The design resistance effect (R*) *has to* be calculated from 0.8 times the part of the dead load tending to resist the instability and the design capacity of the structural components to resist instability (ΦR).

(d) The design resistance effect *has to* be greater than the design action effect.

For the cases involving dead, live and earthquake loadings, the requirements may be expressed by the following:

(i) $1.25\ G^{C} + 1.5\ Q^{C} < 0.8G^{R} + (\Phi R)$

(ii) $1.25\ G^{C} + \psi_{c}Q^{C} + W_{u}{}^{C} < 0.8\ G^{R} + (\Phi^{R})$

(iii) $1.25\ G^{C} + \psi_{c}Q^{C} + 1.0\ F^{C}{}_{eq} < 0.8\ (G + \psi_{c}Q)^{R} + (\Phi R)$

where G^{C}, Q^{C}, $W_{u}{}^{C}$ and $F^{C}{}_{eq}$ are parts of the dead, live, wind and earthquake loads that tend to cause instability G^{R} is the part of the load tending to resist instability and ΦR is the design capacity of the structural component designed to resist instability (if any).

Examples of load components for limit states are given in Table J1.

A1 The load cases A, B and C in Table J1 are as follows:

A Load case A—limit state load case for strength.

B Load case B—limit state load case for stability.

C Load case C—limit state load case for serviceability.

TABLE J1

EXAMPLES OF LOAD COMPONENTS FOR LIMIT STATES (not including wind and earthquake loads)

Load factors	Description	Load case		
		A	B	C
γ_{g1} for γ_1	Dead load (G_1) of structure	1.25	0.8	1
γ_{g2} for γ_2	Dead load (G_2) of fill behind structure	1.25	1.25	1
γ_{g3} for γ_3	Dead load (G_3) of fill on structure	1.25	0.8	1
γ_{g4} for γ_4	Dead load of fill (G_4) in front of structure	0.8	0.8	1
γ_{gw} for γ_w	Weight of water behind or in front of structure	1	1	1
γ_{q1}	Traffic load (Q_1) or other live load on structure	1.5	0	0.7(1) or 0.4(2) or 1.0(3)
γ_{q2}	Traffic load (Q_2) or other live load behind structure	1.5	1.5	0.7(1) or 0.4(2) or 1.0(3)
γ_{q3}	Traffic load (Q_3) or other live load in front of structure	0	0	0

NOTES:

1 Refers to short-term case.

A2 2 Refers to long-term case.

3 Refers to storage load.

J4 LOAD COMBINATIONS FOR SERVICEABILITY LIMIT STATES

The design load for the serviceability limit states *has to* be taken from the factored loads for short-term effects and long-term effects as follows:

(a) For short-term effects:

(i) W_{s}

(ii) $\psi_{s}\ Q$

(iii) $G + W_{s}$

(iv) $G + \psi_{s}\ Q$

(v) $G + \psi_{s}\ Q + Ws$

(b) For long-term effects:

(i) G

(ii) $\psi_l Q$

(iii) $G + \psi_l Q$

ψ_s and ψ_l may be taken from AS 1170.1.

J5 EXAMPLE OF THE USE OF LOAD FACTORS

The elements of a gravity earth-retaining structure are illustrated in Figure J1. The effective earth-retaining structure may include the structural system, its contained earth and any superstructure (including overburden and supported structures) that acts integrally with the structure. Examples of effective earth-retaining structures are illustrated in Figure J2.

The examples of the application of the load combinations for strength, stability and serviceability limit states to the effective earth-retaining structure are illustrated in Figure J3.

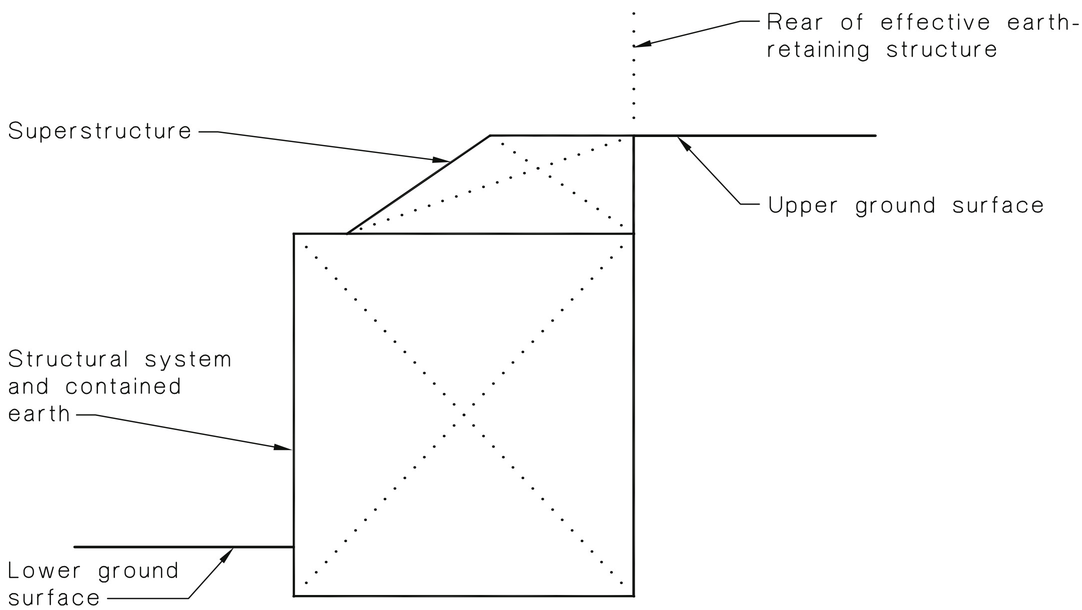
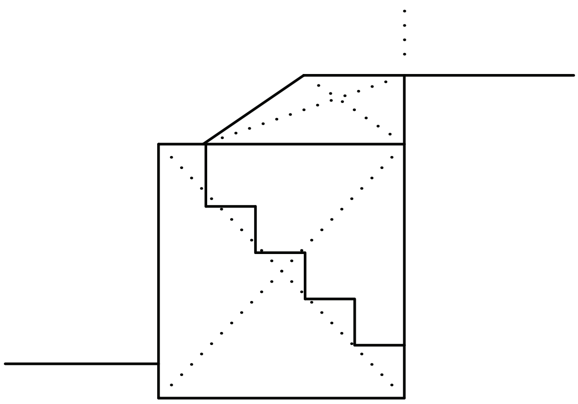


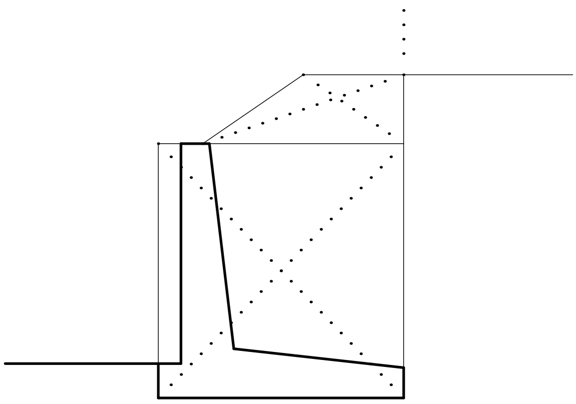
FIGURE J1 ELEMENTS OF A GRAVITY EARTH RETAINING STRUCTURE

J6 LOADING COMBINATIONS FOR SPECIAL STRUCTURES

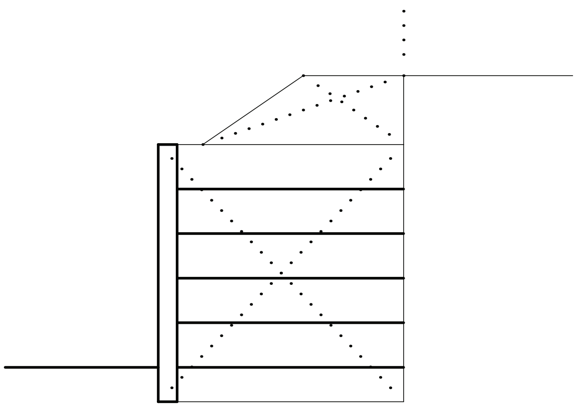
In the calculation of traffic surcharge, the unfactored value *has to* be 20 kPa for roads of functional road classes 1, 2, 3, 6 or 7 (see HB77). For all other functional class roads or temporary roads (e.g. ramps) the unfactored traffic loading *has to* be 10 kPa.



A1 (a) Masonry wall system

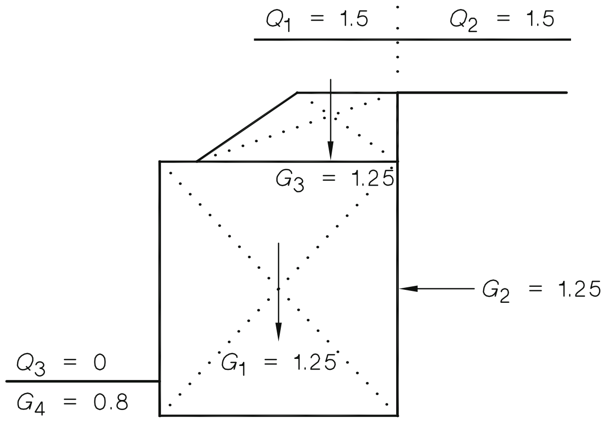


(b) Cantilever wall system

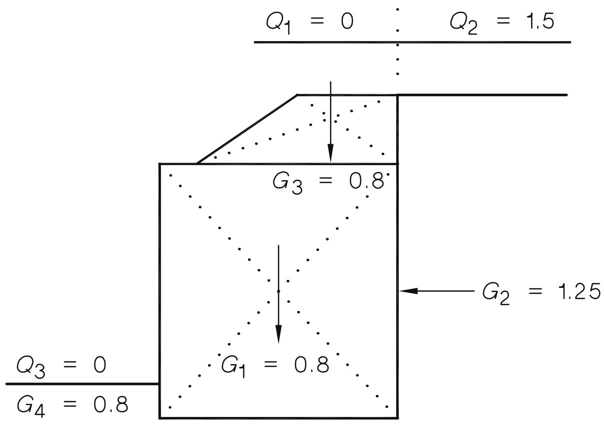


(c) Reinforced soil system

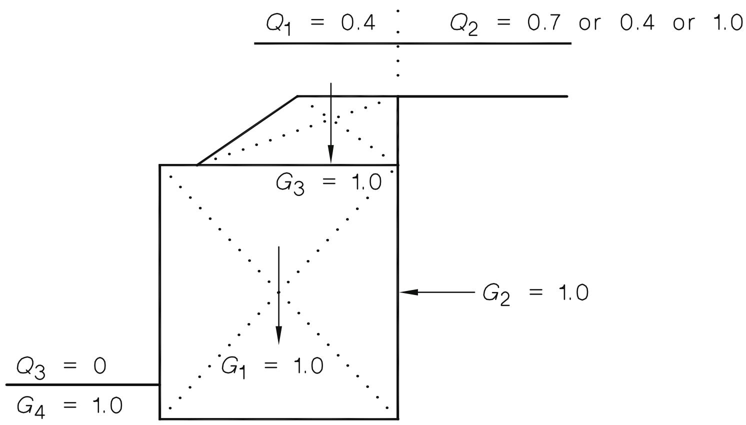
FIGURE J2 EFFECTIVE EARTH-RETAINING STRUCTURES



(a) Strength load combinations



(b) Stability load combinations



(c) Serviceability load combinations

FIGURE J3 LOAD COMBINATIONS
(not including wind and earthquake loads)

J7 ASSESSMENT OF EARTH PRESSURES

J7.1 Earth and material pressures

J7.1.1 *Load factors and pressures*

The factored loads and resistances to be used in the design calculations (including earth pressures, soil friction, soil bearing strength and component strengths) *have to* be those corresponding to the most demanding combination of forces and material properties that can reasonably coexist. For the most common cases such as strength, stability or serviceability of gravity structures, this combination will result from the application of the partial factors given in Sections 4 and 5.

NOTE: For serviceability, the partial load factors and partial material factors selected would usually be equal to 1.0.

Where a more exigent design situation can reasonably be expected to result by using factors that are different from those given in Sections 4 and 5, then they *have to* be considered. An alternative set of partial load or material factors *has to* be considered so as not to diminish the overall safety index of the structure or its components for strength, stability and serviceability.

Having determined the factored loads and resistances that can reasonably be expected to coexist in the most exigent situation, the structural load effects on the wall or reinforced soil structure (i.e. bending moments, shears etc.) *has to* be calculated using equilibrium principles without the application of further load factors.

Where the material to be retained consists substantially of weathered rock or rock-like materials, then the principles of rock mechanics, including the assessment of any 'wedge type' failures, *have to* be used in combination with the properties of backfill material to assess the pressures on the wall or reinforced soil structure.

Information on earth pressures commonly used in the design of conventional retaining walls and reinforced soil structures is given in Paragraph J7.2.

J7.1.2 *Restraint conditions*

As the earth pressure to be resisted by a wall or RSS structure is dependent on the conditions of structural constraint, the designer *has to* give careful consideration to the restraint conditions applying to the particular structure in the assessment of whether the structure will be subjected to active, at rest, or passive earth pressures.

Whilst normally RSS structures are designed for a loading condition equivalent to the 'active' earth pressure condition, there are many situations under which a conventional retaining wall structure can have 'at rest' or 'passive' loading conditions applied.

J7.1.3 *Limit states*

The designer *has to* give consideration to questions of overall slope stability, the possibility of a general rotational failure of the soil mass and the bearing capacity of the foundation strata in the determinations.

Information on the limit states commonly used in the design of conventional retaining walls and RSS structures is given in Paragraph J7.2.

J7.2 Earth pressures

The earth pressure that acts on an earth-retaining structure is strongly dependent upon the lateral deformations that can occur in the soil or weathered rock strata; thus, deformation conditions should be estimated with reasonable accuracy as a part of the design process.

The minimum pressure that can be exerted against the wall is referred to as the 'active' condition and occurs when the wall moves sufficiently far outwards for the soil behind the wall to expand laterally and reach the state of plastic equilibrium.

Similarly, the maximum pressure that can be exerted against the wall is referred to as the 'passive' condition and occurs when the wall movement is towards the soil.

The amount of movement necessary to reach these failure conditions is dependent primarily upon the type of materials behind the wall or RSS including the backfill material.

Some guidelines on these movements are given in Table J2.

For wall displacements less than those necessary to produce the failure conditions, it can be assumed that the magnitude of the pressure upon the wall lies between the extreme values.

In weathered rock strata, the situation is more complex, requiring a consideration of the principles of rock mechanics and how a pressure may be transmitted from the potential failure wedge to the wall or RSS.

TABLE J2

WALL DISPLACEMENTS REQUIRED TO DEVELOP ACTIVE AND PASSIVE EARTH PRESSURES (Ref. 1)

Soil	State of stress	Type of movement	Necessary displacement
Sand	Active	Parallel to wall	0.001 *H*
	Active	Rotation about base	0.001 *H*
	Passive	Parallel to wall	0.05 *H*
	Passive	Rotation about base	> 0.10 *H*
Clay	Active	Parallel to wall	0.004 *H*
	Active	Rotation about base	0.004 *H*
	Passive	—	—

J8 INFLUENCE OF GEOMETRIC SHAPE OF RETAINING STRUCTURE

The geometric shape requirements of a retaining structure will vary depending on the following:

(a) Level and slope of the retained ground surface and below the structure.

(b) Surcharge loadings.

(c) Soil properties in the select fill and general backfill zones.

(d) Bedrock levels and overall geological model assumed.

(e) Type of retaining structure, i.e. reinforced concrete wall, gravity type wall or reinforced soil structure.

(f) Other considerations, e.g. areas of known seepage, soft ground areas or where future excavation adjacent to retaining wall is planned.

For conventional gravity-retaining structures, an initial geometric sizing of base width (*B*) equal to 0.7 times the height (*H*) is generally used to provide the required sliding and overturning resistance and reduce foundation bearing pressures.

For reinforced soil structures, the geometry of the reinforced fill zone may be rectangular or stepped (i.e. trapezoidal) in cross-section. Stepped wall geometries should only be used where the founding conditions of the RSS are on stable rock strata. Figure J4 gives details of minimum sizing of reinforced soil walls with various geometries to satisfy both internal and external stability.

J9 EARTH PRESSURES PRODUCED BY COMPACTION

Proper compaction of the backfill behind conventional retaining walls or within the select fill zone of a reinforced soil structure is essential to ensure adequate shear strength and stiffness. Whilst compaction is essential, its effect may induce large horizontal earth pressures, which are subsequently 'locked into' the soil mass. These pressures can vary both in magnitude and distribution and can exceed (Ref. 2) active earth pressures, particularly in the upper part of the retaining structure.

A simplified method has been proposed by Ingold (Ref. 3) to estimate compaction-induced earth pressures against retaining walls. The distribution of horizontal earth pressures is idealized in Figure J5. Increased earth pressures given by p_{hm} are predicted to occur within h_c (depth below surface of the structure). For depths greater than h_c a linear increase of earth pressure with depth (either using K_a in the case of yielding structures or K_o in the case of unyielding rigid structures) is assumed.

Compaction-induced horizontal stresses can sometimes be a significant contribution to the total pressure against a retaining wall. In construction, it is good practice to place controls on various construction equipment that are allowed to work behind such retaining structures. In the case of a reinforced concrete structure with a thin cantilever stem, it is advisable to use light compaction equipment such as vibratory plate compactors (say 125 kg mass), which would induce compaction pressures of approximately 12 kPa.

A similar situation would apply in the compaction of select backfill in reinforced soil-retaining structures where light compaction plant is used within 1.5 m behind the facings. Heavier construction plant (e.g. 10.2 t static smooth wheel rollers) may induce horizontal stresses up to 20 kPa and, therefore, such equipment should be used outside this zone. For vibratory rollers, the combined effect of the roller dead weight and centrifugal force has to be used in calculating the effective line loads Q_1 imposed by the compaction plant.

J10 EFFECTS OF SURCHARGES

J10.1 Types of surcharges

Surcharges behind reinforced soil walls and retaining structures may be categorized into two main types, as follows:

(a) *Uniformly distributed loads*—these may be continuous loads that act on the surface of retained ground behind the structure, and include traffic loads and loadings from materials stacked behind the retaining structure.

(b) *Concentrated loads*—these may be line loads (e.g. loadings from strip footings), point loads (e.g. loads from isolated footings and piers) and area type loads, which have to be considered in relation to the height and geometry of the wall.

Surcharges behind or on retaining structures may be of a permanent or temporary nature.

Examples of permanent loading are loads due to shallow foundations of an adjacent structure or sill beam loadings of a bridge structure directly supported by reinforced soil walls. Traffic-induced uniform loads for major roads may be classed as permanent. Examples of temporary loads would be loads from construction plant or storage of goods or construction materials that exert a transient load to the structure.

In some cases, the retaining structure may support other amenities such as sound barriers or New Jersey kerbs that may attract loadings, e.g. wind loads or traffic impact loads. The effect of such loads should be checked for the global and internal stability of the retaining structure.

J10.2 Design surcharge loads

The distribution of stresses on a retaining structure due to surcharge loads should be carefully assessed in design. The design load factors are 1.5 for strength and stability limit states, and 0.7 for the serviceability limit state. See Table J1 for examples of load components for each limit state.

J10.3 Assessment of the effect of surcharge for conventional retaining structures

Each pressure induced by surcharge loads will depend on the load spreading properties of the retained earth and the stiffness of the wall. Two approaches may be used to assess the magnitude and distribution of lateral pressures induced by surcharge loading, as follows:

(a) *Rankine active earth pressure theory*—the trial wedge method and associated force polygon methods may be employed (see Figure J6).

(b) *Elasticity theory supported by experimental measurements*—the earth pressure design charts given by Terzaghi (Ref. 6) given in Figure J7 may be used to estimate lateral pressure due to vertical line loads, point loads and horizontal line loads. These have been modified from the Boussinesq (Ref. 5) solution for distribution of stresses in an isotropic semi-infinite elastic medium based on experimental evidence.

A2

Numerical modelling approaches (finite element analysis) may be used to assess the magnitude and distribution of earth pressures for complex loaded retaining structures.

J10.4 Assessment of the effect of applied loadings on reinforced soil structures

For reinforced soil-retaining structures, the internal stresses are derived from two separate load conditions, resulting from—

(a) the supporting function, and

(b) the retaining function.

These stresses should be superimposed for the determination of the total stresses due to applied loadings and for the calculation of reinforcement stresses.

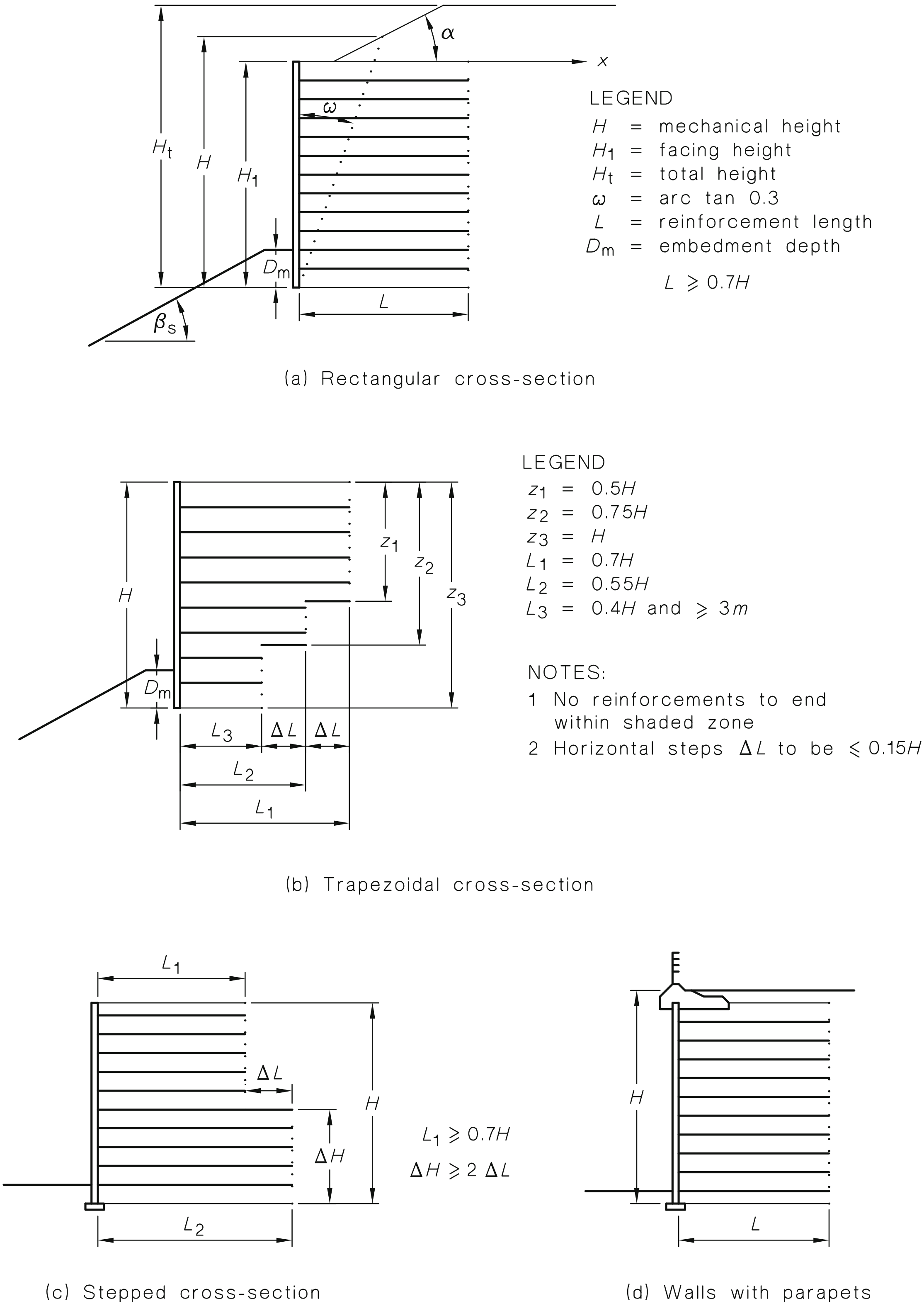
The supporting function requires the determination of the additional vertical and horizontal stresses resulting from the diffusion of the applied loadings.

For vertical loadings, the distribution of vertical strip loadings through the fill may be determined by a lateral dispersion defined by a slope of 2 vertically to 1 horizontally (see Figure J8), or by calculating an appropriate stress distribution such as defined by Boussinesq (Ref. 5). Vertical load dispersion *has to* take into account the effect of the face on the available dispersion area.

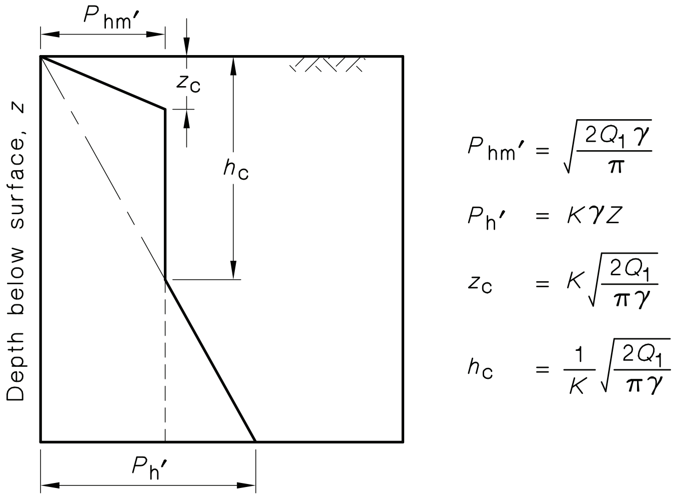
For horizontal loadings, the distribution of horizontal strip loadings through the fill may be determined by a vertical dispersion over the outside surface or facing of the structure defined by a slope from the rear of the contact surface. The horizontal pressure distribution on the outside surface or facing *has to* vary linearly from a maximum at the level of the contact surface to zero at its lowest point (see Figure J8).

The retaining function takes into account the overall retaining structure loadings such as self weight, superstructure and general surcharge loadings. These should then be taken in combination with the overturning moments generated by the applied loads on the structure.

The diffusion of stresses imposed by applied loadings depends on the stiffness characteristics of the structure. Further guidance on the selection and use of appropriate design methods is provided in BS 8006.



A1 FIGURE J4 SIZING OF WALLS WITH VARIOUS GEOMETRIES



LEGEND:

K = earth pressure coefficient (see Note 2)

Q_1 = intensity of effective line load imposed by compaction plant (see Note 3)

z_c, h_c = critical depths as shown

γ = soil unit weight

A1 P_{hm}' = maximum horizontal earth pressure induced by compaction

P_h' = horizontal earth pressure induced by overburden stress

NOTES:

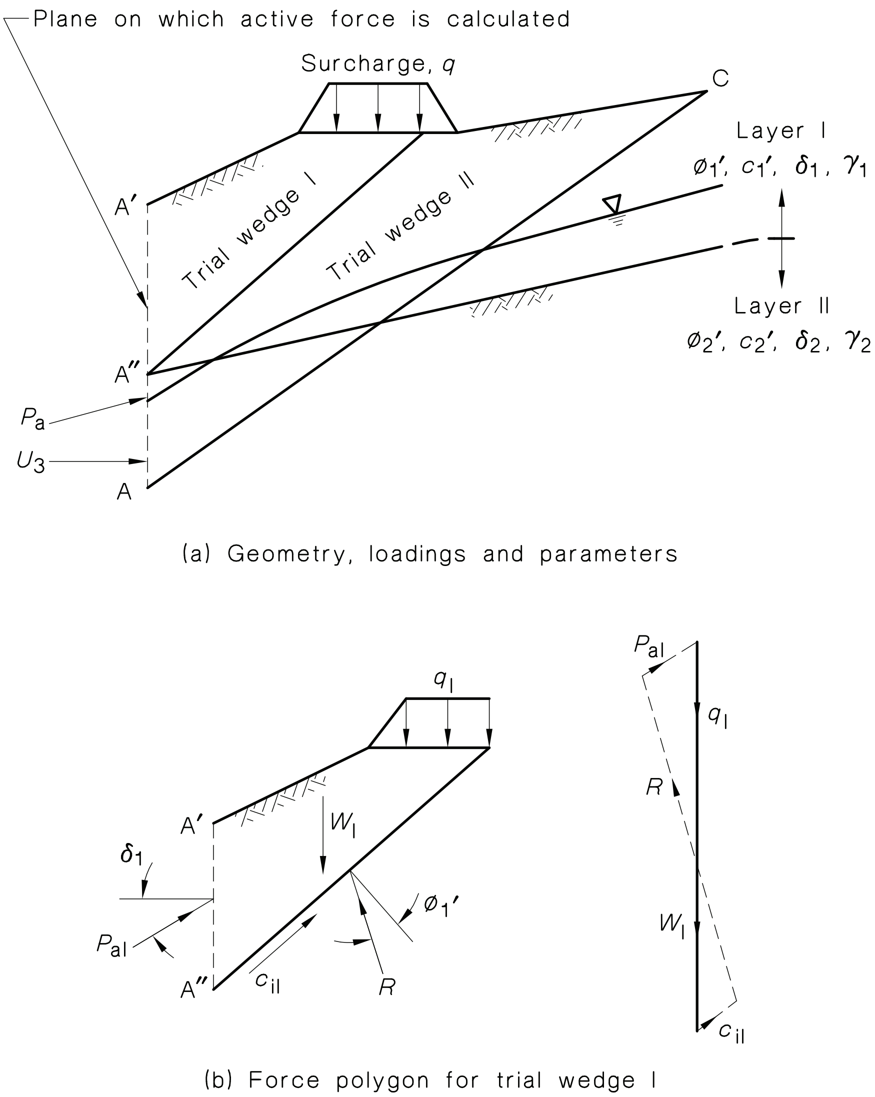
1 Figure based on Ingold (1979a and b).

2 For retaining walls that can move forward sufficiently to mobilize active condition in the fill, $K = K_a$. For unyielding rigid structures, $K = K_o$. For walls supporting a fill slope, it may be assumed that the compaction-induced earth pressure is the same as that given by the diagram above for a horizontal final surface, except z_c should be taken as zero.

3 For dead weight rollers, the effective line load is the weight of the roller divided by its roll width, and for vibratory rollers it should be calculated using an equivalent weight equal to the dead weight of the roller plus the centrifugal force generated by the roller's vibrating mechanism. The centrifugal force may be taken to be equal to the dead weight of the roller in the absence of trade data.

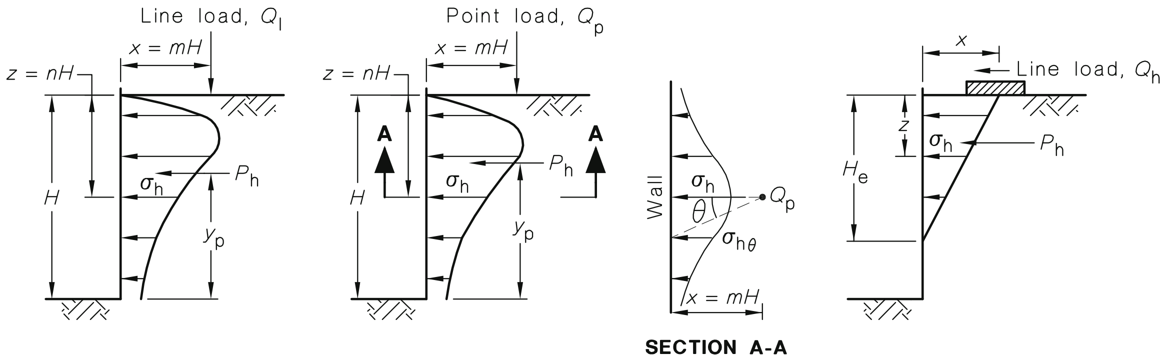
4 The compaction-induced earth pressure assumed in the design should be clearly stated on the drawings.

FIGURE J5 SIMPLIFIED METHOD FOR THE EVALUATION OF COMPACTION-INDUCED EARTH PRESSURES



A1 FIGURE J6 TRIAL WEDGE METHOD OF ANALYSIS FOR SURCHARGE LOADING BEHIND RETAINING STRUCTURES

A2



For $m \leqslant 0.4$

$$\sigma_h \left(\frac{H}{Q_l}\right) = \frac{0.20n}{(0.16 + n^2)^2}$$

$$P_h = 0.55\, Q_l$$

For $m > 0.4$

$$\sigma_h \left(\frac{H}{Q_l}\right) = \frac{1.28 m^2 n}{(m^2 + n^2)^2}$$

$$P_h = \frac{0.64\, Q_l}{(1 + m^2)}$$

(a) Lateral pressure on wall due to vertical line load, Q_l

For $m \leqslant 0.4$

$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{0.28n^2}{(0.16 + n^2)^3}$$

$$P_h = 0.69\, Q_p$$

For $m > 0.4$

$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{1.77 m^2 n^2}{(m^2 + n^2)^3}$$

$$P_h = 0.48\, Q_p \left[\frac{m(1 - m^2)}{(1 + m^2)^2} + \tan^{-1}\left(\frac{1}{m}\right)\right]$$

$$\sigma_{h\theta} = \sigma_h \cos^2(1.1\theta)$$

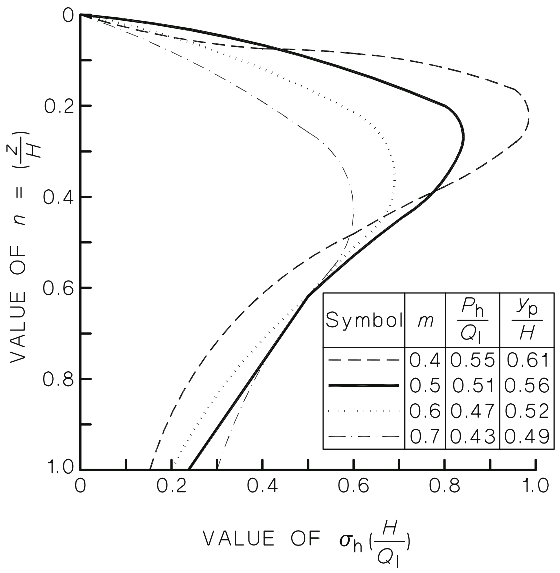
(b) Lateral pressure on wall due to vertical point load, Q_p

$$H_e = x \tan\left(45° + \frac{\phi}{2}\right)$$

$$\sigma_h \left(\frac{H_e}{Q_h}\right) = 2\left(1 - \frac{z}{H_e}\right)$$

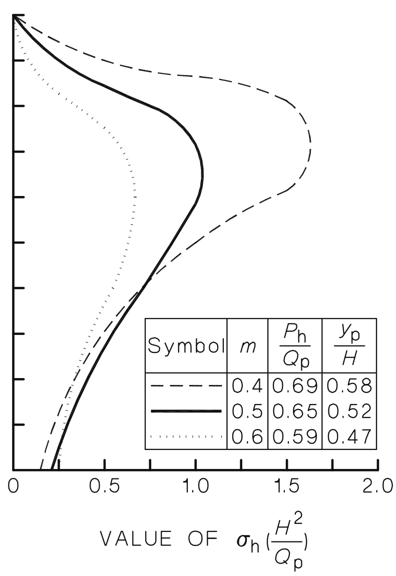
Resultant $P_h = Q_h$

(c) Lateral pressure on wall due to horizontal line load, Q_h



Symbol	m	$\frac{P_h}{Q_l}$	$\frac{y_p}{H}$
– – –	0.4	0.55	0.61
——	0.5	0.51	0.56
······	0.6	0.47	0.52
–·–·–	0.7	0.43	0.49

(d) Pressure distribution due to vertical line load, Q_l



Symbol	m	$\frac{P_h}{Q_p}$	$\frac{y_p}{H}$
– – –	0.4	0.69	0.58
——	0.5	0.65	0.52
······	0.6	0.59	0.47

(e) Pressure distribution due to vertical point load, Q_p

FIGURE J7 CALCULATION OF LATERAL PRESSURE ON A VERTICAL RETAINING WALL DUE TO VERTICAL AND HORIZONTAL LOADS

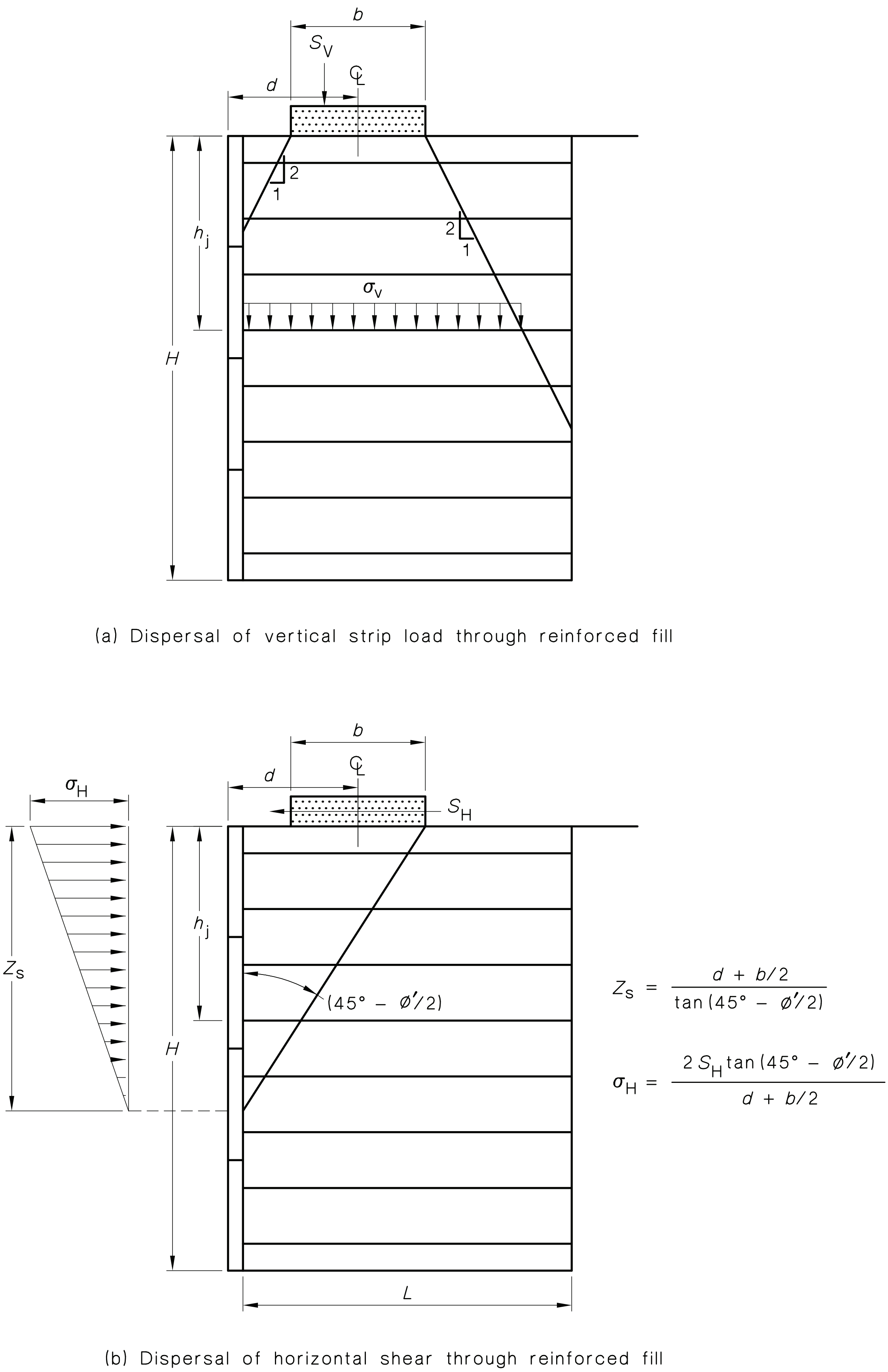


FIGURE J8 APPLIED LOAD DISTRIBUTION IN REINFORCED SOIL STRUCTURES

J11 EFFECTS OF WATER AND SEEPAGE

J11.1 General

Controlling hydrostatic pressures will often be the most important assumption in the design of a retaining structure. None of the following is sufficient to ensure that hydrostatic pressures will be adequately controlled throughout the design life of a structure:

(a) Providing a 'drain' on a cross-sectional drawing.

(b) Assuming or specifying that the backfill be 'free draining'.

(c) Assuming that the wall itself is free draining.

If drains become ineffective, then the loads on a structure will increase substantially, and the resistances will reduce substantially, such that the overall safety of the structure will be halved.

J11.2 Uplift pressures

Hydrostatic pressures causing uplift on the base of retaining structures should be included in design calculations unless positive means are adopted to ensure that they do not develop.

J11.3 Long-term conditions

Hydrostatic pressures behind and below a structure may be reduced by the design of an appropriate drainage system. If such a system is incorporated in the design, then both the construction and maintenance of the drainage system will need to be arranged to ensure that the designer's intent is met.

J12 REFERENCES

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APPENDIX K

PARTIAL MATERIAL STRENGTH FACTOR DETERMINATION FOR SOIL REINFORCEMENT

(Informative)

K1 GENERAL

This Appendix provides recommendations for the determination of material strength reduction factors and uncertainty factors, which should be considered in the derivation of long-term design strength of soil reinforcement, as defined in Section 5.

In the absence of internationally approved test method verified values, the appropriate maximum material strength reduction factors defined in Paragraph K2, K3, K4 and K5 should be adopted.

K2 MANUFACTURING VARIATION FACTOR

The uncertainty factor (product), (Φ_{up}) may be determined from Table K1.

TABLE K1

UNCERTAINTY FACTOR (PRODUCT), (Φ_{up})

Situation	Φ_{up}
R_{st} is a guaranteed minimum	1.00
R_{st} is a characteristic value	0.95

K3 TIME-DEPENDENT STRESS/STRAIN BEHAVIOUR FACTORS

Allowable design load of the reinforcement should be based on real time creep tests for a period not less than 10% of the design life. For design lives greater than 10 years, the allowable design load of the reinforcement should be based on a minimum duration of 10^4 hours.

Extrapolation of two log cycles is permitted provided the extrapolation is based on supported test data derived from real time or accelerated tests, such as those carried out at a range of temperatures.

The reduction factors for the extrapolation in Table K2 should be used.

In the absence of appropriate test results the reduction factors ϕ_{rc} in Table K3 should be used.

TABLE K2
UNCERTAINTY FACTOR (EXTRAPOLATION), (Φ_{ue})

Material	Φ_{ue}			
	Log of cycles of extrapolation			
	0	1	1.5	2
Metals	1.00	1.00	1.00	1.00
Polyester	1.00	1.00	0.88	0.75
Polyethylene	1.00	1.00	0.88	0.75
Polypropylene	1.00	1.00	0.88	0.75

TABLE K3
A1
REDUCTION FACTOR (CREEP), (Φ_{rc})

Material	Service life (years)	Φ_{rc}
All metals	100	1.00
Polyester	30 100	0.60 0.50
Polyethylene	30 100	0.33 0.30
Polypropylene	30 100	0.20 0.17

K4 CONSTRUCTION DAMAGE FACTOR

The reduction factor (construction or installation) (Φ_{ri}) should be determined by reference to the material and the structure of the product, as well as the nature of the backfill and the compactive effort and should be determined from actual tests undertaken in controlled conditions. In the absence of such test results, the values in Table K4 should be used.

TABLE K4
REDUCTION FACTOR (CONSTRUCTION OR INSTALLATION) (Φ_{ri})

Situation	Φ_{ri}
Steel of minimum thickness 4 mm in all backfills	1.00
Geosynthetic material in fine sand	0.90 – 0.8
Geosynthetic material in coarse gravel	0.90 – 0.6

NOTE: Some polyester grids in coarse gravel will suffer damage greater than 40%, thus requiring reduction factors less than 0.6.

K5 ENVIRONMENTAL FACTORS

K5.1 General

The environmental degradation factors that cover chemical and biological attack of the reinforcement material are the following:

(a) Reduction factor (thickness), Φ_{rt}

(b) Reduction factor (strength), Φ_{rs}

(c) Reduction factor (temperature), Φ_{rst}

(d) Uncertainty factor (degradation), Φ_{ud}

K5.2 The reduction factor (thickness), Φ_{rt}

The reduction factor (thickness), (Φ_{rt}) takes into account the loss of material with time, as follows:

A1

$$\Phi_{rt} = (1 - \Delta A / A_o) \quad \text{. . . K5(1)}$$

where

ΔA = loss of section area

A_o = original section area

For steel reinforcements, loss of section area should be estimated from the following equation:

A1

$$\Delta A = 2 A_c t^n \quad \text{. . . K5(2)}$$

where

A_c = corrosion loss at t = 1 year

t = time in years

n = constant representing rate of change of corrosion rate

The maximum loss of section area over the design life of the structure should be

A1 0.33 × original section area.

K5.3 The reduction factor (strength), (Φ_{rs})

The reduction factor (strength), Φ_{rs}, takes into account the alteration of the material itself and the variability or unevenness of the corrosion or degradation and is related to the reduction factor (thickness) as follows:

A2

$$\Phi_{rt} \, \Phi_{rs} = 1 - K(1 - \Phi_{rt}) \quad \text{. . . K5(3)}$$

where

K is given by the equation

A1

$$\Delta T / T_o = K(\Delta A / A_o) \quad \text{. . . K5(4)}$$

and

ΔT = loss of tensile strength

A1 T_o = original tensile strength

For steel, minimum values of K may be taken as follows:

K = 1.5 for galvanized steel in standard environments

= 2.0 for black steel in standard environments

= 2.5 for black steel in aggressive environments

K5.4 Steel reinforcement

For steel reinforcement, a combined reduction factor (thickness and strength loss), $\Phi_{rt}\ \Phi_{rs}$ may be estimated from the corrosion allowance as per Table K5 (flat strip) and Table K6 (round bar). The corrosion allowance for steel reinforcements is dependent on the properties of the backfill and the environment for a given service life and should be estimated from Appendix J.

NOTE: Corrosion allowance is given by the relevant authorities and takes into account the *K* values of the base materials.

K5.5 Polymer reinforcement

For polymer reinforcements, the reduction factor (thickness) (Φ_{rt}), and the reduction factor (strength) Φ_{rs}, will be in the following ranges:

(a) Reduction factor (thickness) (Φ_{rt})..0.90 to 1.00.

(b) Reduction factor (strength) (Φ_{rs}) ....0.50 to 0.90.

K5.6 The reduction factor (temperatures) (Φ_{rst})

The reduction factor (temperature) should be obtained from manufacturers of the reinforcement and should account for projected temperatures at the depth of placement.

K5.7 The uncertainty factor (degradation) (Φ_{ud})

The uncertainty factor (degradation) may be taken as 0.8 for all materials.

A2

TABLE K5

COMBINED REDUCTION FACTOR ($\Phi_{rt}\ \Phi_{rs}$) FOR FLAT STEEL STRIP

Original thickness (mm)	$\Phi_{rt}\ \Phi_{rs}$ Corrosion allowance (mm)			
	0.5	1.0	1.5	2.0
4	0.87	0.75	0.62	0.50
5	0.90	0.80	0.70	0.60
6	0.92	0.83	0.75	0.67

A1
A2

TABLE K6

COMBINED REDUCTION FACTOR ($\Phi_{rt}\ \Phi_{rs}$) FOR ROUND STEEL BAR

Original diameter (mm)	$\Phi_{rt}\ \Phi_{rs}$ Corrosion allowance (mm)			
	0.5	1.0	1.5	2.0
8	0.88	0.77	0.66	0.56
9	0.89	0.79	0.69	0.60
10	0.90	0.81	0.72	0.67

AMENDMENT CONTROL SHEET

AS 4678—2002

Amendment No. 1 (2003)

REVISED TEXT

SUMMARY: This Amendment applies to the Clauses 1.1, 1.2.2, 1.4.1.22 (new), 1.4.1.23 (new), 1.4.1.24 (new), 1.4.1.25 (new), 1.5, 4.1, 4.2, 5.5.1.2, 5.5.1.3, 5.5.1.4, 5.5.1.6, 5.5.2.2, 5.5.3 and 7, Tables 1.1, 2.1, 4.1, 5.1(A), 5.1(B), 5.2, 5.3, 7.1, A1 (new), A2 (new), B1, E1, I3, K3 and K6, Figures 1.2 (new), 3.1(E), A1, E2, J2, J4, J5 and J6, Paragraphs A3 (new), B4.2.4, E2.3, H2, H3, I5, I14, I15, J2, J3, K5.2 and K5.3.

Published on 7 July 2003.

Amendment No. 2 (2008)

CORRECTION

SUMMARY: This Amendment applies to Clauses 1.5, 5.5.1.6 and Appendices D, G, I, J and K.

Published on 15 August 2008.

NOTES

NOTES

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