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Maritime works

Part 1-3: General — Code of practice for geotechnical design

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Contents

Summary of pages

This document comprises a front cover, and inside front cover, pages i to iv, pages 1 to 48, an inside back cover and a back cover.

Foreword

Publishing information

This part of BS 6349 is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 30 April 2021. It was prepared by Technical Committee CB/502, *Maritime works*. A list of organizations represented on this committee can be obtained on request to the committee manager.

Supersession

This part of BS 6349 supersedes BS 6349-1-3:2012, which is withdrawn.

Relationship with other publications

BS 6349 is published in the following parts:

- Part 1-1: *General – Code of practice for planning and design for operations*;
- Part 1-2: *General – Code of practice for assessment of actions*;
- Part 1-3: *General – Code of practice for geotechnical design*;
- Part 1-4: *General – Code of practice for materials*;
- Part 2: *Code of practice for the design of quay walls, jetties and dolphins*;
- Part 3: *Design of dry docks, locks, slipways and shipbuilding berths, shiplifts and dock and lock gates*;
- Part 4: *Code of practice for design of fendering and mooring systems*;
- Part 5: *Code of practice for dredging and land reclamation*;
- Part 6: *Design of inshore moorings and floating structures*;
- Part 7: *Guide to the design and construction of breakwaters*;
- Part 8: *Code of practice for the design of Ro-Ro ramps, linkspans and walkways*.

This part of BS 6349 is intended to be read in conjunction with BS EN 1997-1:2004 and BS EN 1997-2:2007.

Information about this document

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

- general restructuring;
- inclusion of clause on bearing piles, previously in BS 6349-1-4;
- enhanced reference to and alignment with BS EN 1997-1:2004 and BS EN 1997-2:2007;
- removal of general geotechnical guidance, which was more akin to a design manual.

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Users may substitute any of the recommendations in this British Standard with practices of equivalent or better outcome. 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 in this standard are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is "should".

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

Where words have alternative spellings, the preferred spelling of the Shorter Oxford English Dictionary is used (e.g. "organization" rather than "organisation").

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Section 1: General

1 Scope

This part of BS 6349 gives recommendations for geotechnical activities associated with the design and implementation of maritime works. It covers ground investigation and geotechnical design, and gives additional guidance on testing procedures.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes provisions of this document[1]. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 6031, *Code of practice for earthworks*

BS 6349-1-2, *Maritime works – Part 1-2: General – Code of practice for assessment of actions*

BS 6349-1-4, *Maritime works – Part 1-4: General – Code of practice for materials*

BS 6349-2, *Maritime works – Part 2: Code of practice for the design of quay walls, jetties and dolphins*

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

BS 8081, *Code of practice for ground anchorages*

BS EN 1536, *Execution of special geotechnical work – Bored piles*

BS EN 1538, *Execution of special geotechnical works – Diaphragm walls*

BS EN 1992 (all parts), *Eurocode 2 – Design of concrete structures*

BS EN 1995, *Eurocode 5: Design of timber structures*

BS EN 1993-5:2007, *Eurocode 3 – Design of steel structures – Part 5: Piling*

BS EN 1997-1:2004, *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 1998 (all parts), *Eurocode 8 – Design of structures for earthquake resistance*

BS EN 12063:1999, *Execution of special geotechnical work – Sheet pile walls*

BS EN 14731, *Execution of special geotechnical works – Ground treatment by deep vibration*

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

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

[1] Documents that are referred to solely in an informative manner are listed in the Bibliography.

3 Terms, definitions, symbols and abbreviations

3.1 Terms and definitions

For the purposes of this part of BS 6349, the following terms and definitions apply.

3.1.1 deadman anchorage

anchorage that relies on the mobilization of passive resistance

3.1.2 extreme high water (EHW)

highest level that can be predicted to occur as a combination of astronomical tides, positive or negative surges, seiches and freshwater flow

3.1.3 extreme low water (ELW)

lowest level that can be predicted to occur as a combination of astronomical tides, positive or negative surges, seiches and freshwater flow

3.1.4 geotechnical investigation

studies and investigation of the site, including ground investigations, the studies and appraisals of existing structures and the studies of the historical development of the site

3.1.5 ground investigation

direct and indirect investigation of the physical characteristics of the ground at the site

NOTE Ground investigations can include boreholes, penetration testing, trial pits, laboratory testing and geophysical profiling, amongst other things.

3.1.6 highest astronomical tide (HAT)

highest level that can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions

3.1.7 lowest astronomical tide (LAT)

lowest level that can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions

NOTE It is often the level selected as the datum for soundings on navigational charts.

3.1.8 mean high water springs (MHWS)

average, over a long period of time, of the heights of two successive high waters at springs

NOTE This is the average level that would exist in the absence of tides; a long period is typically 18.6 years (one cycle of the moon's nodes).

3.1.9 mean low water springs (MLWS)

average, over a long period of time, of the heights of two successive low waters at springs

NOTE This is the average level that would exist in the absence of tides; a long period is typically 18.6 years (one cycle of the moon's nodes).

3.1.10 neap tide

occasion in a lunar month when the average range of two successive tides is least

3.1.11 range

difference in height between one high water and the preceding or following low water

3.1.12 return period

period that, on average, separates two occurrences

3.1.13 spring tide

occasion in a lunar month when the average range of two successive tides is greatest

3.1.14 waterway

body of water in front of a structure

3.2 Symbols

For the purposes of this part of BS 6349, the following symbols apply.

B_e	breadth of earth-retaining structure
c_{min}	minimum concrete cover
d_p	depth of pile toe from dredge level
H_R	retained height of structure
K_a	coefficient of active earth pressure
K_p	coefficient of passive earth resistance
z	depth at which a calculation is to be made
Δ_c	concrete cover tolerance for deviation
γ	density of soil
γ_w	weight density of groundwater

3.3 Abbreviations

For the purposes of this part of BS 6349, the following abbreviations apply.

CPT	cone penetration test
ELW	extreme low water
EHW	extreme high water
HAT	highest astronomical tide
LAT	lowest astronomical tide
MHWS	mean high water springs
MLWS	mean low water springs

Section 2: Geotechnical investigation

4 General

Geotechnical investigation of the surface and subsurface conditions at and near the site of proposed works should be carried out in a progressive manner as a key feature of the design of maritime structures. The investigation should be carried out in accordance with BS EN 1997-1:2004 and BS EN 1997-2:2007.

Preliminary studies should be undertaken to provide a basis for determining the extent of ground investigation and laboratory testing required. These studies should also include, where possible, the interpretation and evaluation of the information obtained.

The preliminary studies should include an assessment of the characteristics of soil or rock formations that can be retained by, or provide the foundation of, structures, or can be incorporated in or affected by earthworks in the form of dredging and reclamation. The preliminary studies should also include the collection of data on locally available materials for use in constructing the works, including the long-term durability of these materials in the particular location where the works are to be constructed.

NOTE 1 Ground investigation practice is described in BS EN 1997-2:2007.

NOTE 2 Information on geophysical surveying techniques is given in CIRIA Report C562 [1].

NOTE 3 Ground investigation prior to the planning, design and construction of dry docks is of greater importance than for many other structures of similar value or size. The subsoil conditions can greatly influence the choice of construction method.

*NOTE 4 Appropriate methods of calculation, drawing attention to the interdependence and interaction of different types of structure and their surrounding soil masses, are described in general terms in Section **3**.*

Maritime structures normally fall within geotechnical category 2 as defined in BS EN 1997-1:2004, but in situations where complex or unusual engineering is involved, structures or parts of structures should be assumed to be in geotechnical category 3.

NOTE 5 The ground investigation requirements are normally at least those indicated in BS EN 1997-2:2007, but additional investigations and more advanced tests might be required, dependent upon the circumstances that place the structure or structural element in geotechnical category 3.

5 Planning of geotechnical investigations

5.1 Existing data sources

The first stage of the geotechnical investigation should be a desk study of available published and unpublished information, including a thorough visual examination of the site where possible (see **5.2**).

NOTE Sources of information relevant to maritime structures include:

- *Admiralty charts and handbooks (the Pilot series) [2];*
- *bathymetric charts and maps;*
- *Ordnance Survey maps and old maps;*
- *meteorological data;*
- *national and local government records;*

- *existing ground investigation reports and geological records;*
- *geological maps and memoirs;*
- *local and national government and Environment Agency reports and data concerning contamination and other anthropogenic material(s);*
- *aerial and satellite photographs; and*
- *information on existing works in the locality.*

Admiralty charts and Ordnance Survey maps should be studied to identify evidence of changes in seabed levels and in the configuration of the foreshore that can indicate areas of erosion or accretion.

5.2 Site reconnaissance

COMMENTARY ON ***5.2***

Many maritime sites are areas of recent geomorphological development, and such processes are more active if the site is nearer to the shore. The timescale of these changes is often comparable with the lifespan of a project, and they can often be the most significant feature of the ground conditions, affecting many maritime works.

It is important to recognize characteristic features of coastal sediments. Those laid down in estuarine conditions are commonly flocculated silts and clays, although the flocs can also entrap coarser sediments brought into the estuary by coastal currents. Thus, estuarine deposits are often characterized by great depths of very fine soils. Deltaic deposits have been laid down seasonally and therefore usually contain alternating bands of coarse and fine sediments corresponding to the variation in transporting power of the river in winter and in summer. Beach deposits often result from coastal rather than river transport and are usually of medium to coarse size. They can arise from small embayments and are therefore very local, but they can occasionally arise from long lengths of foreshore.

It is also important to recognize the chrematistic features of river, lake and stream sediments. Depending on the velocity and size of the stream/river, alluvial/fluvial deposits can range from fine clays and silts, to coarse sands, gravels, cobbles and boulders. The dynamic nature and evolving geological sequencing and depositions at river deltas are of critical consideration for projects in these environments.

A thorough visual examination of the site should be made, where possible, as part of the desk study (see **5.1**). Exposures of soil or rock on the foreshore or coastal cliffs should be examined to obtain preliminary information on the geology of the site and to obtain any evidence of erosion, accretion or instability. The appearance of existing structures and earthworks should be examined for signs of subsidence or ground heave.

The landforms of the foreshore should be studied in relation to seabed contours and information on littoral currents, in order to delineate areas of active erosion and accretion, and to provide a basis for predicting changes that might result from construction of the new works.

Depending on the size of the proposed project or construction activities, a bathymetric survey should be undertaken where a greater understanding of the erosion and/or accretion activities is required. The bathymetric survey should cover an area that provides sufficient understanding of the seafloor conditions for the project, and for any construction-related activities (e.g. dredging, seaborne transportation). The bathymetric survey should be accompanied by a detailed onshore topographic survey.

Particular attention should be paid to the possibility of leaching and the formation of caverns in organically formed and carbonate-rich deposits, especially where major structures such as breakwaters or dry docks are proposed.

NOTE Such deposits are mainly calcareous, of which coral and calcite (excluding algae that cement detrital material) are examples.

Sedimentary rocks formed by induration are also subject to cavernous formation, so a ground investigation programme arranged to locate such features should be adopted in these types of ground.

Similarly, attention should be paid to carbonaceous organic deposits such as peat or inorganic soils with a modest organic content, as these materials are compressible and crushable and can be difficult to work with.

An assessment of the likelihood of encountering anthropogenic material at the site should also be carried out. Attention should be paid to the presence and likelihood of potentially contaminated material, and a review of the previous site and near-site uses and users should be undertaken as these can provide useful data. Local government or reports by the government environmental agency should also be consulted. Finally, the likelihood of encountering unexploded ordnances should be taken into account via the commission of an appropriate survey.

5.3 Selection of investigation and sampling methods

The types of investigation methods to be carried out should be chosen on the basis of the size and complexity of both the underlying ground conditions and the proposed works. The selection of drilling and sampling methods for soil and rock, and measurement of groundwater, should be in accordance with BS EN ISO 22475-1.

NOTE 1 The types of investigation methods can be classified into two broad categories, which are:

- *non-intrusive: geophysical surveys such as seismic reflection and refraction; and*
- *intrusive: boreholes, cone penetration tests (CPTs), trial excavations.*

The advice of a suitably qualified and experienced geotechnical engineer should be sought when planning the investigation.

NOTE 2 The quality of data, and of soil, rock or groundwater samples, is influenced by the geological and hydrogeological conditions and by the choice and execution of the investigation, drilling and/or sampling method, including the handling, transport and storage of the samples.

5.4 Location and extent of geophysical surveys

A clear purpose should be established for a geophysical survey. The location and extent of geophysical surveys should then be planned such that the relevant information to be obtained on the surface and subsurface conditions to achieve that purpose can be determined. The survey lines should provide full longitudinal and lateral cover of the extent of the works. An orthogonal orientation of survey lines with longitudinal and lateral lines should be adopted in most investigations, with diagonal or offset lines carried out if deemed appropriate or necessary.

Geophysical survey lines should also be undertaken to focus on known or suspected subsurface features that could influence the proposed works, such as faults and dykes (predominantly in rock), paleochannels, remnant or active submarine landslides or any other similar subsurface feature. Geophysical survey lines that are planned to assess the detailed location of a known feature should be carried out orthogonal to the known alignment of the feature, in order to have the best chance of identifying its position and characteristics.

The advice of a specialist marine geophysical engineer, ideally with local knowledge and experience, should be sought at an early stage of the planning and preparation of the geophysical surveys.

5.5 Location of boreholes, CPTs and trial excavations

The layout of exploratory boreholes and CPTs should cover the full longitudinal and lateral extent of the works. Trial excavations, if practicable, should also be undertaken. The existing data (where available) and/or results of surveys (bathymetric, geophysical) should be used to inform the location and spacing of boreholes and CPTs.

For large and complex proposed works, the combination of a borehole and CPT at each investigation location should normally be adopted, unless it is considered that this would be of only marginal benefit.

NOTE 1 For smaller and less complex projects, boreholes without an adjacent CPT might be suitable.

The advice of a suitably qualified and experienced geotechnical engineer should be sought when scoping and planning intrusive investigations. The number and layout of boreholes should be determined, taking into account the expected ground conditions, the results of bathymetric and geophysical surveys, and the size of the structure. In general, unless the ground is very uniform, investigation locations should be not more than 50 m apart.

Intrusive investigations for wharves and quay walls should be spaced at regular intervals along the waterside frontage of the structure. In a transverse direction, locations should be chosen to assess the ground conditions over the width of the works, including any ground that could affect the wall and backfill. The intrusive investigation should also allow for any anchorages on the landward side. The investigation layout should be sufficient to assess possible risks for instability due to a rotational slip.

Investigations for jetties should be sited along the line of the jetty as a whole, including berthing heads, mooring structures and approach structures. At least one investigation location should be within the ground that will be influenced by the particular component of the structure, including mooring or fender structures.

For a simple ship building berth, there might be a need for a retaining wall at the lower end to provide deep water, so the ground investigation should take into account that a retaining wall might be needed at that location.

The remainder of the site should be investigated to ascertain whether the allowable ground pressure is sufficient to support shipyard hard standing areas by direct loading, or whether bearing piles will be required. In the latter case, the ground investigation should provide sufficient information to establish the probable required length of the bearing piles.

For a shiplift, the investigation locations should be positioned on the lines of the piers such that the size and length of the new piles, which might be heavily loaded, can be determined.

The layout of investigation locations for dredging and reclamation work should be based on the extent of the operations required and on the variability of the subsoil.

NOTE 2 If the depth of ground to be removed is limited to a few metres, a relatively simple method of ground investigation, such as vibrocoring, can be used.

If unexpected variations in level, thickness or the properties of various layers of subsoil are revealed during intrusive investigations, the adequacy of the investigation should be reviewed. This should be undertaken in consultation with a suitably qualified and experienced geotechnical engineer. The need to increase the number of the investigation locations should be determined, either as part of the current investigation or as part of a future investigation, to ensure a sufficient understanding of the subsoil conditions can be attained.

5.6 Depth of intrusive investigations

The depth of boreholes and CPTs should be in accordance with BS EN 1997-2:2007.

For all cases, intrusive investigations should extend a sufficient depth below any proposed foundations, dredging surfaces, excavations and likely slip failure surfaces to allow for design to be carried out.

In order to obtain soil parameters to form the basis of slope stability analyses, the depth of intrusive investigations should cover the likely maximum depth and lateral extent of slip surfaces (see Figure 1). The depth of investigations for piled foundations should extend to a sufficient depth to establish the bearing capacity and settlement characteristics of both individual piles and groups of piles.

NOTE 1 For individual piles, this is typically at least 3 m below base of pile, or five times the pile diameter, whichever is the greater. For pile groups, this is typically 1.5 times the width of the pile group.

In order to obtain information on the stability and deformation of the foundations of gravity-type quay walls, such as sheet pile cellular structures, or monoliths, the boreholes should be drilled to a depth below the base of the structure of 1.5 times the width of the structure or pile group supporting the structure (see Figure 2).

Boreholes drilled in rock formations should be taken to a depth sufficient to explore the thickness and characteristics of any rock that could affect the behaviour of slopes and foundations. Where heavily loaded structures depend for their stability on the strength of fresh unweathered rock, the boreholes should be drilled to a depth of 3 m or five times the diameter of foundations below the base of the foundation, whichever is the greater, into such material to establish and confirm its quality and continuity. Advice on the likely depth of weathering in rock should be sought from a suitably experienced and qualified geotechnical engineer or engineering geologist, but in any case, this should be established by the ground investigation.

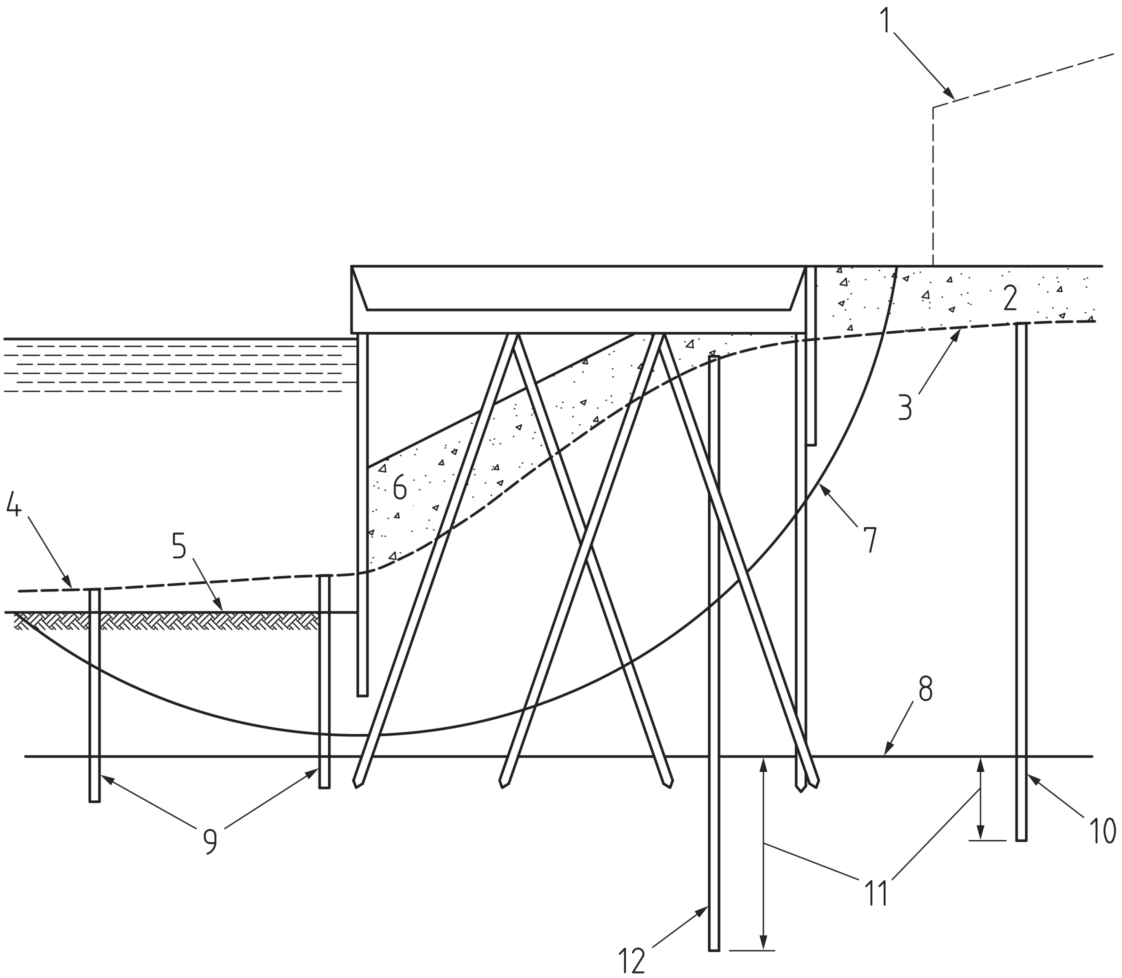
Dredging below the seabed can involve removal of soil from below the designed dredge level, depending upon the excavation technique, and should be taken into account in planning the depth of investigations. The possibility of future deeper dredging should be reviewed. Boreholes in dredging areas should be extended below design dredge level until a stratum of known geological characteristics is encountered or to a depth of 5 m below design dredge level, whichever is the lesser. The investigation should be made within the planned areas of dredging. It is not sufficient or acceptable to rely upon other investigations outside the proposed dredging areas, although the results of such investigations should be examined and made available, where relevant.

The possibility of heave of the floor of the excavation due to swelling of clay soils should be taken into account. Boreholes should be taken to a sufficient depth to establish the thickness and characteristics of swelling soils and to enable the risk of floor heave to be assessed and evaluated.

NOTE 2 Deep excavations can involve a risk of uplift of the base, due to shear failure in soft to firm clays, or to the occurrence of groundwater under pressure in pervious layers underlying impervious soils or rocks at the base of the excavations.

Where groundwater lowering schemes for excavations, or installations to cut off inflow into excavations, are proposed, the boreholes should be drilled completely through the water-bearing formation to locate the underlying impervious stratum, and located far enough into the stratum to establish its continuity.

Figure 1 — *Location and depth of boreholes for piled wharf*



Key

1 Future shoreside structure

2 Reclamation fill

3 Original foreshore level

4 Original seabed level

5 Dredged level

6 Fill

7 Potential sliding surface of rotational slip

8 Top of bearing stratum

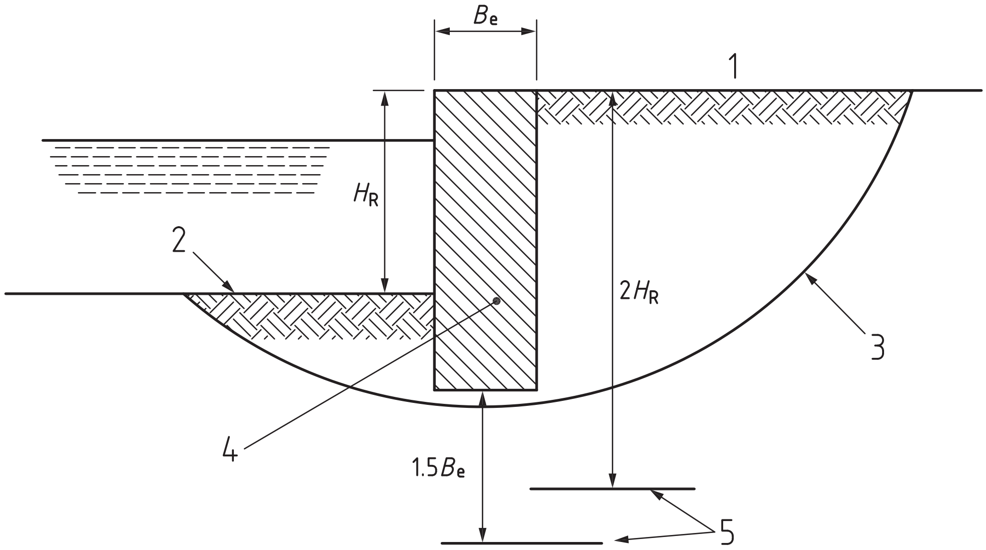
9 Boreholes to obtain information on passive earth resistance, dredging characteristics and overall stability

10 Boreholes to obtain information on reclamation and foundations of shoreside structures

11 Boreholes to be taken to sufficient depth to prove bearing capacity of piles

12 Boreholes to obtain information on active earth pressure, anchorages, pile driving conditions and overall stability

Figure 2 — *Depth of boreholes in relation to retained height of soil and width of quay wall*



Key

1 Ground surface
2 Dredged level
3 Potential surface of sliding
4 Gravity-type earth-retaining structure
5 Minimum depth of boreholes

5.7 Sealing of boreholes

All boreholes should be sealed upon completion in such a way as to prevent leakage into the excavation and contamination of or connections between aquifers. Sealing should be in accordance with BS EN ISO 22475-1, with a material of equal or lower permeability than the original ground.

NOTE 1 The use of grout is preferable.

NOTE 2 There have been cases of leakage into dock excavations from abandoned boreholes which have been extremely difficult to seal.

5.8 Ground investigations over water

*COMMENTARY ON **5.8***

The scope of the work, including the methods of boring, sampling and in-situ testing, requires careful consideration depending on the particular difficulties of the site. Attention is drawn to health and safety requirements, navigational warnings, and the regulations of governmental departments and other authorities that might be relevant when working over water.

General guidance on geotechnical investigation and testing is given in BS EN ISO 22475-1.

5.8.1 General

Ground investigations conducted over water are more difficult and time-consuming than comparable investigations conducted on land, and should be planned well in advance of the time identified to undertake the investigation so that the logistics and constraints of carrying out the investigation can be adequately addressed. Where a phased approach to an investigation is to be adopted, the likely full extent of the ground investigation should be identified at the outset such that adequate allowance is made in the overall project planning.

NOTE 1 Geophysical surveys are extensively used at the planning stage to provide additional geological information for the ground beneath the construction site and the positioning of the investigation boreholes.

NOTE 2 The sinking of boreholes and CPTs below water presents particular difficulties and requires a fixed, floating or heave-compensated working platform.

Suitable measures should be established to protect the investigation string from the effects of currents and waterborne objects.

When planning an overwater investigation, a realistic allowance should be made for the possibility of delays being caused by adverse weather conditions.

NOTE 3 For any given weather condition, the amount of delay depends on the type and size of the installation. In general, the larger the platform or floating craft, the smaller the risk of delay due to adverse weather conditions but, on the other hand, the greater the complexity.

NOTE 4 Sinking boreholes and CPTs between high and low tide levels may be achieved in a number of ways, including:

- *scaffold platforms by using flat-bottomed pontoons;*
- *shallow draft jack-up rigs; and*
- *moving boring rigs to the location when it is dry and exposed during periods permitted by the tides.*

5.8.2 Platforms and barges

The design of all barges and platforms should take into account the capability of the sea-bed strata to withstand the foundation actions. The design should also include the effects of the fluctuating water levels due to tides, waves and swell conditions. Such constructions should be sufficiently strong for the boring operations to resist waves, tidal flow, other currents and floating debris.

Onshore ground investigation equipment should be used only where stable working platforms are available or can be provided, such as oil drilling platforms and jetties or purpose-built scaffold platforms and drilling towers. Where such equipment is used, the current loads on the drill string should be taken into account. When working from existing structures, a cantilever platform on which to mount the boring rig should, if necessary, be constructed over the water. Advice should be sought from a geotechnical investigation engineer who specializes in both onshore and offshore investigations.

NOTE 1 When carrying out intrusive investigations close to the shore in relatively shallow water, the convenience of constructing a scaffold or other tower at the borehole location to avoid working in water can be advantageous.

Where free-standing independent towers are used, a means of transporting the boring equipment to the tower should be provided. The tower should be constructed such that it can be moved from one position to another.

NOTE 2 Platforms and other specialized craft fitted with spud legs are commonly used during overwater investigations. These can be self-propelled or might require a separate transport craft in order to be manoeuvred into position. These fulfil the requirement for a fixed working platform and provide manoeuvrability.

5.8.3 Floating craft

Where floating craft are to be used, the following factors should be taken into account in selecting and determining the suitability of the floating craft:

a) the geotechnical properties of the seabed;

b) the likely weather conditions;

c) the depth of water;

d) the strength of currents;

e) whether the water is sheltered or open; and

f) whether accommodation is required on board for personnel.

In inland water, a small anchored barge might suffice, but in less sheltered waters a barge should be of substantial size, and anchors should be correspondingly heavy.

NOTE In offshore conditions, a ship is often employed, and it might then be possible to accommodate the personnel on board, reducing the need for auxiliary supply vessels.

In order to achieve high quality coring, constant pressure should be maintained between the drill bit and the bottom of the hole. Heave compensation should be adopted where appropriate. The types of sample to be recovered, and in-situ tests to be performed, should be selected on the basis that appropriate equipment will be employed to counter the local environmental effects when boring from floating craft and that the working platform might move.

5.8.4 Setting out and locating investigation positions

A differential global positioning system should be used to set out investigation positions, as the equipment is simple to operate, provides accurate positioning data and eliminates unnecessary risks to personnel in terms of health and safety.

5.8.5 Determination of reduced level of bed and strata boundaries

For correct interpretation and reduction to an appropriate datum level, tidal corrections should be applied to the data obtained, particularly in the near-shore environment, if reduced bed level data cannot be obtained directly. Actual measured data should be used wherever practicable, but standard tide tables may also be used.

If a tide gauge is set up close inshore in order to transfer reduced levels from shore to a boring vessel or operating platform, the gauge should be read at frequent intervals throughout the tidal cycles, with readings of water depth being taken at the same time on the vessel or operating platform.

NOTE Corrections might be necessary to allow for tidal variations when the tide gauge reading and the reading from the vessel or operating platform vary significantly. Some methods of heave compensation on the drilling vessel make this correction automatically. The depth of water can be difficult to determine where the seabed is very soft, and reduced levels of strata boundaries then become less accurate.

6 Groundwater investigations

*COMMENTARY ON CLAUSE **6***

The piezometric head of pore water in soils is a critical factor in the analysis of the stability of excavated slopes and earth-retaining structures. Observations of the salinity of the groundwater at various positions back from the waterside face can indicate the relative influences of saline water and non-saline groundwater on piezometric levels.

General guidance on groundwater sampling and measurement is given in BS EN ISO 22475-1.

Geotechnical investigation on land, or land that might be exposed during variations in sea or river levels, should include measures (e.g. piezometers) to record groundwater pressures over a sustained period. The piezometric head of pore water in soils is a critical factor in the analysis of the stability of excavated slopes and earth-retaining structures, and the design should take into account fluctuations in piezometric head in the groundwater caused by corresponding fluctuations in tidal or seasonal levels in the waterway.

Piezometers should be installed in key strata, where practicable, and provision should be made for their continued reading following the completion of any fieldwork. The period of reading should be as long as possible, but generally not less than a year.

The relationship between the groundwater and waterway levels should be established by means of simultaneous observations of both the levels of the waterway and levels in piezometers, which should be installed on the landward side at various distances back from the waterside face of a structure or slope. The observations should be made to cover periods of spring tides and neap tides. Where practicable, they should cover periods of seasonal peak conditions in waterway and sea levels.

An investigation should be undertaken of the possible existence of groundwater under artesian or sub-artesian pressure within higher permeability soil layers confined by impervious strata. The investigation should be achieved by taking observations in piezometers installed within the higher permeability layers.

Where groundwater lowering schemes are proposed to enable, among other things, the construction of dock basins or lock chambers, the groundwater investigations should include measurements of the permeability of the soil or rock in situ.

NOTE Information on permeability testing of soils is given in BS EN 1997-2:2007.

7 Field tests in soil and rock

7.1 Planning

The following factors should be taken into account when planning field tests:

a) geology/stratification of the ground;

b) type of structure, the possible foundation and the anticipated work during the construction;

c) type of geotechnical parameter required; and

d) intrusive investigation method being adopted.

7.2 Normal field tests

Information about the site should be obtained using as many of the following field tests as are appropriate, either individually or in combination:

a) dissipation tests (CPT only);

b) pressuremeter and dilatometer tests;

c) standard penetration test;

d) dynamic probing;

e) weight sounding test;

f) field vane test; and

g) plate loading test.

NOTE Information on these tests is given in the various parts of BS EN ISO 22476. General guidance on testing methods is given in BS EN 1997-2:2007.

7.3 Other field tests

7.3.1 Determination of earth pressure coefficient at rest

When planning the ground investigations for earth-retaining structures that are to be formed directly against the retained soil, a review should be undertaken of the need for in-situ measurements of horizontal earth pressures. This should include the methods to determine such pressures. Generally, unless the investigation methodology can gain useful and applicable information on in-situ horizontal earth pressures, such specific work should not be undertaken.

NOTE In soils, horizontal earth pressures can be estimated from pressuremeter or dilatometer tests. In rock, the over-coring or hydraulic fracturing method can be employed to obtain in-situ stress results.

7.3.2 Detection of underground movements at depth

Where maritime works are to be constructed on sites where previous instability is suspected, movements in the ground should be monitored at depths where underground movements could affect the new works before construction begins. This should be followed by similar observations during, and for as long as possible after, construction of the project.

NOTE 1 This form of monitoring also forms part of investigations of instability, which might arise as a result of earthworks associated with shoreside structures.

NOTE 2 Trial pits and trenches may be used to determine the location of existing shear surfaces at relatively shallow depths. This method becomes increasingly difficult with depth.

Where there is reason to believe that movement is taking place, or it is suspected that movement is imminent, some form of quantitative monitoring should be established.

NOTE 3 This can include direct surveying techniques, shape arrays, and inclinometers installed in boreholes. Inclinometers, unlike simple slip indicators, can show the precise direction in which the soil has sheared between any two occasions of measurement.

The equipment should be used where movement is anticipated or where the rate of movement is small.

NOTE 4 If used when movement is relatively rapid, the inclinometer probe is soon unable to pass the shear surface and hence all measurement of movement below this level ceases to be possible.

The annular space between ground and the instrumentation access casing should be grouted.

7.3.3 Field tests for dredging

*COMMENTARY ON **7.3.3***

Vibrocoring might give little indication, particularly in sands or gravels, of the in-situ strength or degree of consolidation of the material.

Relatively small differences in strength can have a significant effect on dredging production. In sands, a high degree of consolidation will adversely affect the dredgeability of the soil; in clays, high shear strengths will also result in much lower production, as will greater crushing strength in weak rocks. Soil strengths also influence the stability of side slopes, which can be of particular importance in the dredging of temporary trenches for the laying of pipes, outfalls or other services.

The removal of rock by dredging, even for small quantities, usually involves the mobilization of specialist equipment.

A number of borehole measurements, records, in-situ tests or field assessments, particularly in rock, may be carried out to assess strength or resistance to cutting. These include:

- *detailed borehole logging;*
- *observation of discontinuities;*
- *downhole televiewer and geophysical records;*
- *total core recovery;*
- *solid core recovery;*
- *rock quality designation;*
- *drillability;*
- *point load tests;*
- *velocity of propagation of sound; and*
- *standard penetration tests.*

Further information on in-situ tests and assessments is given in BS EN 1997-2:2007.

Where possible, in-situ penetration testing should be undertaken in addition to the recovery of samples, in order that direct measurement of in-situ material strength can be carried out.

Where the dredging of rock, with or without pre-treatment, is anticipated, a thorough investigation should be undertaken, with adequate sampling and laboratory testing of recovered rock cores, to allow an accurate assessment and description to be made of the condition of the rock in relation to fracture state and rock quality. Information about rock strength, which affects the energy required to effect removal, and abrasiveness, which affects the rate of wear of dredger components, should be obtained through laboratory testing.

NOTE If fracturing is closely spaced and open, pre-treatment might not be necessary to enable dredging. If pre-treatment is necessary, fractured rock can impede drilling by causing the drill to jam.

7.4 Trial dredging

*COMMENTARY ON **7.4***

Trial dredging might be the only satisfactory way of predicting the performance of particular dredging plant on a particular site. However, the practicability, time needed and difficulty of trial dredging can be unacceptably high unless suitable plant is already available and located on the site, but there are situations in which it is recommended.

Trial dredging should be undertaken in the following situations:

a) when the soil conditions within the area to be dredged are known to be extremely complex with a wide variety of soil types and strengths, and a pattern of sampling by boreholes or some other method might not provide a truly representative picture of the overall condition;

b) where trench or channel formations are proposed that would cut across deposits of doubtful stability or across the paths of substantial sediment transport routes; and

NOTE 1 In such cases, the formation of a trial section by dredging, and the careful monitoring of the performance of that trial section, might provide the best means of accurately forecasting the performance of the finished formation.

c) in situations where there is no satisfactory conventional soil investigation method that is capable of sampling the true ground conditions.

NOTE 2 This includes sites that contain particle sizes too large to be recovered intact by normal sampling methods. Trial dredging is therefore appropriate in areas known to contain very coarse soils, namely cobbles and boulders, typically of glacial origin.

The performance of the dredger and of the dredged formation should be monitored throughout the trial. Key parameters such as rate of production, precision and plant wear should be selected before execution and recorded during execution.

Account should be taken of changes in weather conditions during these trials to assess plant performance.

7.5 Sampling of soils, rock and groundwater

The sampling of soils, rock and groundwater should be carried out in accordance with BS EN ISO 22475-1, in the context of geotechnical testing as described in BS EN 1997-1:2004 and BS EN 1997-2:2007.

8 Laboratory tests on soil and rock

The testing of soils and rock should be carried out in accordance with BS EN 1997-2:2007.

NOTE 1 The following properties are important when determining the engineering performance of soil and rocks:

- *bulk density;*
- *water content;*
- *particle density;*
- *consistency limits;*
- *organic content;*
- *plasticity;*
- *particle size grading;*
- *chemistry;*
- *compressive strength;*
- *in-situ stress and earth pressure;*
- *tensile strength;*
- *drained/undrained shear strength;*
- *compressibility;*
- *permeability; and*
- *porosity.*

NOTE 2 The following rock properties are important in providing the information necessary to assess whether, or how, rock can be dredged, with or without pre-treatment:

- *density;*
- *hardness;*
- *abrasiveness;*
- *porosity;*
- *tensile strength; and*
- *compressive strength.*

Section 3: Geotechnical design

9 General

9.1 Basis of geotechnical design

Geotechnical design should be carried out in accordance with BS EN 1997-1:2004. The geotechnical parameters should be established in accordance with BS EN 1997-1:2004 through an appropriate geotechnical investigation. The partial factors applied to the geotechnical parameters should be taken from BS EN 1997-1:2004.

9.2 Geotechnical design report

The geotechnical aspects of the design should be recorded in a geotechnical design report in accordance with BS EN 1997-1:2004.

The design and design report should include recommendations for design verification, including testing, trials and monitoring.

The geotechnical design report should include the geotechnical investigation report as described in BS EN 1997-1:2004 and should indicate the datum adopted for any geotechnical investigation, design calculations and the scheme drawings. It should also indicate the relationship between ordnance datum and chart datum.

9.3 Tides and water level variations

Maritime structures should be designed to safely withstand the effects of the still water level from extreme low water (ELW) to extreme high water (EHW) that are expected to occur during the design life of the structure. These extremes should be established in relation to the purpose of the structure and the accepted probability of occurrence, but should normally have a return period of not less than 50 years for permanent works.

NOTE 1 Extreme water levels, which can be caused by a combination of astronomical tides, positive or negative surges, seiches and freshwater flow, are required for the evaluation of:

- *overtopping;*
- *hydrostatic pressures, including buoyancy effects;*
- *soil pressures on quay walls; and*
- *lines of action of mooring and berthing forces, forces from other floating objects and wave forces.*

The effect of waves and wave run-up should be taken into account in relation to overtopping and hydrostatic pressures.

The potential effects of climate change should be taken into account and allowed for in predicting extreme water levels.

Design for ultimate limit state conditions should be based on extreme water levels on both sides of the structure, with the astronomical tide level being treated as a temporary load and the other elements as transient loads.

Design for serviceability limit state conditions should be based on tidal variation from highest astronomical tide (HAT) to lowest astronomical tide (LAT), but taking into account tidal lag, drainage issues and the effects of freshwater flow, as well as any other known contributors.

NOTE 2 Information concerning waves is given in BS 6349-1-2.

NOTE 3 Guidance on methods of assessing the relationship between astronomical data and surge tide levels can be obtained, in the UK, from the National Oceanography Centre, Joseph Proudman Building, 6 Brownlow Street, Liverpool, L3 5DA, or the appropriate national authority in other countries.

9.4 Earthquakes

COMMENTARY ON ***9.4***

For most engineered and non-engineered structures in the UK, natural hazards, such as wind, flood, ground movements due to moisture change, and extreme temperatures, pose a substantially higher risk of injury and economic loss over the lifetime of a structure than the risk posed by earthquakes. Consequently, in the UK, where seismic activity is low, allowance for earthquake effects is not normally necessary for the design of maritime structures. For maritime works in many other countries, seismic effects are very significant.

The following examples might fall into this category:

- *structures where failure poses a large threat of death or injury to the population. Examples include certain petrochemical installations, such as liquid natural gas (LNG) storage tanks and high pressure gas pipelines;*
- *structures which form part of the national infrastructure and the loss of which would have large economic consequences. An example is a major bridge forming a transportation link vital to the national economy;*
- *structures whose failure would impede the regional and national ability to deal with a disaster caused by a major damaging earthquake; and*
- *strengthening or upgrading of historical structures forming an important part of the national heritage.*

Many countries subject to earthquakes include specific seismic design considerations within their building codes, although there are considerable variations in the approach and some are less complete than others. Few existing codes have been prepared specifically with maritime structures in mind, although guidance is available in the PIANC publication Seismic design guidelines for port structures *[3].*

The damaging effect of earthquakes arises from horizontal and vertical accelerations of the soil mass being transferred to structures above ground level through their foundations, base or pile support. The response of a structure to these accelerations depends upon its type, mass and dimensions and the failure modes to which it might be subject.

For each maritime structure, a review should be carried out to determine whether the structure needs to be designed for seismic effects. The review should take into account the established practice for the country or region in which the structure is located, together with the requirements of BS EN 1998 and any other relevant standards.

The design and construction of maritime engineering works to withstand seismic actions should be in accordance with the relevant part of BS EN 1998.

In seismically active areas, a type of structure should be adopted that has as little sensitivity to seismic action as is reasonably practicable.

The design should also take into account the following factors:

a) fine sandy soils can be vulnerable to liquefaction; and

b) soft clays can be susceptible to cyclic degradation and excess pore pressure build-up.

NOTE Local regulations might apply. Further information is given in PD 6698.

10 Geotechnical data – Selection of parameters for working design

10.1 General considerations

The planning for, collection of, and assessment of information on the ground should be carried out in accordance with BS EN 1997.

This should include a detailed examination of the borehole records, soil samples and preliminary test data, with the object of locating and defining the soil layers that are critical to the stability of the structure. The effects of disturbance caused by construction operations, dynamic loading during the service life of the structure, and the strain-dependent and time-dependent characteristics of the soil should all be taken into account.

Soil characteristics of fill material should meet the recommendations given in **12.1**. Soil characteristics for use in design calculations should be established and selected in accordance with BS EN 1997-1:2004.

10.2 Monitoring during and after construction

*COMMENTARY ON **10.2***

By their nature, marine structures often use soils that are created or significantly modified as part of the works. This is particularly the case with marine reclamation. The pre-works ground investigation does not identify the characteristics of such material placed and formed as part of the new works. Thus, monitoring (both during and after construction) of materials placed as part of the works is more important than with land-based structures.

The process of verification of the performance and the parameters of soils during and after the construction of marine works should be established as part of the design. It should be taken as an extension of the ground investigation process.

*NOTE More general recommendations for verification are given in Clause **16**.*

11 Water

11.1 Single-wall structures

*COMMENTARY ON **11.1***

Tidal variations cause a differential hydrostatic pressure between the waterside face and the landward face of an earth-retaining structure. Fluctuations in the level of the groundwater are normally less than the tidal variation, depending on the type and efficiency of the drainage measures provided in the wall, the permeability of the soil retained by and beneath the wall, and the flow of surface or subsoil water from landward sources.

*In all the cases described in **11.1.2** to **11.1.6**, the assumed groundwater level is taken to be that due to flow in a homogeneous permeable soil.*

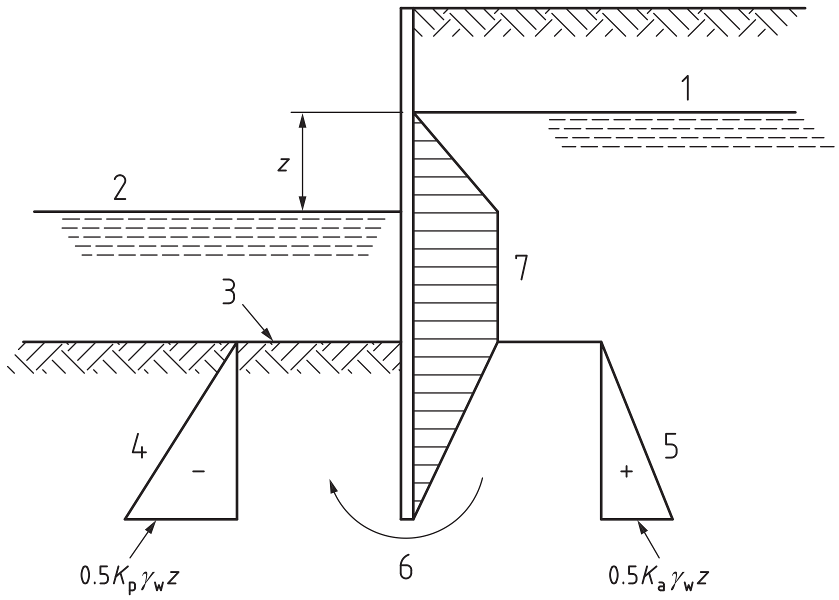
11.1.1 General

For a structure that retains a permeable soil, but where the wall elements are installed into an impermeable soil, acting as a cut-off to flow below dredged level, the differential water pressure should be derived in accordance with **11.1.2** to **11.1.6** as appropriate for the particular situation.

The hydrostatic pressures resulting from a strong subsoil water flow from a landward source should be taken into account. The effects of excess water pressures in permeable layers within a layered or laminated soil or rock formation should also be taken into account.

In cases where permeable soil extends below the toe of the structure, seepage can take place, and the hydrostatic pressure distribution should be modified to allow for this. The effects of seepage causing an increase in active earth pressure and a decrease in passive earth resistance should be taken into account (see Figure 3).

Figure 3 — *Effects on hydrostatic and soil pressure distribution where seepage takes place beneath a retaining structure*



Key

1 Assumed groundwater level

2 Extreme low water

3 Dredged level

4 Decrease in passive resistance

5 Increase in passive pressure

6 Seepage

7 Pressure envelope

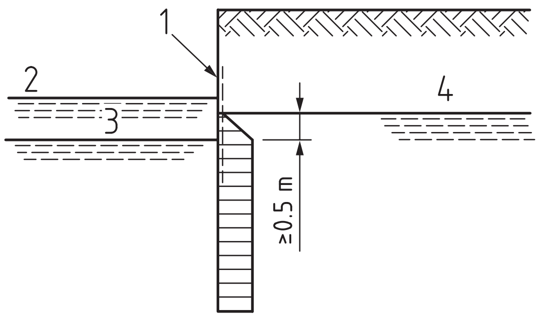
11.1.2 Minor non-tidal water level variations (drainage provided)

In non-tidal waterways where variations are caused by minor seasonal fluctuations, the differential water pressure from the landward to the waterside face should be taken as a minimum of 0.5 m, as shown in Figure 4a). Any other sources of groundwater should be taken into account in the design of the structure.

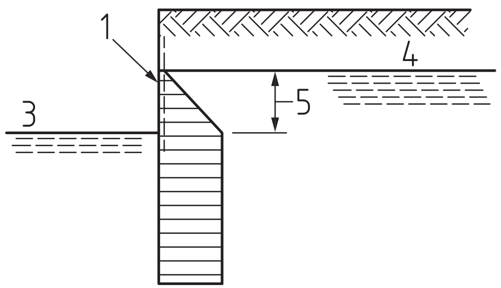
11.1.3 High flood flows in non-tidal rivers (drainage provided)

Where a rapid fall in water level of the waterway occurs at times of recession of floods, the differential water pressure should be taken as equal to the maximum predicted fall in water level over a 24 h period [see Figure 4b)]. The maximum water level in the waterway from which the predicted fall takes place should be assessed on the basis of available flood flow statistics. It should also be selected such that the most unfavourable effects of combined earth and differential water pressure are applied to the structure. Any other sources of groundwater should be taken into account in the design.

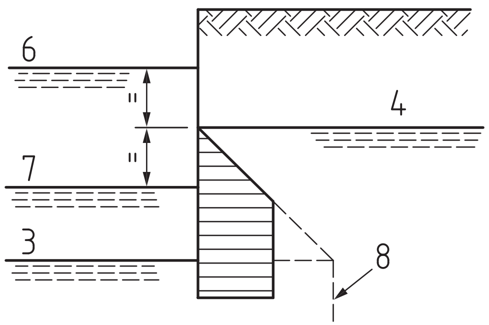
Figure 4 — *Hydrostatic pressure distribution on waterfront structures where soil is retained to full height of structure*



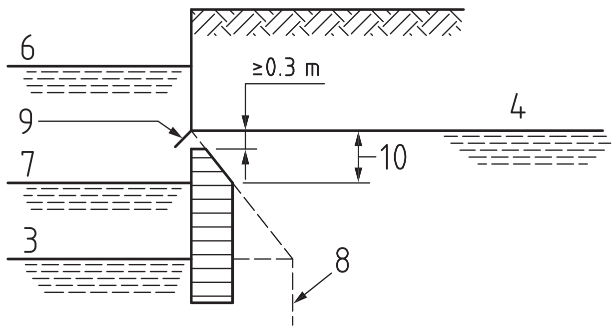
a) Minor, non-tidal



b) Large drawdown after floods



c) Tidal, no drainage



d) Tidal, with flap valves

Key

1 Drainage
2 Mean water level
3 Extreme low water (see 9.3)
4 Assumed groundwater level
5 Max. predicted fall in 24 h for most unfavourable combination of high water and low water in waterway
6 Mean high water springs (MHWS)
7 Mean low water springs (MLWS)
8 Extreme case
9 Flap valve
10 Min. 1 m hydrostatic pressure difference

11.1.4 Large tidal variations (no drainage provided)

The differential between the groundwater level on the retained side of the structure and the tide level in the waterway should be determined. This should take into account the normal groundwater flow, how the new asset is predicted to interrupt the groundwater flow, and whether the groundwater level is expected to be tidally influenced after the asset is constructed.

The groundwater level should be determined in accordance with BS EN 1997-1:2004 and assuming a low tide level. Low tide level should be taken to be in the range between mean low water (MLWS), for normal cases, and an assumed water level, as low as extreme low water (ELW) (see **9.3**). For the latter case, an assessment should be made of the risks of structural failure due to the combined effects of earth pressure and water pressure conditions for extremely low tides [see Figure 4c)], and the results of the assessment should be taken into account in the design.

Any other sources of groundwater should be taken into account in the design.

11.1.5 Large tidal variations (flap valve drainage provided)

For normal cases, the differential water pressure based on a groundwater level at 0.3 m above the invert of the flap valve (i.e. the head required to operate the valve), and a level in the waterway at MLWS, should be adopted in the design.

It should also be established whether an intermediate level down to ELW (see **9.3**) needs to be taken into account. In this case, an assessment should be made of the risks of structural failure due to the combined effects of earth pressure and water pressure conditions for extremely low tides, and the results taken into account in the design [see Figure 4d)].

Any other sources of groundwater should be taken into account in the design.

Flap valves should be designed to allow free drainage of the asset and an appropriate maintenance regime should be established to facilitate continued effective operation. The requirements for maintenance should be set out in the design report.

11.1.6 Embanked soil behind retaining structure (drainage provided)

Where the surface of the soil behind the retaining structures is sloped back and the flow from landward sources is horizontal, the effects of a sloping groundwater profile, as shown in Figure 5, should be taken into account in the design. This should be either in relation to the effective weight of the wedge of soil behind the structure, or within the zone of any anchorages (see **13.4**).

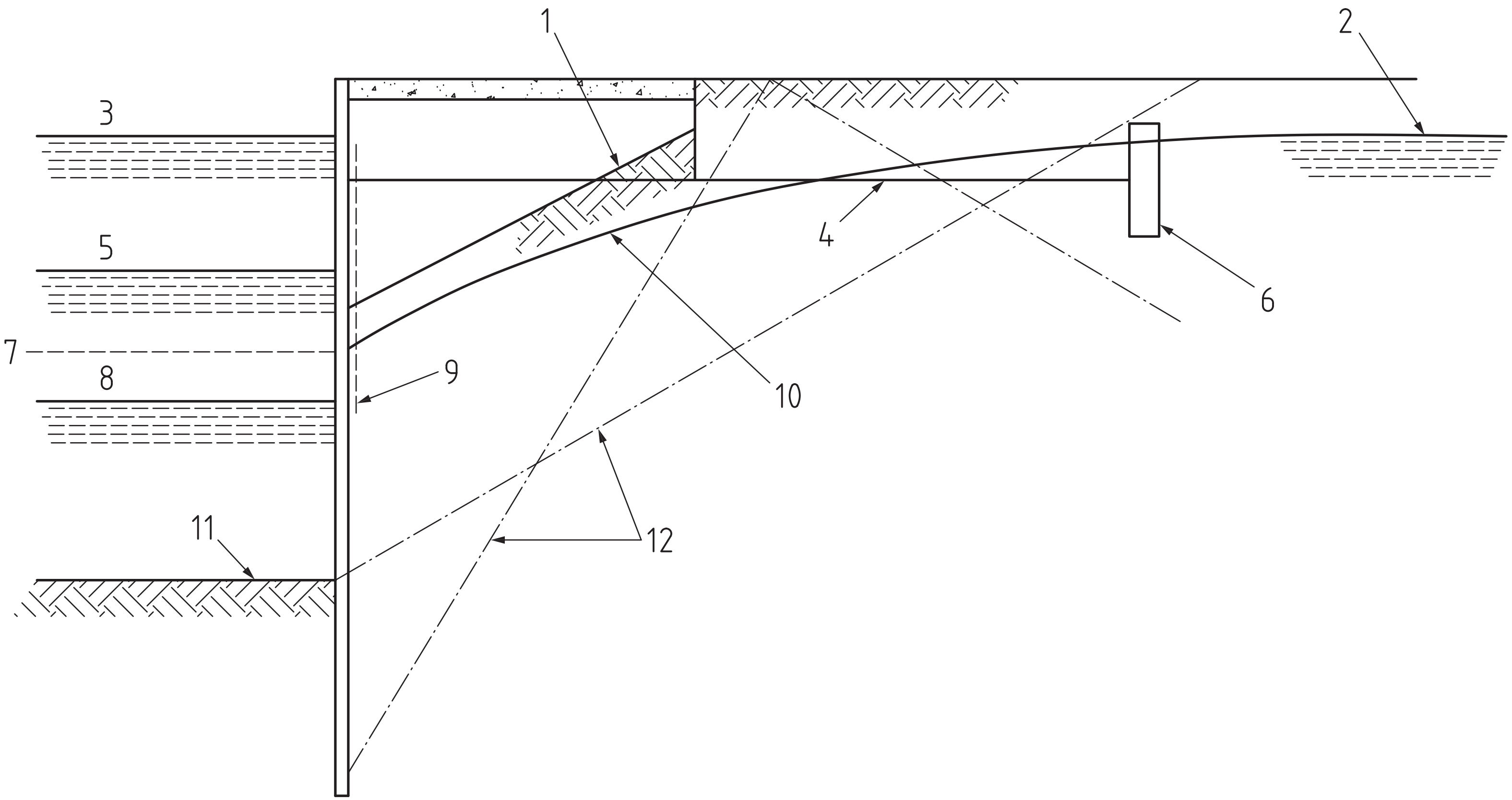
11.2 Double-wall and cellular structures

Free-draining material should be used as the preferred type where practicably available as filling within double-wall or cellular structures. Drainage should be provided in the walls so that the water level within the structure can then be taken as corresponding to the assumed groundwater level for **11.1.2** to **11.1.6**. Where alternative fill materials are used, the effects of any differential pressures acting on opposite sides of a double-wall structure or around the circumference of a cell should be taken into account when assessing global stability.

If hydraulic filling operations are used to place sand in the cells, the rate of pumping the fill can exceed the drainage capacity; in this case, as a temporary condition, the water level within the cells should be taken as the level of the tops of the cells, or at any lower level at which water is permitted to overflow freely.

Where double-wall or cellular structures are employed as breakwaters or as cofferdams subjected to wave action on the waterside face, the tops of the cells should be covered to prevent a rise in internal water level due to overtopping by waves.

Figure 5 — *Hydrostatic pressure distribution on waterfront structure where the soil is embanked behind the structure*



Key

1	Embanked soil	7	Design low water
2	Assumed unchanged groundwater level	8	Extreme low water
3	Mean high water springs (MHWS)	9	Drainage
4	Tie rod	10	Assumed groundwater profile
5	Mean low water springs (MLWS)	11	Dredged level
6	Anchorage	12	Potential failure surface

12 Fill, dewatering, ground improvement and reinforcement

NOTE Requirements concerning filling, dewatering, ground improvement and reinforcement are given in BS EN 1997-1:2004.

12.1 Fill materials

Where practicable, fill placed behind quay walls, in front of and behind deadman anchorages, and in reclamation areas should be granular material, i.e. sands and gravels, or crushed rock, capable of free drainage. Low permeability fill should be used only where the inclusion of such material has been allowed for in the design, there is no practicable granular source and the selected material is shown to correspond to the assumptions made in the design. Loss of material through joints in retaining walls should be prevented by providing a suitable filter behind the wall or by sizing the fill material such that it will not pass through the joints.

The availability of fills often dictates the viability of a scheme, but the additional pressure exerted by cohesive fill on a retaining wall due to the lack of drainage, the uncertainty in the performance of such fills and potential swelling should be taken into account when assessing the suitability of fills.

The availability and nature of materials for fill should be assessed and taken into account as part of the design process.

NOTE 1 Using free-draining material allows self-weight consolidation of the fill material to be maximized in the submerged zone, where additional compaction might not be required. Above water level, the fill may be compacted by conventional means. The compaction can increase the lateral earth pressures on the wall, resulting in additional actions on the wall or anchorage, or in additional seaward deflections. Ground improvement techniques can be applied to the submerged fill if it is necessary to reduce settlements or reduce the risk of liquefaction during a seismic event.

The need to replace weak soils should be assessed. The results of this assessment should then be used to inform the design, such that any vertical and horizontal movements that might affect port operations or the stability of the structures are minimized.

NOTE 2 Such soils, located below, behind or in front of quay walls and anchorages, include soft clay and silt and fine-grained sand, which could cause large settlements or liquefy in earthquakes.

Soil replacement below final seabed level and at in-situ concrete walls should be completed before quay wall construction and/or piling commences.

Where hydraulic fill is placed behind retaining walls, measures should be implemented to prevent pockets of soft material from forming next to the wall.

Fill above relieving platforms and false decks should be compacted granular material.

The need to compact loose materials above or below water level, including the use of deep compaction, should be assessed as part of the design. The density depends on the degree of compaction given to the fill, and the parameters should be selected accordingly.

NOTE 3 The soil parameters to be selected for cohesionless soils depend on the manner of the soils' deposition. Where they are discharged from a pipeline or dumped to fall through water, they are in a loose state of deposition.

NOTE 4 Cohesionless soils can be assumed to be in a medium dense state where drainage can take place, where they are placed above water level by hydraulic fill methods, or where these soils can be tipped and compacted above water level.

NOTE 5 Guidance on deep compaction techniques is given in Piling and ground treatment [4].

Where cohesionless soils are pumped immediately behind a retaining wall, the fill above normal standing water level should be treated as a fluid having a density equal to that of a fluid containing suspended solids.

NOTE 6 This condition applies until such time as drainage takes place to dissipate pore water pressures in the pumped material.

Crushed rocks should be treated as granular soils when selecting shear strength and deformation parameters. When crushed rocks are dumped through water, they should be assumed to be in a loose state of deposition (but see Note 7). High densities with a correspondingly high angle of shearing resistance should be assumed for crushed rocks compacted above water level.

NOTE 7 Time-dependent consolidation might occur as a result of crushing and degradation of points of contact between rock fragments which, on an unyielding surface, can amount to up to 1% of the height of the rockfill. Guidance on construction using crushed rock fill is given in BRE paper 15/71 [5] and in Laboratory compression tests and the deformation of rockfill structures [6].

The effects of weathering during the operational life of the structure, on rocks used as fill, should be taken into account, as softening at the points of contact of rock fragments results in a reduction of the angle of shearing resistance. The parameters for rock used as fill should be selected taking into account whether the rock degrades to a cohesionless or cohesive soil or some intermediate type. The parameters should reflect the likely long-term condition of the material.

NOTE 8 Complete degradation results in the formation of a mass of fill of soil-like consistency, which can result in settlement.

NOTE 9 The causes and effects of settlement of filled areas, and methods for constructing earthworks over compressible soils where settlement might be excessive, are given in BS 6031.

If cohesive materials are to be used, the following should be taken into account in the design of retaining structures:

a) undrained behaviour using total stress parameters and behaviour in terms of effective stress parameters;

b) the way in which the low permeability cohesive fill could affect the groundwater level and water pressures on the wall;

c) the compaction of the cohesive fill and the relationship between optimum moisture content and natural moisture content;

d) relationship of cohesive fill with moisture content and its subsequent behaviour (shrinkage, swell, plastic limit, liquid limit, strength, etc.); and

e) settlements.

Highly over-consolidated clay should not be used as filling behind a retaining structure if this can be avoided due to the high-swelling pressures that are caused when the clay softens.

NOTE 10 Cohesive fills can be subject to significant long-term settlement due to self-weight settlement even after normal compaction. This process can induce additional stresses within tie rods and settlement beneath paving areas.

NOTE 11 While an effective stress approach is appropriate for both long-term situations (where steady and predicable pore pressures might operate) and short-term situations, the latter can be more difficult to apply as the prediction of short-term pore water pressure can be very difficult. Numerical modelling can enable such design analyses to be undertaken.

12.2 Ground improvement

The need for ground improvement techniques should be assessed in reclamation projects where densification of newly placed fills is required, and taken into account in the design. Where ground improvement is assumed in the design, the requirements should be set out in the construction documentation and described in the design report.

NOTE This is often required to increase the density of loose sand layers where seismic effects will lead to liquefaction.

Where vibrators are used to densify existing material or construct granular columns to reinforce the soil, the process described in BS EN 14731 should be followed.

Where vibro-compaction is undertaken, the effect of both horizontal actions on existing structures generated by the vibro process and the displacement of the existing fill materials should be taken into account in the design.

Where other methods are to be used, such as installation of band drains with surcharge or vacuum consolidation, specialist advice should be obtained.

12.3 Reinforcement

Reinforced soil structures should be designed in accordance with BS 8006-1.

13 Retaining structures

*COMMENTARY ON CLAUSE **13***

This clause covers all forms of retaining structures, including gravity walls, embedded structures, and double-wall and cellular cofferdams.

Retaining walls behave either more or less flexibly (the degree of stiffness) dependent upon the dimensions of the wall section and material used.

Although timber, glass-reinforced plastics and reinforced concrete have all been used as flexible walls, the most common material is steel.

Concrete diaphragm and bored pile walls, and tubular steel combined walls, are stiff structures and may be utilized where the ground is weak or where the wall deflection needs to be limited.

Geotechnical aspects of the design of retaining structures are covered in BS EN 1997-1:2004.

13.1 Earth pressures

13.1.1 General

For the purposes of calculating earth pressures, the following recommendations should be met.

a) Variable actions on surfaces should be determined as described in BS 6349-1-2.

b) Water levels should be determined in accordance with **9.3** and Clause **11**.

c) Ground pore-water pressures should be determined with reference to tidal range, soil permeability, drainage provisions and any artesian or sub-artesian groundwater conditions.

d) Allowance should be made for reduced passive resistance caused by overdredging and/or scour.

e) The possibility of high lateral pressures should be taken into account where concrete retaining walls are cast against the excavated face of soil and/or rock with potential swelling characteristics.

f) For flexible structures, such as sheet piled walls, account should be taken of the deformation of the structures that can lead to significant variations in earth pressures.

g) The lateral pressures on retaining structure induced by surcharge applied in the form of uniform, line and patch actions should be taken into account.

NOTE 1 Patch loadings create a 3D pressure distribution, particularly when a long structure is locally loaded. A simplified design methodology to account for those loadings is given in CIRIA Report C760 [7].

If a new groundwater regime is likely to be established in a marine structure (either in any fill or in the original ground), any resulting changes should be assessed and taken into account in the design. The pre-works groundwater regime should be established by appropriate monitoring.

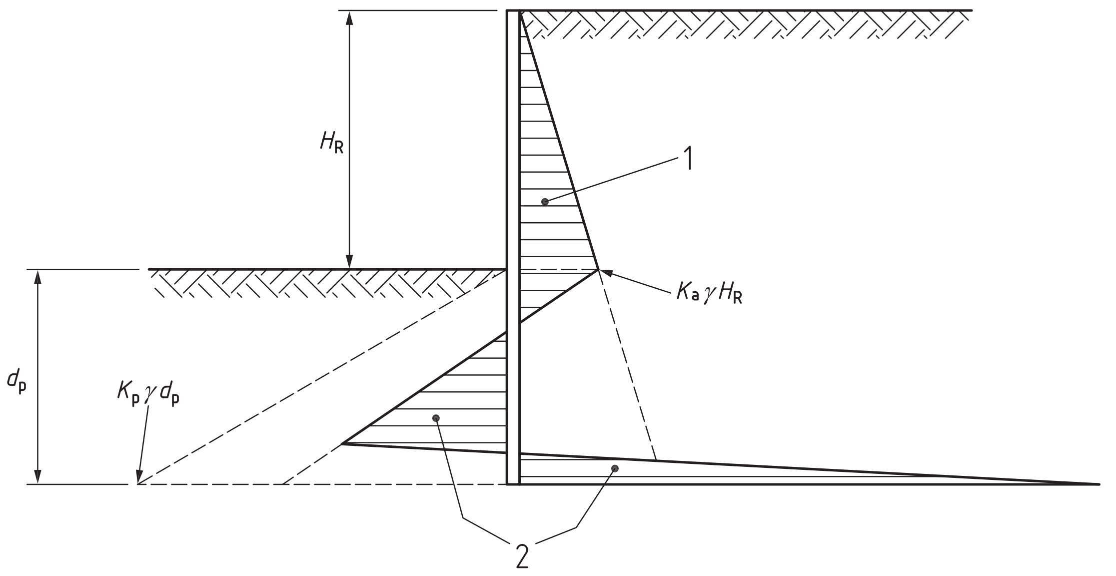
The groundwater design assumptions and the post-construction groundwater regime should be confirmed by appropriate groundwater monitoring. If the regime differs significantly from that assumed in the design, the need for appropriate mitigation and remedial works should be assessed, and implemented where appropriate.

Software capable of modelling soil–structure interaction should be used to determine the earth pressures acting on the structure and consequential load effects, taking into account the flexibility of the wall and allowing for any time-dependent changes to the soil characteristics.

NOTE 2 Cohesionless soils do not normally show time-dependent effects.

NOTE 3 An idealization of earth pressures on a single-wall sheet pile structure is given in Figure 6.

Figure 6 — *Distribution of earth pressure and earth resistance on cantilevered single-wall sheet pile structure*



Key

1 Active pressure

2 Net passive resistance

13.1.2 Arching effects in anchored walls

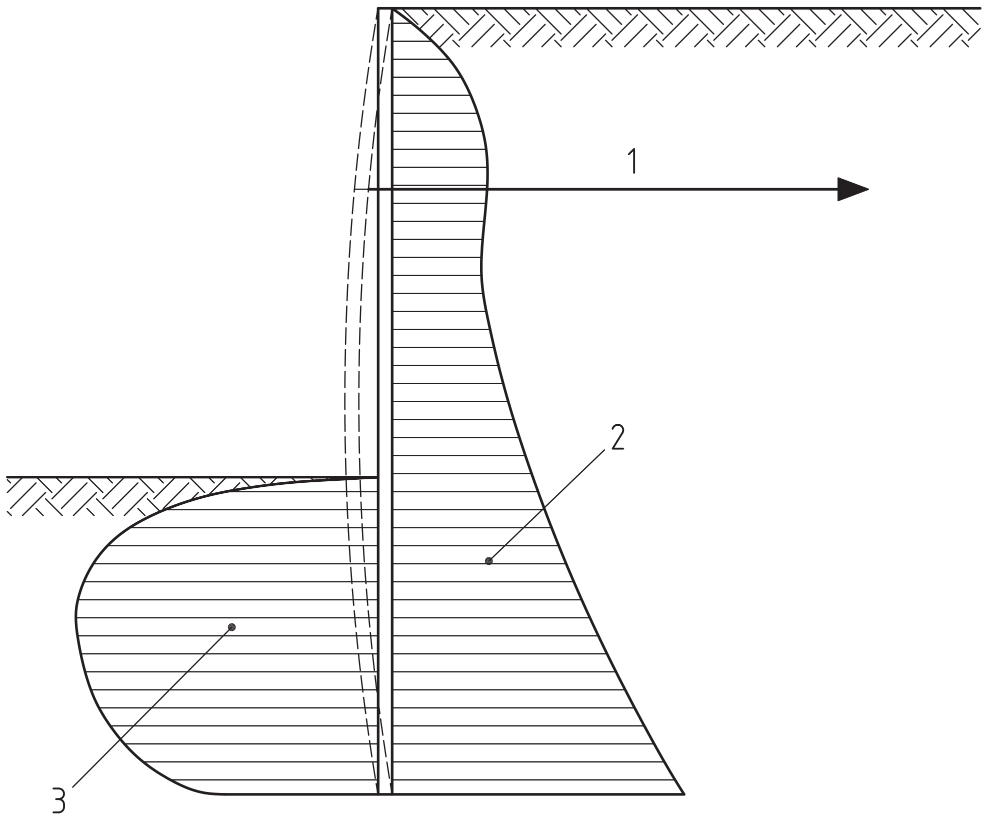
Arching resulting from wall deflection should be taken into account in the design only when the anchor cannot yield.

NOTE 1 An idealization of earth pressures on a single anchored wall is given in Figure 7.

Arching, with the development of high lateral pressures in the upper part of the soil wedge, should be taken into account in cases where cohesionless soil fill is compacted above standing water level and/ or where surcharge is imposed on the ground surface behind the wall.

NOTE 2 It is considered that arching effects are nullified when the yield of the anchorage system is less than 0.1% of the height of a wall in front of which dredging is carried out after completion. This order of movement would normally take place in an anchored sheet pile structure.

Figure 7 — *Distribution of earth pressure and earth resistance on anchored single-wall sheet pile structure*



Key

1 Tie rod

2 Active pressure

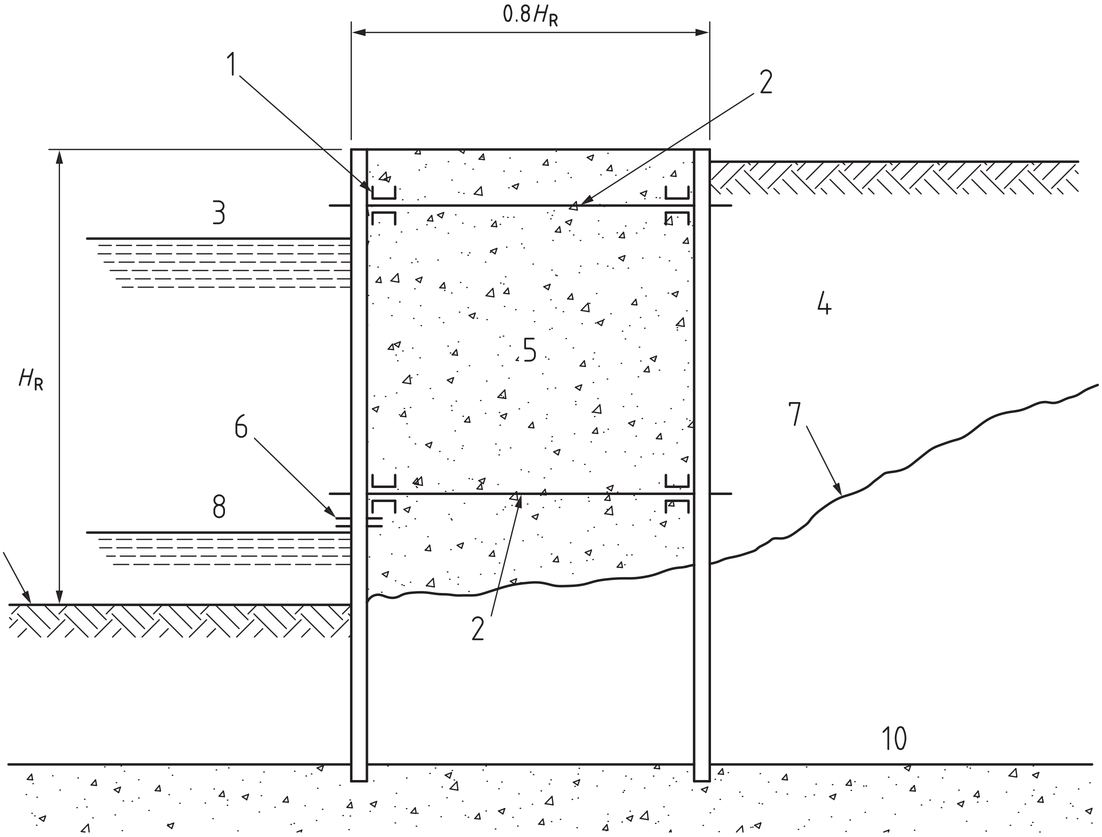
3 Passive pressure

Arching conditions should also be assumed in relation to the lateral pressure distribution on gravity-type, double-wall sheet pile retaining walls (see Figure 8 and Figure 9).

NOTE 3 Results of limited full-scale tests of a dense, uniform, medium-grained, dry sand show that a total translational wall movement of 0.5% of the wall height is required to reduce the level of the pressure resultant from 0.45 times the height of the wall to the linear distribution level of 0.33 times the wall height. At that stage of movement, a slip plane is developed in the surface of the compacted sand backfill.

NOTE 4 Where sheet piles are toed into rock, movement of the sheet piles is prevented, and the distribution of active pressures is modified. The pressures at the toe correspond to at-rest conditions.

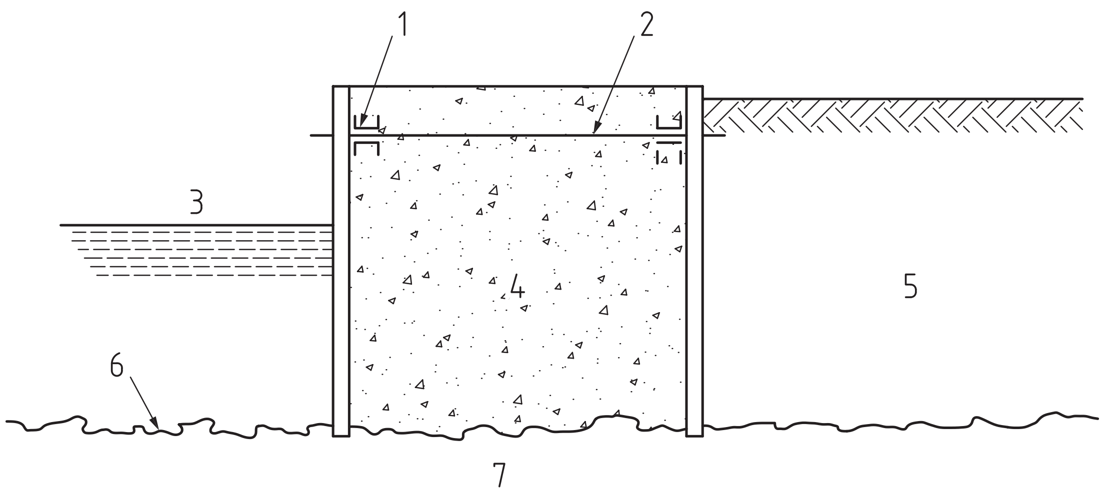
Figure 8 — *Double-wall sheet pile structures – Sheet piles driven into soil below seabed*



Key

1 Walings	6 Drainage
2 Tie rod	7 Original ground level
3 Mean high water springs (MHWS)	8 Mean low water springs (MLWS)
4 Bulk fill	9 Seabed
5 Cell fill	10 Hard stratum

Figure 9 — *Double-wall sheet pile structures – Sheet piles terminated on rock at seabed*



Key

1 Walings

2 Tie rod

3 Mean high water springs (MHWS)

4 Cell fill

5 Bulk fill

6 Seabed

7 Rock

13.1.3 Retaining structures in cohesive deposits

Earth resistance in normally and slightly over-consolidated cohesive soils is likely to be critical for short-term undrained conditions, when a total stress analysis might be appropriate, but a check calculation should be carried out for long-term drained conditions with effective stress parameters. Over-consolidated clays soften with time as a result of wall deflection, and drained conditions should be adopted when calculating earth resistance.

Where the sheet piles are terminated in a cohesive soil, allowance should be made for long-term settlement of the structure due to actions applied to the deck. The design should allow for any reduction in wall friction resulting from relative vertical movement between the structure and the retained soil on the active pressure side, but should not take account of the increase in wall friction on the passive side.

13.1.4 Relieving structures

Where a relieving structure is installed behind a retaining wall and supported by piles driven below the potential surfaces of sliding, it should be assumed that pressures from the actions supported by the piles are not transmitted to the retaining wall. However, earth pressures caused by soil displacement during pile installation and the effects of pore pressures in the soil should be taken into account.

NOTE These pressures are retained by and beneath the wall, and are induced by soil displacement as the piles are driven. Further guidance is given in Ground movements due to pile driving [8].

13.1.5 Cellular structures

Where double-wall and cellular sheet pile structures are provided with a reinforced concrete deck supported by the perimeter piling and capable of carrying heavy, imposed actions, internal lateral pressures from the fill material due to the actions on the deck should not be assumed to act on the retaining structure.

In cellular structures, granular soil that is used to fill the cells should not be placed until the large displacement bearing piles have been driven. Bored and cast-in-place piles should be installed after completion of filling, but measures should be taken to prevent the development of excessive pore pressures in any layered or laminated clays that exist below the base of the structure.

In cases where it is necessary to drive bearing piles within the cells to support heavy deck loading, the effects of soil displacement and pore pressure development in the natural soil within and beneath the cells should be taken into account in the design.

NOTE Guidance on the development of pore pressures during pile installation is given in Failure of foundations and slopes on layered deposits in relation to site investigation practice [9].

Actions on the deck increase the resistance of the structure to sliding and overturning, but favourable variable actions should not be included in stability calculations.

13.1.6 Earth pressure measurements

It should be determined whether additional verification of the development of earth pressure on retaining structures is required. If such verification is required, the earth pressures should be measured by means of either pressure cells interposed between the soil and the face of the retaining structure, or load cells mounted on components such as anchors, struts and shores.

13.2 Sheet piles

*COMMENTARY ON **13.2***

Sheet piling may be used in all types of temporary works and permanent structures, including cofferdams, retaining walls, river frontages, quays, wharves, dock and harbour works, permanent foundations, land reclamation and sea defence works.

13.2.1 General

Where appropriate, low vibration, silent piling or water jetting methods should be used to install sheet piles, thereby reducing noise and marine life disruption. Water jetting should be stopped before the final penetration/embedment level, so as not to soften any clay that might occur at the wall toe.

13.2.2 Steel sheet piles

*COMMENTARY ON **13.2.2***

Piles are available in various shapes. The stiffer sections are capable of being driven to a considerable depth in a wide range of ground conditions and, when necessary, into rock. Driving can be very difficult in rock, and is used only in exceptional circumstances. Silent and vibration-free installation of sheet piles is widely available, and is suitable for use where pile installation using impact or vibratory techniques is expected to result in disturbance of the ground in the vicinity of the site. The interlocks between adjacent piles are relatively watertight, but an interlock sealant might be advantageous when water exclusion is desirable. Early discussions with specialist engineers are recommended.

Steel sheet piling should be designed in accordance with BS 6349-2, BS EN 1993-5:2007 and BS EN 1997, and installed in accordance with BS EN 12063:1999. Corrosion mitigation and protection should be in accordance with BS 6349-1-4.

The type of pile to be used should be selected from the following options, on the basis of section type, strength, driveability and durability, taking into account possible environmental impacts:

a) U-section;

b) Z-section;

c) straight web piles;

d) H- or I-sections; and

e) tubular sections.

NOTE 1 Types a) to c) are rolled sections which are manufactured to interlock together, but types d) and e) require the introduction of interlocking devices to form a continuous structure. It is also possible to combine sheet piles with structural sections or tubes to create a wall with enhanced stiffness and bending resistance. Such systems are referred to as high modulus or combined walls. The difference between these two wall forms is that a high modulus wall comprises interlocking steel elements that have the same geometry, whereas a combined wall is composed of primary and secondary elements.

For combined (combi) walls, the primary elements should be steel tubular piles, I-sections or built-up box types, spaced uniformly along the length of the wall. The secondary elements should normally be steel sheet piles of various types installed in the spaces between the primary elements and connected to them by interlocks.

Type c) sections are designed to act primarily in tension across the webs and should be used in cellular configurations.

NOTE 2 Cross-sections of the various types of sheet pile in common use are shown in BS EN 1993-5:2007.

NOTE 3 Steel sheet piling can be rolled in lengths up to approximately 35 m, though handling facilities do not always permit the use of such long lengths. This is especially the case with straight-web piling. Where it is necessary to increase the pile length during driving, site-welded joints can be used.

13.2.3 Concrete sheet piles

*COMMENTARY ON **13.2.3***

Reinforced concrete sheet piles are used in retaining walls in canal banks, where they are quite short and therefore of acceptable thickness. The height of any earth-retaining structure is limited by the depth to which it is possible to drive the piles without breakage in order to achieve the required passive resistance to earth pressure. It is difficult to achieve a tight interlock because of warping and shrinkage in individual piles. In addition, as the length of concrete piles increases, the thickness of the piles, even if they are pre-stressed, becomes excessive, making them very heavy and therefore complex to manufacture and handle. Their use in UK conditions is generally regarded as uneconomical, but they remain in use in developing countries in structures of modest size.

Steel shoes are not normally required on the toes of concrete sheet piles that are driven through soft or loose soils into dense sands and gravels or firm to stiff clays. A blunt pointed end is all that is required to achieve the desired penetration in these soils, and if the blunt point is cast on a longitudinal rake, it assists in the close driving of adjacent piles.

Concrete sheet piling should be designed in accordance with BS 6349-2, BS EN 1992 and BS EN 1997.

The following aspects of concrete sheet piles should be in accordance with BS 6349-1-4:

a) concrete mixes;

b) cover to reinforcement;

c) taking into account aggressive conditions; and

d) the use of pre-stressed concrete.

13.2.4 Timber sheet piles

Timber sheet piles should be used only for low earth-retaining structures and beach groynes, for replacing damaged sheet piles in existing timber structures, or for temporary works such as cofferdams.

NOTE 1 Interlock can be achieved by tongue and groove joints or by lapping the joints of a double row of planks. Typical details are shown in BS EN 12063:1999, Figure F.1.

Timber sheet piling should be designed in accordance with BS 6349-2, BS EN 1995 and BS EN 1997, and installed in accordance with BS EN 12063:1999.

Materials for timber sheet piles should be selected in accordance with BS 6349-1-4.

NOTE 2 Timber sheet piles are often fitted with steel shoes to prevent splitting of the toe and to assist in driving. Cutting of the toe on a longitudinal rake assists in close driving.

13.3 Diaphragm walls and bored pile walls

*COMMENTARY ON **13.3***

Diaphragm walls are formed in trenches, which are stabilized during excavation by the introduction of a support fluid in the form of bentonite slurry or a manufactured polymer. This fluid applies pressure to the trench walls, thereby obviating the need for support by timbering or sheet piling and allowing construction to considerable depth. The wall is formed by pouring concrete through a tremie pipe, displacing the slurry, which is then reused or discarded.

Bored piles, both contiguous and secant, can be used to form retaining walls, including quay walls. Supporting fluid and casings might be needed to ensure the stability of the pile bores.

13.3.1 General

Diaphragm walls should be designed in accordance with BS 6349-2, BS EN 1992 and BS EN 1997, and constructed in accordance with BS EN 1538.

Bored pile walls should be designed in accordance with BS 6349-2, BS EN 1992 and BS EN 1997, and constructed in accordance with BS EN 1536.

Provision of durability for both diaphragm walls and bored pile walls should be in accordance with BS 6349-1-4.

NOTE Foundations in weak ground for heavy shoreside structures, such as fixed cranes or silos, can be constructed on rectangular or cruciform trenches excavated under support fluid. Further information on the various structural forms that are in common use is given in BS 6349-2.

In all cases, the excavation should be made from the ground surface above the highest level of the groundwater table. Guide walls should be constructed to retain the fluid in permeable ground above the water table and to maintain vertical and horizontal alignment of the concrete substructure. Where quay walls and similar structures are sited on the foreshore or in tidal waters, fill should be placed to form a temporary working surface above the tidal height and to accommodate the guide walls. The filled areas should be protected from erosion on the seaward side by dumping rock fill or by temporary sheet piling.

The potential effect of tidal variation on the casting of concrete should be taken into account in the design.

Keyed joints should be provided between adjacent panels or units, at sufficient intervals to enable uniformity in distribution of earth pressure and earth resistance over the full length and depth of the structure.

13.3.2 Design of excavations supported by fluid

NOTE 1 The general principles that govern the stability of slurry-supported excavations are set out in BS EN 1538. It is assumed that such excavations are formed to enable the construction of diaphragm wall panels.

A head of support fluid should be maintained in the trench above the level of the groundwater table, with a sufficient margin of safety to allow for a rise in groundwater level at the time of high tides or surges.

Allowance should also be made for temporary unplanned lowering of the slurry level due to overbreak in the excavation, or loss of fluid to seepage, either through the soil or through interstices in open gravel or fill material.

The length-to-width ratio of the panels should be designed to utilize the arching action of the soil surrounding the excavated trench. The dimensions of the panels should also take into account the capacity of the available concreting plant, as the concrete has to be placed in a continuous pour.

NOTE 2 In addition to the ratio of the length-to-width of the panel, the head of slurry above groundwater, the nature of the ground itself and the dimensions of the excavating equipment in relation to the length and width of the panel govern the stability of the trench excavation.

Periodic checks should be made on the density and other properties of the fluid, as required by BS EN 1538, to verify that the design assumptions remain valid.

NOTE 3 Guidance on the constituent materials and methods of mixing and testing of the support fluid is given in BS EN 1538 and FPS publication Bentonite support fluids in civil engineering [10].

13.4 Function and location of anchorages

*COMMENTARY ON **13.4***

Anchorage systems are used in maritime structures to restrain the structures against movement caused by earth pressure, hydrostatic pressure, wave-impact forces, berthing-impact forces and mooring-rope pull. Actions due to wind, earthquake, thermal stresses and pipe anchor forces might also need to be taken into account.

Anchorages should be designed in accordance with BS 6349-2, BS 8081 and BS EN 1997-1:2004.

Steel anchors and walings should be designed in accordance with BS EN 1993-5. Concrete elements should be designed in accordance with BS EN 1992. Durability of both steel and concrete elements should be in accordance with BS 6349-1-4.

Allowable stresses in tendon anchorages should be determined, and installation and stressing carried out, in accordance with BS 8081.

In selecting soil parameters for the design of injected tendon anchors to resist vertical uplift, account should be taken of the effects of cyclic loading on the soil.

NOTE Cyclic loading can be caused by variations in hydrostatic pressure beneath an anchored floor slab due to variation in tidal levels.

The design should allow sufficient standoff between the main wall and the anchorage to prevent the active wedge of the main wall from interacting with the passive wedge of the anchorage.

Anchorages should be located at sufficient depth to allow underground services to run above them and to limit the effect of local surcharge from cranes or stored materials.

14 Bearing piles

*COMMENTARY ON CLAUSE **14***

Piles can be categorized into three types:

- *large displacement, typically solid precast concrete, closed ended steel tubes and boxes;*
- *small displacement, open ended steel tubes that are not plugged, H piles; or*
- *non-displacement, bored piles.*

General requirements regarding the design of piles can be found in BS EN 1997-1:2004. Detailed guidance on pile design and construction is given in Pile design and construction practice *[11].*

14.1 Selection of bearing piles

The following recommendations should be met when selecting the most suitable pile type.

a) The availability of the pile section should be taken into account when selecting a type and size of pile.

b) Requirements due to the location and type of structure should be determined.

NOTE 1 For a structure built over water, either tubular, box or H-section steel piles, tubular precast or pre-stressed concrete piles are likely to be most suitable. Solid precast or pre-stressed concrete piles can be used, but the practicalities of handling and lifting need to be assessed, as such piles might become too heavy to handle effectively depending on the depth of water. Timber piles might be suitable, but are limited by considerations of length and cross-section. In exposed maritime conditions, steel tube or box piles are preferable to H-sections because of the smaller drag forces from waves and currents. Large diameter steel tube or box piles are also an effective solution to the problem of dealing with impact forces from waves and berthing ships.

NOTE 2 Piles for inshore or land-based structures can be selected from any of the three bearing pile categories, namely large displacement, small displacement or non-displacement. Of these, bored and cast-in-place piles are often the most practicable, and large diameter bored piles, sometimes with enlarged bases, are capable of carrying very high loads. Bored piles are often specified in environments where ground heave, noise and vibration are to be avoided, but ingress of water to the pile excavation can result in serious structural defects.

c) The durability requirements for the piles should be determined.

NOTE 3 Steel-bearing piles in normal undisturbed soil conditions usually have an adequate resistance to corrosion. The portion of the piles above the seabed in maritime structures, in disturbed ground or in corrosive soils can be provided with a sacrificial thickness which takes into account anticipated corrosion losses over the life of the structure, cathodic protection or a coating. Further information is provided in BS 6349-1-4.

NOTE 4 The durability performance of in situ concrete bored piles is unlikely to be as good as an equivalent pre-formed concrete structure, and the exposure conditions need to be taken into account before using bored and cast-in-place piles, including diaphragm walls where the face of the pile is subsequently directly exposed to the marine/estuarine environment (e.g. becomes the exposed quay wall). Durability of in-situ concrete piled elements when used in a marine environment may be improved by increasing both the minimum cover (c_{min}) and the tolerance for deviation (Δ_c). In the absence of evidence that a smaller value can be routinely achieved, a Δ_c value of 50 mm is normally considered to be appropriate. Further information is provided in BS 6349-1-4.

NOTE 5 Precast concrete piles are not expected to suffer corrosion in saline water below the mean high water, and well compacted concrete can normally withstand attack from quite high concentrations of sulfates in soils and groundwater. Cast-in-place concrete piles are not so resistant to aggressive substances because of difficulties in ensuring complete compaction of the concrete, but protection can be provided against attack by, for example, placing the concrete in permanent linings of coated light gauge metal or plastics.

NOTE 6 Timber piles are liable to decay above groundwater level and, in some situations, they are damaged below water level by marine borers (see BS 6349-1-4). Piles can also be damaged by abrasion (see BS 6349-1-4) and protective measures are necessary, depending on the particular circumstances.

d) The requirements due to ground conditions should be determined.

NOTE 7 Silent and vibration-free installation techniques are available for steel piles, including large diameter tubes. It is also possible to install steel piles into rock using these methods.

NOTE 8 Auger bored piles are suitable for firm to stiff cohesive soils, but augering is more difficult in very soft clays or in loose or water-bearing granular soils, where the loss of support and instability of the side walls is a significant risk. In these cases, driven or driven-and-cast-in-situ piles, casings, support fluid or continuous flight auger (CFA)-type piles could be used as alternatives.

NOTE 9 Concrete driven and driven-and-cast-in-situ piles are not suitable for ground containing boulders, nor in soils where ground heave is to be avoided.

NOTE 10 Driven and cast-in-situ concrete piles, which use a retractable tube, are not suitable for very deep penetrations, because of the limitations of jointing and retrieving the driving tube.

NOTE 11 Thin-walled steel piles are prone to tearing when being driven through soils containing boulders. For hard driving conditions in boulder clays or gravely soils, a thick-walled steel tube pile or a steel H-section can withstand heavier driving than a precast or pre-stressed concrete pile of solid or hollow section.

14.2 Steel bearing piles

*COMMENTARY ON **14.2***

Types of steel-bearing piles include tubes, box sections and H-sections, which are supplied in a wide range of sizes and thicknesses. They can be readily cut down or extended where the level of the bearing stratum varies and the head of a pile that buckles during driving can be cut down and re-trimmed for further driving. They have a good resilience and high resistance to buckling and bending forces. They are light to handle compared with precast concrete piles and this can be particularly advantageous in maritime construction over water; their high strength to weight ratio facilitates their use as raking piles.

Guidance on the strength and durability of steel elements in a marine environment is given in BS 6349-1-4.

The design of steel bearing piles should be in accordance with BS EN 1993-5:2007.

14.3 Concrete bearing piles

*COMMENTARY ON **14.3***

Precast concrete piles are often used in maritime structures because of the simplicity of manufacture and their suitability in a wide range of applications. Their main disadvantage is the difficulty of extending them when necessary.

Particularly in maritime works, pre-stressed concrete piles are normally preferable to ordinary reinforced concrete piles because they are able to tolerate larger bending loads and are more durable. Their higher bending strength permits manufacture and handling in a wider range of lengths and greater resistance to lateral forces in operation. They have a greater resistance to tensile stresses caused by uplift. A further advantage of pre-stressed concrete is that any cracks that occur during handling and driving are likely to close up. This, combined with the high quality concrete necessary for pre-stressing, gives the pre-stressed pile increased durability in the maritime environment. However, pre-stressed piles are less resistant to impact from harbour craft or lighters than reinforced concrete piles.

Concrete used for piles should be in accordance with BS 6349-1-4.

The working stresses in the concrete during lifting, handling and pitching of precast piles should not exceed those given in BS EN 1992. High stresses, which can exceed the handling stresses, can occur during driving, and the serviceability limit of cracking should be taken into account. The cover to reinforcement for piles exposed directly to seawater should be in accordance with BS 6349-1-4.

14.4 Timber bearing piles

Timber piles should be designed in accordance with BS EN 1995. When calculating the working stress on a pile, allowance should be made for bending stresses due to eccentric and lateral loading, and to eccentricity caused by deviations in the straightness and inclination of a pile.

Allowance should also be made for reductions in the cross-sectional area due to drilling or notching and to the taper on a round log.

15 Slopes

15.1 Design considerations for slopes and embankments

NOTE 1 Guidance on the methods of analysing the stability of slopes is given in BS EN 1997-1:2004, BS EN 14475 and BS 8006-1.

Generally, both short-term and long-term stability of dredged slopes and embankments should be taken into account in the design. Analyses should be made in terms of effective stresses in both cohesionless and cohesive soils, unless it can be confidently assured that only an undrained analysis will be reliable.

Stability analyses in terms of total stresses using undrained shear strength parameters should be limited to cases where only short-term stability needs to be assessed. The risks of a loss of undrained conditions should be assessed, and if there is any doubt, or the consequence of failure is unacceptable and cannot be managed, analysis should also be undertaken using an effective stress approach.

NOTE 2 The assessment of the time for which short-term conditions apply, and for which undrained strength might be used, is both difficult and entails risks in terms of the time that undrained conditions might persist.

In cuttings, account should be taken of the effects of removal of overburden pressure due to dredging or above-water excavation to the design formation level.

When constructing embankments on cohesive soils, the gain in shear strength due to consolidation of the soil under the imposed actions should be taken into account and the rate of placing the embankment fill should be controlled.

NOTE 3 This is to allow time for the pore pressures to dissipate, which results in strengthening of the soil.

Geotechnical instrumentation (including piezometers, inclinometers and settlement gauges) should be installed to monitor and control rates of consolidation, movement and embankment construction.

The possibility of local slips or falls occurring on the face of the slope should be taken into account when preparing designs for the alignment and profile of a slope that has been formed by dredging or by placing material to form an embankment. The overall stability against the various modes of failure described in **15.3** should also be taken into account.

NOTE 4 Local slips or falls can occur due to the presence of random pockets of weak or erodible soils, or thin layers of weak or shattered rocks. In the case of underwater slopes, occurrences of local instability cannot be readily detected, and remedial works are limited to placement of material to form a stable profile or to surface protection by mattresses.

A conservative approach should be adopted in the selection of a profile for the permanently submerged area.

*NOTE 5 Local instability can be detected in the areas above high water and in inter-tidal zones. If local instability is found, appropriate remedial action, as described in **15.10** and **15.11**, can be taken.*

15.2 Slope stability and protection – Environmental factors

15.2.1 Design principles

NOTE 1 The general principles for design of slopes are set out in BS 8006-1.

When assessing the stability of slopes in the long term, the cohesion in terms of effective stress of all soil types should be taken as zero, unless reliable evidence including the performance of existing slopes in the same or similar material shows otherwise. Account should be taken of the effect of tidal variations on pore pressures in the soil behind the slope.

NOTE 2 In some analytical models, setting cohesion to zero can be problematic and a very small value can be adopted, e.g. 0.1 kPa.

In the case of partly submerged slopes, the effects of variation in level of the groundwater from landward sources should also be taken into account.

NOTE 3 The fabric of the soil, which is the presence of fissures, layers or laminations of permeable soil interbedded with impermeable soils, has an effect on variations of pore pressure.

Variations of water level due to wave action on a slope can also cause variations in pore pressure behind the slope, and the depth of soil affected by these rapid fluctuations in pore pressure should be assessed. In the case of submarine slopes in deep water, numerical analyses should be used to determine the wave-induced pore pressures and effective stresses.

NOTE 4 Guidance on the response of a porous elastic bed to water waves is given in Wave-induced pressure, stress and strain in sand beds [12].

NOTE 5 An increase in pore pressure in the soil behind a slope can be caused by constructional operations such as displacement of the soil by pile driving, or by dumping materials on to or beyond the crest of a slope.

15.2.2 Changes in slope profile

*COMMENTARY ON **15.2.2***

*Steepening can be the result of erosion of the toe of a slope by tidal or river currents, or the wash from ships (see **15.9**). Wave action can also cause changes in slope profile due to the effects of undercutting and deposition of loose material by upwards surge of waves.*

Steepening of the upper part of the slope can result from material being dumped at the crest, or, alternatively, soil being deposited by the natural processes of accretion.

The possibility of instability due to the gradient of a slope becoming steeper should be taken into account in the design.

15.2.3 Other effects

In areas of known seismic activity, the effects of earthquakes on the stability of slopes in soils sensitive to reduction in shear strength by disturbance should be reviewed, assessed and taken into account in the design where appropriate. Pre- and post-earthquake properties of clay should be investigated by means of advanced cyclic testing.

Sensitive clay should be specifically investigated, and remoulded and degraded values for soil strength parameters selected for design.

Soil susceptible to liquefaction should be identified, replaced or remediated to accommodate design requirements.

NOTE 1 See BS EN 1998 and BS 8006-1 for further information on design for earthquakes.

NOTE 2 In tidal waters, blocks of ice adhering to the soil at the water line can cause degradation of a slope on the falling tide.

15.3 Modes of failure

NOTE Modes of failure of unreinforced slopes are described in BS 6031.

15.3.1 Instrumentation to warn against instability

Where uncertainty exists over slope stability, or the consequence of failure will be significant, instrumentation should be installed to provide early warning of any movements, thereby allowing mitigation and remedial measures to be undertaken before failure occurs. The need for monitoring and its scope and nature should be developed as part of the design, or as part of the assessment or preliminary studies if an existing slope is being assessed.

15.3.2 Monitoring surface and sub-surface movements

Monitoring of ground surface and sub-surface movement in both horizontal and vertical planes should be carried out by appropriate field survey methods. The particular methods to be used should be determined according to the scale of the slopes, degree of accuracy required and the general access and logistical constraints.

15.4 Safety and risks of failure

Account should be taken of the consequences of underwater slope failure on maritime structures.

NOTE 1 A slip caused by dredging for a berth can result in collapse of a jetty installation or quay wall. Similarly, blockage of a dredged channel can result in closure of a port.

NOTE 2 Mobilization of equipment and materials for remedial works involving dredging and restoration of profiles by dumping can be slow, and will only be applied to the small volume of material involved in a slip.

Slope design should be carried out in accordance with BS EN 1997-1:2004, applying the partial factors described in NA to BS EN 1997-1:2004 to actions and material properties (see Note 3).

NOTE 3 BS 6031 indicates that designers need to ensure that the risk and consequences of failure have been adequately considered during design, and points out that BS EN 1990 and BS EN 1997-1:2004 permit the variation of the relevant partial factors where the consequence of failure is either higher or lower than normal.

Where embankments are constructed to form breakwaters on weak soils, the consequence of shear failure and subsidence of the embankment followed by overtopping by waves at times of storms should be assessed in relation to the effects on harbour installations protected by the breakwater. The results of the assessment should be taken into account in the design.

15.5 Slope profile

*NOTE 1 The required slope angles are obtained by the analytical methods referred to in **15.1** or by empirical methods. It might be desirable to choose angles flatter than those that are required as a minimum, in order to avoid frequent maintenance dredging, or to meet aesthetic criteria for above-water slopes.*

Where underwater slopes are formed in erodible loose sands and silts, the design profile should take into account the likelihood of local steepening caused by erosion. The required slope profile should be established from local knowledge and experience based on the geometry of the underwater excavations and the presence of obstructions to flow, such as piles, moored ships, quays, etc. (see **15.1**). The need for measures to control erosion should be assessed and taken into account in the design.

Where slopes are formed in layers of soil or rock of significantly differing characteristics, the slope angles should be appropriate for the engineering behaviour of each individual formation.

NOTE 2 Slope angles can be varied in previously water-bearing soils by adopting a steep slope approaching the angle of repose of the soil located above the highest groundwater level. Alternatively, the highest level affected by tides or uprush of waves can be used as a break point, with a steeper slope being adopted above the break point, and a flatter slope being adopted in the zone affected by varying tidal levels and wave action.

Where steep upper slopes are adopted, account should be taken of the overall stability of the earthworks. Where necessary, a berm should be introduced between the two differing slope profiles.

In above-water slopes, a berm should be provided at the level of the interface between an impervious formation and an overlying water-bearing soil. An open channel or piped drain should be provided on the berm to collect seepage from the upper slope. The surface of the berm should be sloped back to prevent water spilling down the lower slope at times of heavy surface water run-off.

A berm or other space should be provided at the toe of rock or steep earth cliffs, to trap boulders or falls of soil from the face of the cliff where such falls would cause danger to persons or property.

NOTE 3 Guidance on the required width of the berm or debris trap is given in Transportation Research Board Report 29 [13].

If insufficient space is available for the calculated width, a suitable fence or wall should be constructed along the outer margin.

NOTE 4 The profile of the slope required for the face of a breakwater or training wall is governed by two considerations. The first is the factor of safety against failure in the underlying soil and of differential water pressure within and on each side of the embankment. The second is the need to avoid erosion and overtopping of the structure by wave action. Guidance on the design of breakwaters is given in BS 6349-7 and CIRIA Report C683 [14].

15.6 The effects of construction procedure

The procedure adopted for dredging of berths and channels should not be such as to endanger the stability of slopes. In particular, the usual practice of dredging in a series of vertically sided steps allowing the slope to slump to its natural angle of repose should not be followed if it results in a general weakening of the soil behind the slope such that the design profile cannot be maintained.

Where fill is placed on an existing slope for the purpose of reclaiming ground from the foreshore or behind a wharf or quay, geometric limits on the fill should be established and adopted to avoid excessive surcharge by placement of fill on or beyond the crest of the slope that could lead to failure or unacceptable distress of the slope.

Embankments and training walls constructed by placement of fill on to weak soils below the seabed should be undertaken only in situations where a period of time can be allowed between successive stages of filling for the purpose of dissipating excess pore pressure. In such cases, the placing of the underwater fill should be undertaken in such a way as to minimize local surcharge.

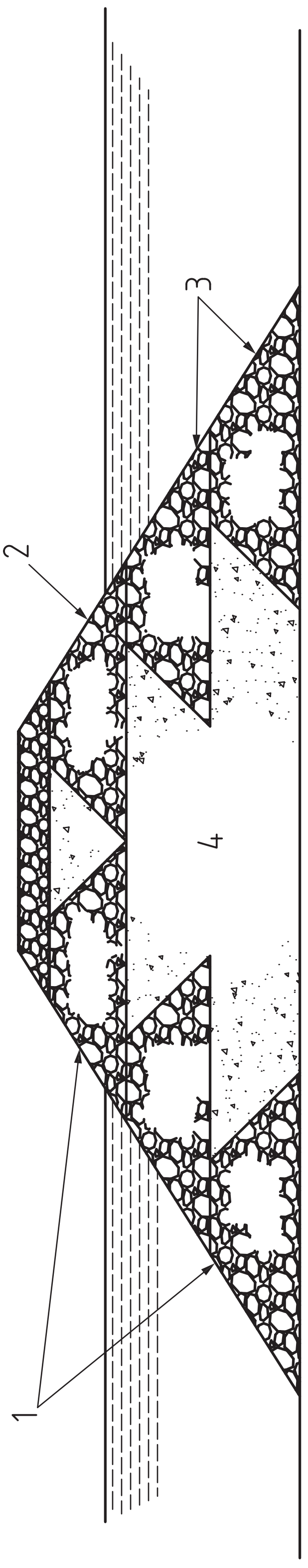
NOTE It might be necessary to place the main mass or core of the fill between outer embankments designed to retain the filling and to protect the core material against wave attack and erosion (see Figure 10).

Where the outer protective embankments are placed in successive stages as shown in Figure 10, the height of each stage should be controlled to prevent the formation of mud waves that could become trapped within the core material and cause instability of the embankment.

Any proposals for constructing embankments by end tipping from the shore should take account of the consequences of surcharge due to dumping material on to a steep slope, and to erosion of the seabed soil beneath the advancing toe of the slope.

Where dredging or reclamation is undertaken in weak, unstable soils, the effects of rapid pore pressure increase due to blasting or pile driving for associated works should be taken into account.

Figure 10 — *Embankment built in stages with core material protected by dumped stone*



Key

1 Large stone on outer face for protection against wave action and erosion

2 Stone placed above water level by end tipping

3 Stone dumped below water level from barge

4 Core material

Suitable drainage measures should be taken to prevent accumulation of surface water or diversion of subsoil water onto areas at or near the crest of above-water slopes, if the resulting rise in pore water pressure would have an adverse effect on the stability of partly completed or completed earthworks.

15.7 Drainage

An assessment should be carried out in accordance with BS 6031 to determine whether pre-earthworks drainage is required.

NOTE Drainage might be required at the top of a cutting slope to intercept surface water flowing towards the excavation preventing the water from discharging down the slope. This drainage can take the form of open channels, ditches or piped drains.

The gradient of the drains should, as far as possible, be the optimum for the particular type, and should not be flatter than 1 in 300 unless the drains have the main purpose of providing storage capacity for run-off, when flatter gradients might be acceptable.

The capacity of the drains should be determined by the nature of the soil, the contour of the ground (e.g. whether sidelong or otherwise), and the existence of springs, agricultural drains or water channels which can be interfered with by the cutting.

15.8 Monitoring stability

*COMMENTARY ON **15.8***

Where experience or stability analyses give reasonable assurance of stability, no special measures are required for monitoring stability. It is good practice, however, to make periodic inspections during construction and in the early months following completion, when grassing or other methods used to control erosion in above-water slopes are attaining the desired conditions of stable growth.

Careful inspection of steeply cut temporary slopes for foundation excavations or trenches is required to verify safe working conditions for operatives and to avoid damage to partly constructed works or existing structures adjacent to the excavation.

Instrumentation that is suitable for installation in earthworks above high-water mark, including apparatus for pore water pressure observation, is unlikely to be practicable for underwater slopes, particularly in areas where access for vessels is required. In these areas, monitoring might need to be limited to detecting deformations by taking soundings or making observations on beacon poles.

The design or assessment of a new or existing slope should establish the need for, and scope and nature of, any monitoring. This should include an assessment of the need for periodic inspections. Periodic inspections should be adopted, unless it can be shown that these can be replaced by remote sensing and instrument monitoring.

Where it has been determined that monitoring should be undertaken, a monitoring plan should be developed and implemented. This should also establish how monitoring will be reported and what actions should be undertaken, depending on the results of monitoring.

Where required as part of the monitoring, periodic inspections should be undertaken, including checking for the following conditions:

a) deformation – settlements in the upper part of the slope and bulging towards the toe can indicate incipient failure by a rotational shear slide (see BS 6031);

b) cracking – a series of cracks parallel to the crest and parallel ridging towards the toe can indicate incipient translation failure on above-water slopes (see BS 6031). Hexagonal or random pattern cracking indicates drying shrinkage of cohesive soils;

c) fissuring – opening of joints and fissures in a rock slope can indicate incipient translational failure or toppling failure (see BS 6031);

d) seepage – water carrying soil particles seeping from a slope is indicative of internal or seepage erosion (see BS 6031);

e) gullying – channels eroded on a slope face indicate the need for protection against surface erosion; and

f) scour – erosion at the toe of the slope that could undermine slope stability, especially close to obstructions or vessel propellers.

Inspections should be carried out after storms, periods of heavy rain, snow or severe frost. Clay slopes should be inspected during or immediately after rainfall or wave attack following a period of dry weather, to assess the effects of water entering surface cracks.

NOTE Inspection of the position and inclination of pegs or beacon poles driven into a slope is a simple means of detecting gross deformations.

15.9 Slope protection

15.9.1 General

Measures should be taken to protect the overall stability of the slope against the modes of failure referred to in **15.3**, and to protect the surfaces of slopes against erosion by currents, waves, and surface and subsoil water.

15.9.2 Underwater slopes

Account should be taken of the effects of vortex formation on the geometry of slopes formed by dredging schemes for berths and navigable channels or by reclamation from the foreshore. Sharply projecting spurs or re-entrant slopes should be avoided where practicable.

The effect of scour on underwater slopes should be taken into account in the design.

NOTE Conditions giving rise to severe scour can occur in the presence of moored ships, as a result of restriction in the area of flow alongside and beneath the hull and vortex formation at the bow or stern. Moving ships can cause wave action due to the bow wave or propeller wash. Bow thrusters can pose particular problems. Deep scour can occur around obstructions to flow, such as piles or the protecting corners of quay walls. Protection of the seabed in the form of dumped rock or prefabricated mattresses might be needed in these areas if the currents are strong. Guidance on the design of anti-scour aprons is given in BS 6349-7.

15.9.3 Above-water and partly submerged slopes

*COMMENTARY ON **15.9.3***

Erosion and instability of slopes within the influence of the rise and fall of tides and wave action are caused by:

- *currents with associated vortex formation as described in **15.9.2**;*
- *scour by waves and wash from ships;*
- *movement of soil particles due to the egress of water on a falling tide or retreat of waves;*
- *egress of subsoil water;*
- *action of winds; and*
- *action of surface water.*

Where slope surfaces are protected by a layer of rock or precast concrete blocks on the slope, the effects of a varying water pressure on each side of the protective layer should be taken into account in the design.

NOTE 1 Protection against wave action and severe scouring conditions might require the provision of large blocks of rock or precast concrete shapes to absorb and dissipate wave energy. Such cases require the provision of means *to prevent the flow of finer soil particles into the interstices of the large units. These means could be, for example, a blanket of filter material interposed between the rock or precast concrete armouring and the soil forming the slope.*

If a filter is included, it should be designed to prevent the movement of the finest particles from the soil flowing out of the slope on the falling tide or on retreat of waves. One of the two following methods should be used, with option a) being the preferred method.

a) As the interstices between the blocks forming the armouring to the slope are large, the filter may consist of several granular layers graded from coarse to fine material. Each layer should be designed such that the finest filter material does not move into the adjacent coarser filter layer under the influence of flowing water.

b) Alternatively, where option a) is not practicable, the filter may consist of other materials (e.g. geosynthetic mesh or brushwood mats) protected by a layer of crushed and graded stone and overlaid with stone or precast concrete block armouring (see Figure 11).

NOTE 2 Guidance on the design of single or multi-layer filters is given in CIRIA Report C683 [14] and Soil mechanics in engineering practice [15].

NOTE 3 Ground surfaces beyond the influence of waves or tidal water movements can be protected against the erosive effects of winds and surface water by blanketing with stone, paving with concrete, covering with bituminous materials or planting vegetation.

15.10 Maintenance of earthworks

NOTE 1 In general it is normal to expect that earthworks will be subject to little planned maintenance.

*NOTE 2 Guidance on inspection and maintenance of above-water slopes is given in **15.9.3**.*

NOTE 3 In the zone subject to wave attack, water that cannot drain immediately has a disruptive effect on the structure. The design of any filter layers, for maintenance and during construction, therefore demands careful attention.

A maintenance and inspection regime should be developed as part of the design. It should be reported in the design documentation. This should take into account the requirements of the owner of the infrastructure in which the earthworks are situated, and any existing maintenance and inspection requirements.

In general, the design should take into account the maintenance regime where it is established, but in any case, it should be such as to minimize the requirement for maintenance.

The design of underwater slopes in areas of accretion should take into account the difficulty of carrying out maintenance, other than dredging. However, periodic soundings should be taken to determine the trend of changes in seabed levels following the construction of maritime works and to enable the appropriate remedial action to be taken before major instability develops.

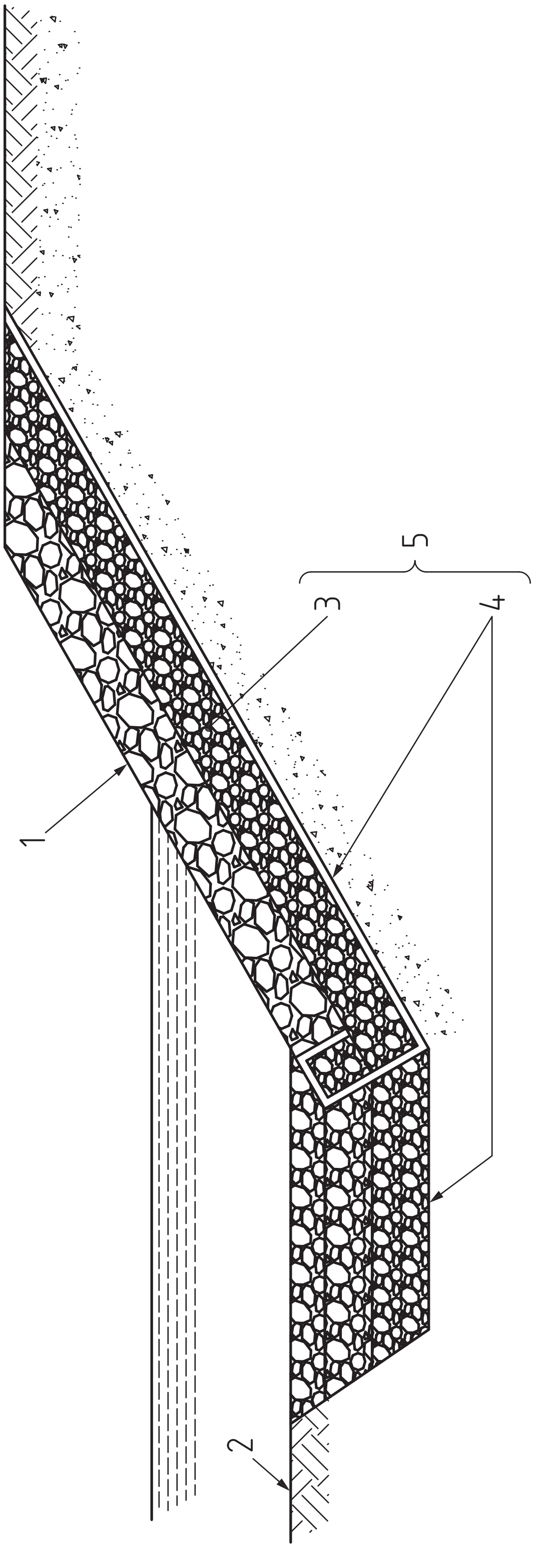
15.11 Remedial works

*NOTE 1 Guidance on treatment to remedy failure of above-water slopes is given in **15.9.3**.*

In the case of submerged slopes, the selection of the type of remedial treatment to be adopted should be based on the consequences of failure and the effects of remedial work on navigable depths in berths and channels. Thus, dumping rock fill at the toe of a partly submerged slope to apply a counterweight against a rotational shear slide should not be adopted if this prevents access to a berth. In such a case, an alternative method should be adopted, e.g. restraining the slip by anchored sheet piling.

NOTE 2 Toe weighting can be used as a method for temporarily improving stability if the slide endangers important shoreside works.

Figure 11 — *Slope protection by rock or concrete armouring backed by filter layer*



Key

1 Large stone or concrete block armouring

2 Seabed

3 Crushed and graded stone

4 Geogrid

5 Alternative: multi-layer graded stone or gravel filter

NOTE 3 Where scouring of seabed material from the toe of a slope causes a rotational shear slide, it is usually feasible to dump rock into the scour hole without adverse effect on navigation.

The desirability and practicability of laying mattresses on the seabed to control further erosion around the affected area should be assessed and taken into account in the design.

NOTE 4 Adjustment of the slope profile by removing material at the crest of the slipped area might be a feasible remedial treatment for submerged or partly submerged slopes.

Groynes or training walls should be provided, where appropriate, to control scour and encourage accretion in areas where there is a trend towards increased scour that could result in instability of slopes.

16 Verification

Verification of the construction works should be carried out as the final stage of the design process.

The basis for the verification of the works should be established in the design. This should include verifying the assumptions made in the design and that the performance of the works matches the predictions made in the design.

A verification plan should be established prior to construction. This plan should address both individual elements of the works, e.g. fill materials, and the overall performance of the work, e.g. settlement of reclamation. It should also include the form of report required and the reporting intervals, both of which should be appropriate to the scale and nature of the work.

Verification should cover all aspects of the works, and should take into account the scale and importance of the works.

NOTE A verification plan might typically cover:

- *quality of fill material placed;*
- *settlement of reclamation;*
- *settlement and deflections of structures;*
- *extent of dredging;*
- *performance of dewatering, both temporary and permanent; and*
- *environmental impacts.*

Where possible, verification should be carried out and reported at the planned intervals as the works proceed.

If the performance of any aspects of the works, based on the verification process, falls outside the limits set as part of the design, the need for a remedial plan or further monitoring should be established. Where remedial works are needed, these should be subject to the same verification process as the original work.

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