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Code of practice for earth retaining structures

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Contents

Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 96, an inside back cover and a back cover.

Foreword

Publishing information

This British Standard is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 30 June 2015. It was prepared by Technical Committee B/526, *Geotechnics*. A list of organizations represented on this committee can be obtained on request to its secretary.

Supersession

Together with BS EN 1997-1:2004+A1:2013, this British Standard supersedes BS 8002:1994, which is withdrawn.

Relationship with other publications

BS 8002 gives non-contradictory, complementary information for use with BS EN 1997 and its UK National Annexes.

Information about this document

This is a full revision of the standard, which introduces the following principal changes:

- the revised text is fully compatible with the current version of Eurocode 7 (BS EN 1997);
- guidance is given on designing earth retaining structures according to limit state principles using partial factors;
- guidance is given on the selection of design parameters for soils;
- guidance is given on model factors to be applied to prop loads determined by calculation;
- the revised text reflects advances in earth retaining structure technology over the past 30 years.

Use of this document

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

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

Presentational conventions

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

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

The word "should" is used to express recommendations of this standard. The word "may" is used in the text to express permissibility, e.g. as an alternative to the primary recommendation of the clause. The word "can" is used to express possibility, e.g. a consequence of an action or an event.

Notes and commentaries are provided throughout the text of this standard. Notes give references and additional information that are important but do not form part of the recommendations. Commentaries give background information.

Contractual and legal considerations

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

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

1 Scope

This British Standard gives recommendations for the design and construction of earth retaining structures to support ground at slopes steeper than the ground would naturally assume. It provides non-contradictory, complementary information for use in conjunction with BS EN 1997 and its UK National Annex.

Clause **4** gives general recommendations for the design and construction of all types of earth retaining structures; Clause **5**, Clause **6**, and Clause **7** give specific recommendations for the design and construction of gravity walls, semi-gravity walls, and embedded walls (respectively); and Clause **8** gives specific recommendations for the design and construction of cofferdams, basements, and strutted excavations.

Annex A gives specific recommendations for the design and construction of deadman anchors.

Annex B gives information about specific geological formations encountered in the UK.

NOTE 1 This standard does not cover the design and construction of anchors (other than deadman anchors), for which see BS 8081.

NOTE 2 This standard does not cover the design and construction of earthworks, for which see BS 6031.

NOTE 3 This standard does not cover the design and construction of foundations, for which see BS 8004.

NOTE 4 This standard does not cover the design and construction of maritime works, for which see BS 6349.

NOTE 5 This standard does not cover the design and construction of earth retaining structures constructed using strengthened or reinforced soil walls, for which see BS 8006.

2 Normative references

Standards publications

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

BS 65, *Specification for vitrified clay pipes, fittings and ducts, also flexible mechanical joints for use solely with surface water pipes and fittings*

BS 437, *Specification for cast iron drain pipes, fittings and their joints for socketed and socketless systems*

BS 4449, *Steel for the reinforcement of concrete – Weldable reinforcing steel – Bar, coil and decoiled product – Specification*

BS 4660, *Thermoplastics ancillary fittings of nominal sizes 110 and 160 for below ground gravity drainage and sewerage*

BS 4729, *Clay and calcium silicate bricks of special shapes and sizes – Recommendations*

BS 4962, *Specification for plastics pipes and fittings for use as subsoil field drains*

BS 5480, *Specification for glass reinforced plastics (GRP) pipes, joints and fittings for use for water supply or sewerage*

BS 5481, *Specification for unplasticized PVC pipe and fittings for gravity sewers*

BS 5642-2, *Sills, copings and cappings – Part 2: Specification for copings and cappings of precast concrete, cast stone, clayware, slate and natural stone*

BS 5837, *Trees in relation to design, demolition and construction – Recommendations*

BS 5911 (all parts), *Concrete pipes and ancilliary concrete products*

BS 5930, *Code of practice for site investigation*

BS 5975, *Code of practice for temporary works procedures and the permissible stress design of falsework*

BS 6031:2009, *Code of practice for earthworks*

BS 6349 (all parts), *Maritime works* [1)]

BS 8004:2015, *Code of practice for foundations*

BS 8006-1:2010, *Code of practice for strengthened/reinforced soils and other fills*

BS 8006-2, *Code of practice for strengthened/reinforced soils – Part 2: Soil nail design*

BS 8081, *Code of practice for ground anchors*

BS 8102, *Code of practice for protection of below ground structures against water from the ground*

BS 8215, *Code of practice for design and installation of damp-proof courses in masonry construction*

BS 8417, *Preservation of wood – Code of practice*

BS 8500-1:2015, *Concrete – Complementary British Standard to BS EN 206-1 – Part 1: Method of specifying and guidance for the specifier*

BS 8500-2 [2)], *Concrete – Complementary British Standard to BS EN 206-1 – Part 2: Specification for constituent materials and concrete*

BS 10175, *Investigation of potentially contaminated sites – Code of practice*

BS EN 206:2013, *Concrete – Specification, performance, production and conformity*

BS EN 295 (all parts), *Vitrified clay pipe systems for drains and sewers*

BS EN 335:2013, *Durability of wood and wood-based products – Use classes: definitions, application to solid wood and wood-based products*

BS EN 350-2, *Durability of wood and wood-based products – Natural durability of solid wood – Part 2: Guide to the natural durability of and treatability of selected wood species of importance in Europe*

BS EN 351-1, *Durability of wood and wood-based products – Preservative-treated solid wood – Part 1: Classification of preservative penetration and retention*

BS EN 460, *Durability of wood and wood-based products – Natural durability of solid wood – Guide to the durability requirements for wood to be used in hazard classes*

BS EN 598, *Ductile iron pipes, fittings, accessories and their joints for sewerage applications – Requirements and test methods*

BS EN 771-1, *Specification for masonry units – Part 1: Clay masonry units*

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1) Specific references are made to the following part: BS 6349-2:2010, *Maritime works – Part 2: General – Code of practice for the design of quay walls, jetties and dolphins*

2) Informative reference is made to BS 8500-2:2015.

BS EN 771-2, *Specification for masonry units – Part 2: Calcium silicate masonry units*

BS EN 771-3, *Specification for masonry units – Part 3: Aggregate concrete masonry units*

BS EN 771-4, *Specification for masonry units – Part 4: Autoclaved aerated concrete masonry units*

BS EN 771-5, *Specification for masonry units – Part 5: Manufactured stone masonry units*

BS EN 771-6, *Specification for masonry units – Part 6: Natural stone masonry units*

BS EN 845-1, *Specification for ancillary components for masonry – Part 1: Wall ties, tension straps, hangers and brackets*

BS EN 1401 (all parts), *Plastic piping systems for non-pressure underground drainage and sewerage*

BS EN 1536, *Execution of special geotechnical works – Bored piles* [3)]

BS EN 1538, *Execution of special geotechnical works – Diaphragm walls* [4)]

BS EN 1852-1, *Plastics piping systems for nonpressure underground drainage and sewerage – Polypropylene (PP) – Part 1: Specifications for pipes, fittings and the system*

BS EN 1916, *Concrete pipes and fittings, unreinforced, steel fibre and reinforced*

BS EN 1990:2002+A1:2005, *Eurocode – Basis of structural design*

BS EN 1991, *Eurocode 1: Actions on structures* [5)]

BS EN 1992 (all parts), *Eurocode 2: Design of concrete structures* [6)]

BS EN 1993 (all parts), *Eurocode 3: Design of steel structures* [7)]

BS EN 1995 (all parts), *Eurocode 5: Design of timber structures* [8)]

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3) Informative reference is made to BS EN 1536:2010.

4) Informative reference is made to BS EN 1538:2010.

5) Specific references are made to the following parts:

- BS EN 1991-1-1:2002, *Eurocode 1: Actions on structures – Part 1-1:Densities, self-weight, imposed loads for buildings;*
- BS EN 1991-1-6:2005, *Eurocode 1: Actions on structures – Part 1-6: General actions – Actions during execution;*
- BS EN 1991-2:2003, *Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges.*

6) Specific references are made to the following parts:

- BS EN 1992-1-1:2004+A1:2014, *Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings;*
- BS EN 1992-3, *Eurocode 2: Design of concrete structures – Part 3: Liquid retaining and containing structures.*

7) Specific references are made to the following parts:

- BS EN 1993-1-1, *Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings*;
- BS EN 1993-5, *Eurocode 3: Design of steel structures – Part 5: Piling*.

8) Specific references are made to the following part: BS EN 1995-1-1, *Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings.*

BS EN 1996 (all parts), *Eurocode 6: Design of masonry structures* [9)]

BS EN 1997-1:2004+A1:2013, *Eurocode 7: Geotechnical design – Part 1: General rules*

BS EN 1997-2:2007, *Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing*

BS EN 10025, *Hot rolled products of structural steels*

BS EN 10080, *Steel for the reinforcement of concrete – Weldable reinforcing steel – General*

BS EN 10210, *Hot finished structural hollow sections of non-alloy and fine grain steels*

BS EN 10218-1:2012, *Steel wire and wire products – General – Part 1: Test methods*

BS EN 10218-2:2012, *Steel wire and wire products – General – Part 2: Wire dimensions and tolerances*

BS EN 10219, *Cold formed welded structural hollow sections of non-alloy and fine grain steels*

BS EN 10223-3:2013, *Steel wire and wire products for fencing and netting – Part 3: Hexagonal steel wire mesh products for civil engineering purposes*

BS EN 10223-8:2013, *Steel wire and wire products for fencing and netting – Part 8: Welded mesh gabion products*

BS EN 10244-2:2009, *Steel wire and wire products – Non-ferrous metallic coatings on steel wire – Part 2: Zinc or zinc alloy coatings*

BS EN 10245-1, *Steel wire and wire products – Organic coatings on steel wire – Part 1: General Rules*

BS EN 10245-2, *Steel wire and wire products – Organic coatings on steel wire – Part 2: PVC finished wire*

BS EN 10245-3, *Steel wire and wire products – Organic coatings on steel wire – Part 3: PE coated wire*

BS EN 10245-5, *Steel wire and wire products – Organic coatings on steel wire – Part 5: Polyamide coated wire*

BS EN 10248, *Hot rolled sheet piling of non alloy steels*

BS EN 10249, *Cold formed sheet piling of non alloy steels*

BS EN 12063, *Execution of special geotechnical work – Sheet pile walls*

BS EN 12666-1, *Plastics piping systems for non-pressure underground drainage and sewerage – Polyethylene (PE) – Part 1: Specifications for pipes, fittings and the system*

BS EN 13369, *Common rules for precast concrete products*

BS EN 13670, *Execution of concrete structures*

BS EN 14199, *Execution of special geotechnical works – Micropiles*

BS EN 15258, *Precast concrete products – Retaining wall elements*

[9)] Specific references are made to the following parts:

- BS EN 1996-1-1, *Eurocode 6: Design of masonry structures – Part 1-1: General rules for reinforced and unreinforced masonry structures;*
- BS EN 1996-2, *Eurocode 6: Design of masonry structures – Part 2: Design considerations, selection of materials and execution of masonry.*

BS EN ISO 1461, *Hot dip galvanized coatings on fabricated iron and steel articles – Specifications and test methods*

BS EN ISO 14688-1, *Geotechnical investigation and testing – Identification and classification of soil – Part 1: Identification and description*

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

BS EN ISO 14689-1, *Geotechnical investigation and testing – Identification and classification of rock – Part 1: Identification and classification*

prEN ISO 22477 (all parts), *Geotechnical investigation and testing – Testing of geotechnical structures* [10)]

BS ISO 5667-11, *Water quality – Sampling – Part 11: Guidance on sampling of groundwaters*

NA to BS EN 1991-2:2003, *UK National Annex to Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges*

NA to BS EN 1997-1:2004+A1:2013, *UK National Annex to Eurocode 7: Geotechnical design – Part 1: General rules*

NA to BS EN 1993-1-1, *UK National Annex to Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings*

NA to BS EN 1993-5, *UK National Annex to Eurocode 3: Design of steel structures – Part 5: Piling*

PD 6694-1:2011, *Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004*

PD 6697:2010, *Recommendations for the design of masonry structures to BS EN 1996-1-1 and BS EN 1996-2*

Other publications

[N1] PREENE, M., ROBERTS, T.O.L., POWRIE, W., and DYER, M.R. *Groundwater control: design and practice,* 2nd edition (CIRIA Report C750). London: CIRIA, 2015, ISBN 978-0860175155.

[N2] BURLAND, J.B., BROMS, B.B., and de MELLO, V.F.B. *Behaviour of foundations and structures*, State-of-the-Art Report, Session 2, Proc. 9th International Conference on Soil Mechanics and Foundation Engineering, 1977, Tokyo, Vol. 2, pp 495–546.

[N3] GABA, A.R., SIMPSON, B., POWRIE, W., and BEADMAN, D.R. *Embedded retaining walls – guidance for economic design* (CIRIA C580), London: CIRIA, 2003, ISBN 0-86017-580-4. [11)]

[N4] INSTITUTION OF CIVIL ENGINEERS. *ICE Specification for piling and embedded retaining walls* (2nd edition, 2007), London: Thomas Telford Publishing, ISBN 978-0-7277-3358-0.

10) In preparation.

11) It is anticipated that a revised version of C580 (with a new number, C760) will be published in 2016.

3 Terms and definitions

For the purposes of this British Standard, the terms and definitions given in BS EN 1990, BS EN 1997, and the following apply.

NOTE Symbols are defined locally to the equations where they are used.

3.1 cofferdam

3.1.1 cofferdam
watertight enclosure pumped dry to permit construction below the waterline

3.1.2 cellular cofferdam
cofferdam consisting of a series of filled cells of circular or other shape in plan

3.1.3 double-wall cofferdam
cofferdam formed by a wall consisting of two parallel walls tied together, with filling between them and which is usually self-supporting against external pressure

3.1.4 land cofferdam
cofferdam constructed from a land surface

3.1.5 water cofferdam
cofferdam constructed in open water

3.2 crib wall

3.2.1 crib wall
retaining wall built up of individual elements to form a series of box-like cells into which infill (which acts as an integral part of the structure) is placed

3.2.2 crib cell
box-like structure formed from headers and stretchers into which infill is placed

3.2.3 header
element that runs normal to the line of a crib wall

3.2.4 stretcher
element that runs along the line of a crib wall

3.2.5 infill
material placed within crib cells

3.2.6 backfill
material located behind a crib wall

3.3 earth pressure coefficients

3.3.1 earth pressure coefficient, K
ratio of horizontal effective stress to vertical effective stress

3.3.2 active earth pressure coefficient, K_a
earth pressure coefficient under active conditions

3.3.3 passive earth pressure coefficient, K_p
earth pressure coefficient under passive conditions

3.3.4 at-rest earth pressure coefficient, K_0
earth pressure coefficient under at-rest conditions

3.4 earth pressure

3.4.1 active earth pressure
minimum value of earth pressure that occurs when the structure moves away from the ground

NOTE The movement required to mobilize active earth pressure is usually within the structure's serviceability limit state.

3.4.2 passive earth pressure
maximum earth pressure generated by the ground when the structure moves towards the ground

NOTE The movement required to mobilize passive earth pressure is often outside the structure's serviceability limit state.

3.4.3 at-rest earth pressure
earth pressure acting on a structure when there is no relative movement between it and the ground

3.5 working platform
temporary structure that provides a foundation for construction plant

4 General rules

4.1 Choice and design of earth retaining structure

4.1.1 General

4.1.1.1 The design of earth retaining structures should conform to BS EN 1997-1 and this clause (**4**).

NOTE Guidance on geotechnical design, construction, and verification can be found in the ICE manual of geotechnical engineering (2012), Volume II *[1].*

4.1.1.2 The following should be taken into account when choosing a suitable earth retaining structure:

- the location of the wall, its position relative to other structures, and the amount of construction space available;
- the necessity or otherwise to confine the support system within the site boundaries;
- the proposed height of the wall and the topography of the ground, both before and after construction;
- ground conditions;
- groundwater, standing water, flood and tidal conditions;
- the extent of acceptable ground movement during construction and in service;
- the effect of movement of the earth retaining structure on existing services and supported structures;
- external live loading;
- the availability of materials;
- appearance;
- the required design life; and
- maintenance requirements.

NOTE 1 Guidance on the selection of a suitable earth retaining structure can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 62 *[1] and in* Earth Pressure and Earth-Retaining Structures (3rd edition), Chapter 6 *[2] and in* CIRIA Report C580 *[N3]* [12)].

NOTE 2 Guidance on geotechnical engineering principles, problematic soils, and site investigation can be found in the ICE manual of geotechnical engineering (2012), Volume I *[3].*

4.1.1.3 The design of deadman anchors should conform to Annex A.

4.1.1.4 The design of maritime works should conform to BS 6349.

4.1.1.5 The design of reinforced soil retaining walls and abutments should conform to BS 8006-1:2010, Section 6.

4.1.2 Gravity retaining walls

The design of gravity retaining walls should conform to **4.1.1.1** and Clause **5**.

NOTE Gravity retaining walls are earth retaining structures that depend primarily on their own self-weight (and that of any enclosed material) to support the retained ground and any structures or loads placed upon it.

4.1.3 Semi-gravity retaining walls

The design of semi-gravity retaining walls should conform to **4.1.1.1** and Clause **6**.

NOTE Semi-gravity retaining walls are earth retaining structures that depend partly on their own self-weight (and that of any enclosed material) and partly on their structural resistance to support the retained ground and any structures or loads placed upon it.

4.1.4 Embedded retaining walls

The design of embedded retaining walls should conform to **4.1.1.1** and Clause **7**.

NOTE Embedded retaining walls are earth retaining structures that depend primarily on lateral earth resistance and their own structural resistance (plus that of any structural supports) to support the retained ground and any structures or loads placed upon it.

4.1.5 Cofferdams

The design of cofferdams should conform to **4.1.1.1** and Clause **8**.

NOTE Cofferdams are composite earth retaining structures that act in some ways like a gravity retaining wall and in other ways like an embedded retaining wall.

4.1.6 Basements and strutted excavations

The design of basements and strutted excavations should conform to **4.1.1.1** and Clause **8**.

NOTE Basements and strutted excavations are underground constructions that incorporate earth retaining structures.

4.2 Basis of geotechnical design

NOTE Attention is drawn to The Construction (Design and Management) Regulations 2015 *[4], with regards to health and safety requirements for construction works, in particular:*

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12) It is anticipated that a revised version of C580 (with a new number, C760) will be published in 2016.

- *Regulation 9, duties of designers;*
- *Regulation 11, duties of the principal designer in relation to the pre-construction phase; and*
- *Regulation 22, excavations.*

4.2.1 Design requirements

4.2.1.1 Geotechnical data and investigation

4.2.1.1.1 Geotechnical data should be obtained in accordance with BS EN 1997-1:2004+A1:2013, Clause **3**.

4.2.1.1.2 Particular attention should be given to the requirements of BS EN 1997-1:2004+A1:2013, **3.1**.

4.2.1.1.3 Geotechnical investigations should conform to BS EN 1997-1:2004+A1:2013, **3.2**.

4.2.1.1.4 Ground investigations should conform to BS EN 1997-2 and BS 5930. Information about the site from the desk study should be used to supplement that obtained in any new ground investigation.

4.2.1.1.5 Sampling and testing of contaminated ground and groundwater should conform to BS 10175 and BS ISO 5667-11.

4.2.1.1.6 The spacing and depth of site investigations for earth retaining structures should conform to BS EN 1997-2:2007, **B.3**, and this subclause (**4.2.1**).

4.2.1.1.7 The location of investigation points for an earth retaining structure should be representative of the line of the structure and should encompass both the retained and the retaining material. The number of investigation points should be sufficient to establish ground conditions, and any variability in those conditions, along the length of the earth retaining structure.

4.2.1.1.8 The spacing of investigation points along the length of an earth retaining structure should be no greater than 20 m, unless the ground conditions can be shown to be sufficiently uniform to justify a greater spacing.

4.2.1.1.9 For gravity and semi-gravity earth retaining structures, the depth of investigation below the planned base of the foundation (z_a) should conform to recommendations given in BS 8004:2015, **4.2.1.1**, for low-rise buildings, namely:

$$z_a \geq \begin{cases} 2b_f \\ 3\text{m} \end{cases} \tag{1}$$

where:

b_f is the smaller side length of the wall's foundation (on plan).

4.2.1.1.10 For embedded earth retaining structures, the depth of investigation below the base of the excavation (z_a) should satisfy:

$$z_a \geq \begin{cases} 2H & \text{for an unsupported structure} \\ H + 2\text{ m} & \text{for a supported structure} \end{cases} \tag{2}$$

where:

H is the retained height.

4.2.1.1.11 If ground anchors are proposed to support an embedded retaining wall, the investigation should be of sufficient depth and lateral extent to provide data on the ground in which the anchors attain their bond lengths in accordance with BS 8081.

4.2.1.1.12 The presence of trees and large shrubs should be recorded during the course of the ground investigation, so that decisions can be taken concerning their retention or subsequent removal.

NOTE Guidance on site investigation can be found in the ICE manual of geotechnical engineering (2012), Volume I, Section 4: Site investigation *[3] and in the* NHBC Design Guide, Efficient design of piled foundations for low rise housing (2010), Section 4: Site investigation [5].

4.2.2 Design situations

4.2.2.1 Design situations should be specified in accordance with BS EN 1997-1:2004+A1:2013, **2.2**.

4.2.2.2 Design situations for earth retaining structures should include:

- the conditions given in BS EN 1997-1:2004+A1:2013, **9.3.3**;
- the geometry of the structure and neighbouring ground;
- material characteristics of the structure, e.g. following corrosion;
- effects due to the environment within which the design is set;
- groundwater levels, including their variations due to the effects of dewatering, possible flooding, or failure of any drainage system;
- overdredging of river or sea beds;
- deepening of river or sea beds due to scour;
- increase in hydrostatic head due to drawdown; and
- water levels in the tidal or flood range.

To conform to BS EN 1990, exceptional conditions involving local failure (such as a burst water pipe that continues to leak after the burst) should be classified as accidental design situations.

4.2.3 Design considerations

*COMMENTARY ON **4.2.3***

*Design and construction considerations for earth retaining structures are given in BS EN 1997-1:2004+A1:2013, **9.4.1**. The design considerations given in this subclause also include more specific examples of the issues that can affect the performance of earth retaining structures.*

4.2.3.1 General

The design of an earth retaining structure should consider:

- the design considerations given in BS EN 1997-1:2004+A1:2013, **9.4.1**;
- the influence of tidal and flood conditions on groundwater, both for waterfront structures and also for near-shore structures;
- the maximum tidal and flood range to waterfront structures;
- potential or possible surge tides and flood conditions;
- the height, length and angle of approach of waves, and the resulting forces on the structure;
- softening and subsequent loss of strength of fine-grained backfill behind the retaining structure;
- softening and subsequent loss of strength of fine soil beneath formation level in front of the retaining structure;
- ingress of water into fissures formed during hot, dry spells;

- the effect of frost action behind the structure;
- the effect on the earth pressures acting against the structure owing to climatic variations, including:
  - diurnal and seasonal temperature changes (particularly ground freezing);
  - short- and long-term rainfall variations and resulting moisture content changes in the ground;
  - artificially induced climatic changes such as those produced in boiler houses or cold stores;
- trafficking by heavy plant;
- seepage flow around the structure, taking into account layers of markedly different permeability; and
- the possibility that the level of any compacted fill placed against the back of an earth retaining structure might differ during construction from the level of any adjoining fill.

4.2.3.2 Drainage

*COMMENTARY ON **4.2.3.2***

Provision of suitable drainage is vital to ensure the acceptable performance of an earth retaining structure. Groundwater control is separate from the control of surface water, such as rainfall run-off.

4.2.3.2.1 Drainage should be provided to prevent:

- surface water from entering and eroding the face of any excavations;
- build-up of excessive water pressures during construction in case they have harmful effects upon the foundations;
- instability of slopes.

4.2.3.2.2 Measures taken to prevent groundwater entering a structure should conform to BS 8102. Alternatively, a properly designed and ventilated air space may be provided.

4.2.3.2.3 Sub-surface drainage used to lessen the risk of water ingress into a structure should be designed in accordance with BS 8102.

4.2.3.2.4 Surface water drains should be constructed using one of the types of pipe listed in **4.3.10**.

4.2.3.2.5 Drainage systems should be designed for ease of maintenance and renewal during the design working life of the structure.

4.2.3.2.6 Where the safety and serviceability of the works depend on the successful performance of the drainage system, the consequences of failure should be considered and one of the following conditions (or a combination of them) should be applied:

- a maintenance programme specified;
- a drainage system specified that will perform adequately without maintenance; or
- a secondary (“backup”) system specified – for example, a pipe or channel that encloses the primary system – that will prevent any potential leakage from entering the ground beneath or next to the structure.

4.2.3.2.7 The design of drainage systems for earth retaining structures should consider:

- the design considerations given in BS EN 1997-1:2004+A1:2013, **9.4.2**;
- changes in groundwater levels (temporary or permanent) owing to construction of the structure.

4.2.3.2.8 A drainage layer might be required to prevent one or more of the phenomena listed in **4.2.3.2.1**. Wherever practical, a structure that retains cohesive soil or backfill of medium to low permeability (2×10^{-5} m/s or less) should be provided with a drainage layer at its back face.

4.2.3.2.9 Drainage layers may be constructed from:

- a blanket of rubble or coarse aggregate, clean gravel, or crushed stone;
- hand-placed pervious blocks as dry walling;
- a graded filter drain, where the backfilling consists of fine-grain material;
- a geotextile filter, in combination with a permeable granular material; or
- a geotextile composite (fin-drain), consisting of a geotextile filter fixed to one or both faces of a permeable core.

4.2.3.2.10 The design of filters should conform to CIRIA C750 (2015), Section 3.3 [N1].

4.2.3.2.11 A drainage system should be designed so that water freely exits from drainage layers, through porous land drains or pipes laid at the bottom of the drainage layers, or through weepholes.

4.2.3.2.12 Any impermeable wall or facing should have weepholes provided unless an alternative drainage system is provided or they are designed to resist the water pressure.

NOTE 1 Guidance on surface water control can be found in Groundwater lowering in contruction: a practical guide *[6],* Construction dewatering and groundwater control: new methods and applications *[7], and* CIRIA C750 *[N1].*

NOTE 2 Guidance on groundwater control can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 80 *[1].*

4.2.3.3 Cyclic loading

The following should be taken into account during the design of earth retaining structures subject to cyclic loading:

- degradation of ground strength (leading to ultimate limit states being exceeded at loads below those expected from verifications based on static strength);
- degradation of ground stiffness, leading to an accumulation of permanent foundation displacement ("ratcheting" effects); and
- amplification of loads or movements owing to resonance.

NOTE Guidance on foundations subjected to cyclic and dynamic loads can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 60 *[1].*

4.2.3.4 Trees

*COMMENTARY ON **4.2.3.4***

Earth retaining structures built adjacent to existing trees might suffer deleterious effects from the penetration of root-systems. These effects include increased loading on the structure and penetration of roots into joints or drainage systems.

4.2.3.4.1 The principles and procedures to be applied to achieve a harmonious and sustainable relationship between trees (including shrubs and hedges) and structures should conform to BS 5837.

4.2.3.4.2 Earth retaining structures should be designed to accommodate any volumetric changes in clay soils that might be caused by the presence of nearby trees.

NOTE Guidance on building near trees can be found in NHBC Standards, Chapter 4.2 *[8].*

4.2.3.5 Environmental considerations

The following should be taken into account during the design of earth retaining structures:

- their effect on sensitive species;
- generation and control of noise and dust during construction;
- generation, reuse and disposal of waste materials;
- minimizing the amount of material to be disposed of;
- contaminants entering watercourses (for example, as a result of excavation);
- reuse of clayey soils and weak rocks (such as marls and shales) might require particular attention and expedients, including treatment with cement;
- management of water on site;
- stability of earthworks;
- generation and control of contaminated run-off;
- managing vegetation, especially the risks associated with allowing vegetation to grow unchecked; and
- the carbon footprint of the construction and the use of the structure.

4.2.3.6 Ground improvement

Guidance on ground improvement can be found in BS 8004.

4.3 Materials

NOTE For guidance on specific formations (e.g. London clay, Lambeth group, glacial soils and tills, problematic soils, chalk and Mercia mudstone group), see Annex B.

4.3.1 Soils

4.3.1.1 General

*COMMENTARY ON **4.3.1.1***

The design of earth retaining structures usually involves effective stress analysis, although, in some circumstances ,total stress analysis might be appropriate or necessary for the design of foundations in fine soils. Soil properties are determined as part of the site investigation process but might be supplemented by data from back analysis of comparable earth retaining structures in similar ground conditions.

4.3.1.1.1 The identification and description of soil should conform to BS EN ISO 14688-1.

4.3.1.1.2 The classification of soil should conform to BS EN ISO 14688-2.

4.3.1.1.3 Soil properties should be determined in accordance with BS EN 1997-2 and BS 5930.

4.3.1.1.4 Characteristic soil parameters should be selected in accordance with BS EN 1997-1, based on the results of field and laboratory tests, complemented by well-established experience.

NOTE 1 Guidance on soil description can be found in Soil and rock description in engineering practice [9].

NOTE 2 Information about the behaviour of soils as particulate materials can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 14 *[3].*

NOTE 3 Information about the strength and deformation behaviour of soils can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 17 *[3].*

4.3.1.2 Very coarse soils (cobbles and boulders)

*COMMENTARY ON **4.3.1.2***

Very coarse soils contain a majority (by weight) of particles >63 mm in size. Cobbles are between 63 mm and 200 mm in size; boulders are greater than 200 mm.

Very coarse soils should be identified and classified in accordance with BS EN ISO 14688 and BS 5930.

4.3.1.3 Coarse soils (sands and gravels)

*COMMENTARY ON **4.3.1.3***

Coarse soils contain a majority (by weight) of particles ≤63 mm in size and do not stick together when wet. Sand particles are between 0.063 mm and 2 mm in size; gravels are between 2 mm and 63 mm. Coarse soils cannot be remoulded.

4.3.1.3.1 When establishing the values of parameters for coarse soils, the following should be considered:

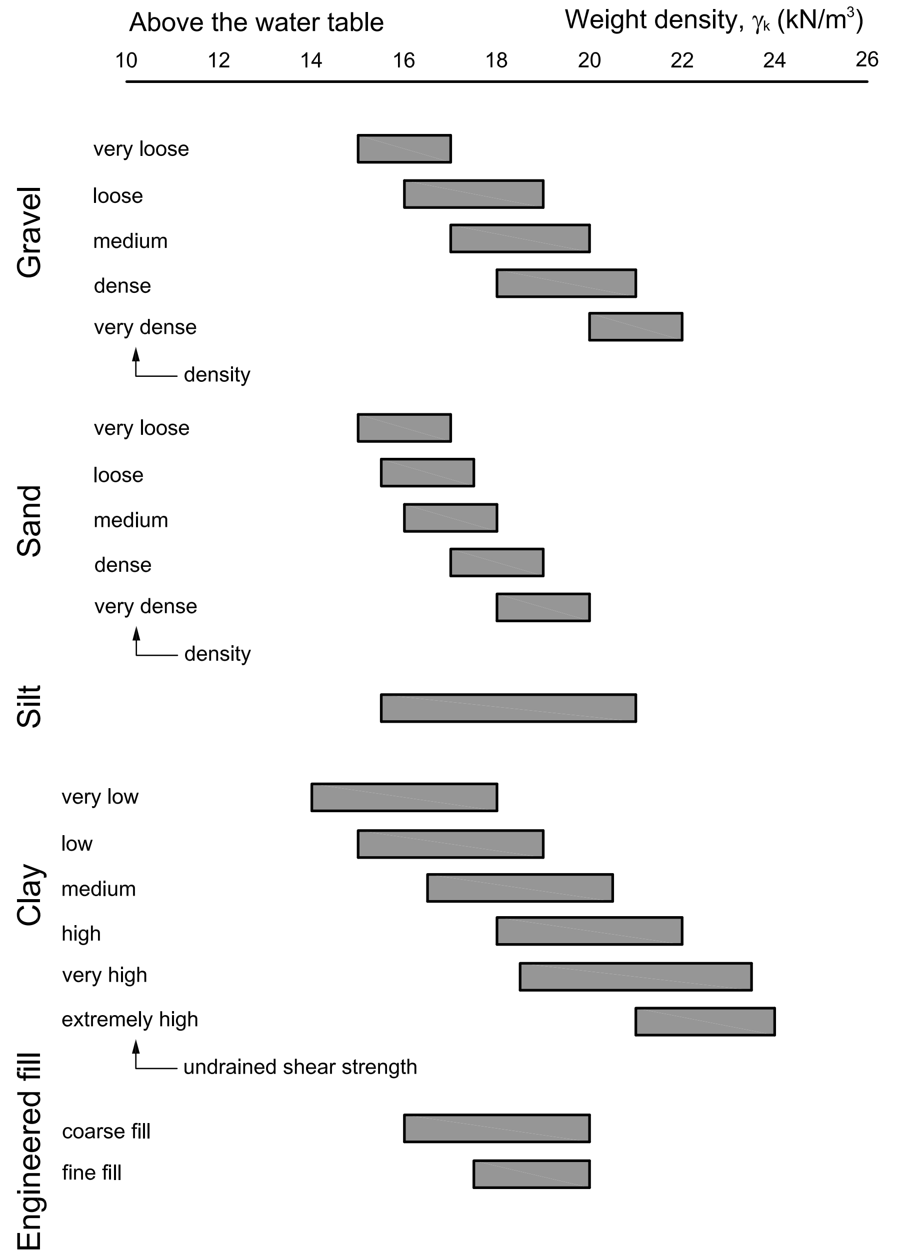
- the items listed in BS EN 1997-1:2004+A1:2013, **2.4.3**(5);
- the weakening of collapsible soils above the groundwater table, owing to percolation or a rise in groundwater levels;
- disturbance of dense deposits owing to unsuitable construction methods; and
- the presence of weaker material.

4.3.1.3.2 For coarse soils above the groundwater table, the suggested values for characteristic weight density given in Figure 1 may be used in the absence of reliable test results.

4.3.1.3.3 For coarse soils below the groundwater table, the suggested values for characteristic weight density given in Figure 2 may be used in the absence of reliable test results.

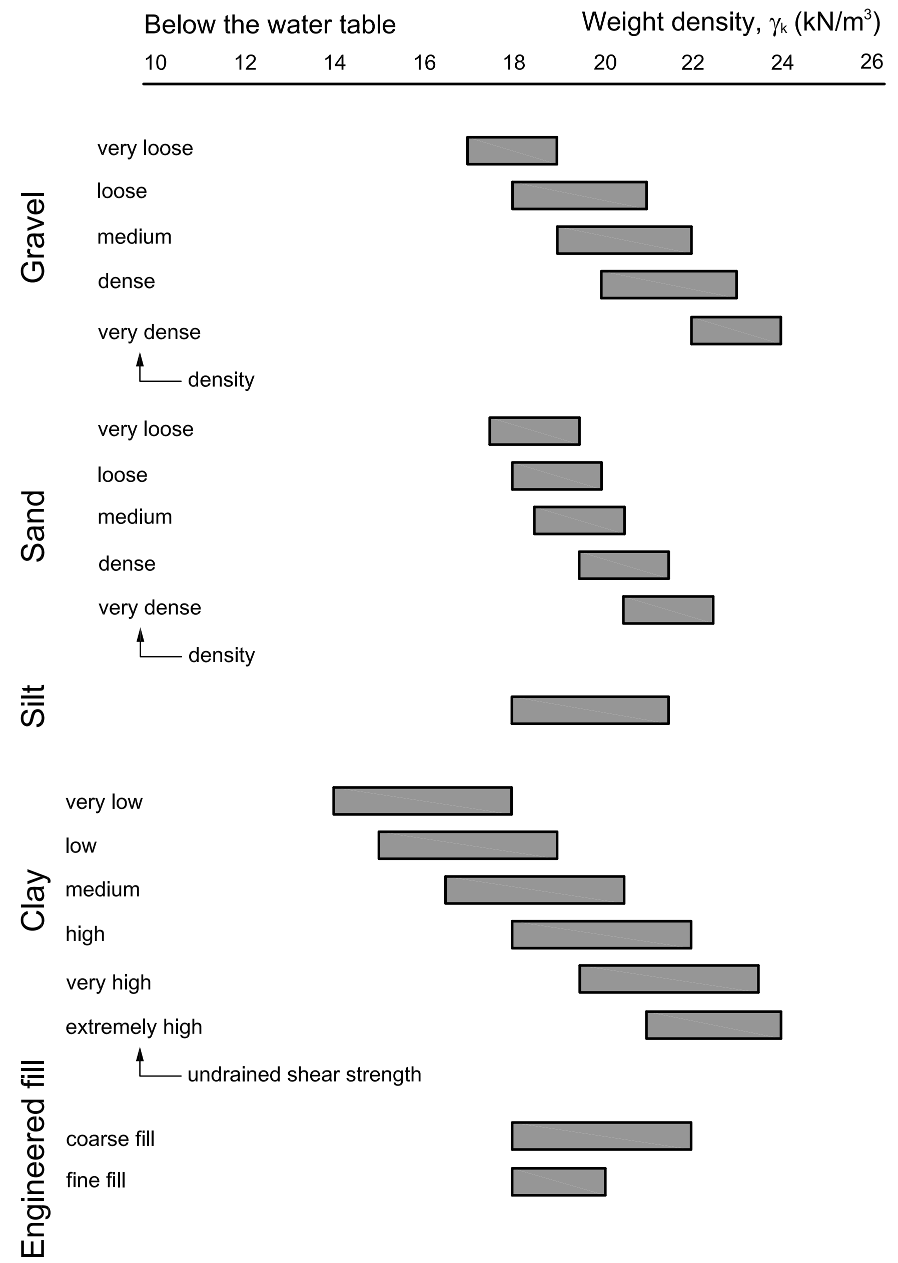
4.3.1.3.4 A superior value of weight density should be selected when a high value is unfavourable; an inferior value should be selected when a low value is unfavourable.

Figure 1 **Suggested values for characteristic weight density of soils above the groundwater table**



Source: Data from Bond [10].

Figure 2 **Suggested values for characteristic weight density of soils below the groundwater table**



Source: Data from Bond [10].

4.3.1.3.5 For siliceous sands and gravels, the characteristic constant volume (also known as critical state) effective angle of shearing resistance ($\varphi'_{cv,k}$) may – in the absence of reliable test results – be estimated from:

$$\varphi'_{cv,k} = 30° + \varphi'_{ang} + \varphi'_{PSD} \tag{3}$$

where:

φ'_{ang} is contribution to $\varphi'_{cv,k}$ from the angularity of the particles; and

φ'_{PSD} is contribution to $\varphi'_{cv,k}$ from the soil's particle size distribution.

Values of φ'_{ang} and φ'_{PSD} are given in Table 1.

4.3.1.3.6 For siliceous sands and gravels with a fines content less than 15%, the characteristic peak effective angle of shearing resistance ($\varphi'_{pk,k}$) may be estimated from:

$$\varphi'_{pk,k} = \varphi'_{cv,k} + \varphi'_{dil} \tag{4}$$

where:

φ'_{dil} is contribution to $\varphi'_{pk,k}$ from soil dilatancy.

Values of φ'_{dil} are given in Table 1.

4.3.1.3.7 The value of φ'_{dil} may alternatively be estimated from:

$$\varphi'_{dil} = nI_R = n\left[I_D \times \ln\left(\sigma_c/\sigma'_f\right) - 1\right] \tag{5}$$

where:

n is 3 for triaxial strain or 5 for plane strain;

I_R is the soil's relative dilatancy index;

I_D is the soil's density index (defined in BS EN ISO 14688-2);

σ_C is the aggregate crushing stress; and

σ'_f is mean effective stress in the soil at peak strength.

NOTE 1 Bolton [11] defines the relative dilatancy index as $I_R = I_D(Q - l_n\ p') - 1$, where $Q = l_n\ \sigma_c$ and $p' = \sigma'_f$.

NOTE 2 Many geotechnical problems can be simplified into a two-dimensional form where the foundation or structure is significantly long in one direction in comparison with other dimensions. Hence, a large number of stability problems involving embankments and cuttings, retaining walls and strip footings are commonly analysed by assuming a plane strain condition in which no deformation occurs in the direction of the long dimension of the foundation or structure.

4.3.1.3.8 The value of σ_C may be taken as 20 MPa for quartz sands, but can be substantially larger for quartz silts, and substantially smaller for carbonate sands (see *The strength and dilatancy of sands* [11]).

4.3.1.3.9 If the fines content of the coarse soil exceeds 25%, then φ'_{dil} should be assumed to be zero, unless testing demonstrates otherwise. For coarse soils with fines content between 15% and 25%, φ'_{dil} may be determined by linear interpolation.

4.3.1.3.10 If shearing is matrix (i.e. fines) controlled, then φ'_{ang} should be assumed to be zero, unless testing demonstrates otherwise. For coarse soils with fines content between 15% and 25%, φ'_{ang} may be determined by linear interpolation.

4.3.1.3.11 The characteristic angle of shearing resistance (φ'_k) for coarse soils with fines content exceeding 25% should be determined as for fine soils (see **4.3.1.4**).

Table 1 **Values of φ'_{ang}, φ'_{PSD} and φ'_{dil} to obtain values of $\varphi'_{pk,k}$ and $\varphi'_{cv,k}$ for siliceous sands and gravels with fines content not exceeding 15%**

Soil property	Determined from	Classification	Parameter D)
Angularity of particles A)	Visual description of soil	Rounded to well-rounded	$\varphi'_{ang} = 0°$
		Sub-angular to sub-rounded	$\varphi'_{ang} = 2°$
		Very angular to angular	$\varphi'_{ang} = 4°$
Uniformity coefficient, C_U B)	Soil grading	$C_U < 2$ (evenly graded)	$\varphi'_{PSD} = 0°$
		$2 \le C_U < 6$ (evenly graded)	$\varphi'_{PSD} = 2°$
		$C_U \ge 6$ (medium to multi graded)	$\varphi'_{PSD} = 4°$
		High C_U (gap graded), with C_U of fines < 2 E)	$\varphi'_{PSD} = 0°$
		High C_U (gap graded), with $2 \le C_U$ of fines < 6E)	$\varphi'_{PSD} = 2°$
Density index, I_D C)	Standard penetration test blow count, corrected for energy rating and overburden pressure $(N_1)_{60}$	$I_D = 0\%$	$\varphi'_{dil} = 0°$
		$I_D = 25\%$	$\varphi'_{dil} = 0°$
		$I_D = 50\%$	$\varphi'_{dil} = 3°$
		$I_D = 75\%$	$\varphi'_{dil} = 6°$
		$I_D = 100\%$	$\varphi'_{dil} = 9°$

A) Terms for defining particle shape can be found in BS EN ISO 14688-1.
B) The uniformity coefficient C_U is defined in BS EN ISO 14688-2.
C) The density index I_D is defined in BS EN ISO 14688-2. Density terms may be estimated from the results of field tests (e.g. Standard Penetration Test, Cone Penetration Test) using correlations given in BS EN 1997-2.
D) Values of φ'_{dil} are appropriate for siliceous sands and gravels reaching failure at a mean effective stress up to 400 kPa. For non-siliceous sands, see *The strength and dilatancy of sands* [11].
E) "Fines" refers to that fraction of the soil whose particle size is less than 0.063 mm.

4.3.1.4 Fine soils (silts and clays)

*COMMENTARY ON **4.3.1.4***

Fine soils contain a majority (by weight) of particles ≤63 mm in size and stick together when wet. Silt particles are between 0.002 mm and 0.063 mm in size; clay particles are smaller than 0.002 mm. Fine soils can be remoulded.

Clay soils with plasticity indices greater than about 20% might exhibit considerably lower angles of shearing resistance than observed at the critical state, if their particles become fully aligned with one another. This phenomenon is termed "sliding shear" to distinguish it from "rolling shear" observed in other soils (including coarse soils and fine soils with plasticity indices less than 20%). The angle of shearing resistance exhibited during sliding shear is called the "residual angle of shearing resistance".

4.3.1.4.1 When establishing the values of parameters for fine soils, the following should be considered, as a minimum:

- the items listed in BS EN 1997-1:2004+A1:2013, **2.4.3**(5);
- pre-existing slip surfaces;
- dessication; and
- any changes in stress state, either induced by construction or resulting from the final design condition.

4.3.1.4.2 For fine soils above the groundwater table, the suggested values for characteristic weight density given in Figure 1 may be used in the absence of reliable test results.

4.3.1.4.3 For fine soils below the groundwater table, the suggested values for characteristic weight density given in Figure 2 may be used in the absence of reliable test results.

4.3.1.4.4 A superior value of weight density should be selected when a high value is unfavourable; an inferior value should be selected when a low value is unfavourable.

4.3.1.4.5 In the absence of reliable test data, the characteristic undrained shear strength of a fine soil ($c_{u,k}$) may be estimated from:

$$\frac{c_{u,k}}{p'_v} = k_1 R_O^{k_2} = k_1 \left(\frac{p'_{v,max}}{p'_v} \right)^{k_2} \qquad (6)$$

where:

p'_v is the effective overburden pressure in the ground;

$p'_{v,max}$ is the maximum effective overburden pressure that the soil has previously been subjected to;

R_O is the soil's overconsolidation ratio; and

k_1 and k_2 are constants.

NOTE The ratio c_u/p'_v normally varies with depth (i.e. it is not a constant).

4.3.1.4.6 In the absence of reliable test data, the values of k_1 and k_2 in equation (6) may be taken as 0.23 ±0.04 and 0.8 respectively (following *New developments in field and laboratory testing of soils* [12]).

4.3.1.4.7 When determining the characteristic undrained strength of high strength fine soils, due allowance should be made for:

- the detrimental effect of any sand or silt partings containing free groundwater;
- the influence of sampling;
- the influence of the method of testing; and
- likely softening on excavation.

4.3.1.4.8 For fine soils, the characteristic constant volume (also known as critical state) effective angle of shearing resistance ($\varphi'_{cv,k}$) may, in the absence of reliable test results, be estimated from:

$$\varphi'_{cv,k} = \left(42° - 12.5 \log_{10} I_P\right) \quad \text{for} \quad 5\% \leq I_P \leq 100\% \qquad (7)$$

where:

I_P is the soil's plasticity index (entered as a %).

NOTE 1 Equation (7) is based on an expression proposed by Santamarina and Díaz-Rodriguez [13], which fits data presented by Terzaghi, Peck, and Mesri [14].

NOTE 2 Values of $\varphi'_{cv,k}$ based on this expression are given in Table 2.

Table 2 **Values of $\varphi'_{cv,k}$ for fine soils from plasticity index**

Plasticity index, I_P	**Characteristic constant volume angle of shearing resistance** $\varphi'_{cv,k}$
%	degrees (°)
15	27
30	24
50	21
80	18

NOTE Values of φ'_{cv} in excess of 40° have been observed for clays that classify as highly plastic but show signs of bioturbation or the presence of microfossils.

4.3.1.4.9 The characteristic constant volume effective cohesion ($c'_{cv,k}$) should be taken as zero.

4.3.1.4.10 The peak effective angle of shearing resistance (φ'_{pk}) may be related to the constant volume effective angle of shearing resistance (φ'_{cv}) by:

$$\varphi'_{pk} = \varphi'_{cv} + \varphi'_{dil} \tag{8}$$

where:

φ'_{cv} is the soil's constant-volume angle of shearing resistance; and

φ'_{dil} is the contribution to $\varphi'_{pk,k}$ from soil dilatancy.

NOTE 1 The value of φ'_{dil} for fine soils is not the same as that for coarse soils. For fine soils, it is typically in the range of 0°–4°. No specific guidance is given in this standard for values of φ'_{dil} for fine soils.

NOTE 2 Values of φ'_{dil} are known to increase with a fine soil's overconsolidation ratio and are greater than or equal to zero.

4.3.1.4.11 When a clay soil is able to undergo "sliding shear" – normally only where pre-existing slip surfaces exist in the ground – then the operational angle of shearing resistance is the clay's residual value (φ'_{res}):

$$\varphi'_{res} \le \varphi'_{cv} \le \varphi'_{pk} \tag{9}$$

where:

φ'_{cv} is the soil's constant-volume angle of shearing resistance; and

φ'_{pk} is the soil's peak angle of shearing resistance.

NOTE Guidance on the undrained strength and the residual shear strength of clay soils can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 17 *[3].*

4.3.1.5 Mixed soils

*COMMENTARY ON **4.3.1.5***

Some deposits (especially glacial deposits) can comprise a mixture of fine and coarse soils, e.g. sandy clays and clayey sands. The behaviour of mixed soils is typically intermediate between that of coarse and fine soils.

For mixed soils with clay fraction less than 50% (plasticity index less than 30% or liquid limit less than 60%), $\varphi'_{cv,k}$ may, in the absence of reliable test data, be estimated from *Drained shear strength parameters for analysis of landslides* [15].

4.3.1.6 Soil stiffness

*COMMENTARY ON **4.3.1.6***

Young's modulus of elasticity of an isotropic soil (E) is related to its shear modulus (G) by:

$$E = 2G(1+v) \tag{10}$$

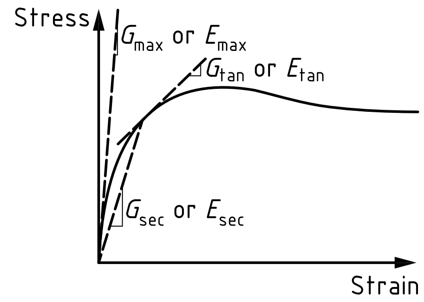
where:

v *is the soil's Poisson's ratio.*

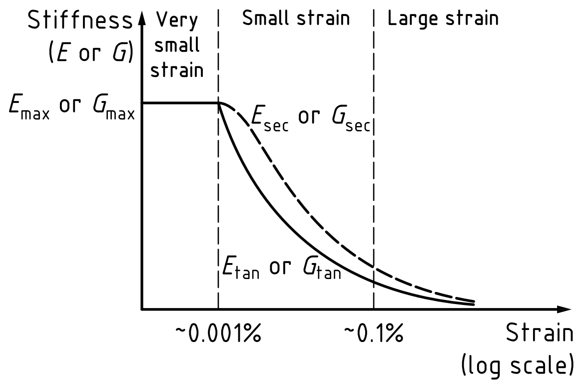
Stiffness parameters for a soil depend on the level of strain (axial ε or engineering shear strain γ) applied to the soil, as illustrated in Figure 3:

- *at very small strains (ε or $\gamma \leq \sim 0.001$ %), the soil's moduli of elasticity (E and G) reach their maximum values (E_{max} and G_{max}), also known as "very-small-strain" values; E_{max} and G_{max} are normally measured using dynamic methods, such as laboratory tests using bender-elements or field tests using seismic methods;*
- *at small strains (~0.001 % $\leq \varepsilon$ or $\gamma \leq$~0.1%), E and G decrease rapidly with increasing strain, as shown in Figure 3; their values are measured using advanced methods, such as laboratory tests with local gauges;*
- *tangent values of small-strain stiffness (E_{tan} and G_{tan}) are commonly used in numerical methods of design;*
- *at large strains (ε or $\gamma >$~0.1%), E and G decrease less rapidly with increasing strain and are commonly taken to be constant in value; those values are measured using conventional laboratory testing;*
- *secant values of large-strain stiffness (E_{sec} and G_{sec}) are commonly used in routine methods of design;*
- *soil stiffness parameters are not normally isotropic. Simple isotropic models are to be used with caution, especially if their adequacy has not been demonstrated by back analysis. Depending on the soil's deposition history, different values of stiffness might be appropriate for vertical strains (E_v) compared to horizontal strains (E_h) and for shearing in different directions (G_{hh}, G_{hv}).*

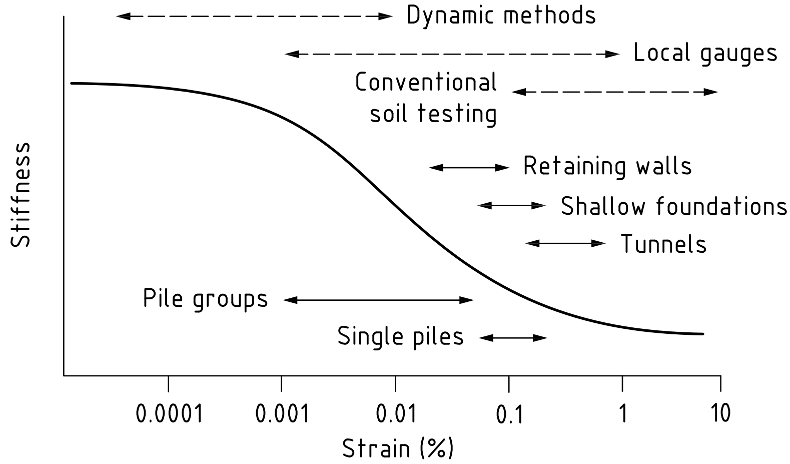
Figure 3 **Stiffness parameters for non-linear soil**



a) Definition of tangent and secant moduli



b) Definition of very small, small and large strain stiffnesses



c) Typical strain ranges for common geotechnical constructions (solid arrows) and laboratory tests (dashed arrows)

NOTE Based on Non-linear soil stiffness in routine design *[16] and the* ICE manual of geotechnical engineering (2012), Volume II, Chapter 52 *[1].*

4.3.1.6.1 The difference between the direction of loading and the direction of measurement of soil stiffness should be taken into account in the assessment of soil stiffness.

4.3.1.6.2 The secant shear modulus of a soil, G_{sec}, may be estimated from (see also *Stiffness of sands through a laboratory database* [17]):

$$\frac{G_{sec}}{G_{max}} = \left[1 + \left(\frac{\gamma - \gamma_e}{\gamma_{ref}}\right)^m\right]^{-1} \leq 1 \qquad (11)$$

where:

G_{max} is the soil's very-small-strain shear modulus;

γ is the engineering shear strain in the soil;

γ_e is the elastic threshold strain beyond which shear modulus falls below its maximum value;

γ_{ref} is a reference value of engineering shear strain (at which G_{sec}/G_{max} = 0.5); and

m is a coefficient that depends on soil type.

4.3.1.6.3 In the absence of reliable test results, the values of the parameters in equation (11) may be taken from Table 3.

Table 3 **Values of parameters for use with equation (11)**

Soil type	**Parameter**			**Reference**
	γ_{ref} %	m	γ_e %	
Sand	0.02–0.1 (0.044[A)])	0.88	0.02% + 0.012 γ_{ref}	Oztoprak and Bolton [18]
Clays and silts	0.0022 I_P [B)]	0.736 ±0.122 [C)]	0 (assumed)	Vardanega and Bolton [19]

[A)] Mean value.
[B)] I_P is the soil's plasticity index.
[C)] ± value indicates standard error.

4.3.1.6.4 In the absence of reliable test results, the very-small-strain shear modulus of a soil G_{max} may be estimated from (see *Non-linear soil stillness in routine design* [16] and *Stiffness at small strain: research and practice* [20]):

$$\frac{G_{max}}{p_{ref}} = \frac{k_1}{(1+e)^{k_2}}\left(\frac{p'}{p_{ref}}\right)^{k_3} \qquad (12)$$

where:

e is the soil's voids ratio;

p' is the mean effective stress in the soil;

p_{ref} is 100 kPa; and

k_1, k_2, and k_3 are coefficients that depend on soil type.

4.3.1.6.5 In the absence of reliable test results, the values of the parameters in equation (12) may be taken from Table 4.

Table 4 **Values of parameters for use with equation (12)**

Soil type	Parameter				Reference
	k_1	k_2	k_3	p_{ref}	
Fine soil	2 100 A)	0	0.6–0.8 A)	1 kPa	Viggianni and Atkinson [21]
Sand	370–5 760 B)	3	0.49–0.86 C)	100 kPa	Oztoprak and Bolton [18]
Clays and silts	20 000 ±5 000	2.4	0.5	1 kPa	Vardanega and Bolton [19]

A) Depends on the soil's plasticity index, I_P.
B) Decreasing with strain.
C) Increasing with strain.

4.3.2 Rocks and rock masses

*COMMENTARY ON **4.3.2***

The engineering properties of rock relevant in design are controlled by the extent and orientation of the bedding planes and joints within the rock mass, together with the water pressures on the discontinuity planes. The site investigation needs to establish the strength and orientation of the discontinuity planes.

Weak rocks, particularly weakly cemented sandstones, fissured shales and chalk, are often difficult materials to sample and test.

4.3.2.1 The identification and classification of rock should conform to BS EN ISO 14689-1.

4.3.2.2 Rock properties should be determined in accordance with BS EN 1997-2 and BS 5930, as well as BS EN 1997-1:2004+A1:2013, **3.3.2**, **3.3.8** and **3.3.9**.

4.3.2.3 Characteristic rock parameters for intact rock should be selected in accordance with BS EN 1997-1, based on the results of field and laboratory tests, complemented by well-established experience.

4.3.2.4 Design parameters for the rock mass should take into account the properties of the intact rock and any discontinuities. See BS EN 1997-1:2004+A1:2013, **3.3.8**.

4.3.2.5 The following non-destructive tests may be used to determine rock mineralogy and composition in order to predict the rock's performance during excavation (in particular, if it will slurry or smear rather than be broken up into smaller pieces):

- microscope;
- X-ray computerized tomography; and
- spectroscopy.

NOTE 1 Guidance on rock description in engineering practice can be found in Soil and rock description in engineering practice *[9].*

NOTE 2 Information about the behaviour of rocks can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 18 *[3].*

NOTE 3 Information about mudrocks, clay, and pyrite (and their issues) can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 36 *[3].*

4.3.3 Fill

*COMMENTARY ON **4.3.3***

The term "fill" refers to artificially deposited material, for example, in an excavation or as ground made by human activity. These materials are also termed "artificial ground".

"Non-engineered" fill is material that is dumped with little control and in deep lifts. It is often poorly compacted, and thus in a loose state, and has varying geotechnical properties, both horizontally and vertically. Non-engineered fill is commonly referred to as "made ground".

"Engineered" fill is material that is placed with some degree of control to ensure that its geotechnical properties conform to a predetermined specification. Engineered fill is commonly referred to as just "fill".

On geological maps, "made ground" refers to material placed above and "fill" to material placed below original ground level. On recent British Geological Survey (BGS) maps, these terms have been altered to "made up ground" and "infilled ground".

4.3.3.1 All materials intended to be placed behind earth retaining structures should be properly investigated and classified.

4.3.3.2 Non-engineered fill, such as industrial, chemical, and domestic wastes, should not be placed behind earth retaining structures.

4.3.3.3 Engineered fill comprising selected coarse granular soils, such as well-graded small rockfills, gravels, and sands, may be placed behind earth retaining structures.

4.3.3.4 Engineered fill should be appropriate to the intended application. The fill should be classified and an earthworks specification provided detailing acceptability criteria, compliance testing, and compaction requirements. The general specification of earthworks fill should conform to BS 6031.

NOTE 1 Guidance on the description of made ground can be found in Soil and rock description in engineering practice, Chapter 14 *[9].*

NOTE 2 Information about non-engineered fills can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 34 *[3].*

NOTE 3 Information about fill formation and deposits, covering opencast mining backfill, colliery spoil, pulverized fly ash, industrial and chemical wastes, urban fill, domestic refuse, infilled docks, pits, and quaries, and hydraulic fill, can be found in BRE Report 424 (2nd edition), Chapter 2 *[22].*

4.3.4 Earthworks

The properties of earthworks should be determined in accordance with BS 6031.

4.3.5 Groundwater

4.3.5.1 Groundwater pressures should be determined by considering hydrological, hydrogeological and environmental information.

4.3.5.2 To conform to BS EN 1997-1:2004+A1:2013, **2.4.6.1**(6)P, design values of groundwater pressure at the serviceability limit state should be the most unfavourable values that could occur during normal circumstances.

4.3.5.3 To conform to BS EN 1997-1:2004+A1:2013, **2.4.6.1**(6)P, design values of groundwater pressure at the ultimate limit state should be the most unfavourable values that could occur during the design lifetime of the structure.

NOTE The relationship between characteristic and design water pressures can be determined by applying a geometrical margin.

4.3.5.4 If suitable statistical data are available, then design values of groundwater pressure at the serviceability limit state should be chosen with a return period at least equal to the duration of the design situation, and design values at the ultimate limit state should be chosen such that there is a 1% probability that they are exceeded during the design situation.

4.3.5.5 Where groundwater pressures are not hydrostatic, the design should take into account:

- the worst credible combination of heterogeneity and anisotropy of permeability;
- effects of layering, fissuring and other heterogeneity;
- any geometrical features that could cause pressures to concentrate (such as in corners of excavations).

4.3.5.6 Long-term changes in groundwater that are likely to occur during the design working life of the structure (including those due to climate change and rising groundwater) should be taken into account.

4.3.5.7 Increased groundwater pressures owing to burst pipes and other failures of engineered systems should be classified as accidental actions if the event that causes the increase in groundwater pressure is unlikely to occur during the design working life of the structure.

4.3.5.8 The design of an earth retaining structure should be based on the most adverse water pressure conditions that can be anticipated.

NOTE Guidance on the selection of water tables and seepage forces can be found in CIRIA Report C580 *[N3].*

4.3.5.9 Design water pressures should take into account the effect of tides on water levels in the ground.

NOTE Guidance on tides and water level variations can be found in BS 6349-1-3.

4.3.5.10 If the equilibrium level of the water table is well defined and measures are taken to prevent it changing during heavy rain or flood, the design water pressures can be calculated from the position of the equilibrium water table, making due allowance for possible seasonal variations, otherwise the most adverse water pressure conditions that can be anticipated should be used in design.

4.3.5.11 Equilibrium water levels in fine soils should be determined from piezometric readings taken over an adequate length of time.

4.3.5.12 Allowance should be made in undrained (i.e. total stress) analyses for water pressures due to the temporary filling of cracks in fine soils.

4.3.5.13 Water pressures used in drained (i.e. effective stress) analyses should be determined for the groundwater regime in the vicinity of the structure.

4.3.5.14 Where a difference in water pressures exists on opposite sides of an earth retaining structure, allowance should be made for seepage around the wall. Where layers of markedly different permeability exist, the water levels relevant to each permeable stratum should be taken into account.

4.3.5.15 The distribution of pore water pressures may be determined from a flow net, provided it adequately represents the hydraulic and permeability conditions in the vicinity of the structure.

4.3.6 Concrete

4.3.6.1 Concrete incorporated into earth retaining structures should conform to BS EN 1992-1-1, BS EN 206, and BS 8500-2.

4.3.6.2 Concrete incorporated into earth retaining structures should be specified in accordance with BS EN 206 and BS 8500-1.

4.3.6.3 Steel reinforcement for concrete earth retaining structures should conform to BS EN 10080 and BS 4449.

4.3.7 Steel

4.3.7.1 Steel incorporated into earth retaining structures should conform to BS EN 1993-1-1, BS EN 1993-5, and BS 8081, as appropriate.

4.3.7.2 The values of steel parameters should be determined in accordance with BS EN 1993-1-1 and BS EN 1993-5, and their UK National Annexes.

4.3.7.3 Hot rolled steel products should conform to BS EN 10025.

4.3.7.4 Cold formed hollow steel sections should conform to BS EN 10219.

4.3.8 Timber

4.3.8.1 Timber incorporated into earth retaining structures should conform to BS EN 1995-1-1.

4.3.8.2 The values of timber parameters should be determined in accordance with BS EN 1995-1-1.

4.3.9 Masonry

4.3.9.1 Masonry incorporated into earth retaining structures should conform to BS EN 1996-1-1.

4.3.9.2 The values of masonry parameters should be determined in accordance with BS EN 1996-1-1.

4.3.9.3 The following masonry units should conform to the relevant part of BS EN 771:

- clay masonry units (Part 1);
- calcium silicate masonry units (Part 2);
- aggregate concrete masonry units (Part 3);
- autoclaved aerated concrete masonry units (Part 4);
- manufactured stone masonry units (Part 5); and
- natural stone masonry units (Part 6).

4.3.9.4 The dimensions of clay and calcium silicate brick of special shapes and sizes should conform to BS 4729.

4.3.9.5 Mortars used in masonry earth retaining structures should conform to PD 6697.

4.3.9.6 Damp proof courses used in masonry earth retaining structures should conform to BS 8215.

4.3.9.7 Wall ties should conform to PD 6697.

4.3.10 Pipes

Pipes used in drainage systems for earth retaining structures should conform to one of the following standards, as appropriate:

- vitrified clay pipes – BS 65 and BS EN 295;
- concrete pipes – BS 5911 and BS EN 1916;
- glass reinforced plastics (GRP) pipes – BS 5480;
- cast iron – BS 437;
- ductile iron – BS EN 598;
- unplasticized polyvinyl-chloride (PVC-U) – BS 4660 or BS 5481 or BS EN 1401;
- polypropylene (PP) – BS EN 1852-1;
- polyethylene (PE) – BS EN 12666-1;
- thermoplastics structure wall pipes – BS 4962;
- geotextile wrapped land drains – BS 4962.

4.4 Durability

4.4.1 General

The durability of earth retaining structures should conform to BS EN 1990.

4.4.2 Concrete

4.4.2.1 The durability of concrete should conform to BS EN 1992-1-1.

4.4.2.2 Exposure classes for concrete should be determined in accordance with BS EN 206 and BS 8500-1.

4.4.2.3 For the purpose of specifying concrete to be used in earth retaining structures, ground conditions should be classified in accordance with BS 8500-1:2015, Table A.2.

NOTE 1 BS 8500-1:2015 and BS 8500-2:2015 are complementary British Standards to BS EN 206:2013.

NOTE 2 Guidance on concrete in aggressive ground can be found in BRE Special Digest 1 *[23].*

4.4.3 Steel

4.4.3.1 The durability of steel should conform to BS EN 1993-1-1.

4.4.3.2 The durability of steel reinforcement in reinforced concrete should conform to BS EN 1992-1-1.

NOTE Guidance on corrosion at bi-metallic contacts and its remediation can be found in PD 6484.

4.4.4 Timber

4.4.4.1 General

NOTE 1 Information on the biological agents that can attack wood can be found in BS EN 335:2013, Annex C.

NOTE 2 Guidance on shipworm (various species of the genera Teredo and Banksia), Martesia, and gribble (various species of the genus Limnoria) can be found in BRE Technical Note 59 *[24].*

4.4.4.1.1 The preservative treatment of timber should conform to BS 8417.

4.4.4.1.2 Components should be machined so that they contain a high proportion of permeable sapwood.

NOTE Wood species can be selected for permeability and sapwood content from the information given in BS EN 350-2.

4.4.4.1.3 The durability of timber should conform to BS EN 1995-1-1, which requires timber and wood-based materials to either:

- have adequate natural durability conforming to BS EN 350-2 for the particular hazard class defined in BS EN 335-1:1992, BS EN 335-2:1992, and BS EN 335-3:1992, or
- be given a preservative treatment conforming to BS EN 351-1 and BS EN 460.

NOTE BS EN 335-1:1992, BS EN 335-2:1992, and BS EN 335-3:1992 have been superseded by BS EN 335:2013.

4.4.4.1.4 All machining of timber, including notching, should be undertaken before applying preservative treatment.

4.4.4.2 Service classes

Service classes for timber should be determined in accordance with BS EN 1995-1-1.

4.4.4.3 Use classes

Use classes for timber should be determined in accordance with BS EN 335:2013.

NOTE 1 Use classes relevant to timber in earth retaining structures are summarized in Table 5.

NOTE 2 BS EN 1995-1-1:2004 requires timber structures to be assigned to one of three "service classes". BS EN 335:2013 defines five "use classes" for wood and wood-based products. BS EN 335:2013, Annex A provides a possible mapping of service classes to use classes.

Table 5 **Use classes relevant to timber in earth retaining structures**

Use class	Situation	Attack is possible by:
UC 4	Wood is in direct contact with the ground or fresh water	Fungi and wood-destroying fungi Wood-boring insects Termites (in countries where these present a hazard) Bacterial decay
UC 5	Wood is permanently or regularly submerged in salt water (i.e. sea water and brackish water)	Invertebrate marine organisms Wood-destroying fungi Growth of surface moulds and staining fungi Wood-boring insects (above water)

4.4.5 Masonry

The durability of masonry earth retaining structures should conform to BS EN 1996-1-1, BS EN 1996-2, and PD 6697.

4.5 Geotechnical analysis

4.5.1 Actions

4.5.1.1 General

4.5.1.1.1 The actions assumed in the geotechnical analysis of earth retaining structures should conform to BS EN 1991.

4.5.1.1.2 The actions assumed in the design of the reinforcement used in earth retaining structures constructed using strengthened or reinforced soils should conform to BS 8006.

4.5.1.1.3 The actions assumed in the geotechnical analysis of earth retaining structures (other than reinforced soil structures) subject to traffic loading should additionally conform to PD 6694-1.

NOTE The design of reinforced soil structures is explicitly excluded from the scope of PD 6694-1.

4.5.1.1.4 To conform to BS EN 1990:2002+A1:2005, **6.4.3.3(4)**, combinations of actions for accidental design situations should include either the accidental action itself or actions that occur after the accidental event.

4.5.1.1.5 The design of an earth retaining structure should take into account the effects of dynamic and cyclic loads on its performance.

4.5.1.2 Surcharges

4.5.1.2.1 The surcharge on the retained ground surface behind an earth retaining structure should conform to BS EN 1997-1:2004+A1:2013, **9.3.1.3**.

4.5.1.2.2 Imposed loads on vehicle traffic areas adjacent to earth retaining structures (excluding walls adjacent to bridges) should conform to the recommendations given in BS EN 1991-1-1:2002, **6.3.3**, for identical loads inside buildings. The uniformly distributed load (q_k) should be used to assess general effects of the loading on the structure. Characteristic values of q_k should be taken from Table 6.

Table 6 **Imposed loads on vehicle traffic areas (excluding walls adjacent to bridges or where the gross vehicle weight is greater than 160 kN)**

Category [A] **of traffic areas**	**Gross vehicle weight, W** kN	**Imposed load,** [B] q_k kPa	**Examples**
F	W ≤ 30	2.5	Garages, parking areas, parking halls
G	30 < W ≤ 160	5.0	Access routes, delivery zones, zones accessible to fire engines

[A] As defined in BS EN 1991-1-1.
[B] Values taken from the UK NA to BS EN 1991-1-1.

4.5.1.3 Construction loads on the retained ground surface behind an earth retaining structure should conform to BS EN 1997-1:2004+A1:2013, **9.3.1.3**. The weight density of construction materials may be obtained from BS EN 1991-1-1:2002, Annex A.

4.5.1.4 The weight density of stored materials may be obtained from BS EN 1991-1-1:2002, Annex A.

4.5.1.5 Load models for abutments and walls adjacent to public highways and railways

*COMMENTARY ON **4.5.1.5***

BS EN 1991-1-2 defines "Load Model 1" (LM1) for general and local verifications of structures subject to the effects of traffic of lorries and cars. LM1 comprises two components: a tandem system (TS) with two concentrated loads and a uniformly distributed load (UDL).

*As suggested in Note 1 to BS EN 1991-2:2003, Paragraph **4.9.1**(1), the tandem system (TS) loads defined in Load Model 1 may be replaced by an equivalent uniformly distributed load (q_{eq}) spread over an appropriate relevant rectangular surface depending on the dispersal of the loads through the backfill or earth. For normal traffic, the UK National Annex to BS EN 1991-2 provides the vehicle model (Figure NA.6) to be used for vertical loads on backfill behind abutments and wingwalls.*

The equivalent surcharge values given in this subclause are more onerous than Load Models given in the UK National Annex to BS EN 1991-2, but simpler to apply in design.

4.5.1.5.1 Load models for the design of abutments and wing walls adjacent to road bridges, footbridges, and railway bridges should conform to the UK National Annex to BS EN 1991-2:2003, **NA.2.34.**

4.5.1.5.2 The characteristic vertical and horizontal pressures in the backfill behind a bridge abutment or wing wall and other horizontal actions on such structures should conform to the UK National Annex to BS EN 1991-2:2003.

4.5.1.5.3 Horizontal earth pressure due to traffic loading behind a bridge abutment or wing wall should additionally conform to PD 6694-1.

4.5.1.5.4 Alternatively, traffic loading behind wing walls and other earth retaining structures may be modelled as a uniformly distributed load conforming to **4.5.1.5.5**.

4.5.1.5.5 Where the centre-line of the nearest traffic wheel load is located at least 0.5 m away from the rear face of an earth retaining structure, the load model defined in the UK National Annex to BS EN 1991-2:2003, **NA.2.34** may be replaced by a uniformly distributed load (q_k) over the entire ground surface from the back of the structure.

4.5.1.5.6 Characteristic values of q_k should be taken from Table 7.

Table 7 **Imposed loads on vehicle traffic areas (next to wing walls and other earth retaining structures)**

Category of traffic areas		**Imposed load, q_k** kPa
Footways, cycle tracks, and footbridges		5
Normal vehicle traffic	≥1 m from rear face of structure	10
	≥0.5 m but <1 m from rear face of structure	20
Special vehicle traffic (vehicles conforming to STGO [A)] [LM3 SV models] or SO [B)] Regulations)	≥1 m from rear face of structure	20
	≥0.5 m but <1 m from rear face of structure	30
Normal rail traffic (on areas occupied by the tracks)		50
Light rail traffic		[C)]

A) Special Types General Order (see BS EN 1991-2).
B) Special Order (see BS EN 1991-2).
C) To be determined for the specific project.

4.5.1.5.7 Where the centre-line of the nearest traffic wheel load is located closer than 0.5 m away from the rear face of an earth retaining structure, the load models defined in the UK National Annex to BS EN 1991-2:2003, **NA.2.34** should be used.

4.5.1.5.8 The vertical loading and earth pressure effects of normal traffic on mainline railways may be modelled as a uniformly distributed load over a width of 3 m at a level 0.7 m below the running surface of the track, as detailed in BS EN 1991-2:2003, **6.3.6.4**.

4.5.2 Earth pressures

4.5.2.1 General

4.5.2.1.1 Earth pressures should be calculated in accordance with BS EN 1997-1:2004+A1:2013, **9.5**.

4.5.2.1.2 For earth retaining structures in rock, the calculation of earth pressures should take into account discontinuities in the rock mass. The rock should be considered to have the potential to fail either as a rock mass or on planes of discontinuity.

NOTE It might be beneficial to seek specialist advice when designing earth retaining structures in rock.

4.5.2.1.3 The model factor on earth pressure coefficients that is recommended in PD 6694-1 should only be applied to gravity and semi-gravity walls that form part of or are adjacent to bridges and are subject to traffic loading.

NOTE The purpose of the model factor ($\gamma_{Sd;K}$ = 1.2) that is recommended in PD 6694-1 is to maintain a level of reliability similar to that achieved in the withdrawn BS 5400 for the design of steel, concrete, and composite bridges.

4.5.2.2 At-rest earth pressures

*COMMENTARY ON **4.5.2.2***

Where there is no movement of an earth retaining structure relative to the ground acting against it, the retained material is said to be in an "at-rest" state.

At-rest earth pressures should be calculated in accordance with BS EN 1997-1:2004+A1:2013, **9.5.2**.

4.5.2.3 Active earth pressures

*COMMENTARY ON **4.5.2.3***

Where an earth retaining structure moves away from the ground sufficiently for the earth pressure against it to reach its minimum value, the ground is said to be in an "active" state. This condition is mainly relevant to ground located behind the structure (i.e. the retained ground).

4.5.2.3.1 In a drained analysis, when the earth retaining structure moves away from the ground, the effective earth pressure acting normal to the face of the structure (P'_n) may be calculated from:

$$P'_n \geq P'_a = K_a P'_v - K_{ac} c' \quad (13)$$

where:

P'_a	is the effective active earth pressure acting normal to the structure;
P'	is the effective overburden pressure in the ground;
c'	is the soil's effective cohesion; and
K_a and K_{ac}	are active earth pressure coefficients.

4.5.2.3.2 Values of the active earth pressure coefficient K_a may be calculated from the analytical procedure given in BS EN 1997-1:2004+A1:2013, **C.2**.

4.5.2.3.3 For earth retaining structures with vertical faces, values of the active earth pressure coefficient, K_a, may be estimated from the figures given in BS EN 1997-1:2004+A1:2013, **C.1**.

*NOTE The formulae in BS EN 1997-1:2004+A1:2014, **C.1**, give similar values of K_a to BS EN 1997-1:2004+A1:2014, **C.2**.*

4.5.2.3.4 The value of K_{ac} may be calculated from:

$$K_{ac} = 2\sqrt{K_a(1 + a/c')} \leq 2.56\sqrt{K_a} \qquad (14)$$

where:

a is the effective adhesion at the interface between the soil and structure.

4.5.2.3.5 In an undrained analysis, when the earth retaining structure moves away from the ground, the total pressure acting normal to the face of the structure (p_n) may be calculated from:

$$p_n \geq p_a = p_v - K_{ac,u}c_u \qquad (15)$$

where:

p_a is the total active pressure acting normal to the structure;

p_v is the total overburden pressure in the ground;

c_u is the soil's undrained shear strength; and

$K_{ac,u}$ is an undrained active earth pressure coefficient.

4.5.2.3.6 The value of $K_{ac,u}$ may be calculated from:

$$K_{ac,u} = 2\sqrt{1 + a/c_u} \leq 2.56 \qquad (16)$$

where:

a is the total adhesion at the interface between the soil and structure.

4.5.2.3.7 In both drained and undrained analyses, the total pressure (p_h) acting normal to the face of the structure should additionally satisfy:

$$p_h \geq p_{h,min} = \begin{cases} k_{min}z & \text{in the absence of water} \\ \gamma_w z & \text{in the presence of water} \end{cases} \qquad (17)$$

where:

$p_{h,min}$ is the minimum total pressure to be assumed in the design;

k_{min} is a constant equal to 5 kPa/m;

γ_w is the weight density of groundwater; and

z is depth below ground surface.

NOTE The product $k_{min}z$ is traditionally known as the "minimum equivalent fluid pressure (MEFP)".

4.5.2.4 Passive earth pressures

*COMMENTARY ON **4.5.2.4***

Where an earth retaining structure moves towards the ground sufficiently for the earth pressure against it to reach its maximum value, the ground is said to be in a "passive" state. This condition is mainly relevant to ground located in front of the structure (i.e. the restraining ground).

4.5.2.4.1 In a drained analysis, when the earth retaining structure moves towards the ground, the effective earth pressure acting normal to the face of the structure (p'_n) may be calculated from:

$$p'_n \le p'_p = K_p p'_v + K_{pc} c' \tag{18}$$

where:

p'_p	is the effective passive earth pressure acting normal to the structure;
p'_v	is the effective overburden pressure in the ground;
c'	is the soil's effective cohesion; and
K_p and K_{pc}	are passive earth pressure coefficients.

4.5.2.4.2 Values of the passive earth pressure coefficient,K_p, may be calculated from the analytical procedure given in BS EN 1997-1:2004+A1:2013, **C.2**.

4.5.2.4.3 For earth retaining structures with vertical faces, values of the passive earth pressure coefficient, K_p, may be estimated from the figures given in BS EN 1997-1:2004+A1:2013, **C.1**.

*NOTE The formulae in BS EN 1997-1:2004+A1:2013, **C.1**, give similar values of K_p to BS EN 1997-1:2004+A1:2014, **C.2**, except for large angles of shearing resistance and interface friction, when the values from **C.2** are more conservative (see* Eurocode 7 – a commentary *[25]). The use of the figures in **C.1** is not recommended for interface friction angles greater than two-thirds of the soil's angle of shearing resistance.*

4.5.2.4.4 The value of K_{pc} may be calculated from:

$$K_{pc} = 2\sqrt{K_p(1 + a/c')} \le 2.56\sqrt{K_p} \tag{19}$$

where:

a	is the effective adhesion at the interface between the soil and structure.

4.5.2.4.5 In an undrained analysis, when the earth retaining structure moves towards the ground, the the total pressure acting normal to the face of the structure (p_n) may be calculated from:

$$p_n \le p_p = p_v + K_{pc,u} c_u \tag{20}$$

where:

p_p	is the total passive pressure acting normal to the structure;
p_v	is the total overburden pressure in the ground;
c_u	is the soil's undrained shear strength; and
$K_{pc,u}$	is an undrained passive earth pressure coefficient.

4.5.2.4.6 The value of $K_{pc,u}$ may be calculated from:

$$K_{pc,u} = 2\sqrt{1 + a/c_u} \le 2.56 \tag{21}$$

where:

a is the total adhesion at the interface between the soil and structure.

4.5.2.5 Compaction earth pressures in coarse soils and fill

COMMENTARY ON ***4.5.2.5***

Earth retaining structures can retain coarse soils or engineered fill that is placed and compacted behind the wall. Compaction can lock in higher horizontal stresses in the retained soil or fill than are calculated assuming at rest or active conditions.

4.5.2.5.1 Compaction pressures and earth pressure due to live loading should not be applied simultaneously.

4.5.2.5.2 Filling adjacent to earth retaining structures should conform to BS 6031:2009, **7.6.10**.

4.5.2.5.3 To prevent excessive horizontal forces on earth retaining structures, both the compaction plant and method of compaction used for filling adjacent to structures may differ from those used in general earthworks construction.

4.5.2.5.4 The effects of compaction behind an earth retaining structure should be considered in the calculation of bending moments and shear forces in the structure.

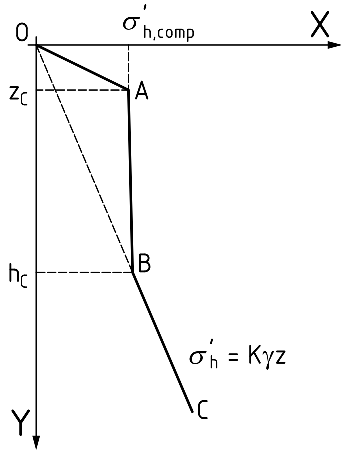
4.5.2.5.5 The horizontal effective stress in coarse soils and fill that are placed and compacted behind an earth retaining structure may be estimated from Figure 4 and the following expressions (see also *The pressure of compacted fill on retaining walls* [26]):

$$\sigma'_\mathrm{h} = \begin{cases} \dfrac{\sigma'_\mathrm{v}}{K_0} & \text{for } z \le z_\mathrm{c} = K_0\sqrt{\dfrac{2P}{\pi\gamma}} \\ \sigma'_\mathrm{h,comp} = \sqrt{\dfrac{2P\gamma}{\pi}} & \text{for } z_\mathrm{c} \le z \le h_\mathrm{c} \\ K_0\sigma'_\mathrm{v} & \text{for} z \ge h_\mathrm{c} = \dfrac{1}{K_0}\sqrt{\dfrac{2P}{\pi\gamma}} \end{cases} \qquad (22)$$

where:

σ'_h and σ'_v	are the horizontal and vertical effective stresses (respectively) at depth z below ground surface;
K_0	is the soil's at-rest earth pressure coefficient;
P	is effective line load per unit width of the roller of the compaction plant (= mass of compaction plant per unit width m × acceleration due to gravity, g);
γ	is weight density of the retained material;
z_c	is critical depth; and
h_c	is critical height.

Figure 4 **Maximum effective horizontal earth pressure from compaction theory**



Key

X Lateral earth pressure, σ'_h

Y Depth below crest of wall, z

4.5.2.5.6 Values of $\sigma'_{h,comp}$, z_c, and h_c may be taken from Table 8.

Table 8 **Values of $\sigma'_{h,comp}$, z_c, and h_c for different weights of compaction plant**

Dead-weight of roller per width of roll, *m*	Critical depths		Effective horizontal earth pressure, $\sigma'_{h,comp}$, based on type of compaction A)	
	z_c	h_c	Static	Dynamic B)
kg/m	mm	mm	kPa	kPa
75	80	310	3	6
100	90	360	4	7
500	200	800	8	16
1 000	280	1 130	11	23
1 300	320	1 290	13	16

A) Assuming γ = 20 kN/m^3, K_0 = 0.5, and g = 10 m/s^2.
B) Assuming the mass of a dynamic roller is twice its dead weight.

NOTE 1 Even for relatively large rollers and rigid walls, compaction pressures, σ'_h, are not likely to exceed 20 kPa to 30 kPa and the depth to which compaction pressures are significant (h_c) is not likely to exceed 3 m to 4 m.

NOTE 2 Background theory for the expressions given in this subclause can be found in Earth Pressure and Earth-Retaining Structures (3rd edition)*, Section 9.2.4 [2] and* CIRIA Report C516*, Section 6.4 [27].*

4.5.3 Water pressures

4.5.3.1 Water pressures should be calculated in accordance with BS EN 1997-1:2004+A1:2013, **9.6**.

4.5.3.2 Actions caused by water pressures should conform to BS EN 1991-1-6:2005, **4.9**, which recommends they should be represented as static pressures and/or dynamic effects, whichever gives the most unfavourable effects.

4.5.3.3 Water pressures may be classified as permanent, variable or accidental actions, depending on the applicable environmental conditions.

4.5.4 Interface friction and adhesion

NOTE Guidance on the selection of a suitable value of interface friction (also known as wall friction) can be found in PD 6694-1:2011, ***7.2.2****.*

4.5.5 Tension cracks

COMMENTARY ON ***4.5.5***

Where there is a low equilibrium groundwater table, tension cracks might form behind the retaining structure in a retained clay soil and extend over the full retained height of the wall.

4.5.5.1 Earth retaining structures in fine soils should be designed to withstand water pressures arising from fully- or partially-filled tension cracks that might form behind the wall.

NOTE The tension crack may be considered closed at the depth where total earth pressure in the ground exceeds the water pressure in the tension crack.

4.5.5.2 Allowance for the formation of a tension crack should be made by one or more of the following:

- re-assessing earth pressures acting over the depth of the tension crack on the basis of effective stress parameters, assuming the characteristic effective cohesion $c'_k = 0$;
- assuming a minimum total pressure on the wall in accordance with **4.5.2.3.7**.

4.5.5.3 The depth of a tension crack may be limited to the retained height of the wall.

4.6 Ultimate limit states

4.6.1 General

4.6.1.1 The ultimate limit state design of an earth retaining structure should conform to BS EN 1997-1:2004+A1:2013, **2.4.7**, **9.2**, and **9.7**.

4.6.1.2 The overall stability of an earth retaining structure should be verified in accordance with BS EN 1997-1:2004+A1:2013, **9.7.2**.

4.6.2 Design values of geotechnical parameters

COMMENTARY ON ***4.6.2***

The UK National Annex to BS EN 1997-1:2004+A1:2013 states that "it might be more appropriate to determine the design value of φ'_{cv} directly, rather than apply the partial factor γ_φ (= 1.25 for limit state GEO, Set M2) to its characteristic value".

4.6.2.1 In accordance with BS EN 1997-1:2004+A1:2013, **2.4.5.2**, the characteristic value of a geotechnical parameter should be selected as a cautious estimate of the value "affecting the occurrence of the limit state". The value of φ'_k may therefore be selected as a peak value, a constant volume value, a residual value or an intermediate value (as appropriate).

4.6.2.2 The design values of geotechnical parameters should conform to BS EN 1997-1:2004+A1:2013, **2.4.6.2**.

4.6.2.3 When the peak angle of shearing resistance is the value that affects the occurrence of the limit state, the design value of shearing resistance (φ'_d) should either be assessed directly or obtained from:

$$\tan\varphi'_d = \frac{\tan\varphi'_{pk,k}}{\gamma_\varphi} \qquad (23)$$

where:

$\varphi'_{pk,k}$ is the characteristic value of the soil's peak angle of shearing resistance; and

γ_φ is the partial factor specified in the UK National Annex to BS EN 1997-1:2004+A1:2013.

4.6.2.4 If it is anticipated that there can be significant post-peak softening of the soil's shearing resistance, together with significant straining of the soil, then the peak angle of shearing resistance should not be selected as the value that affects the occurrence of the limit state.

4.6.2.5 When the constant volume angle of shearing resistance is the value that affects the occurrence of the limit state, the design value of shearing resistance (φ'_d) should either be assessed directly or obtained from:

$$\tan\varphi'_d = \min\begin{cases}\dfrac{\tan\varphi'_{pk,k}}{\gamma_\varphi} \\ \dfrac{\tan\varphi'_{cv,k}}{\gamma_{\varphi,cv}}\end{cases} \tag{24}$$

where:

$\varphi'_{pk,k}$ and γ_φ are as defined for equation (23);

$\varphi'_{cv,k}$ is the characteristic value of the soil's constant volume angle of shearing resistance; and

$\gamma_{\varphi,cv}$ is a partial factor whose value is 1.0.

4.6.2.6 When the residual angle of shearing resistance is the value that affects the occurrence of the limit state, the design value of shearing resistance (φ'_d) should either be assessed directly or obtained from:

$$\tan\varphi'_d = \min\begin{cases}\dfrac{\tan\varphi'_{pk,k}}{\gamma_\varphi} \\ \dfrac{\tan\varphi'_{res,k}}{\gamma_{\varphi,res}}\end{cases} \tag{25}$$

where:

$\varphi'_{pk,k}$ and γ_φ are as defined for equation (23);

$\varphi'_{res,k}$ is the characteristic value of the soil's residual angle of shearing resistance; and

$\gamma_{\varphi,res}$ is a partial factor whose value is 1.0.

4.6.3 Minimum surcharge

*COMMENTARY ON **4.6.3***

It has been traditional in UK practice to apply a minimum surcharge of 10 kPa on the retained surface behind an earth retaining structure. This surcharge was intended to cater for unplanned additional loading from traffic, stored materials, goods, and containers, etc.

There is no requirement in BS EN 1997-1 for a minimum surcharge to be applied to earth retaining structures.

4.6.3.1 Design situations should take into account all imposed loads that are relevant to the areas over which they are likely to occur during the corresponding time period, as specified in BS EN 1991-1-1:2002, **6.3**.

4.6.3.2 The design surcharge, q_d, acting over the surface of the retained ground should satisfy:

$$q_d \geq \begin{cases} \left(\dfrac{H_d}{H_{ref}}\right) \times q_{d,min} & \text{for} \quad H_d < H_{ref} \\ q_{d,min} & \text{for} \quad H_d \geq H_{ref} \end{cases} \qquad (26)$$

where:

H_d is the design value of the retained height of the earth retaining structure;

H_{ref} is a reference height, equal to 3 m; and

$q_{d,min}$ is the minimum design surcharge, equal to 10 kPa.

4.6.3.3 The value of $q_{d,min}$ may be reduced when:

- the ground surface behind the retaining structure is subject to adequate engineering supervision throughout the design situation to prevent a surcharge greater than $q_{d,min}$ being applied to it, and that supervision is specified in the Geotechnical Design Report (GDR) and in the extract from it that is provided to the client in accordance with BS EN 1997-1-1:2004+A1:2013, **2.8(6)**P; or
- it is physically impossible for the retained surface to be subjected to surcharge loading greater than $q_{d,min}$ during its design lifetime.

4.6.4 Unplanned excavation

*COMMENTARY ON **4.6.4***

*BS EN 1997-1:2004+A1:2013, **9.3.2.2(2)** recommends that the level of the soil in front of a retaining wall "should be lowered below the nominally expected level by an amount Δa", where Δa equals 10% of the retained height of the wall below its lowest support (if any), subject to a maximum of 0.5 m. This allowance, which BS EN 1997-1 specifies for "normal site control", is typically considered to provide for the effects of so-called "unplanned excavation" in front of the wall.*

4.6.4.1 The value of Δa specified in BS EN 1997-1-1:2004+A1:2013, **9.3.2.2(2)** may be reduced to zero when the surface level in front of the retaining wall:

- is subject to adequate engineering supervision throughout the design situation and that supervision is specified in the Geotechnical Design Report and in the extract from it that is provided to the client in accordance with BS EN 1997-1-1:2004+A1:2013, **2.8(6)**P; or
- cannot deviate from its nominal level owing to the presence of structural elements (such as floor slabs or pavement).

4.6.4.2 Planned excavations should be considered in specific design situations, separately from any provision for unplanned excavations.

4.7 Serviceability limit states

*COMMENTARY ON **4.7***

A serviceability limit state is a condition beyond which specific service requirements for the structure or foundation are no longer met. Serviceability requirements for earth retaining structures are commonly expressed as limiting criteria for wall deflection, settlement of the ground behind the structure, cracking in the retaining structure, or strain in nearby services and buildings.

4.7.1 The serviceability limit state design of an earth retaining structure should conform to BS EN 1997-1:2004+A1:2013, **2.4.8**, **9.2**, and **9.8**.

4.7.2 The terminology used to describe ground movements should conform to BS EN 1997-1:2004+A1:2013, Annex H(1) and Figure H.1.

4.7.3 Damage to masonry walls owing to ground movement should be classified in accordance with *Behaviour of foundations and structures* [N2] .

NOTE Guidance on building response to ground movements can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 26 *[3].*

4.8 Structural design

4.8.1 The structural design of an earth retaining structure should conform to BS EN 1997-1:2004+A1:2013, **9.7.6**.

4.8.2 The structural design of concrete earth retaining structures should conform to BS EN 1992.

4.8.3 The structural design of steel earth retaining structures should conform to BS EN 1993.

4.8.4 The structural design of timber earth retaining structures should conform to BS EN 1995.

4.8.5 The structural design of masonry earth retaining structures should conform to BS EN 1996.

4.8.6 The shear stress in masonry retaining walls should be assessed in accordance with PD 6697:2010, **6.3.4**.

4.8.7 The structural design of an earth retaining structure should take into account the likelihood that the most severe stresses in the structure might not occur when the structure is at an ultimate limit state of geotechnical failure.

4.8.8 The structure should be designed to withstand the largest stresses that it will encounter during the design situation.

4.9 Execution

NOTE For information on archaeological finds, see BS 8004:2015, Annex D.

4.9.1 General

4.9.1.1 The execution of concrete earth retaining structures should conform to BS EN 13670.

4.9.1.2 The execution of steel earth retaining structures should conform to BS EN 12063.

4.9.1.3 The execution of masonry earth retaining structures should conform to BS EN 1996-2 and PD 6697.

4.9.1.4 The execution of gravity retaining walls should also conform to **5.10**.

4.9.1.5 The execution of semi-gravity retaining walls should also conform to **6.10**.

4.9.1.6 The execution of embedded retaining walls should also conform to **7.10**.

4.9.1.7 When a design necessitates a particular sequence of operations, these should be clearly indicated on the drawings or in the specification.

NOTE 1 Information about construction processes can be found in the ICE manual of geotechnical engineering (2012), Volume II, Section 8 *[3].*

NOTE 2 Attention is drawn to The Construction (Design and Management) Regulations, 2015 *[4], with regards to health and safety requirements for construction works, in particular Regulations 13 and 15, which deal with duties of contractors.*

4.9.2 Temporary works

4.9.2.1 The procedural controls that should be applied to all aspects of temporary works should conform to BS 5975.

4.9.2.2 The design and construction of temporary excavations, trenches, pits and shafts should conform to BS 6031.

4.9.3 Working platforms

Working platforms should conform to BS 8004.

4.9.4 Compaction plant

The dead weight of compaction plant that is used within 2 m of the back of an earth retaining structure should be no greater than the limits given in Table 9.

Table 9 **Maximum weight of compaction plant to be used within 2 m of an earth retaining structure**

Type of compaction plant	**Maximum dead-weight per width of roll** kg/m	**Maximum total mass** kg
Vibratory roller	1 300	1 000
Vibrating plate compactor	—	1 000
Vibro-tamper	—	75

4.10 Testing

Tests on earth retaining structures should conform to prEN ISO 22477, where appropriate.

4.11 Supervision, monitoring and maintenance

4.11.1 Supervision of construction

Supervision of construction of earth retaining structures should conform to BS EN 1997-1:2004+A1:2013, **4.2**.

NOTE Guidance on technical supervision of site works can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 96 *[1].*

4.11.2 Monitoring

4.11.2.1 Monitoring of earth retaining structures should conform to BS EN 1997-1:2004+A1:2013, **4.5**.

4.11.2.2 Where serviceability criteria have been specified, or it is otherwise appropriate, the settlement and deformation of the following should be monitored systematically, in order to assess the performance of:

- the supported structure;
- ground surface;
- adjacent infrastructure; and
- any buried infrastructure or utilities.

4.11.2.3 Where appropriate, instrumentation should be installed to:

- confirm design assumptions and check the predicted behaviour of the earth retaining structure; and
- confirm that the structure continues to perform as required following construction.

4.11.2.4 Monitoring needed to implement the observational method should conform to BS EN 1997-1.

NOTE Guidance on the principles of geotechnical monitoring can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 94 *[1] .*

4.11.3 Maintenance

4.11.3.1 Maintenance of earth retaining structures should conform to BS EN 1997-1:2004+A1:2013, **4.6**.

4.11.3.2 If the design of an earth retaining structure relies on a dewatering system, a maintenance programme for dewatering should be specified.

4.12 Reporting

*COMMENTARY ON **4.12***

*BS EN 1997-1:2004+A1:2013, **2.8(1)P**, requires the "assumptions, data, methods of calculation and result of the verification of safety and serviceability" to be recorded in the Geotechnical Design Report (GDR).*

*Additionally, BS EN 1997-1:2004+A1:2013, **3.4.1(1)P**, requires the results of a geotechnical investigation to be compiled in a Ground Investigation Report (GIR), which "shall form a part of the Geotechnical Design Report'".*

This standard extends this reporting regime to include a Geotechnical Feedback Report (GFR) that contains full records of the works constructed. These as-built records include information that will assist with future maintenance, design of additional works and decommissioning of the works. The GFR could also go some way to satisfying the requirements of CDM Regulations *[4] with regards to preparation of a health and safety file.*

None of these reports is the same as a geotechnical baseline report, which may be used on geotechnical projects for contractual purposes.

4.12.1 Ground Investigation Report

The Ground Investigation Report for an earth retaining structure should conform to BS EN 1997-1:2004+A1:2013, **3.4**.

4.12.2 Geotechnical Design Report

The Geotechnical Design Report for an earth retaining structure should conform to BS EN 1997-1:2004+A1:2013, **2.8**.

4.12.3 Geotechnical Feedback Report

*COMMENTARY ON **4.12.3***

The Geotechnical Feedback Report is also known as a "close-out report".

4.12.3.1 On completion of the works, a Geotechnical Feedback Report (GFR) should be prepared that covers the following broad classes of information:

- a record of construction and any changes to its design; and
- results of monitoring and testing conducted during construction.

4.12.3.2 The GFR should be tailored to suit the size and complexity of the works.

4.12.3.3 The record of construction should include, as appropriate:

- a general description of the works, including ground and groundwater conditions encountered;
- instability problems, unusual ground conditions and groundwater problems, including measures to overcome them;
- contaminated and hazardous material encountered on site and the location of disposal, both on and off site;
- temporary works and foundation treatment, including drainage measures and treatment of soft areas and their effectiveness;
- types of imported and site-won materials and their use;
- any aspect of the specification or standards used that should be reviewed in view of problems encountered on site;
- any requirements for ongoing monitoring or abnormal maintenance requirements;
- any unexpected ground conditions that required changes to design;
- problems not envisaged in the Geotechnical Design Report and the solutions to them; and
- as-built drawings.

4.12.3.4 The results of monitoring and testing should include:

- details of any in-situ testing;
- test logs and test results;
- summary of site laboratory testing;
- location and details of instruments;
- readings from instruments (with dates) and predicted values;
- the results of compliance testing (e.g. in-situ density measurement, plate load tests); and
- data from monitoring instruments (e.g. piezometers, inclinometers, settlement gauges).

NOTE 1 The preparation of the GFR is particularly important where the observational method of design has been used.

NOTE 2 Guidance on the preparation of close-out reports can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 101 *[1].*

5 Gravity retaining walls

*COMMENTARY ON Clause **5***

This clause applies to the design and construction of:

- *mass concrete retaining walls;*
- *unreinforced masonry retaining walls;*
- *gabion walls; and*
- *crib walls.*

*This clause does not cover the design and construction of reinforced concrete retaining walls (which are covered by Clause **6**) or reinforced soil walls.*

In the UK, gravity walls have been built up to 13 m in height. However, other solutions might be more economic when the height of ground to be retained is greater than 8 m.

5.1 Choice and design of gravity retaining walls

5.1.1 General

5.1.1.1 The design of gravity retaining walls should conform to BS EN 1997-1:2004+A1:2013, Clause **9**, and Clause **4** and Clause **5** of this standard.

5.1.1.2 The design of gravity walls in quay and jetty construction should conform to BS 6349-2:2010, Clause **7**.

5.1.1.3 The design of reinforced soil walls should conform to BS 8006-1.

5.1.1.4 Where a gravity wall is combined with soil reinforcement, the design should conform to BS 8006-1.

5.1.1.5 The design of foundations for gravity retaining walls should conform to BS 8004.

5.1.1.6 Selection of a suitable type of gravity retaining wall should take into account:

- soil conditions;
- loads;
- drainage;
- existing and proposed structures and roads;
- groundwater conditions;
- the availability of space; and
- the geometry of the structure.

5.1.1.7 Gravity retaining walls for maritime works should conform to BS 6349-2:2010, **7.3**.

NOTE 1 Guidance on the selection of a suitable gravity retaining wall can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 62 *[1] and in* Earth Pressure and Earth-Retaining Structures, Chapter 6 *[2].*

NOTE 2 Guidance on the selection of low-height modular retaining walls (including unreinforced masonry, gabion, and crib walls) can be found in CIRIA Report C516, Chapter 3 *[27].*

5.1.2 Mass concrete retaining walls

*COMMENTARY ON **5.1.2***

Mass concrete retaining walls are suitable for retained heights typically up to 3 m. They can be designed satisfactorily for greater heights, but as the height increases other types of wall become more economic. The cross-sectional shape of the wall can be affected by factors other than stability, such as the use of the space in front of the wall, considerations of appearance or by the method of construction.

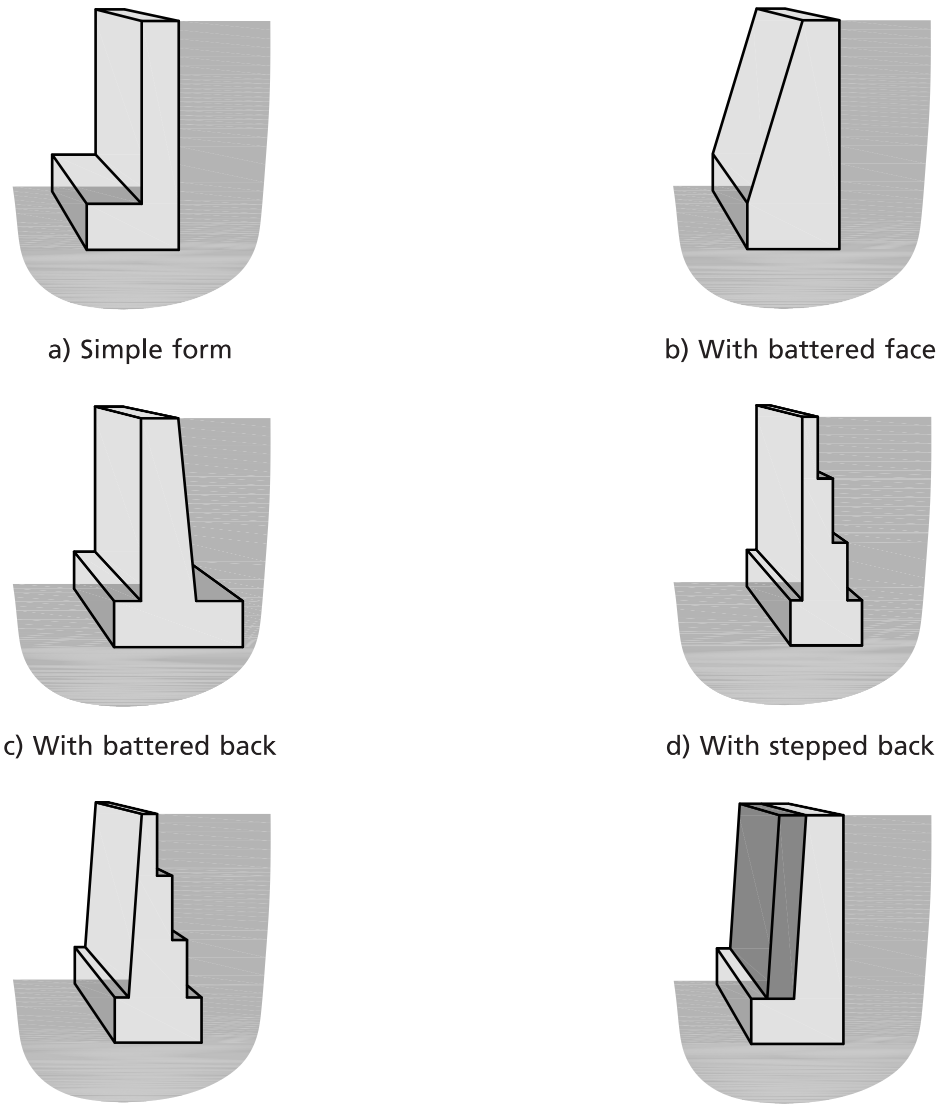
Figure 5 shows examples of typical mass concrete retaining walls.

The simple form is suitable for retained heights up to about 1.5 m.

Greater economy of material might be achieved if either the wall's front or back face is stepped or inclined.

Mass concrete retaining walls constructed under water for maritime works should conform to BS 6349-2:2010, **7.9**.

Figure 5 **Examples of mass concrete retaining walls**



a) Simple form

b) With battered face

c) With battered back

d) With stepped back

e) With stepped back and inclined face

f) With battered masonry face

5.1.3 Unreinforced masonry retaining walls

COMMENTARY ON ***5.1.3***

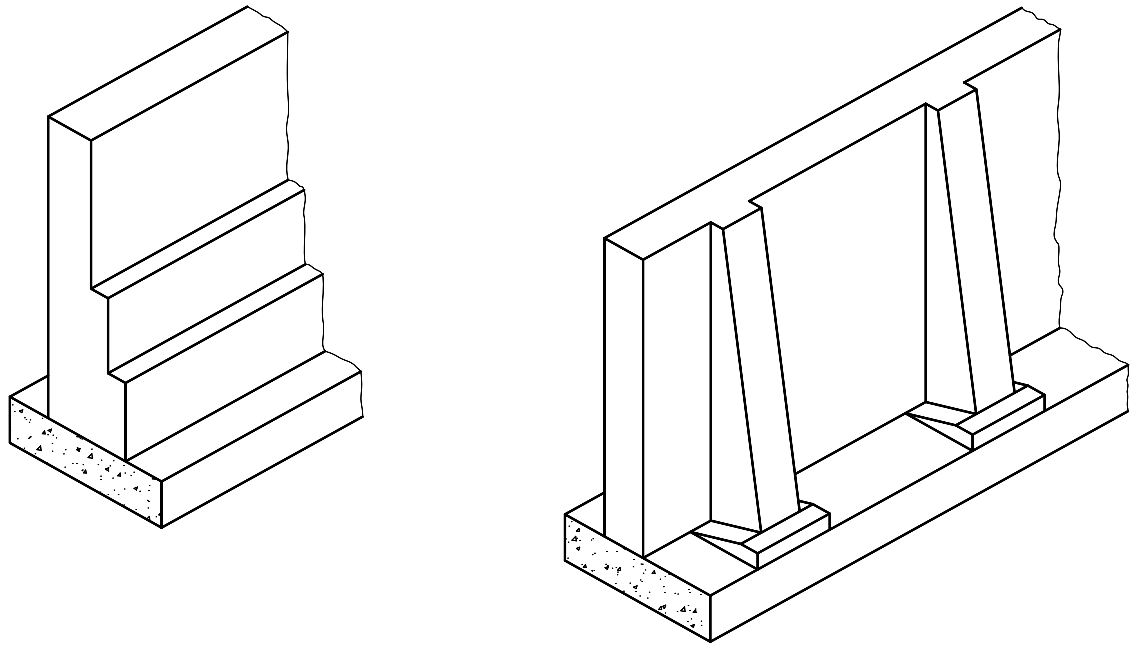
Unreinforced masonry retaining walls are suitable for retained heights typically up to 1.5 m (in the case of a simple stem wall) or greater heights (in the case of stepped or buttressed walls). An advantage of masonry retaining walls is they require minimal construction plant.

"Dry-stack" masonry retaining walls typically consist of precast concrete blocks that interlock to form a wall with a solid face.

Figure 6 shows examples of typical unreinforced masonry retaining walls.

Concrete blockwork retaining walls for maritime works should conform to BS 6349-2:2010, **7.4**.

Figure 6 **Examples of unreinforced masonry retaining walls**



a) Stepped b) Buttressed

5.1.4 Gabion walls

*COMMENTARY ON **5.1.4***

Gabion walls are suitable for retained heights typically up to about 10 m. Gabions are large rectangular cages or baskets, made of hexagonal woven steel wire or square welded mesh, filled with stone. Gabions are used to build retaining walls, revetments, and anti-erosion works. Box gabions are normally available in half metre modules of length 2 m to 6 m, width 1 m to 2 m, and in depths of 0.3 m, 0.5 m, and 1 m.

The permeability and flexibility of gabions make them suitable where the retained ground is likely to be saturated and the underlying ground's bearing resistance is low.

Gabions cages can bulge and deform under load, particularly if they have not been filled correctly, and this might affect the serviceability of the structure and the use of the ground it is retaining. Welded gabions are commonly supplied with an increased diameter wire mesh to the front face to stiffen the structure and provide more support.

Figure 7 shows examples of typical gabion retaining walls.

Further information about gabion walls can be found in CIRIA Report C516 *[27] and in* Earth Pressure and Earth-Retaining Structures (3rd edition) *[2].*

5.1.5 Crib walls

*COMMENTARY ON **5.1.5***

Crib walls are suitable for retained heights typically up to about 12 m. They are built with individual units, made of timber or precast concrete, that, when assembled, create a series of box-like structures into which suitable free draining coarse fill is placed. The fill acts in conjunction with the cribwork to support the retained ground.

Crib walls are flexible, open-faced structures that can accommodate differential settlement and movement caused, for example, by seasonal moisture changes.

The depth of the crib units is chosen to suit the overall height of the crib wall and the actions imposed upon it. In some design situations (including those with large retained heights, high loadings or weak soils), units of different depths are used.

Crib walls are normally built with a 1 (horizontal) to 4 (vertical) batter. However, this can vary to suit design requirements and the available space on site. Crib walls are formed with front and rear stretchers that are tied together with headers having notches or rebates.

A precast concrete crib wall uses less concrete than an equivalent mass concrete wall.

Figure 8 shows examples of typical crib walls.

Crib walls may be constructed of timber or precast concrete units.

NOTE Further information about crib walls can be found in CIRIA Report C516 *[27] and in* Earth Pressure and Earth-Retaining Structures *[2].*

Figure 7 **Examples of gabion retaining walls** (1 of 2)

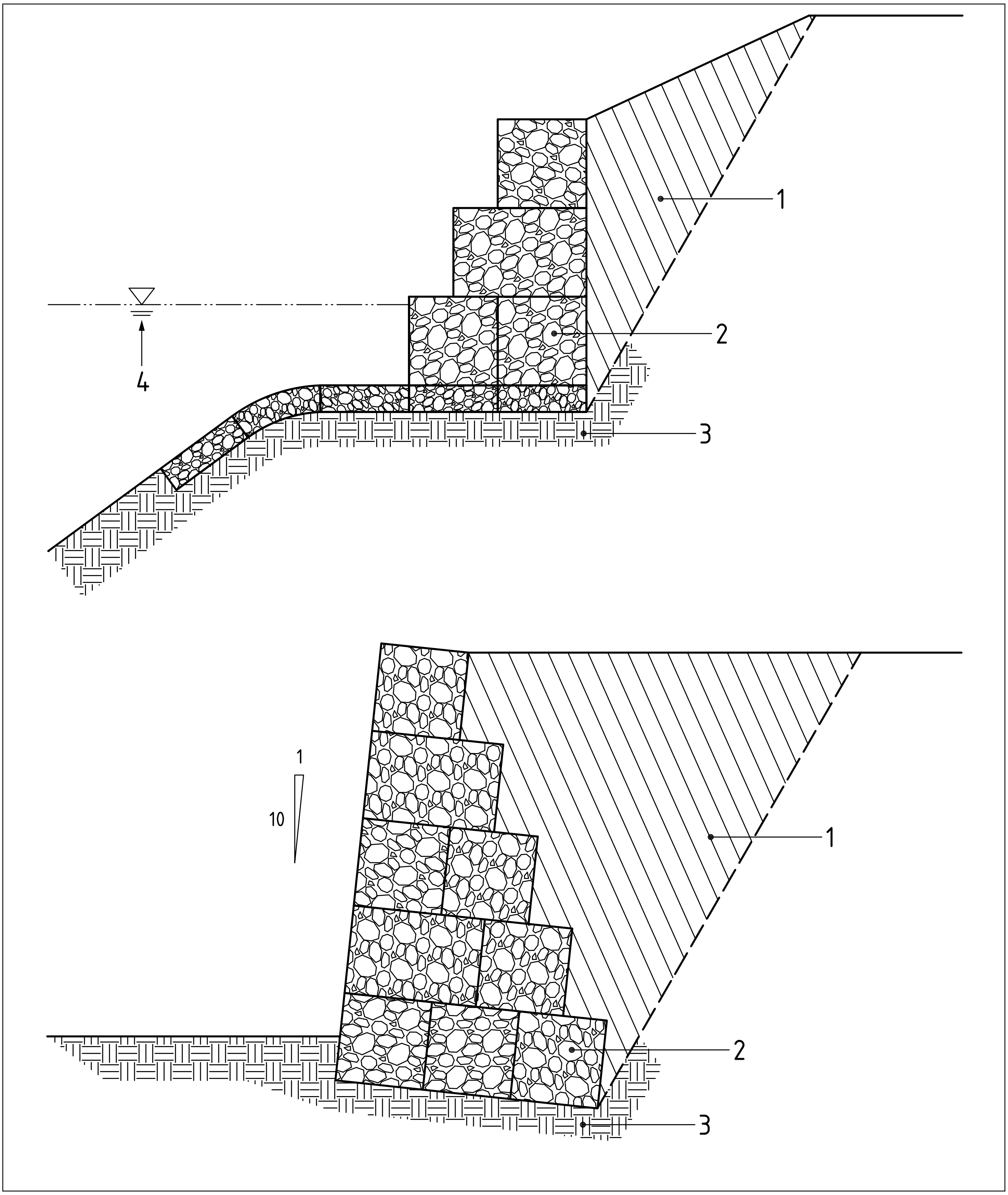
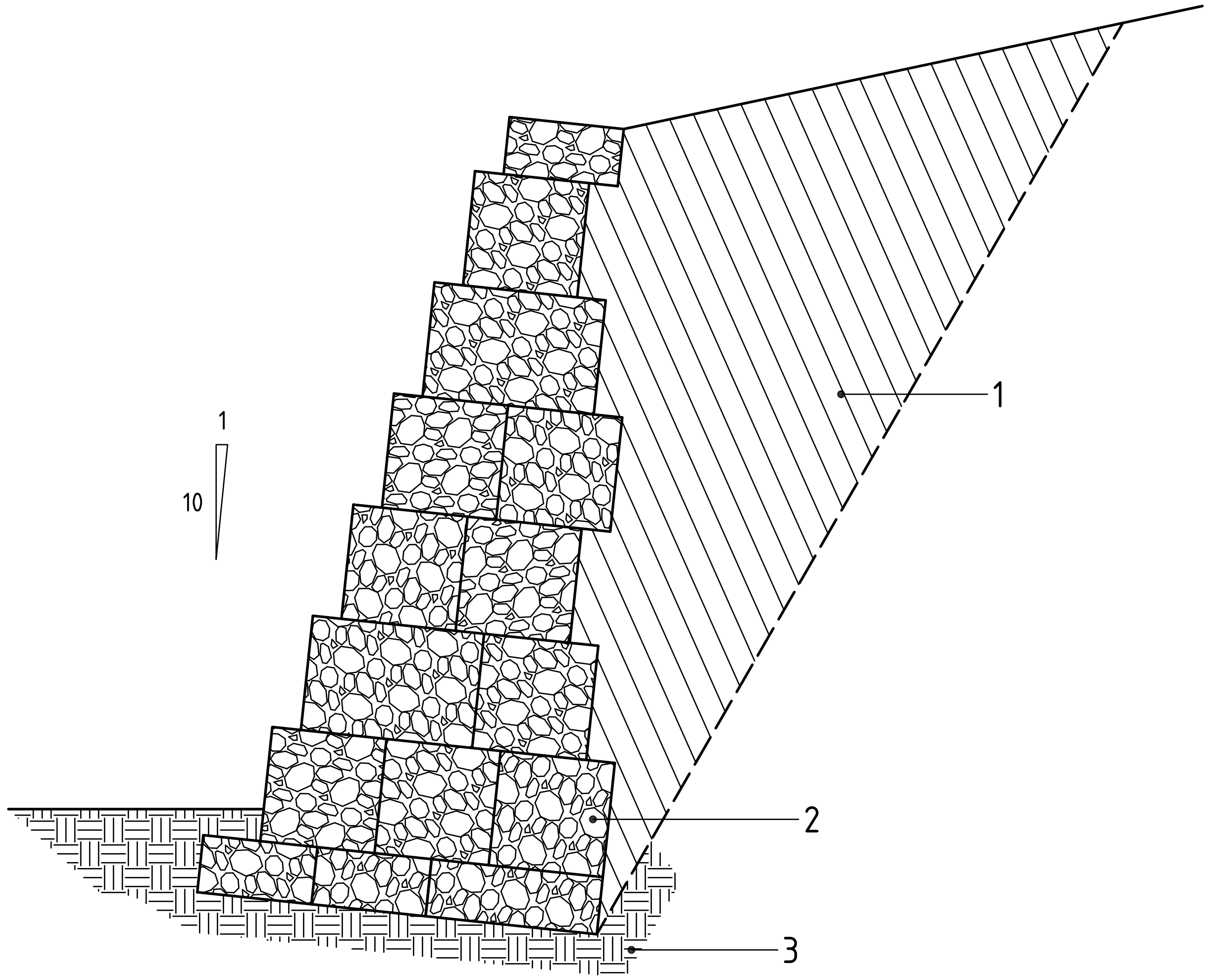


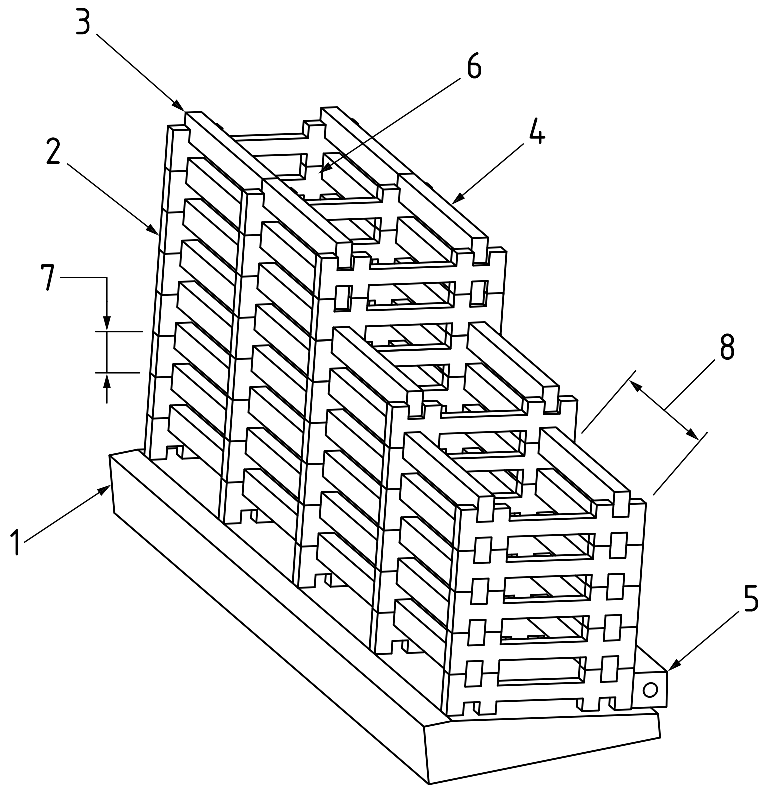
Figure 7 **Examples of gabion retaining walls** (2 of 2)



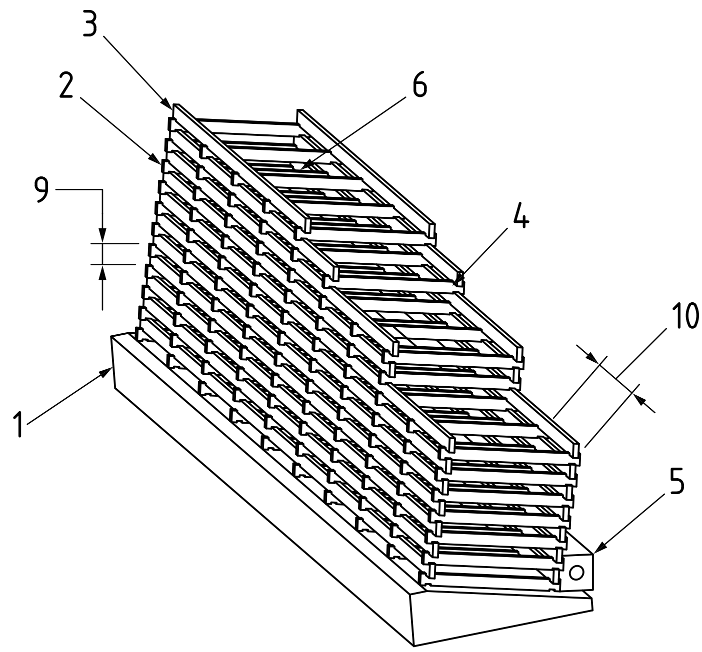
Key

1 Backfill
2 Gabion cage
3 Foundation
4 Standing water level

Figure 8 **Examples of crib walls**



a) Precast concrete



b) Timber

Key

1	Concrete foundation	6	Crib units filled with a suitable granular fill
2	Header	7	Typical course height 300 mm
3	Front stretcher	8	Typical centres 1 500 mm – 1 800 mm
4	Rear stretcher	9	Typical course height 160 mm
5	Drain where required	10	Typical centres 600 mm – 800 mm

5.2 Actions and design situations

5.2.1 Actions for gravity retaining walls should conform to **4.5.1**.

5.2.2 Design situations for gravity retaining walls should conform to **4.2.2**.

5.3 Design considerations

5.3.1 General

To avoid the illusion of tilting forward, the front face of a mass concrete retaining wall should be battered backwards at no less than 1 in 50.

5.3.2 Drainage

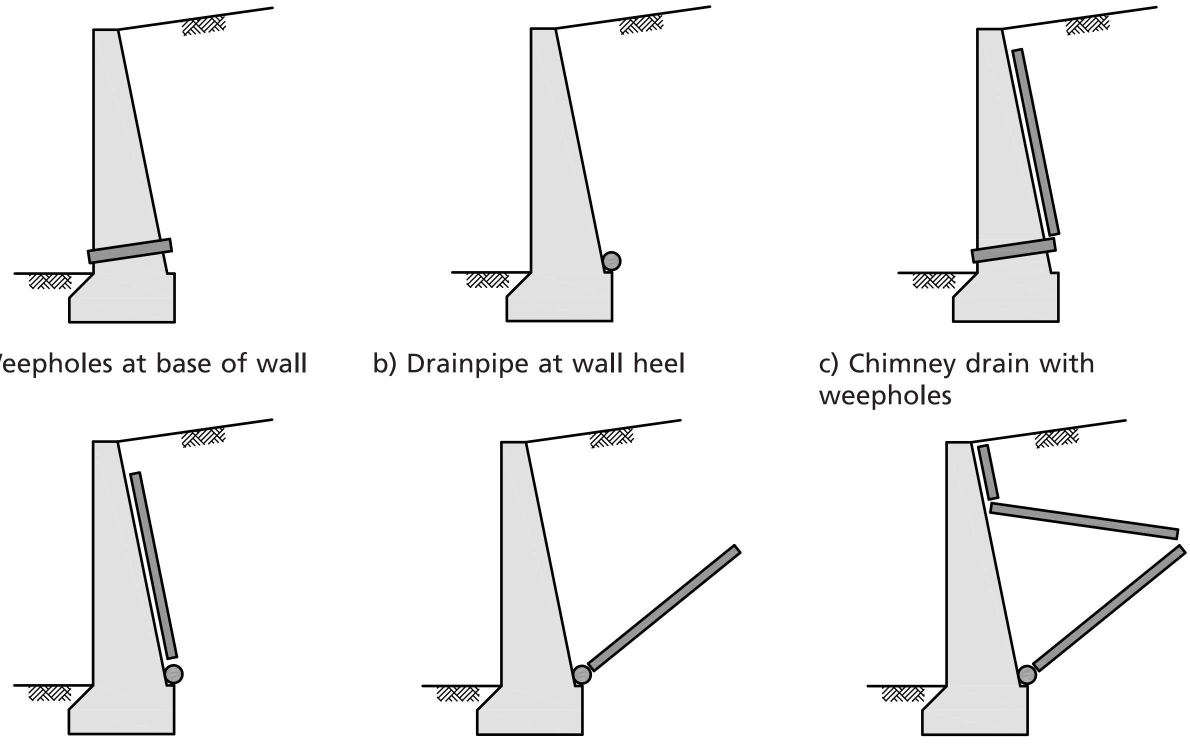
*COMMENTARY ON **5.3.2***

The provision of a drainage system behind a gravity wall can reduce the total pressures that the wall has to resist possibly by more than 30% in granular material.

Figure 9 shows some typical drainage systems behind a gravity retaining wall.

Pipes used in drainage systems behind gravity retaining walls should conform to Table 10.

Figure 9 **Typical drainage systems behind gravity retaining walls**



a) Weepholes at base of wall
b) Drainpipe at wall heel
c) Chimney drain with weepholes
d) Chimney drain with drainpipe
e) Inclined drain
f) Interceptor drains with drainpipe

NOTE Based on Earth Pressure and Earth-Retaining Structures *[2].*

Table 10 **Drainage systems for gravity retaining walls**

Location of pipe	Type of pipe	Requirements
Rear of wall	Vitrified clay	Perforated with flexible mechanical joints
	Unplasticized polyvinylchloride (UPVC), polypropylene (PP) or polyethylene (PE)	Diameter 80 mm to 150 mm Perforated with not less than 1 000 mm^2/m of holes per unit length of pipe
	Thermoplastics structured wall pipe	
	Geotextile wrapped land drain	
Drainage connections	Vitrified clay Thermoplastics structured wall pipe	Normal or surface water pipes

5.3.3 Gabion walls

5.3.3.1 The width of the horizontal tread of the steps should not exceed the depth of the gabion.

5.3.3.2 A gabion wall should be built to a batter to increase its resistance to overturning and sliding.

5.3.3.3 Counterforts or buttresses may be incorporated in the construction of gabion walls.

5.3.3.4 In large walls where the cross section is greater than 4 m wide, consideration should be given to using a cellular form of construction.

5.3.3.5 The outer and inner gabion faces should be tied by bulkheads of gabions and the cells between them filled with stone. The size and shape of the cells should be proportioned to achieve internal stability.

5.3.3.6 In rivers and in tidal waters, consideration should be given to installing a filter behind the wall, to prevent the leaching of fines.

5.3.3.7 Gabion boxes with cages longer than 1.5 m should be fitted with transverse vertical diaphragm panels at 1 m centres to prevent undue distortion and stone migration. The edges of any diaphragm panels should be fixed to the sides by lacing or clips with 2.2 mm minimum binding wire, galvanized or PVC coated, to match the gabion mesh.

5.3.3.8 Gabion units should bear down fully on the gabion below and not overhang the unit at the back by more than 150 mm, except in the case of a stepped revetment. Where gabion units do overhang, care should be taken to compact the backfill in the vicinity of and beneath the overhang.

5.3.3.9 Traffic loads may be ignored in the verification of wall stability if they are a distance greater than the wall's retained height behind the back of the wall.

5.3.3.10 Gabion walls may be designed to support traffic loads, provided the wall's flexibility is taken into account.

5.3.3.11 Gabion walls intended to support an existing slope should be built into the slope, after suitable trimming.

5.3.3.12 Foundations for buildings or other structures should not impose loading onto a gabion wall or its foundation.

5.3.4 Crib walls

5.3.4.1 The batter of a crib wall should normally be between 1-in-6 and 1-in-4 (horizontal to vertical).

5.3.4.2 A crib wall retaining up to 2 m of soil may be built with a nominal batter (i.e. near vertical) if its depth is greater than its retained height.

5.3.4.3 Timber crib walls may be built using whole logs or sawn timbers. If whole logs are used, plane faces should be formed at the points of contact to distribute the load and provide anchorage between adjacent members.

5.3.4.4 Timber crib walls should be formed with front and rear stretcher units tied at intervals by headers across the thickness of the wall. Headers should be anchored by notching or spikes so as to tie together the stretcher courses.

5.3.4.5 The face stretchers of a precast reinforced concrete crib wall should be positively anchored over the full thickness of the wall by interlocking headers. The headers should be aligned vertically to transmit load directly throughout the height of the wall without inducing bending moments in the supporting stretcher units.

5.3.4.6 Traffic loads may be ignored in the verification of wall stability if they are a distance greater than the wall's retained height behind the back of the wall.

5.3.4.7 Crib walls may be designed to support traffic loads, provided the wall's flexibility is taken into account.

5.3.4.8 Crib walls intended to support an existing slope should be built into the slope, after suitable trimming.

5.3.4.9 Foundations for buildings or other structures should not impose loading onto a crib wall or its foundation.

5.3.4.10 Weepholes should be provided if the infill is not free draining (for example if lean mix concrete infill has been used to increase stability). The infill zone immediately behind the wall should be built with free-draining material.

5.4 Calculation models

5.4.1 Virtual back of wall

Gravity retaining walls that do not have a planar back surface may be designed with a planar "virtual back", where the virtual back is:

- for a gravity wall with a stepped back: the vertical plane extending from the heel of the wall base to ground surface;
- for a crib wall: along the back of the cribs; and
- for a gabion wall: along a line from the inner bottom corner of the lowest basket to the inner top corner of the top basket.

5.4.2 Interface friction between wall and ground

*COMMENTARY ON **5.4.2***

In gabion walls, separation of the backfill stone and the retained soil is normally achieved by placing a geotextile membrane along the rear face of the wall. The presence of this membrane can reduce interface friction along the rear face.

5.4.2.1 Owing to their surface roughness, the design angle of interface friction, δ_d, between the rear of a gabion wall and the retained ground should be limited to:

$$\delta_d \leq k_{membrane} \times \varphi'_d \tag{27}$$

where:

φ'_d	is the design peak angle of shearing resistance of the ground; and
$k_{membrane}$	is a factor that accounts for the reduction of friction caused by the presence of a membrane placed against the rear face of the gabion wall.

5.4.2.2 For geotextile membranes, in the absence of reliable test data, the value of $k_{membrane}$ should be taken as 0.75 where the rear face of the wall is planar; otherwise it may be taken as 1.0. For other types of membrane, $k_{membrane}$ should be determined from test data.

5.4.3 "No tension" criterion for bearing pressure

The cross section of a gabion or crib wall should be proportioned so that the resultant force at any horizontal section lies within the middle third of that section.

5.5 Materials

5.5.1 Concrete

5.5.1.1 Plain (i.e. unreinforced) concrete incorporated into mass concrete retaining walls should conform to BS EN 1992-1-1:2004+A1:2014, Clause **12**.

5.5.1.2 Precast concrete blocks incorporated into crib walls should conform to BS EN 13369.

5.5.2 Steel

5.5.2.1 Steel and related products incorporated into gabion walls should conform to **4.3.7** and this subclause (**5.2.2**).

NOTE Gabions may be constructed using hexagonal woven wire mesh or welded wire mesh.

5.5.2.2 Wire mesh incorporated into gabion walls should conform to BS EN 10218-1 and BS EN 10218-2.

5.5.2.3 Hexagonal steel wire mesh products incorporated into gabion walls should conform to BS EN 10223-3.

5.5.2.4 Welded mesh gabion products incorporated into gabion walls should conform to BS EN 10223-8.

5.5.2.5 Welded joints in gabions should be designed to account for differential settlement of the cages.

5.5.2.6 Organic coatings on steel wire incorporated into gabion walls should conform to BS EN 10245-1, BS EN 10245-2, BS EN 10245-3 and BS EN 10245-5.

5.5.3 Timber

Timber and related products incorporated into crib walls should conform to **4.3.8**.

5.5.4 Masonry

Masonry and related products incorporated into unreinforced masonry retaining walls should conform to **4.3.9**.

5.5.5 Fill

5.5.5.1 Fill placed inside or behind gravity retaining walls should conform to **4.3.3**.

5.5.5.2 Naturally occurring rounded stone or quarried stone may be used as fill material in gabions. The size of the stone should be selected so that it cannot pass though the mesh, but otherwise should be as small and as uniform as possible. The maximum size of fill should not exceed 200 mm.

5.5.5.3 Stone whose size is smaller than the mesh opening may be used provided it is not used at external faces. A mesh panel or fabric should be provided to separate the smaller stone from stone that is larger than the mesh opening.

5.5.5.4 Stone fill used in gabion walls should be hard, angular to round, durable, and of such quality that it cannot deteriorate or fragment due to exposure to water or weathering during the design life of the wall.

5.5.5.5 The density of the infill used in gabion and crib walls should be assessed according to the gradings and proportion of voids after placing.

5.5.5.6 Infill material for cribwork should be durable, inert and free draining. Coarse sand, gravel and rock rubble should be used whenever obtainable. Loss of fill through the openings of the cribwork should be prevented.

NOTE Fill placed behind gravity walls may include lightweight fill, treated marginal fill, recycled fill, pulverized fuel ash, or colliery spoil.

5.6 Durability

5.6.1 Concrete

The durability of gravity concrete retaining walls should conform to **4.4.2**.

5.6.2 Steel

5.6.2.1 The durability of steel used in gravity walls should conform to **4.4.3**.

5.6.2.2 Uncoated wire gabions should only be used for temporary works, unless the wire diameter is at least 5 mm and the expected life of the unprotected steel is sufficient for such gabions to be used for permanent works.

5.6.2.3 Galvanized gabions may be used where the expected life of the galvanized wire exceeds the design life of the structure. If the expected life is insufficient, then polymer (PVC or PA6) coated wire or stainless steel mesh should be used instead.

5.6.2.4 Polymer coated gabions should not be used on coastal foreshores, where large shingle or heavy abrasive material is likely to be thrown against or washed over the structure by wave action.

5.6.2.5 Hexagonal woven mesh gabions should be made from galvanized wire conforming to BS EN 10244-2.

5.6.2.6 The panels of welded mesh gabions should either be made from zinc (95%) aluminium (5%) coating conforming to BS EN 10244-2:2009 (Class A) or should be hot dip galvanized after welding, in accordance with BS EN ISO 1461.

5.6.2.7 Polymer coatings for hexagonal woven wire mesh gabions should conform to BS EN 10223-3:2013, **6.5**, Table 4, and **6.7.3**.

5.6.2.8 PVC coatings for welded wire mesh gabions should conform to BS EN 10223-8:2013, **6.4** and **7.6.3**.

5.6.2.9 Coating diameter, thickness, and concentricity should conform to BS EN 10218-2:2012, Table 2 for both woven and welded gabions.

5.6.3 Timber

5.6.3.1 The durability of timber crib walls should conform to **4.4.4**.

5.6.3.2 Before being assembled into the crib, timbers for crib walls should normally be treated with a preservative conforming to BS 8417.

5.6.4 Masonry

The durability of unreinforced masonry retaining walls should conform to **4.4.5**.

5.7 Ultimate limit state design

5.7.1 General

The ultimate limit state design of gravity retaining walls should conform to **4.6** and this subclause (**5.7**).

5.7.2 Bearing

The ultimate bearing resistance of the foundation of a gravity retaining wall should conform to BS 8004:2015, **5.7.2**.

5.7.3 Sliding

5.7.3.1 The ultimate sliding resistance of the foundation of a gravity retaining wall should conform to BS 8004:2015, **5.7.3**.

5.7.3.2 The ultimate sliding resistance of a gabion wall should be verified at multiple levels above its base to prevent shear failure through the wall. The beneficial effect of the wire mesh should be ignored in this verification.

5.7.4 Rotation

The verification of ground resistance to rotation of a gravity retaining wall should conform to BS EN 1997-1:2004+A1:2013, **2.4.7.3**, using expressions (2.6a) and (2.7a) with partial factors for Design Approach 1 (as required by the UK National Annex to BS EN 1997-1).

5.8 Serviceability limit state design

The serviceability limit state design of gravity retaining walls should conform to **4.7**.

5.9 Structural design

5.9.1 General

The structural design of gravity retaining walls should conform to **4.8** and this subclause (**5.9**).

5.9.2 Mass concrete retaining walls

5.9.2.1 General

5.9.2.1.1 The design of plain (i.e. unreinforced) concrete gravity walls should conform to BS EN 1992-1-1:2004+A1:2014, Section 12.

5.9.2.1.2 Vertical joints should be provided at intervals, depending on the expected temperature range and the shape of the structure. Expansion joints should be lined with a resilient jointing material about 10 mm to 20 mm thick and sealed with a sealing compound.

5.9.2.1.3 Joints should be provided at significant steps in foundation level. Joints should also be provided where there are significant changes in retained height of the wall or where the nature of the foundation changes.

5.9.2.1.4 Joints should be provided where the ground beneath the foundation's base changes (e.g. from one soil type to another).

5.9.2.1.5 Joints should be provided between the wing walls of bridges and bridge abutments.

5.9.2.1.6 Movement joints in the masonry facing or cladding should be positioned and detailed in accordance with the recommendations of PD 6697 for stone masonry.

5.9.2.2 Facing units

5.9.2.2.1 Mass concrete retaining walls may be fitted with masonry cladding where a good appearance and good weathering qualities are important. To allow differential movement between it and the concrete, the cladding should be separated from the structural wall by cavity construction and provided with appropriate movement joints (see Figure 10).

5.9.2.2.2 This form of construction should not be used where there is a risk of impact damage, for example, where the wall adjoins a highway.

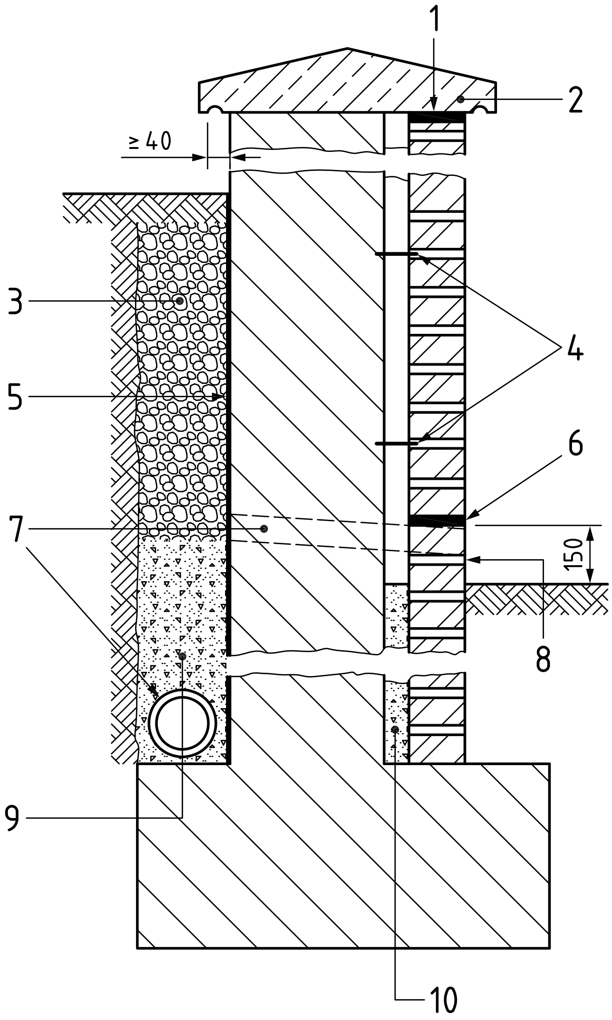
5.9.2.2.3 Care should be taken in the design and detailing of connections to ensure that differential stresses are not induced by the connections between the two surfaces.

5.9.2.2.4 Wall ties should be cast or fixed into the mass concrete retaining wall and subsequently built into the masonry cladding. Ties should be spaced at intervals not greater than 900 mm centres horizontally and 450 mm centres vertically, alternate rows staggered, and should preferably be stainless steel dovetail slot type with fishtail anchors to ensure correct coursing with the masonry.

5.9.2.2.5 Wall ties should conform to BS EN 845-1 and be used in accordance with PD 6697.

Figure 10 **Masonry clad mass concrete wall with cavity**

Dimensions in millimetres



Key

1 Damp proof course if required
2 Coping in reconstructed or natural stone or precast concrete conforming to BS 5642-2
3 Graded drainage media
4 Wall ties
5 Waterproofing if required
6 Damp proof course or two course damp proof course quality bricks
7 Drainage effected either by weepholes through wall or land drain at rear of wall
8 Cross joints left open at 900 mm centres to drain cavity (first course above ground level)
9 Concrete or puddle clay if weephole used
10 Concrete or mortar filling

5.9.3 Unreinforced masonry retaining walls

5.9.3.1 The structural design of unreinforced masonry retaining walls should conform to BS EN 1996-1-1.

5.9.3.2 Unreinforced masonry retaining walls should incorporate movement joints conforming to BS EN 1996-2 and PD 6697.

5.9.3.3 Where buttressed walls are used, a buttress should be provided each side of the movement joint.

5.9.4 Cribwalls

5.9.4.1 Units should be designed for the maximum loading condition at the base of the wall, so that all the units can be of the same size throughout the wall.

5.9.4.2 Units should be detailed and manufactured to provide plane bearing surfaces that are sufficiently large to prevent crushing failure from the loading involved. Due allowance should be made for any reduction in bearing area owing to manufacturing and erection tolerances.

5.9.4.3 Header units should be designed as beams over their unsupported length, to carry a load equal to the weight of the superimposed fill with maximum consolidation.

5.9.4.4 Stretchers should be designed to resist bending caused by the horizontal component of the earth pressure behind the cribwall, together with pressures induced by compaction of the fill.

5.9.4.5 The connection between the headers and stretchers should be designed to resist the reactions from the stretchers mainly by mechanical interlock, which is normally provided by using recesses or dowels.

5.9.4.6 The structural design of the headers and stretchers of a crib wall should consider:

- headers and stretchers spanning between joints, considered both as simply supported and with fixed ends;
- bearing failure at joints; and
- rupture of the headers at joints between the crib elements.

5.10 Execution

5.10.1 General

The execution of gravity walls should conform to **4.9** and this subclause (**5.10**).

5.10.2 Mass concrete retaining walls

5.10.2.1 Falsework and formwork used to construct mass concrete retaining walls should conform to BS EN 13670.

5.10.2.2 Horizontal joints in walls with stepped profiles should coincide with the position of the steps. A longitudinal groove may be formed to generate resistance to the shearing force at the joint.

5.10.2.3 Vertical construction joints should be at approximately 10 m centres and should coincide with contraction or expansion joints.

5.10.2.4 Where a wall is subject to hydrostatic pressures, hydrostatic uplift should be taken into account at construction joints. Horizontal construction joints should be designed to be watertight wherever possible, but if they cannot be watertight, the design should cater for full hydrostatic uplift.

NOTE 1 Guidance on the design and construction of joints in concrete structures can be found in CIRIA Report R146D *[28].*

NOTE 2 Guidance on concreting deep lifts and large volume pours can be found in CIRIA Report R135D *[29].*

5.10.3 Unreinforced masonry retaining walls

The execution of unreinforced masonry retaining walls should conform to BS EN 1996-2 and PD 6687.

5.10.4 Gabions walls

5.10.4.1 Gabions made of welded wire mesh should be electrically welded at every intersection, giving a minimum average weld shear strength of 70% of the minimum ultimate tensile strength of the wire.

5.10.4.2 When gabions are constructed on site from rolls or sheets of mesh, care should be taken to prevent rupture of the outer surface of the mesh (which can lead to progressive failure of the gabion).

5.10.4.3 Empty cages may be placed singly or joined together in groups. Woven wire mesh gabions should be stretched with a small winch before being wired to adjacent units that have already been filled. The tension should be maintained until the units are filled. Underwater gabions should be pre-filled before they are placed.

5.10.4.4 Cages should be tightly filled with some overfilling to allow for subsequent settlement. The top layer of stone may be of a smaller size than is used elsewhere in the cage. When filled, the gabion lids should be properly closed, without gaps, and wired down. Horizontal internal bracing wires should be fitted between the outer and inner faces at 330 mm centres in both woven mesh and welded mesh gabions which are deeper than 500 mm. When filled, the gabion lids should be properly closed without gaps, and wired down.

5.10.4.5 Horizontal internal bracing wires should be fitted between the outer and inner faces of the cage in both woven mesh and welded mesh gabions that are vertically deeper than 500 mm.

5.10.4.6 The vertical joints between individual units may be staggered in adjacent courses, to give a better appearance and to prevent the formation of weak vertical shear planes. Curves and angles in the face of the structure may be formed by cutting and folding the wire mesh to make specially shaped units.

5.10.4.7 Where gabions are subjected to wave action, there should be a minimal amount of movement of the stone filling inside the baskets. The filling should be tightly packed and the wire mesh should be taut. Baskets should be opened after a few tides have passed through the work and stone added to make good any settlement that has occurred in the filling. Any loose stone left over after construction should be removed and not left on the foreshore.

5.10.5 Crib walls

5.10.5.1 Where the infill material to a crib wall has a different grading to the backfill material, the rear wall drainage pipe should be pre-wrapped with a suitable geotextile prior to installation.

5.10.5.2 Where the ground's bearing resistance is adequate, the crib wall may be erected without a separate concrete foundation. It should be built off header units set on a granular bed.

5.10.5.3 The first row of headers should be positioned to line and level on the prepared foundation and held in place by the interlocking stretcher units; the batter should be checked. On sloping ground the foundation should be stepped to follow the slope, the steps being spaced to suit the unit module lengths.

5.10.5.4 The crib units are built in multiple courses, then infilled and backfilled in layers to suit the materials used. The fill should be placed to minimize the development of voids and to avoid disturbing the alignment of the crib units.

5.11 Testing

Testing of a gravity retaining wall should conform to prEN ISO 22477.

5.12 Supervision, monitoring, and maintenance

Supervision, monitoring, and maintenance of a gravity retaining wall should conform to **4.11**.

5.13 Reporting

Reports for gravity retaining walls should conform to **4.12**.

6 Semi-gravity retaining walls

*COMMENTARY ON Clause **6***

This clause applies to the design and construction of:

- *reinforced concrete retaining walls;*
- *reinforced and prestressed masonry retaining walls; and*
- *basement walls.*

6.1 Choice and design of semi-gravity retaining walls

6.1.1 General

6.1.1.1 The design of semi-gravity retaining walls should conform to BS EN 1997-1:2004+A1:2013, Clause **9**, and Clause **4**, and Clause **6** of this standard.

NOTE Reinforced concrete and reinforced and prestressed masonry retaining walls are gravity structures that support the retained ground by a combination of wall's weight and that of the backfill on its base. The stem of a semi-gravity wall resists the horizontal thrust of the retained ground in bending.

6.1.1.2 The design of the foundation to a semi-gravity retaining wall should conform to BS 8004.

6.1.1.3 Selection of a suitable type of semi-gravity retaining wall should take into account:

- soil conditions;
- loads;
- drainage;
- existing and proposed structures and roads;
- groundwater conditions;
- the availability of space; and
- the geometry of the structure.

NOTE 1 Guidance on the selection of a suitable semi-gravity retaining wall can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 62 *[1] and in* Earth Pressure and Earth-Retaining Structures (3rd edition), Chapter 6 *[2].*

NOTE 2 Guidance on the selection of low-height modular retaining walls (including precast reinforced concrete stem walls) can be found in CIRIA Report C516, Chapter 3 *[27].*

6.1.2 Reinforced concrete retaining walls/stem walls

*COMMENTARY ON **6.1.2***

Reinforced concrete retaining walls are suitable for retained heights typically up to about 8 m; for greater heights, a counterfort or buttress is normally needed to prevent the wall's stem becoming uneconomically thick.

Reinforced concrete retaining walls may be cast-in-place or precast in a factory. Precast walls are available in standard sizes up to 4 m high.

The main types of reinforced concrete retaining walls are:

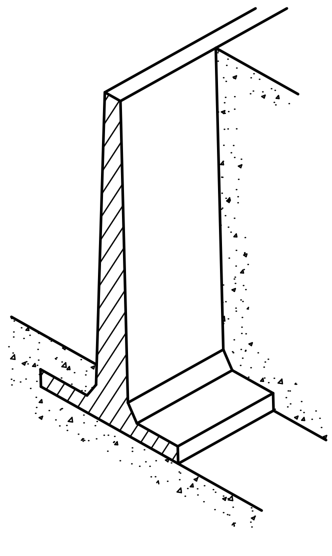
- *cantilever or stem walls (also known as "T-shaped" walls);*
- *reverse cantilever walls (also known as "L-shaped" walls);*
- *counterfort walls;*
- *buttressed walls.*

Buttressed reinforced concrete retaining walls are seldom used.

Figure 11 shows examples of typical reinforced concrete walls.

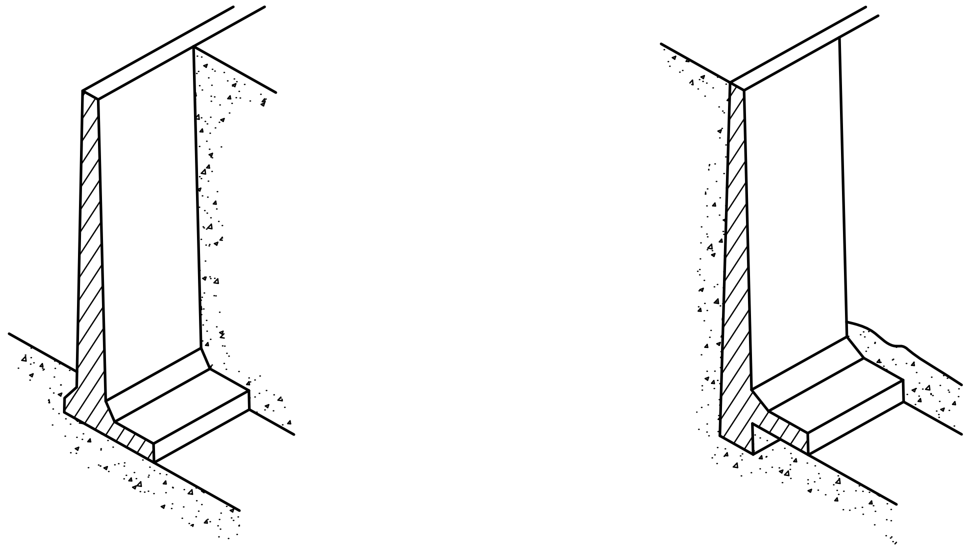
Precast reinforced concrete retaining walls for maritime works should conform to BS 6349-2:2010, **7.5**.

Figure 11 **Examples of reinforced concrete retaining walls** (1 of 2)

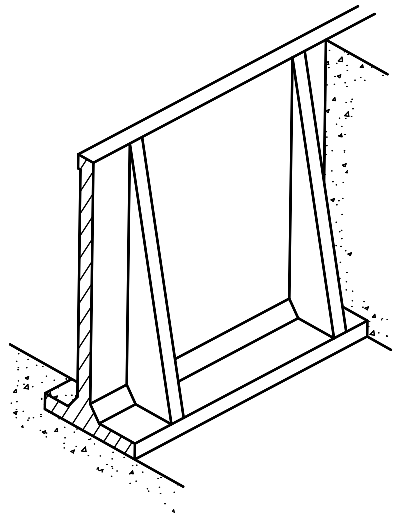


a) T-shaped wall

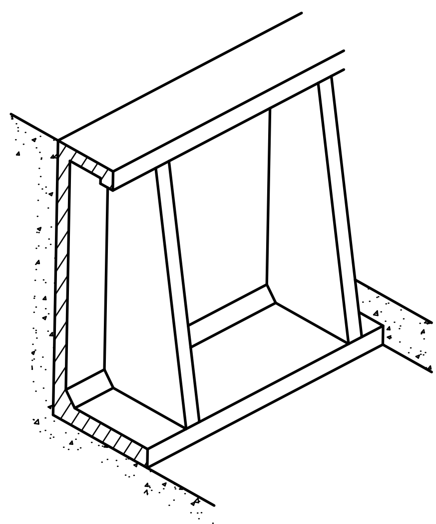
Figure 11 **Examples of reinforced concrete retaining walls** (2 of 2)



b) L-Shaped wall (without key and with key)



c) Reinforced concrete wall with counterforts



d) Reinforced concrete wall with buttresses

6.1.3 Reinforced and prestressed masonry retaining walls

*COMMENTARY ON **6.1.3***

Reinforced masonry retaining walls are suitable for retained heights typically up to about 3 m; prestressed masonry retaining walls are suitable typically up to about 5 m. Masonry walls benefit from a good appearance and good weathering capability.

Prestressed masonry has greater resistance to lateral loading owing to its increased flexural tensile capacity compared to non-prestressed masonry.

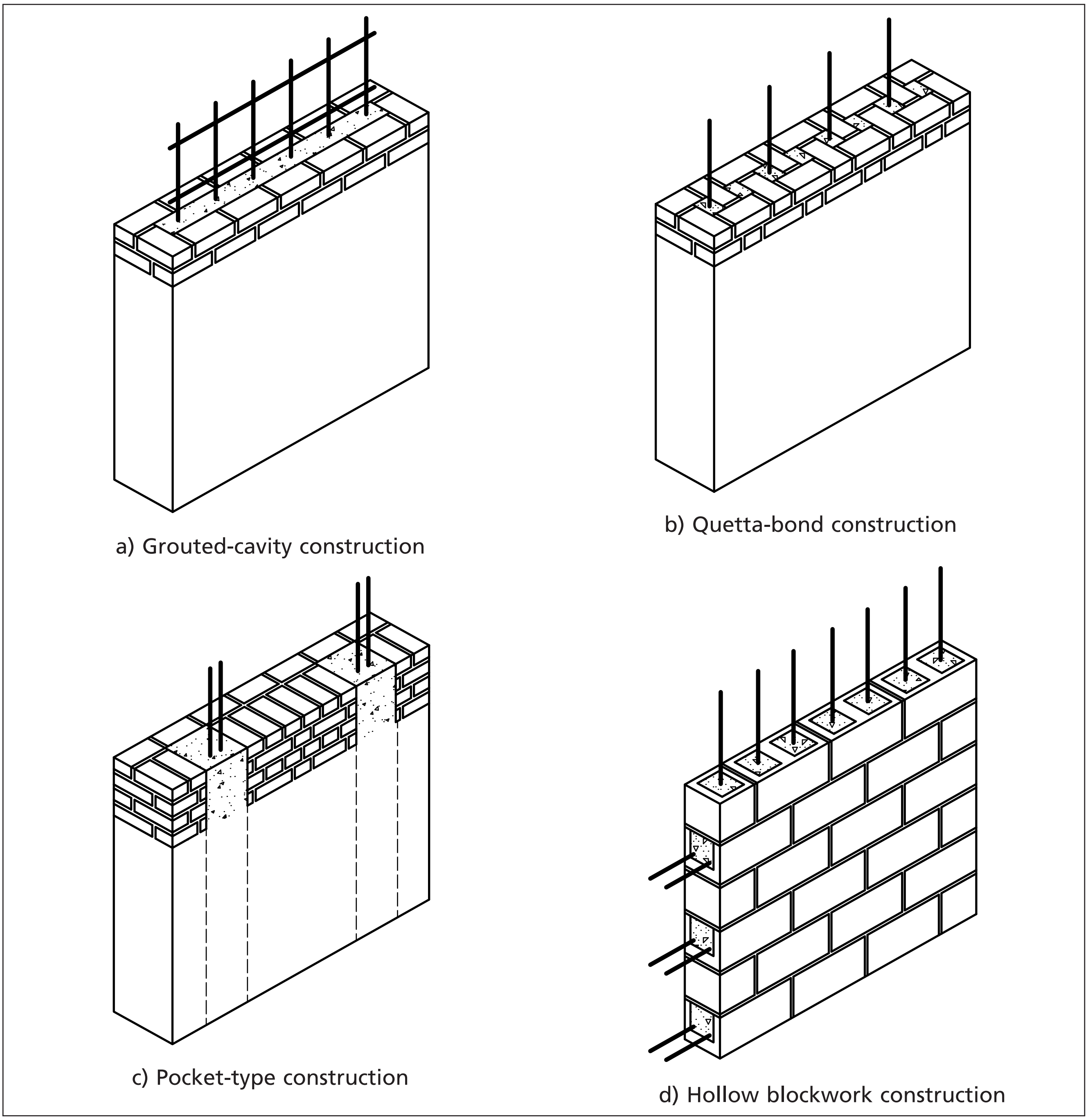
The main types of reinforced masonry retaining walls are of:

- *grouted cavity construction;*
- *quetta-bond construction;*
- *pocket-type construction; or*
- *hollow blockwork construction.*

Figure 12 shows examples of typical reinforced masonry retaining walls.

Figure 13 shows a post-tensioned masonry retaining wall of diaphragm construction.

Figure 12 **Examples of reinforced and prestressed masonry retaining walls**



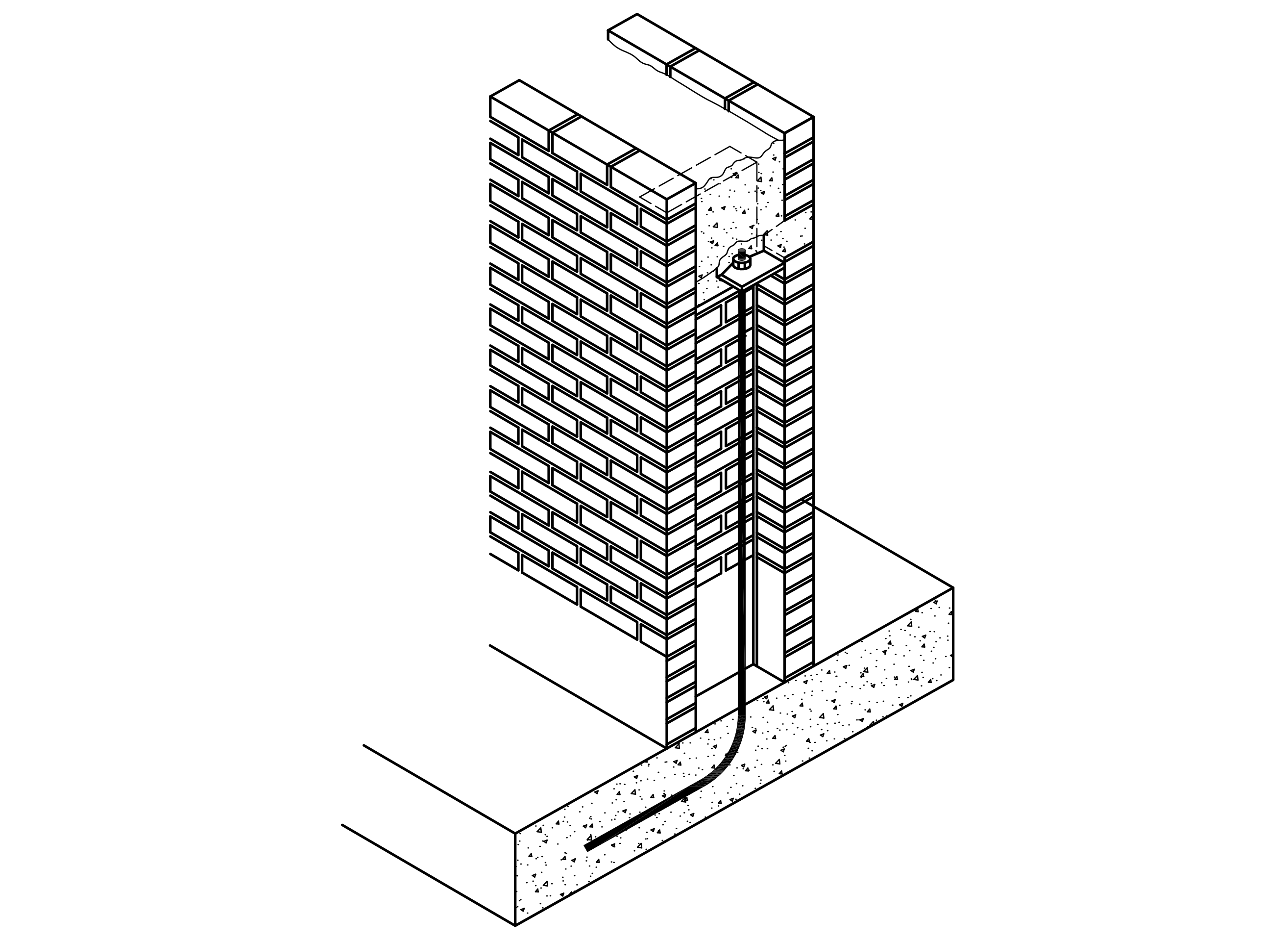
a) Grouted-cavity construction

b) Quetta-bond construction

c) Pocket-type construction

d) Hollow blockwork construction

Figure 13 **Post-tensioned masonry diaphragm wall construction**



6.2 Actions and design situations

6.2.1 Design situations for semi-gravity retaining walls should conform to **4.2.2**.

6.2.2 Actions for semi-gravity retaining walls should conform to **4.5.1**.

6.3 Design considerations

6.3.1 General

To avoid the illusion of tilting forward, the exposed face of a semi-gravity wall should lean back into the retained ground at a batter no less than 1 (horizontal) in 50 (vertical).

6.3.2 Drainage

Drainage systems behind semi-gravity walls should conform to **5.3.2**.

6.4 Calculation models

Calculation models for semi-gravity retaining walls should conform to BS EN 1997-1.

6.5 Materials

6.5.1 Concrete

6.5.1.1 Concrete and related products incorporated into reinforced concrete retaining walls should conform to **4.3.6**.

NOTE Reinforced concrete retaining walls may be cast-in-place or precast.

6.5.1.2 Precast concrete retaining wall elements should conform to BS EN 15258.

6.5.2 Steel

Steel and related products incorporated into semi-gravity walls should conform to **4.3.7**.

6.5.3 Masonry

6.5.3.1 Masonry and related products incorporated into unreinforced masonry retaining walls should conform to **4.3.9**.

6.5.3.2 Reinforcement in masonry retaining walls should conform to PD 6697.

6.5.4 Fill

Fill placed behind semi-gravity retaining walls should conform to **4.3.3**.

6.6 Durability

6.6.1 Concrete

The durability of reinforced concrete retaining walls should conform to **4.4.2**.

6.6.2 Steel

The durability of steel and related products used in semi-gravity walls should conform to **4.4.3**.

6.6.3 Masonry

The durability of reinforced and prestressed masonry retaining walls should conform to **4.4.5**.

6.7 Ultimate limit state design

6.7.1 General

The ultimate limit state design of semi-gravity retaining walls should conform to **4.6** and this subclause (**6.7**).

6.7.2 Bearing

The ultimate bearing resistance of the foundation of a semi-gravity retaining wall should conform to BS 8004:2015, **5.7.2**.

6.7.3 Sliding

The ultimate sliding resistance of a semi-gravity retaining wall should conform to BS 8004:2015, **5.7.3**.

6.8 Serviceability limit state design

The serviceability limit state design of semi-gravity retaining walls should conform to **4.7**.

6.9 Structural design

6.9.1 General

The structural design of semi-gravity retaining walls should conform to **4.8** and this subclause (**6.9**).

6.9.2 Reinforced concrete retaining walls

6.9.2.1 General

6.9.2.1.1 The structural design of reinforced concrete retaining walls should conform to BS EN 1992-1-1.

6.9.2.1.2 Where reinforced concrete walls rely on other structures for support, the construction sequence of the structures and the wall and possible changes in use which might affect the supports should be taken into account in the design.

6.9.2.1.3 The quantity and spacing of reinforcement in a reinforced concrete wall should also be selected to control cracking due to shrinkage and/or flexure.

6.9.2.1.4 To prevent cracking owing to differential settlement, vertical joints should be provided at intervals along the wall, depending on:

- the type of foundation;
- the condition of exposure;
- the shape of the structure; and
- the quantity and spacing of reinforcement.

6.9.2.1.5 Where necessary, the joints should be lined with a resilient jointing material about 10 mm to 20 mm thick and sealed with a proprietary sealing compound. Waterbars might also be required, depending on groundwater conditions. Reinforcement should be curtailed each side of the joint.

6.9.2.1.6 Joints should be provided at significant steps in foundation level. Joints should also be provided where there are significant changes in retained height of the wall or where the nature of the foundation changes.

6.9.2.1.7 Joints should be provided between the wing walls of bridges and bridge abutments.

6.9.2.1.8 In counterfort walls, a counterfort should be provided each side of the joint.

6.9.2.1.9 Details of expansion, contraction and movement joints and their spacing reference should conform to BS EN 1992-3.

6.9.2.2 Wall stem

6.9.2.2.1 The wall stem should be designed as a cantilever, taking into account the forces and pressures acting on it.

6.9.2.2.2 The diameter of the vertical bars in the upper part of the wall stem may be reduced in proportion to the bending moment carried by that part of the stem. The length of the lower (heavier) bars should be chosen so that they can be properly handled on site. The upper (lighter) bars should be spliced onto the lower bars using staggered splices.

6.9.2.3 Base slab

6.9.2.3.1 The toe and heel portions of the base slab should be designed as cantilevers, taking into account the forces and pressures acting on the base.

6.9.2.3.2 If a splay is provided at the junction of the stem, toe and heel of the wall, the critical bending moments and shear forces should be calculated at the ends of the splays.

NOTE The cost of splays is often high compared with the saving in material they allow.

6.9.2.4 Counterforts and buttresses

6.9.2.4.1 Counterforts should be designed as cantilevers of T-section and the wall stem as a continuous slab. The design should transfer the major part of the earth thrust from the slab to the counterfort.

6.9.2.4.2 The lower portion of the wall slab should be designed as cantilevering from the base and simultaneously spanning between the counterforts.

6.9.2.4.3 Buttresses should be designed in a manner similar to counterforts.

6.9.2.5 Facing units

NOTE Non-structural precast concrete facing panels may be fitted to retaining walls to ensure that the completed structure blends with the surrounding environment.

Reinforced concrete retaining walls may be fitted with masonry cladding where a good appearance and good weathering qualities are important. Such cladding should conform to **5.9.2**.

6.9.3 Reinforced and prestressed masonry retaining walls

6.9.3.1 The structural design of reinforced and prestressed masonry retaining walls should conform to BS EN 1996-1-1.

6.9.3.2 Reinforced and prestressed retaining walls should incorporate movement joints conforming to BS EN 1996-2 and PD 6697.

6.9.3.3 Where buttressed walls are used, a buttress should be provided each side of the movement joint.

6.10 Execution

6.10.1 General

The execution of semi-gravity walls should conform to **4.9** and this subclause (**6.10**).

6.10.2 Reinforced concrete retaining walls

6.10.2.1 The execution of reinforced concrete retaining walls should conform to **4.9.1** and this subcclause (**6.10.2**).

6.10.2.2 Construction joints should be kept to a minimum.

6.10.2.3 To facilitate construction, construction joints should be provided between the wall base (or splay) and the wall stem, with additional horizontal joints in the wall stem to suit the lifts of the formwork.

6.10.2.4 Vertical joints should be positioned at points of minimum shear and at approximately 10 m centres. Reinforcement should pass through the joints.

6.10.3 Reinforced and prestressed masonry retaining walls

The execution of reinforced and prestressed masonry retaining walls should conform to **4.9.1**.

6.11 Testing

Testing of a semi-gravity retaining wall should conform to prEN ISO 22477.

6.12 Supervision, monitoring, and maintenance

Supervision, monitoring, and maintenance of a semi-gravity retaining wall should conform to **4.11**.

6.13 Reporting

Reports for semi-gravity retaining walls should conform to **4.12**.

7 Embedded retaining walls

*COMMENTARY ON Clause **7***

This clause applies to the design and construction of:

- *sheet pile walls;*
- *bored cast-in-place concrete pile walls;*
- *diaphragm walls; and*
- *soldier/king pile walls.*

7.1 Choice and design of embedded retaining walls

7.1.1 General

7.1.1.1 The design of embedded retaining walls should conform to BS EN 1997-1:2004+A1:2013, Clause **9**, and Clause **4** and Clause **7** of this standard.

7.1.1.2 Selection of a suitable type of embedded retaining wall should take into account the guidance given in *Embedded retaining walls – guidance for economic design* (C580) [N3], including:

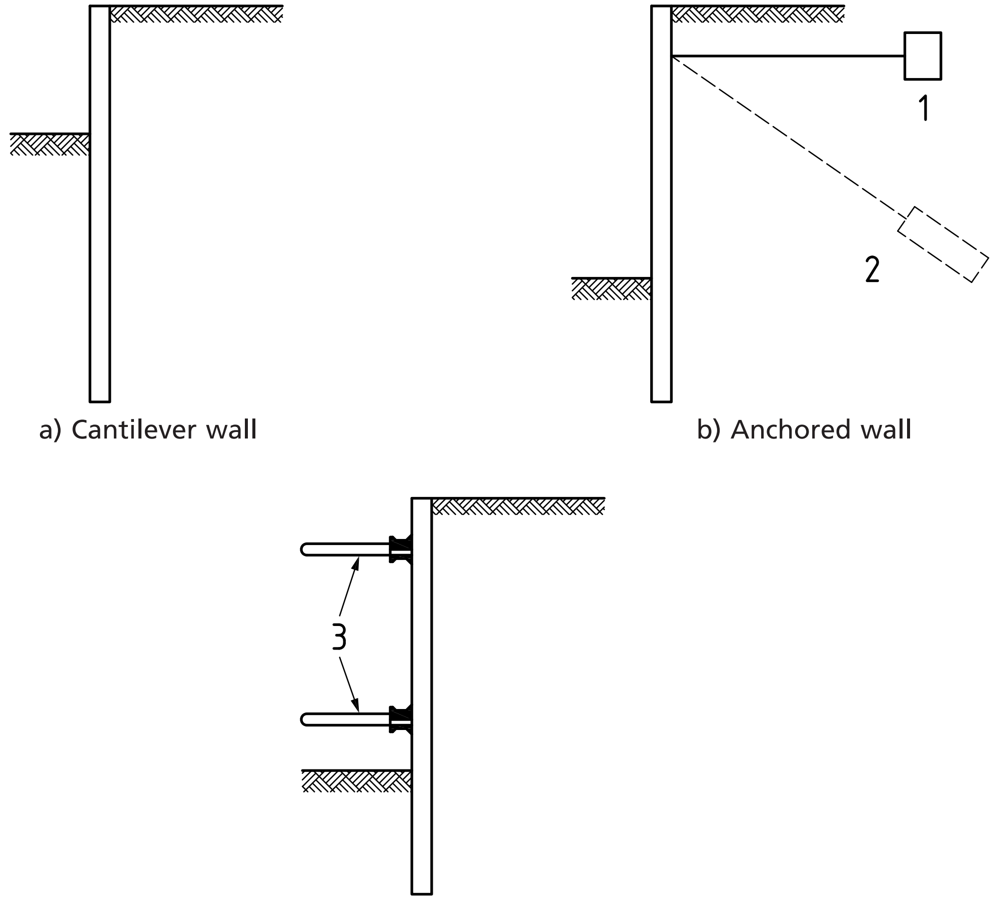
- cost of construction;
- ground and groundwater conditions;
- the need to restrict ground movements to within acceptable limits;
- extent of site to accommodate construction plant;
- compatibility with the permanent works;
- durability;
- contaminated ground;
- environmental issues; and
- speed of construction.

NOTE 1 Guidance on the choice of a suitable embedded retaining wall can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapters 62 and 85 *[1] and in* Earth Pressure and Earth-Retaining Structures (3rd edition), Chapter 6 *[2].*

NOTE 2 See Figure 14 for examples of embedded retaining walls.

7.1.1.3 The design and construction of anchors used to support embedded retaining walls should conform to BS 8081.

Figure 14 **Examples of embedded retaining walls**



a) Cantilever wall b) Anchored wall

c) Propped wall

Key

1 Deadman

2 Alternative ground anchor

3 Struts

7.1.2 Sheet pile walls

*COMMENTARY ON **7.1.2***

Sheet pile walls are constructed by driving, vibrating or pressing sheet piles into the ground without any material being removed. They are traditionally installed using hammers, which can cause a significant noise and vibration. In recent years, installation using pre-augering in dense coarse soil and press-in systems in fine soils have become more prevalent in urban environments, to overcome noise and vibration hazards.

7.1.2.1 Steel sheet pile walls

7.1.2.1.1 Hot rolled steel sheet piles should be manufactured in accordance with BS EN 10248.

7.1.2.1.2 Cold formed steel sheet piles should be manufactured in accordance with BS EN 10249.

7.1.2.2 Timber sheet pile walls

7.1.2.3 Precast concrete sheet pile walls

7.1.3 Bored cast-in-place concrete pile walls

*COMMENTARY ON **7.1.3***

There are two main arrangements of bored cast-in-place concrete pile walls:

- *"contiguous" (or "close bored"), which are built with piles at centres slightly greater than their diameter; and*
- *"secant", which are built with piles at centres less than their diameter (alternate piles are full circular section and intervening piles are cut away in part during construction).*

Bored cast-in-place concrete piles may be constructed in supported or unsupported borings that are created by rotary drilling or continuous flight auger (CFA) equipment.

Unless special measures are taken to provide a seal between adjacent piles, contiguous pile walls should not be used to retain water-bearing granular soils.

NOTE Secant pile walls can provide a watertight retaining wall provided the tolerances of position and verticality are sufficient to eliminate gaps between adjacent piles.

7.1.4 Diaphragm walls

Diaphragm retaining walls for maritime works should conform to BS 6349-2:2010, **7.11**.

NOTE Diaphragm walls are formed from interlocking reinforced concrete panels that may be cast-in-place or precast.

7.1.5 Soldier/king pile walls

*COMMENTARY ON **7.1.5***

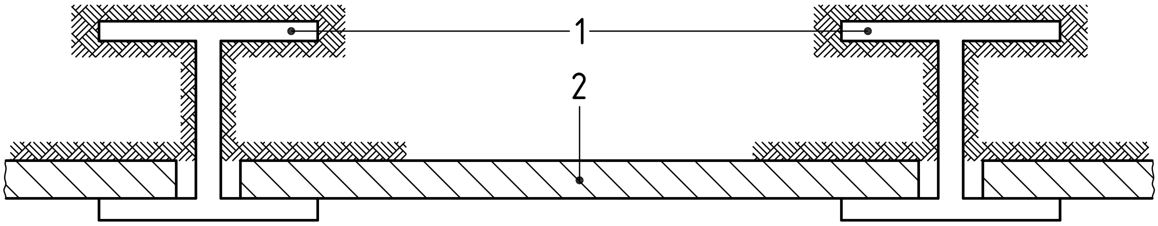
Soldier/king pile walls comprise isolated posts that are installed along the line of the wall with the ground between the posts being supported by railway sleepers, concrete panels, wooden or steel sheeting, or cast-in-place or spray concrete.

King piles (or "posts") are also called "solder piles".

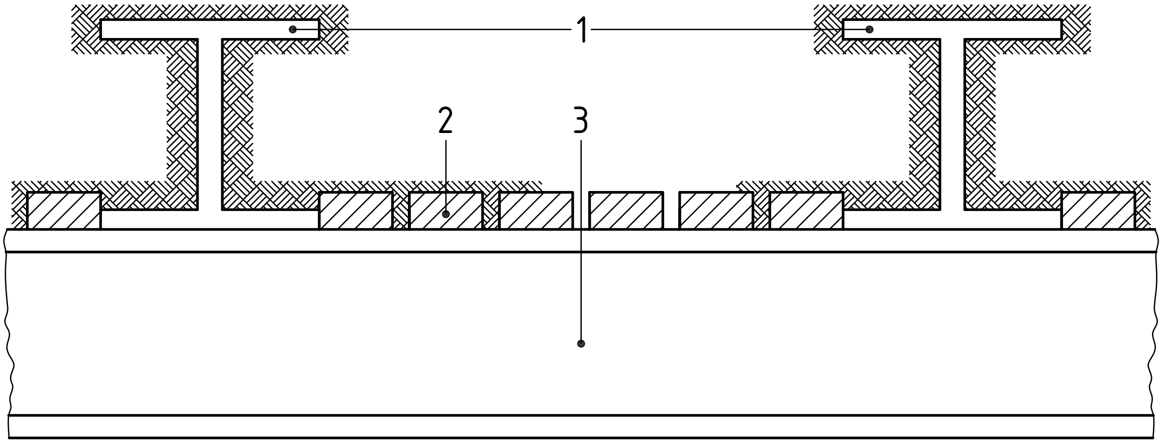
King pile walls are also known as "Berlin walls".

The lagging between adjacent soldier/king piles may be horizontal or vertical (as shown in Figure 15).

Figure 15 **Lagging between adjacent soldier/king piles**



a) Horizontal sheeting



b) Vertical sheeting

Key

1 Soldier piles

2 Sheeting (lagging)

3 Waling

7.2 Actions and design situations

7.2.1 Actions for embedded retaining walls should conform to **4.5.1**.

7.2.2 Design situations for embedded retaining walls should conform to **4.2.2**.

7.3 Design considerations

7.3.1 General

*COMMENTARY ON **7.3.1***

The use of flexible cantilever retaining walls (such as sheet pile walls) is typically acceptable up to a maximum retained height of about 5 m, depending on ground conditions. Where soft or loose soils occur in front of the wall, this maximum retained height might need to be reduced significantly.

The use of stiff cantilever retaining walls (such as diaphragm walls) can be acceptable up to a maximum retained height of about 8 m, depending on the design situation.

The use of very stiff cantilever retaining walls (such as combined and high-modulus cantilever steel sheet pile walls and composite diaphragm walls) can be acceptable up to a maximum retained height of about 12 m, depending on the design situation.

Where services are located behind embedded retaining walls (particularly cantilever walls), the effect of wall deflection on these services should be considered.

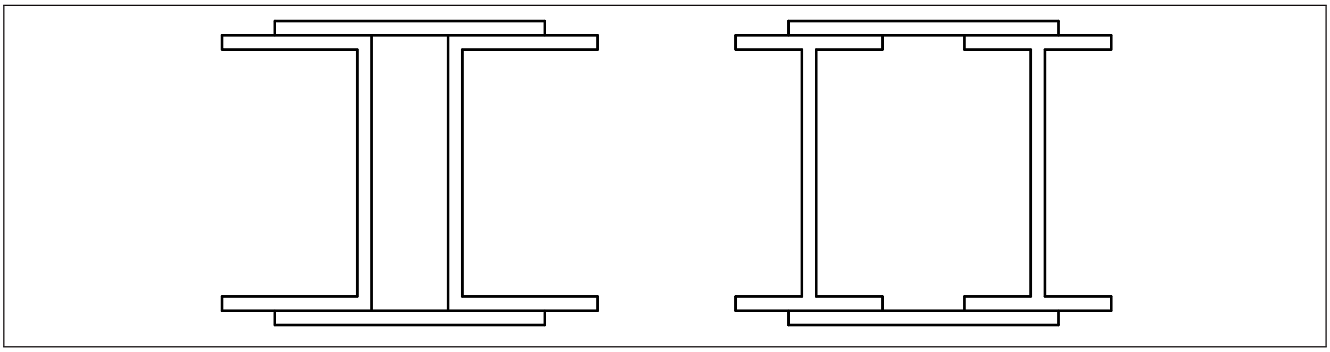
7.3.2 Soldier/king pile walls

7.3.2.1 Piles may be of reinforced concrete or steel sections used either singly or in pairs.

7.3.2.2 Where steel sections are used, they may be set into bored concrete piles below the depth of excavation.

7.3.2.3 Where steel sections are used in pairs to form soldier piles, adequate batten plates or spacers should be provided to ensure the composite action of both piles, as shown in Figure 16.

Figure 16 **Solider piles comprising a pair of steel sections with end plates**



7.3.2.4 The ground between individual soldier piles should be supported by horizontal sheets, which should be fabricated from:

- reinforced or prestressed precast concrete planks;
- cast-in-place concrete planks or sprayed concrete;
- steel trench sheets; or
- timber lagging.

NOTE 1 Unreinforced precast concrete planks might exhibit brittle failure without visible or audible warning.

NOTE 2 Guidance on suitable thicknesses of rough-cut timber lagging can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 85 *[1].*

NOTE 3 The most common material for supporting the ground is timber lagging.

7.3.3 Waling beams

7.3.3.1 Where props or anchors are used to support an embedded retaining wall at various levels, waling beams should be provided along the face of the wall at these support levels.

7.3.3.2 Waling beams may be designed as horizontally spanning beams.

7.3.3.3 Where steel beams are used, a satisfactory method of wedging or infilling should be provided between individual piles and the waling beam, to take up gaps that occur because of surface irregularities or deviations from true verticality and position of individual piles.

7.3.4 Capping beams

A continuous structural beam should be used to connect pile heads together, thereby unifying their behaviour as an earth retaining wall and in order to distribute any vertical load which might also be applied.

7.4 Calculation models

*COMMENTARY ON **7.4***

Embedded retaining walls may be designed using a variety of calculation models, including:

- *the limiting equilibrium method;*
- *discrete spring models; or*
- *continuum models.*

Discrete spring and continuum models are commonly termed "soil-structure interaction" (SSI) analyses.

NOTE Guidance on calculation models for the design of embedded retaining walls can be found in CIRIA Report C580 *[N3] and* Earth Pressure and Earth-Retaining Structures (3rd edition), Chapters 8 and 10 *[2].*

7.4.1 Limiting equilibrium method

*COMMENTARY ON **7.4.1***

The limiting equilibrium method (LEM) is a traditional method of retaining wall design that has been used successfully to design mainly cantilever and propped-cantilever (i.e. single-propped) retaining walls.

In the LEM, the ground surrounding the embedded retaining wall is assumed to be in a state of collapse, with its strength fully mobilized on both sides of the wall. The wall itself is assumed to be about to rotate as a rigid body about a fixed point, which, in the case of cantilever wall, is determined from the analysis and lies below formation level; or, in the case of a propped-cantilever wall, is coincident with the location of the prop or support.

The LEM can be used to determine the ultimate limit state of cantilever and propped-cantilever embedded retaining walls, but not their serviceability limit state. For propped-cantilever walls, a more efficient design might be achieved using a method that takes account of ground-structure interaction or a method of stress-redistribution.

7.4.1.1 The limiting equilibrium method may be used to determine the required depth of embedment of a cantilever or propped-cantilever (i.e. single-propped) embedded retaining wall, and the resulting bending moments and shear forces in it.

NOTE Further information about the limiting equilibrium method can be found in Earth Pressure and Earth-Retaining Structures (3rd edition), Chapter 8.5 *[2] and in* CIRIA Report C580 *[N3].*

7.4.1.2 The bending moments and prop forces that are calculated for propped-cantilever (i.e. single-propped) embedded retaining walls by the limiting equilibrium method may be adjusted for relative wall-soil stiffness using the methods presented by Rowe [30] or Potts and Bond [31].

7.4.2 Discrete spring models

*COMMENTARY ON **7.4.2***

A discrete spring model is a type of soil-structure interaction analysis, in which the ground is modelled by discrete springs and the wall by a continuous beam, thereby allowing a traditional beam-on-spring analysis to be performed.

There are two main forms of discrete spring model: one involving linear-elastic ("Winkler") springs with no interaction between them; the other involving non-linear ("intelligent") springs with interaction. In both cases, earth pressures are limited by their active and passive values. Winkler spring models are also known as "subgrade reaction" models. Intelligent spring models are also known as "pseudo finite element" models.

Discerete spring models can be used to determine both the ultimate and serviceability limit states of embedded retaining walls.

7.4.2.1 Discrete spring models may be used to determine the required depth of embedment and horizontal displacement of an embedded retaining wall, and the resulting bending moments and shear forces in it.

7.4.2.2 Discrete spring models used for embedded retaining wall design should take into account both the stiffness of the ground and limiting earth pressures.

NOTE Further information about discrete spring models can be found in Earth Pressure and Earth-Retaining Structures (3rd edition), Chapter 8.6 *[2] and in* CIRIA Report C580 *[N3].*

7.4.3 Continuum models

*COMMENTARY ON **7.4.3***

A continuum model is a type of soil-structure interaction analysis, in which the ground and the wall are modelled by individual elements that have different engineering properties throughout the domain of the problem. The analysis is performed numerically to bring the wall into equilibrium under the applied loads, while ensuring strain compatibility between the wall and the ground.

Continuum models include techniques such as:

- *the finite element method;*
- *the boundary element method; and*
- *the finite difference method.*

Continuum models can be used to determine both the ultimate and serviceability limit states of an embedded retaining wall.

Continuum models may be used to determine the required depth of embedment and horizontal displacement of an embedded retaining wall, and the resulting bending moments and shear forces in it.

NOTE Further information about continuum models can be found in the book Earth Pressure and Earth-Retaining Structures (3rd edition), Chapter 8.7 *[2] and in* CIRIA Report C580 *[N3].*

7.4.4 Successive stage analysis

*COMMENTARY ON **7.4.4***

The limiting equilibrium method (LEM) can be extended to include the design of multi-propped embedded retaining walls using a technique known as "successive stage analysis" (see Figure 17). In this method, hinges are introduced into the wall at the levels of the props so that two separate calculations can be performed, one a simple beam analysis of the upper part of the wall and the other a propped-cantilever analysis of the bottom part of the wall.

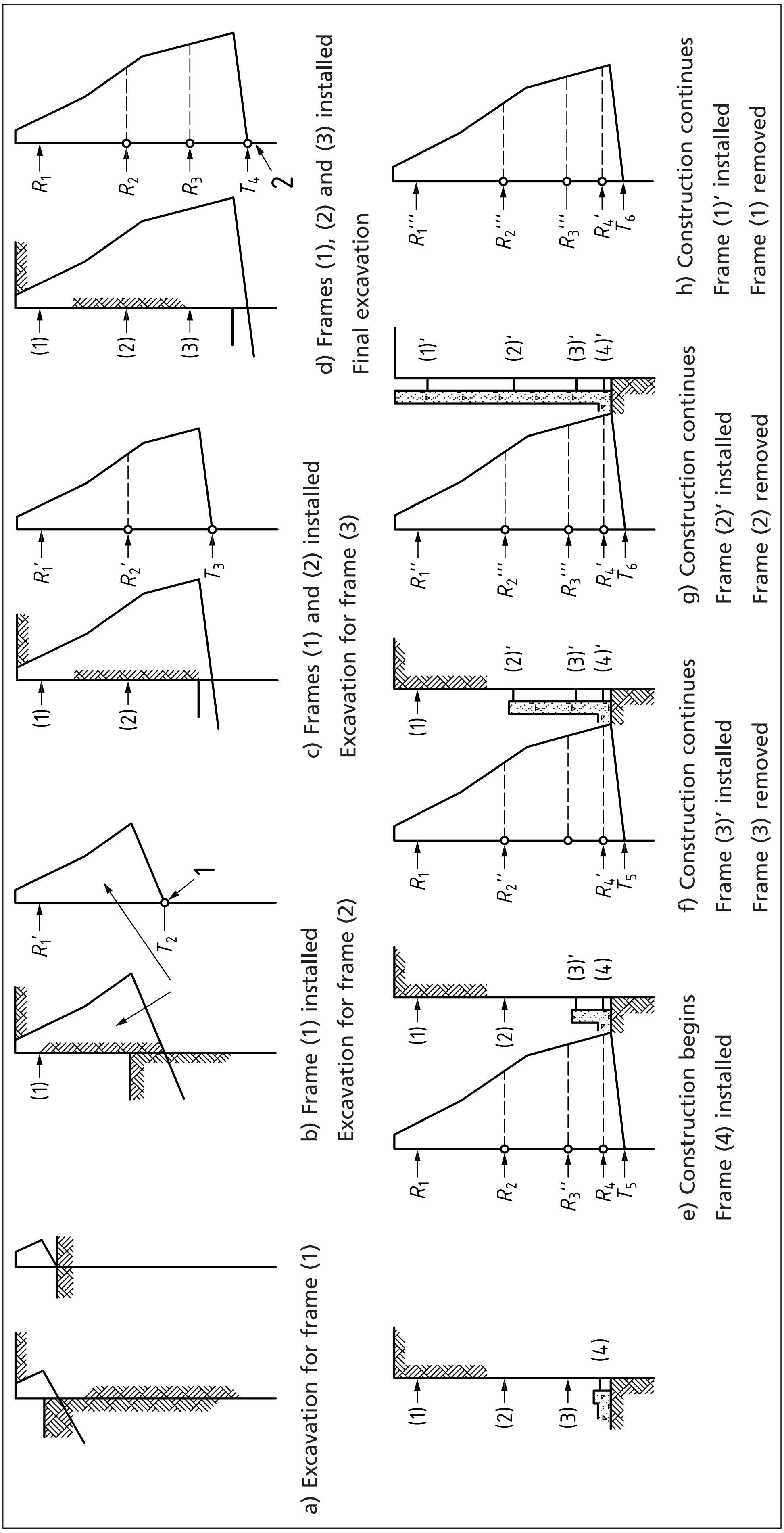
Successive stage analysis is also known as the "hinge" method.

A successive stage analysis can be used to determine the ultimate limit state of multi-propped embedded retaining walls, but not their serviceability limit state.

Successive stage analysis may be used to determine the required depth of embedment of a multi-propped embedded retaining wall, and the resulting bending moments and shear forces in it.

NOTE Further information about successive stage analysis can be found in the Piling Handbook (8th Edition) *[32] and* CIRIA Special Publication 95 *[33]*

Figure 17 **Outline of the successive stage method of analysis**



7.4.5 Equivalent anchor method

*COMMENTARY ON **7.4.5***

The equivalent anchor method can be used to determine the ultimate limit state of embedded walls supported by two props or anchors, but not their serviceability limit state.

The equivalent anchor method (see *Wall design* [34]) may be used to determine the required depth of embedment of an embedded retaining wall with two levels of support, and the resulting bending moments and shear forces in it.

7.4.6 Distributed prop load method

*COMMENTARY ON **7.4.6***

The design of propping to support embedded retaining walls next to deep excavations has traditionally been based on prop loads determined from "apparent earth pressure" diagrams that were first proposed by Peck [35]. These diagrams are also commonly known as "Peck's envelopes". An updated version of Peck's method, in which the term "distributed prop load" is preferred to "apparent earth pressure", was proposed by CIRIA C517 *[36].*

The distributed prop load method (see *CIRIA C517* [36]) may be used to determine the loads in temporary propping to an embedded retaining wall.

NOTE Further information about the distributed prop load method can be found in CIRIA C517 *[36] and* Earth Pressure and Earth-Retaining Structures (3rd edition), Section 10.4.1 *[2].*

7.4.7 Tension cracks

An allowance for tension cracks should be made in accordance with **4.5.5**.

7.5 Materials

7.5.1 Concrete

7.5.1.1 Concrete and related products incorporated into embedded retaining walls should conform to **4.3.6**.

7.5.1.2 Materials and products incorporated into bored cast-in-place concrete piles should also conform to BS EN 1536 and BS EN 206:2013, Annex D.

7.5.1.3 Materials and products incorporated into diaphragm walls should also conform to BS EN 1538 and BS EN 206:2013, Annex D.

NOTE 1 BS EN 206:2013, Annex D, includes the normative rules for concrete for special geotechnical work that were previously given in BS EN 1536:2010 and BS EN 1538:2010.

NOTE 2 At the time of publication, amendments to BS EN 1536:2010 and BS EN 1538:2010 are in preparation to remove the rules that are now contained in BS EN 206:2013, Annex D.

7.5.2 Steel

7.5.2.1 Steel and related products incorporated into embedded retaining walls should conform to **4.3.7**.

7.5.2.2 Steel sheet piling should also conform to BS EN 1993-5.

7.5.2.3 Hot finished steel tubular piles used in combined walls should also conform to BS EN 10210.

7.5.2.4 Cold formed steel tubular piles in combined walls should also conform to BS EN 10219.

7.5.2.5 Hot rolled steel sheet piles should also conform to BS EN 10248.

7.5.2.6 Cold formed steel sheet piles should also conform to BS EN 10249.

7.5.3 Timber

7.5.3.1 Timber and related products incorporated into embedded walls should conform to **4.3.8**.

NOTE Timber piles may be designed with no protection against decay if they are kept below a moisture content of 22% or they are buried in the ground below the lowest permanent water table.

7.5.3.2 Timber used in permanent sheet pile walls should be of high durability.

NOTE Tropical hardwood might meet the durability requirements without any preservation.

7.5.3.3 Coniferous species, when used in waterfront structures, should be impregnated by a preservation fluid pressed into the wood under vacuum conditions.

7.6 Durability

7.6.1 Concrete

7.6.1.1 The durability of precast concrete sheet pile walls should conform to **4.4.2**.

7.6.1.2 The durability of bored cast-in-place concrete pile walls should conform to **4.4.2.**

7.6.1.3 The durability of diaphragm walls should conform to **4.4.2**.

7.6.2 Steel

7.6.2.1 The durability of steel used in embedded walls should conform to **4.4.3**.

7.6.2.2 The durability of steel sheet piling should also conform to BS EN 1993-5.

NOTE The UK National Annex to BS EN 1993-5 gives UK values for loss of thickness per face due to corrosion of steel sheet piles in soils, with or without groundwater.

7.6.3 Timber

7.6.3.1 The durability of timber sheet pile walls should conform to **4.4.4**.

7.6.3.2 The durability of timber used in soldier/king post walls should conform to **4.4.4**.

7.7 Ultimate limit state design

7.7.1 General

The ultimate limit state design of embedded retaining walls should conform to **4.6** and this subclause (**7.7**).

7.7.2 Bearing

7.7.2.1 To prevent excessive vertical movement, the bearing resistance of an embedded retaining wall should conform to BS EN 1997-1:2004+A1:2013, **9.7.5**.

7.7.2.2 Bearing resistance should be calculated in accordance with BS 8004:2015.

7.7.2.3 Allowance should be made for the adverse effect on bearing resistance of interaction between adjacent piles, which are typically closely-spaced, within an embedded retaining wall.

7.7.3 Rotation

7.7.3.1 To prevent rotational failure, the penetration of an embedded retaining wall into the ground should conform to BS EN 1997-1:2004+A1:2013, **9.7.4**.

7.7.3.2 The verification of ground resistance to rotation of an embedded retaining wall should conform to BS EN 1997-1:2004+A1:2013, **2.4.7.3**, using expressions (2.6a) and (2.7a) with partial factors for Design Approach 1 (as required by the UK National Annex to BS EN 1997-1).

7.7.3.3 For Design Approach 1 Combination 1, the partial factors on actions should be applied to the effects of actions in the embedded retaining wall, as allowed by BS EN 1997-1:2004+A1:2013, **2.4.7.3.2(2)**, and reduced partial factors should be applied to the actions themselves, as follows:

- a partial factor $\gamma_{G,red}$ = 1.0 should be applied to permanent actions, G;
- a partial factor $\gamma_{Q,red}$ should be applied to variable actions, Q; and
- a partial factor γ_E (= γ_G) = 1.35 should be applied to the effects of actions (i.e. resulting bending moments and shear forces) in the wall, E.

7.7.3.4 The value of the partial factor $\gamma_{Q,red}$ should be taken as:

$$\gamma_{Q,red} = \frac{\gamma_Q}{\gamma_E} = \frac{1.5}{1.35} = 1.11 \tag{28}$$

7.7.4 Lateral movement

To prevent excessive horizontal movement, the lateral movement of an embedded retaining wall should conform to BS EN 1997-1:2004+A1:2013, **9.7.4**.

7.7.5 Design of support system

*COMMENTARY ON **7.7.5***

The forces generated in props, struts and anchors do not necessarily reach their maximum values when an earth retaining structure reaches an ultimate limit state. Particularly with stiff earth retaining systems, the maximum force in the support might occur when the structure reaches (or even before it reaches) a serviceability limit state.

7.7.5.1 The design axial resistance of a prop, strut, or anchor that is used to support an earth retaining structure should be at least equal to a design axial force (F_d) given by:

$$F_d = \max \begin{cases} F_{Ed,ULS} = \gamma_{Sd} \times P_{d,ULS} \\ \gamma_F \times F_{Ed,SLS} = \gamma_F \times \gamma_{Sd} \times P_{d,SLS} \end{cases} \tag{29}$$

where:

$F_{Ed,ULS}$ is the design force (effect) in the support at the ultimate limit state;

$F_{Ed,SLS}$ is the design force (effect) in the support at the serviceability limit state;

$P_{d,ULS}$ is the axial force (i.e. reaction), calculated using ultimate limit state (ULS) design parameters, that is required to prevent the earth retaining structure exceeding an ultimate limit state; and

$P_{d,SLS}$ is the axial force (i.e. reaction), calculated using serviceability limit state (SLS) design parameters that is required to prevent the earth retaining structure exceeding a serviceability limit state; and

γ_{Sd} is a model factor for load-effects.

NOTE 1 "Ultimate limit state" and "servicability limit state" are defined in BS EN 1990.

NOTE 2 $P_{d,ULS}$ is the higher value from DA1-1 and DA1-2.

7.7.5.2 The value of γ_{Sd} should be taken from Table 11.

Table 11 **Values of the model factor for load-effects for props, struts and anchors**

Number of wall supports	**Level of support**	**Model factor, γ_{Sd}, for different methods of analysis** A)		
		Limiting equilibrium method	**Discrete spring or continuum models**	**Distributed prop load method**
≥1	Top	1.3	1.0	1.0
	Other	1.15	1.0	1.0

A) See **7.7.5.4** for an alternative value of γ_{Sd} for ground anchors.

7.7.5.3 The value of γ_F should be taken as γ_G for the design of props and struts and as γ_{Serv} for the design of grouted anchors. The values of γ_G and γ_{Serv} should conform to BS EN 1997-1.

7.7.5.4 For grouted anchors that are subjected to acceptance tests in accordance BS 8081, the value of γ_{Sd} may be reduced to 1.0.

*NOTE 1 The symbols $F_{ULS;d}$ and $F_{Serv;k}$ in BS EN 1997-1:2004+A1:2013, Clause **8**, are equivalent to $F_{Ed,ULS}$ and $F_{Ed,SLS}$ in equation (29).*

*NOTE 2 The symbol $F_{Serv;d}$ in BS EN 1997-1:2004+A1:2013, Clause **8** is equivalent to $\gamma_{Sd} \times F_{Ed,SLS}$ in equation (29).*

7.8 Serviceability limit state design

The serviceability limit state design of embedded retaining walls should conform to **4.7** and this subclause (**7.8**).

NOTE Guidance on values of stiffness parameters for displacement calculations may be found in CIRIA Report C580 *[N3].*

7.9 Structural design

7.9.1 General

The structural design of embedded retaining walls should conform to **4.8** and this subclause (**7.9**).

7.9.2 Steel sheet pile walls

7.9.2.1 The structural design of steel sheet pile walls should conform to BS EN 1993-5.

7.9.2.2 The structural design of steel sheet pile walls should take account of:

- stresses imparted to the sheet piles during driving (see **7.10.2.1**);
- any loss of thickness required to meet the durability recommendations of **7.6.2**.

NOTE The UK National Annex to BS EN 1993-5 gives UK values for the reduction factors that are to be applied in the calculation of bending resistance of U-shaped steel sheet piles.

7.10 Execution

7.10.1 General

The execution of embedded walls should conform to **4.9** and this subclause (**7.10**).

7.10.2 Sheet pile walls

7.10.2.1 Steel sheet pile walls

7.10.2.1.1 The execution of steel sheet pile walls should conform to BS EN 12063.

7.10.2.1.2 The execution of steel sheet pile walls should also conform to the *ICE Specification for piling and embedded retaining walls* (SPERW [N4]), *Sections B12 and C12, Steel sheet piles*.

7.10.2.1.3 To prevent damage, sheet piles should be designed to withstand the stresses that arise during their installation.

NOTE Sheet piles may be installed by impact, vibratory or hydraulic drivers.

7.10.2.1.4 Sheet piles should be installed in panels whenever possible.

NOTE Panel driving improves the accuracy of sheet pile installation and minimizes the risk of piles de-clutching in hard ground.

7.10.2.2 Timber sheet pile walls

The execution of timber sheet pile walls should conform to BS EN 12063.

7.10.3 Bored cast-in-place concrete pile walls

7.10.3.1 The execution of bored cast-in-place walls should conform to BS EN 1536.

NOTE Additional recommendations can be found inthe ICE Specification for piling and embedded retaining walls *(SPERW [N4]),* Sections B9 and C9, Secant pile walls*, and* Sections B10 and C10, Contiguous pile walls*, as appropriate.*

7.10.3.2 The execution of bored cast-in-place walls constructed using drilled micropiles should conform to BS EN 14199.

7.10.4 Diaphragm walls

7.10.4.1 The execution of diaphragm walls should conform to BS EN 1538.

7.10.4.2 The execution of diaphragm walls should also conform to the *ICE Specification for piling and embedded retaining walls* (SPERW [N4]), *Sections B8 and C8, Diaphragm walls and barrettes.*

7.10.5 Soldier/king pile walls

The execution of soldier/king pile walls should conform to the *ICE Specification for piling and embedded retaining walls* (SPERW [N4]), *Sections B11 and C11, King post walls.*

7.11 Testing

Testing of an embedded retaining wall should conform to prEN ISO 22477.

7.12 Supervision, monitoring, and maintenance

Supervision, monitoring, and maintenance of an embedded retaining wall should conform to **4.11**.

7.13 Reporting

Reports for embedded retaining walls should conform to **4.12**.

8 Cofferdams, basements, and strutted excavations

*COMMENTARY ON Clause **8***

This clause applies to the design and construction of:

- *cofferdams;*
- *basements;*
- *strutted excavations; and*
- *buried concrete structures.*

NOTE 1 Guidance on the design of basements can be found in SCI Publication P275 *[37], the* Concrete Centre's guide to concrete basements *[38],* IStructE's report on deep basements *[39], and* CIRIA Report 139 *[40].*

NOTE 2 Attention is drawn to The Construction (Design and Management) Regulations 2015 *[4], with regards to health and safety requirements for construction works, in particular Regulation 23 which deals with cofferdams.*

8.1 Choice and design of structure

8.1.1 General

8.1.1.1 The design of cofferdams, basements, and strutted excavations should conform to BS EN 1997-1:2004+A1:2013, Clause **9**, and Clause **4** and Clause **8** of this standard.

8.1.1.2 The design of buried concrete structures should conform to PD 6694-1:2011, Clause **10**, and Clause **4** and Clause **8** of this standard.

8.1.1.3 Protection of below ground structures against water from the ground should conform to BS 8102.

8.1.1.4 The design and construction of anchors used to support basement retaining walls should conform to BS 8081.

8.1.2 Cofferdams

8.1.2.1 The design of cofferdams should conform to **8.1.1.1** and BS 6349.

NOTE Guidance on the design and construction of sheet-piled cofferdams can be found in CIRIA Special publication 95 *[33].*

8.1.2.2 The choice of a suitable cofferdam type should consider:

- whether a land or water cofferdam is required;
- the nature of the permanent structure to be built within the cofferdam;
- plan dimensions of the working area required inside the cofferdam;
- total depth of soil and/or water to be retained;
- soil conditions below and above foundation level;
- groundwater levels and their fluctuation and, for water cofferdams, the tidal, seasonal, and flood levels;

- for water cofferdams, the strength of the current, wave action and scour before, during and after construction;
- possible effects of cofferdam construction on existing buildings or other structures close to it;
- availability of materials;
- methods of constructing and dismantling the cofferdam;
- time available for construction of the cofferdam;
- noise, vibration, fumes and fire risk;
- relative total costs of the cofferdams where two or more types are otherwise suitable; and
- accessibility, especially for cofferdams in water.

NOTE The function of a cofferdam is to provide working space for the execution of permanent works below water level. A cofferdam is typically a temporary structure, all or part of which is removed after construction.

8.1.2.3 Single-wall cofferdams

*COMMENTARY ON **8.1.2.3***

A single-wall (also known as "single-skin") cofferdam is usually supported either internally, by bracing within the cofferdam, or externally, by anchors around its perimeter.

8.1.2.3.1 Single-wall cofferdams may be constructed using:

- flexible walls, such as steel sheet pile walls; or
- diaphragm walls.

8.1.2.3.2 Sheet piling used to form a cofferdam should be capable of being driven to the depths required.

8.1.2.3.3 Sheet pile sections should be chosen so that they do not suffer mechanical damage during installation.

8.1.2.3.4 Steel sheet pile walls used to construct single-wall cofferdams should be designed in accordance with Clause **7**.

8.1.2.3.5 Diaphragm walls used to construct single-wall cofferdams should be designed in accordance with Clause **7**.

NOTE Guidance on the design of single skin cofferdams can be found in the Piling Handbook, Section 9.4 *[32].*

8.1.2.4 Double-wall cofferdams

*COMMENTARY ON **8.1.2.4***

A double-wall (also known as "double-skin") cofferdam is a self-supported gravity structure, formed by two parallel lines of sheet piling connected together by a system of walings and tie rods at one or more levels. The space between the walls is normally filled with coarse or very coarse soil, such as sand, gravel, or broken rock. Cellular cofferdams are also "double-walled", but are treated as a separate type of cofferdam in this standard.

Double-wall sheet pile cofferdams for maritime works should conform to BS 6349-2:2010, **7.8**.

NOTE Guidance on the design of double skin wall cofferdams can be found in the Piling Handbook, Section 9.10 *[32].*

8.1.2.5 Cellular cofferdams

*COMMENTARY ON **8.1.2.5***

A cellular cofferdam is a self-supported gravity structure, constructed from straight web steel sheet piles to form cells of various shapes. The cells are normally filled with coarse or very coarse soil, such as sand, gravel or broken rock. Cellular cofferdams shared many similarities to double-wall cofferdams. The stability of a cellular cofferdam depends upon the tensile strength of the sheet piling (especially the clutches), properties of the fill,the shape and size of the cells, and the foundation materials. Outward pressure from the fill is resisted by high circumferential tensile forces in the straight web piles.

Cellular sheet pile cofferdams for maritime works should conform to BS 6349-2:2010, **7.7**.

NOTE Guidance on the design of cellular cofferdams can be found in the Piling Handbook, Chapter 10 *[32].*

8.1.3 Basements

The design of basements should conform to **8.1.1.1** and BS 8102.

NOTE 1 See also Design and construction of deep basements including cut-and-cover structures *[39].*

NOTE 2 Guidance on the construction of water-resisting basements can be found in CIRIA Report 139 *[40].*

NOTE 3 See also Concrete basements – guidance on the design and construction of in-situ concrete basement structures *[38].*

8.2 Actions and design situations

8.2.1 Actions for cofferdams, basements and strutted excavations should conform to **4.5.1**.

8.2.2 Design situations for cofferdams, basements and strutted excavations should conform to **4.2.2**.

8.3 Design considerations

Design considerations for cofferdams, basements and strutted excavations should conform to **4.2.3.**

8.4 Calculation models

Calculation models for cofferdams, basements and strutted excavations should conform to BS EN 1997-1.

8.5 Materials

8.5.1 Concrete

Concrete and related products incorporated into cofferdams, basements and strutted excavations should conform to **4.3.6**.

8.5.2 Steel

Steels and related products incorporated into cofferdams, basements and strutted excavations should conform to **4.3.7**.

8.5.3 Masonry

8.5.3.1 Masonry and related products incorporated into cofferdams, basements and strutted excavations should conform to **4.3.9**.

8.5.3.2 Reinforcement in masonry basement walls should conform to PD 6697.

8.6 Durability

8.6.1 Concrete

The durability of concrete incorporated into cofferdams, basements and strutted excavations should conform to **4.4.2**.

8.6.2 Steel

The durability of steel and related products incorporated into cofferdams, basements and strutted excavations should conform to **4.4.3**.

8.6.3 Masonry

The durability of masonry incorporated into cofferdams, basements and strutted excavations should conform to **4.4.5**.

8.7 Utimate limit state design

The ultimate limit state design of cofferdams, basements and strutted excavations should conform to **4.6**.

8.8 Calculation models

Calculation models for semi-gravity retaining walls should conform to BS EN 1997-1.

8.9 Structural design

The structural design of cofferdams, basements and strutted excavations should conform to **4.8**.

NOTE The simplified calculation method for basement walls subject to lateral earth pressure given in BS EN 1996-3 may be used for the structural design of unreinforced masonry basement walls.

8.10 Testing

Testing of cofferdams, basements and strutted excavations should conform to pr EN ISO 22477.

8.11 Supervision, monitoring, and maintenance

Supervision, monitoring, and maintenance of cofferdams, basements, and strutted excavations should conform to **4.11**.

8.12 Reporting

Reports for cofferdams, basements and strutted excavations should conform to **4.12**.

Annex A (normative) Deadman anchors

*COMMENTARY ON **Annex A***

Anchors for retaining walls are of three general types:

- *ground anchors (including rock and soil anchors);*
- *tension piles; and*
- *deadman anchors.*

This annex applies to the design and construction of deadman anchors comprising:

- *continuous walls (made of steel or concrete);*
- *mass concrete blocks; and*
- *friction slabs.*

This annex does not apply to the design and construction of grouted (also known as ground) anchors or tension piles.

A.1 General

A.1.1 The design of deadman anchors should conform to BS EN 1997-1:2004+A1:2013, Clause **9**, and Clause **4** of this standard and this annex.

A.1.2 The design of grouted anchors should conform to BS 8081.

A.1.3 The design of tension piles should conform to BS 8004.

A.2 Choice of structure

NOTE See Figure A.1 for types of deadman anchor.

A.2.1 General

A.2.1.1 The choice of a suitable deadman anchor should take into account:

- the suitability of the proposed installation technique for the prevailing ground conditions;
- their possible effect on adjoining buildings; and
- the rights of adjoining owners under whose building or land the anchors are to be inserted.

NOTE 1 Deadman anchors may be continuous or comprise a series of separate units.

NOTE 2 Continuous walls may be constructed using steel sheet piles, cast-in-place reinforced concrete or precast reinforced concrete.

A.2.1.2 Continuous walls should be provided with suitable drainage measures to prevent differential hydrostatic pressures acting on them.

A.2.2 Continuous walls made of steel or concrete

*COMMENTARY ON **A.2.2***

Continuous walls provide resistance to horizontal movement of the supported structure by mobilizing passive earth pressure in front of the wall. These walls require a waling to distribute anchor tie forces to the ground.

A.2.3 Mass concrete blocks

*COMMENTARY ON **A.2.3***

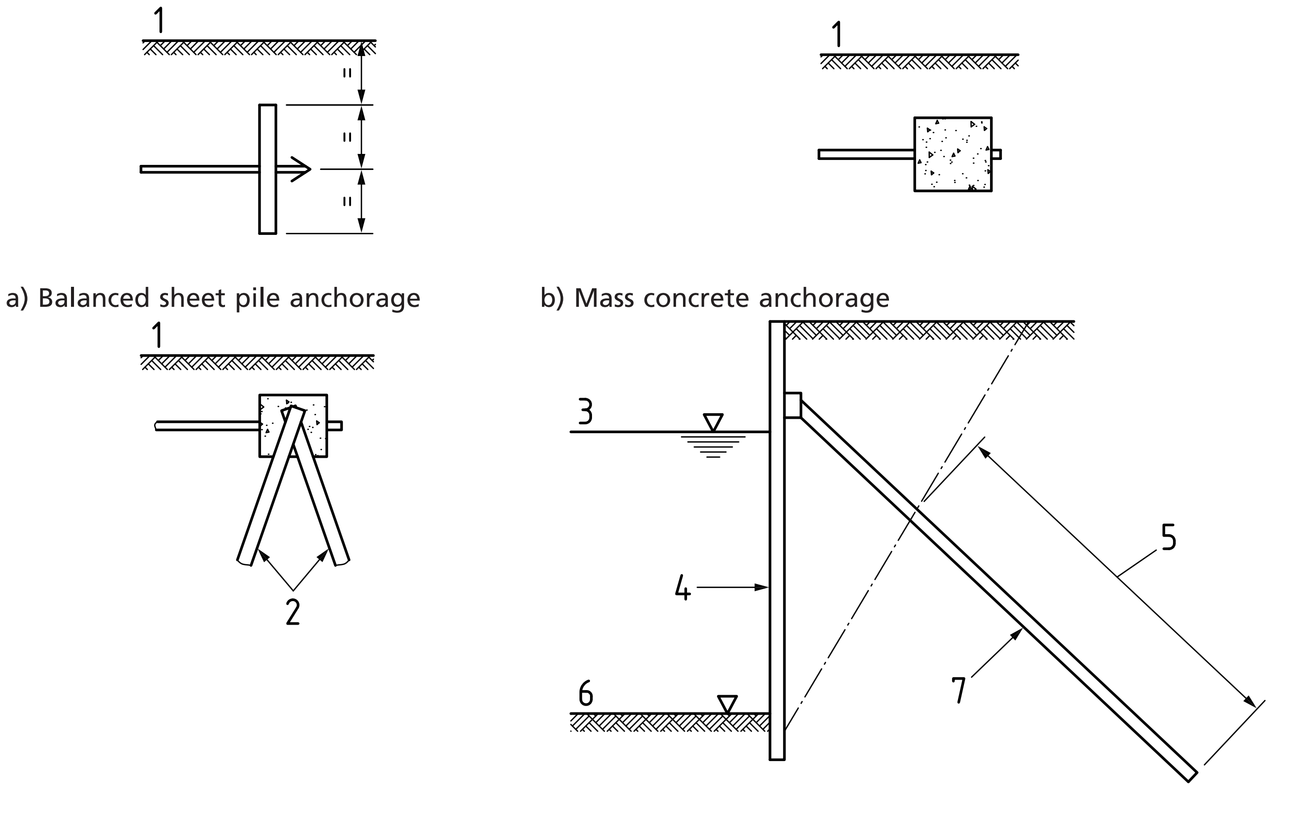
Mass concrete blocks provide resistance to horizontal movement of the supported structure partly through friction at their base and partly by mobilizing passive earth pressure in front of the block. They are generally less efficient than continuous walls in providing horizontal resistance and are used mainly where tie forces are not very large.

A.2.4 Friction slabs

*COMMENTARY ON **A.2.4***

Friction slabs, which may be buried or at ground surface, provide resistance to horizontal movement of the supported structure mainly through friction along their base. They are generally less efficient than walls in providing horizontal resistance and are used mainly where tie forces are not very large.

Figure A.1 **Types of deadman anchor**



a) Balanced sheet pile anchorage

b) Mass concrete anchorage

c) Raking pile ancorage

d) Ground anchor

Key

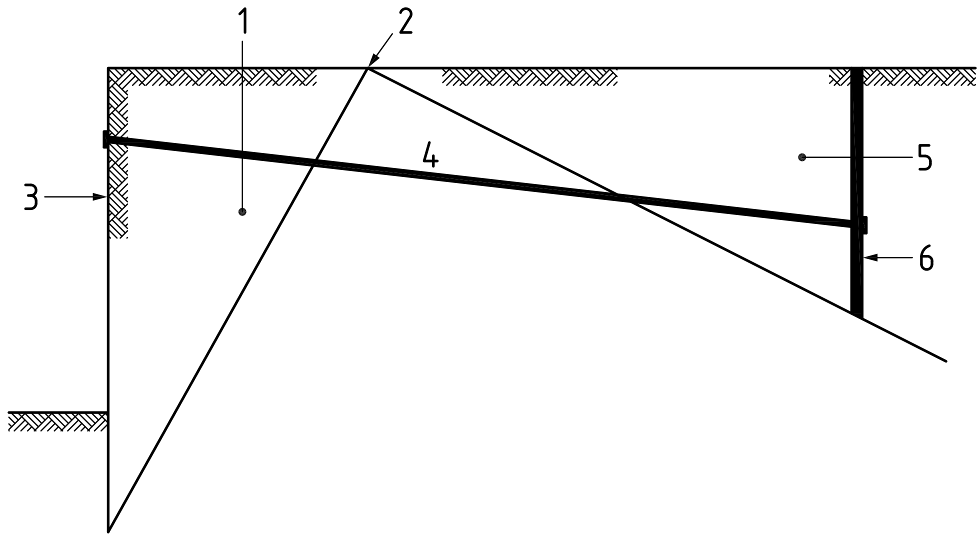
1 Ground level
2 Tension piles
3 Water level
4 Sheet piles
5 Working length for anchor pile
6 Dredged level
7 Anchor pile or ground anchor alternative

NOTE Examples of deadman anchors can be found in BS 6349-2:2010, Figure 13 and Figure 14.

A.3 Calculation models

Deadman anchors may be designed using the calculation model shown in Figure A.2.

Figure A.2 **Non-interference of zones for anchored deadman wall**



Key

1 Active zone to anchored wall
2 Non-interference of active zone to anchored wall and passive zone to deadman
3 Anchored wall
4 Tie
5 Passive zone to deadman
6 Deadman

NOTE Guidance on the use of the calculation model shown in Figure A.2 can be found in the Piling Handbook (9th edition), Chapter 7 *[32].*

A.4 Materials

A.4.1 Concrete

A.4.1.1 Concrete and its products incorporated into deadman anchors should conform to **4.3.6**.

A.4.1.2 Materials and products incorporated into diaphragm walls should also conform to BS EN 1538 and BS EN 206:2013, Annex D.

NOTE 1 BS EN 206:2013, Annex D, includes the requirements for concrete for special geotechnical work that were previously given in BS EN 1536:2010 and BS EN 1538:2010.

NOTE 2 At the time of publication, an amendment to BS EN 1538:2010 is in preparation to remove the rules that are now contained in BS EN 206:2013, Annex D.

A.4.2 Steel

A.4.2.1 Steel and related products incorporated into deadman anchors should conform to **4.3.7**.

A.4.2.2 Steel sheet piling should also conform to BS EN 1993-5.

A.4.2.3 Hot rolled steel sheet piles should also conform to BS EN 10248.

A.4.2.4 Cold formed steel sheet piles should also conform to BS EN 10249.

A.5 Durability

A.5.1 Concrete

A.5.1.1 The durability of precast concrete anchors should conform to **4.4.2**.

A.5.1.2 The durability of concrete diaphragm walls should conform to **4.4.2**.

A.5.2 Steel

A.5.2.1 The durability of steel and related products used in deadman anchors should conform to **4.4.3**.

A.5.2.2 The durability of steel sheet piling should also conform to BS EN 1993-5.

NOTE The UK National Annex to BS EN 1993-5 gives UK values for loss of thickness per face due to corrosion of steel sheet piles in soils, with or without groundwater.

A.6 Reporting

Reports for deadman anchors should conform to **4.12**.

Annex B (informative) Specific formations

B.1 London clay

London Clay was deposited in marine conditions in the Eocene epoch (30 million years ago). London Clay is made up of various silty clay and sandy clayey silt units, separated by glaucontic rich horizons. Very fine sand and silt dustings, partings and lenses are frequent in the siltier clays, and sand layers occur in the sandy clayey silts. Phosphatic and claystone nodules are quite common throughout the deposit.

The London Clay in central London is one of the most highly investigated soils in the world.

NOTE A summary of the characteristics of London Clay can be found in Some characteristics of London Clay *[41] and* The London Clay at T5 *[42].*

B.2 Gault Clay

The Gault Clay comprises a sequence of clays, mudstones, and thin siltstones with bands of phosphatic nodules of Middle and Upper Albian age. It outcrops in East Anglia, Wessex, Dorset, North East Kent, Surrey and Hampshire.

Gault Clay causes a number of serious geotechnical problems, including ancient and recent landslides, and can contain sufficient sulfate and sulfuric acid for potential chemical attack on concrete. Seasonal shrinkage and swelling of this highly expansive soil can result in damage to buildings.

NOTE Guidance on the engineering geology of Gault Clay can be found in BGS Technical Report WN/94/31, Engineering Geology of British Rocks and Soils – Gault clay *[43].*

B.3 Lambeth Group

The Lambeth Group (previously known as the Woolwich and Reading Beds) is a complex sequence of gravels, sands and clays that vary considerably both horizontally and vertically and whose properties range between those of an engineering soil and a rock. The Lambeth Group underlies much of south-east England, particularly in London and Hampshire, and, as a result, is frequently encountered in major construction projects. CIRIA identified the Lambeth group as one of the "economically important UK soils and rocks" (*CIRIA C583* [44]).

NOTE 1 Guidance on the engineering properties of the Lambeth Group can be found in CIRIA C583 *[44].*

NOTE 2 Guidance on the engineering geology of the Lambeth Group can be found in BGS Open Report OR/13/006, Engineering Geology of British Rocks and Soils – Lambeth Group *[45].*

B.4 Glacial soils and tills

Glacial deposits are widespread throughout the world and are frequently encountered in the upland parts of the United Kingdom. Glacial tills and soil are amongst the most difficult to engineer, owing to their marked variation in both thickness and engineering properties.

NOTE 1 Guidance on the classification of glacial tills can be found in Chapter 4 of CIRIA C504 *[46].*

NOTE 2 Guidance on the engineering properties of glacial tills can be found in Chapter 5 of CIRIA C504 *[46].*

NOTE 3 Information about issues relevant to glacial soils can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 31 *[3].*

B.5 Problematic soils

B.5.1 Problematic soils include materials that display significant volume change, a distinct lack of strength, or are potentially corrosive. Problematic soils are profoundly influenced by the climatic regime in which they were developed. (See *ICE manual of geotechnical engineering, Volume II* [1] for further information).

B.5.2 Problematic soils include (but are not limited to): arid soils, tropical soils, glacial soils, alluvial soils, collapsible soils, expansive soils, non-engineered fills, organics/peat soils, sulfate/acid soils, and soluble ground.

NOTE 1 Information about problematic soils and their issues can be found in the ICE manual of geotechnical engineering (2012), Volume I, Section 3 *[3].*

NOTE 2 Information about shrinkable (also known as expansive) soils can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 33 *[3].*

B.6 Chalk

Chalk forms the downland of southern England, the Wolds of eastern England, and the white cliffs of Antrim, East Yorkshire, Dover, and from the Seven Sister to Dorset. The chalk is the UK's most important aquifer for potable water supply. A great deal of construction and infrastructure development is built on chalk.

NOTE 1 Guidance on the description and classification of chalk can be found in CIRIA C574, Chapter 3 *[47].*

NOTE 2 Guidance on the mechanical properties of chalk can be found in CIRIA C574, Chapter 4 *[47].*

B.7 Mercia Mudstone Group

The Mercia Mudstone Group of rocks underlies much of northern, central and southern England and parts of Northern Ireland. The engineering properties of the rocks and the derived soils are important as they are frequenctly encountered in excavations and as founding strata.

NOTE 1 Guidance on the description and classification of Mercia mudstone can be found in CIRIA C570, Section 2, Geological background *[48].*

NOTE 2 Guidance on the engineering properties of Mercia mudstone can be found in CIRIA C570, Section 5, Correlation of engineering properties to the SPT and Section 6, In-situ properties and behaviour of Mercia mudstone *[48].*

NOTE 3 Guidance on the engineering geology of Mercia mudstone can be found in BGS Report RR/01/02, Engineering geology of British rocks and soils: Mudstones of the Mercia Mudstone Group *[49].*

B.8 Lias Group

The Lias Group encompasses an important group of geological materials, comprising clay-rich mudstones interlayered with limestones. The outcrop of the Lias extends in a continuous band from the coast of Dorset in a north-north-easterly direction to Yorkshire, with outlying areas in Somerset and South Wales.

NOTE Guidance on the engineering geology of the Lias Group can be found in BGS Internal Report OR/12/032, Engineering Geology of British Rocks and Soils – Lias Group *[50].*

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