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Maritime works

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

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Contents

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Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 64, 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 September 2012. It was prepared by Technical Committee CB/502, *Maritime works*. A list of organizations represented on this committee can be obtained on request to its secretary.

Supersession

Together with BS 6349-1-1, BS 6349-1-2 and BS 6349-1-4, this part of BS 6349 supersedes BS 6349-1:2000, which will be withdrawn when all four of the new subparts have been published.

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*; [1)]
- Part 1-2: *General – Code of practice for assessment of actions*; [1)]
- Part 1-3: *General – Code of practice for geotechnical design*;
- Part 1-4: *General – Code of practice for materials*; [1)]
- 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 and BS EN 1997-2.

Information about this document

A full revision of BS 6349-1:2000 has been undertaken and the principal change is to split the document into four smaller parts:

- BS 6349-1-1: *Code of practice for planning and design for operations*;
- BS 6349-1-2: *Code of practice for assessment of actions*;
- BS 6349-1-3: *Code of practice for geotechnical design*;
- BS 6349-1-4: *Code of practice for materials*.

The principal change in respect of the geotechnical content is that the document has been edited to be compatible with relevant Eurocodes.

The new BS 6349-1-3 is split into two main sections covering site investigation and geotechnical design, with annexes containing informative text regarding site investigation and testing procedures and typical ground properties.

[1)] In preparation.

Use of this document

As a code of practice, this part of BS 6349 takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

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

Presentational conventions

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

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

Contractual and legal considerations

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

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

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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 site investigation and geotechnical design, and gives additional guidance on testing procedures and typical ground properties.

2 Normative references

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

BS 6031 *Code of practice for earthworks*

BS 6349-1:2000, *Maritime structures – Part 1: Code of practice for general criteria*

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

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

BS EN 1990, *Eurocode – Basis of structural design*

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

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

BS EN 1998 (all parts), *Eurocode 8 – Design of structures for earthquake resistance*

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*

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 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

3.1.2 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

3.1.3 highest astronomical tide (HAT)
highest level that can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions

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2) This standard also gives informative references to BS EN 1997-2.

3.1.4 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.

3.1.5 mean high water springs (MHWS)
average, over a long period of time, of the heights of two successive high waters at springs

3.1.6 mean low water springs (MLWS)
average, over a long period of time, of the heights of two successive low waters at springs

3.1.7 mean sea level (MSL)
average level of the sea surface over a long period

NOTE This is preferably 18.6 years (one cycle of the moon's nodes), or the average level that would exist in the absence of tides.

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

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

3.1.10 return period
period that, on average, separates two occurrences

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

3.2 Symbols

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

B_e	breadth of earth-retaining structure
c_u	undrained shear strength of soil
c'	effective cohesion
d_p	depth of pile toe from dredge level
d_t	depth of tension crack
H_R	retained height of structure
K_A	coefficient of active earth pressure
K_p	coefficient of passive earth resistance
P_A	total active force
P_P	total passive force
z	depth at which a calculation is to be made
γ	density of soil
γ_s	effective bulk weight density of saturated soil
γ_w	weight density of groundwater
δ_{max}	maximum angle of friction between soil and structure

φ_r	angle of soil shearing resistance
φ'	effective angle of soil shearing resistance

3.3 Abbreviations

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

DGPS	differential global positioning system
ELW	extreme low water
EHW	extreme high water
GPS	global positioning system
GWL	groundwater level
HAT	highest astronomic tide
LAT	lowest astronomic tide
MHWS	mean high water springs
MLWS	mean low water springs
RQD	rock quality designation
SCR	solid core recovery
SLS	serviceability limit state
SPT	standard penetration test
TCR	total core recovery
ULS	ultimate limit state

Section 2: Site investigation

4 General

A study of the surface and subsurface conditions at and near the site of proposed works should be carried out as an essential preliminary to the design of maritime structures. Much can be learnt from an early assessment of the basic geology of the area, with particular attention to existing morphological processes. This study should provide a basis for determining the extent of geotechnical investigation and laboratory testing required and aid in the interpretation and evaluation of the information obtained.

The study should include assessment of the characteristics of soil or rock formations, which can be retained by structures or provide their foundations, or which can be incorporated in or affected by earthworks in the form of dredging and reclamation. It 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 maritime environment.

NOTE 1 Site 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 2 Ground investigation practice is described in BS EN 1997-2. Many of the methods and tools widely employed in the investigation of ground on land can be adapted to marine applications. The geology of the coastal margins is almost invariably complex. The apparent savings that can result from carrying out a reduced ground investigation are in many cases outweighed by increased disruption of the works due to unforeseen ground conditions. Preliminary guidance is given in Annex A.

NOTE 3 Information on geophysical surveying techniques is provided in CIRIA Report C562 [1].

These studies all form part of a site investigation for the works, and the geological, geophysical and geotechnical aspects of the investigation should be carried out in accordance with Clause **5**, Clause **6** and Clause **7**. The collection of data for the site should be followed by the selection of design parameters (see Clause **12**) from which the behaviour of soils and rocks can be predicted. The next stage in the geotechnical design process is the calculation of earth pressures and earth resistance, which should then be used, with other actions, to check the adequacy of the proposed works.

*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 Clause **10**.*

*NOTE 5 Descriptions of particular methods of construction are not appropriate to this part of BS 6349, and the effects of constructional methods and procedures are normally referred to only where these could affect stability or design actions. However, because of its essentially geotechnical nature, the use of support fluids in excavations has been included and is described in **16.2.3**.*

Maritime structures will normally fall within geotechnical category 2 as defined in BS EN 1997-1, but in situations where complex or unusual engineering is involved, structures or parts of structures should be assumed to be in geotechnical category 3. The ground investigation requirements will normally be at least those indicated in BS EN 1997-2, 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 ground investigations

5.1 Existing data sources

The first stage of the site investigation should be a desk study of available published and unpublished information.

NOTE Sources of information relevant to maritime structures include:

- *Admiralty charts and handbooks (the Pilot series) [2];*
- *Ordnance Survey maps and old maps;*
- *meteorological data;*
- *national and local government records;*
- *geological maps and memoirs;*
- *aerial and satellite photographs;*
- *information on existing works in the locality.*

Admiralty charts and Ordnance Survey maps covering a long period of years should be studied for 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

A thorough visual examination of the site should be made. 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 can result from construction of the new works.

NOTE 1 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 the project and they can often be the most significant feature of the ground conditions affecting many maritime works.

NOTE 2 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 would therefore be very local, whereas they can occasionally be from long lengths of foreshore.

Organically formed deposits are frequently of significance. They are mainly calcareous, of which coral and calcite – excluding algae that cement detrital material – are two examples. All such materials are subject to leaching and therefore caverns can form. Particular attention should be given to such a possibility, especially where major structures such as breakwaters or dry docks are proposed. Sedimentary rocks formed by induration are also subject to cavernous formation, so a site investigation programme arranged to locate such features should be adopted in these types of soils.

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 can be difficult to work with.

5.3 Selection of sampling methods

The quality of a soil, rock or groundwater sample is influenced by the geological and hydrogeological conditions, the choice and execution of the drilling and/or the sampling method, handling, transport and storage of the samples. 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.

5.4 Layout of boreholes and trial excavations

The layout of exploratory boreholes, trial pits and shafts should cover the full longitudinal and lateral extent of the works. Boreholes for wharves and quay walls should be spaced at intervals along the waterside frontage of the structure. In a transverse direction, the boreholes should be sited to include exploration of the ground conditions for any anchorages on the landward side. The possible risks for instability due to a rotational slip should also be investigated.

NOTE 1 This encompasses the shoreside works and any dredging of the seabed for berths on the water side of the site (see Figure 1).

NOTE 2 General guidance on the spacing of boreholes is given in BS EN 1997-2.

Boreholes for jetties should be sited along the line of the berthing heads and mooring structures and along the line of the approach structures. At least one borehole should be close to the planned position of each major component of the structure and, in general, boreholes should not be more than 50 m apart.

For dry docks, the number and layout of boreholes cannot be defined with certainty before the investigation work is started, but ten equally spaced along the line of each dock wall and along the centre if dewatering works are envisaged should be the minimum. If variations in level, in thickness or in the properties of various layers of subsoil are revealed, the number of the boreholes should be increased until a sufficient understanding of the subsoil conditions can be attained.

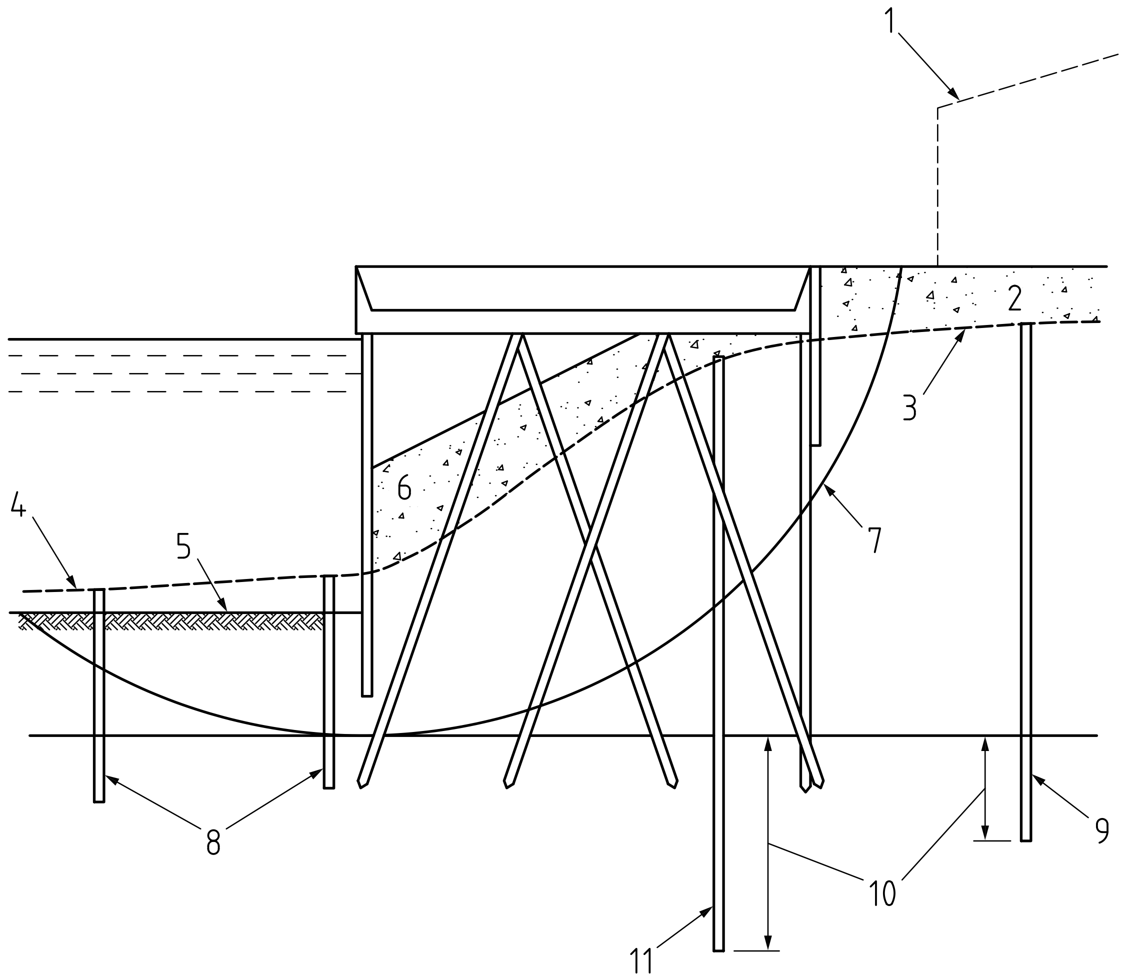
For a simple ship building berth there might be a need for a retaining wall at the lower end to provide deep water, so the site investigation should take into account that sheet piling might be needed at that location.

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 and if so, the site investigation should establish the likely length of such piles.

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

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

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



Key

1 Future shore-side structure

2 Reclamation fill

3 Original foreshore level

4 Original seabed level

5 Dredged level

6 Fill

7 Potential sliding surface of rotational grip

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

9 Boreholes to obtain information on reclamation and foundations of shore-side structures

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

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

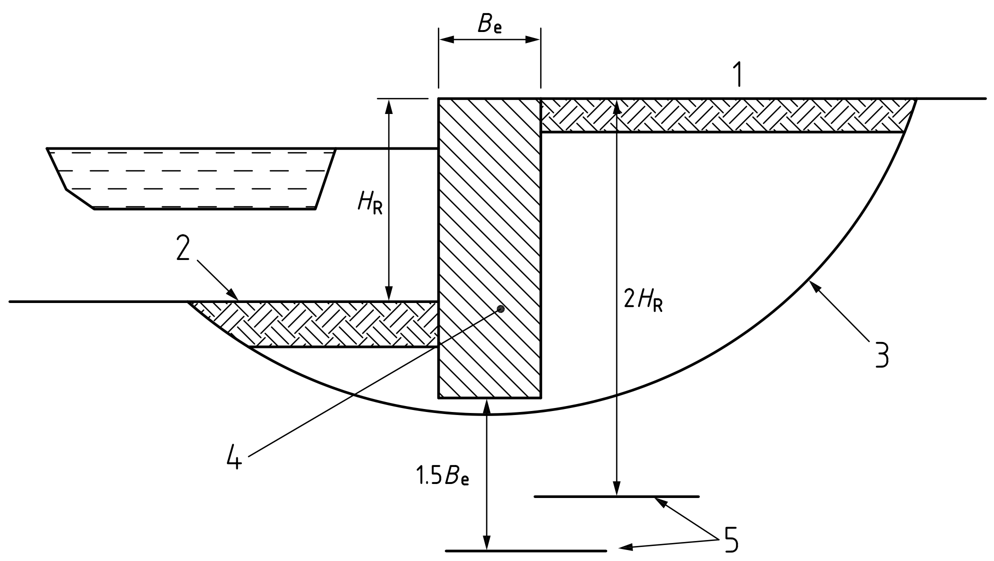
5.5 Depth of boreholes

NOTE Guidance on the depth of boreholes is given in BS EN 1997-2.

In order to obtain soil parameters to form the basis of slope stability analyses, the depth of the boreholes should cover the likely maximum depth and lateral extent of slip surfaces (see Figure 1).

Boreholes should be drilled to sufficient depth to investigate the foundation conditions and stability against a rotational shear slide (see Figure 2).

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

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 weathered 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 at least 3 m into such material to prove its quality and continuity.

In most cases where dredging is required, the depth of ground to be removed is limited to a few metres. In these instances, a relatively simple method of ground investigation, such as vibrocoring, might be adequate. In contrast, where difficult materials such as rock have to be removed by dredging, samples should be obtained by drilling from a floating or fixed structure.

Dredging below the seabed can involve removal of soil from below the designed dredge level to conform to the requirements of a particular excavation technique, and the possibility of future deeper dredging should be taken into account. Therefore 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 normally sufficient to rely upon other investigations outside the proposed dredging areas, although the results of such investigations should be examined and made available where relevant.

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; the possibility of heave of the floor of the excavation due to swelling of clay soils might also need to be taken into account. Boreholes should be taken to a sufficient depth to investigate the presence or absence of such hazards.

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 horizon, and far enough into the stratum to prove continuity.

5.6 Sealing of boreholes

Where boreholes are required in the area of a deep excavation for structures such as drydocks or other deep basins, they should be properly sealed before being abandoned, 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 less permeability than the original ground but preferably concrete.

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

5.7 Ground investigations over water

5.7.1 General

Site investigations conducted over water are more difficult and time-consuming than comparable investigations conducted on land, and there might be a temptation to economize by reducing the scope of the investigation. The extent of the requirement for ground investigations should be realistically assessed, since economies in this direction can turn out to be false. 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.

The sinking of boreholes below water presents special difficulties, particularly with regard to rotary diamond core drilling. Working platforms can be fixed, floating or heave-compensated. A conductor pipe should be suspended from the platform to protect the flexible investigation string from the force of currents. Percussion boring and rotary drilling techniques, both conventional and wireline, may be employed. Geophysical logging techniques are often employed to augment the information obtained from the borehole programme.

Overwater work is always subject to a risk of delay because of unsuitable weather. For any given weather condition, the amount of delay depends on the type and size of the installation. In general, the larger the staging or floating craft, the smaller the risk of delay due to weather but, on the other hand, the greater the operating costs. When planning an overwater investigation, a realistic allowance should be made for the possible cost of weather delays.

Sinking boreholes between high and low tide levels may be achieved using scaffold stagings or platforms (see **5.7.2**), by using flat-bottomed pontoons or shallow draft jack-up rigs or by moving boring rigs to the location during periods permitted by the tides.

NOTE 1 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.

NOTE 2 General guidance on geotechnical investigation and testing is given in BS EN ISO 22475-1.

5.7.2 Stages and platforms

NOTE 1 Where stable working platforms are available or can be provided, such as oil drilling platforms and jetties or purpose-built scaffold stages and drilling towers, it is normally possible to use conventional dry-land site investigation boring equipment and conventional methods of sampling and in-situ testing. When working from existing structures, it might be necessary to construct a cantilever platform over the water on which to mount the boring rig.

When boring 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 should be considered, although a means of transporting the boring equipment to the tower should be provided.

NOTE 2 Some towers are so constructed that they can be moved from one borehole location to another without having to be dismantled. Jack-up platforms and special craft fitted with spud legs can be floated into position and then jacked out of the water to stand on their legs. These fulfil the requirement for a fixed working platform and provide manoeuvrability.

The design of all staging, towers 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, and it is essential that such constructions be sufficiently strong for the boring operations to resist waves, tidal flow, other currents and floating debris.

5.7.3 Floating craft

The suitability of floating craft as an operating platform should be based on:

- the geotechnical properties of the sea bed;
- the likely weather conditions;
- the depth of water;
- the strength of currents;
- whether the water is sheltered or open; and
- 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. For the deeper waters, particularly with swells, special techniques of heave compensation should be adopted. The types of sample to be recovered and in-situ tests to be performed should be selected on the basis that special equipment is employed when boring from floating craft and that the working platform might move.

5.7.4 Setting-out and locating borehole positions

Modern positioning systems, such as the global positioning system (GPS) and the differential global positioning system (DGPS) should be adopted where practicable, because they are simple to operate, provide accurate surveying data and eliminate unnecessary risks to personnel in terms of health and safety. Close inshore, it is often possible to set out boreholes satisfactorily by using measurements to features onshore or by lining in from previously placed shore markers. Where necessary, theodolite observations can be taken from land-based stations.

Further offshore, it is necessary to use electronic methods of position fixing. Electronic methods are also advantageous in poor visibility conditions.

5.7.5 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. Standard tide tables for the area of interest may be used, but reduced levels may also be transferred to a boring vessel from shore by setting up a tide gauge close inshore; this is read at frequent intervals throughout the tidal cycles, and readings of water depth are taken at the same time on the operating platform.

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

6 Groundwater investigations

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

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. Fluctuations in tidal or seasonal levels in the waterway can cause corresponding fluctuations in piezometric head in the groundwater. The effects on groundwater levels of meteorological surges in sea levels should also be taken into account. It is therefore important to establish the relationships by means of simultaneous observations of waterway levels and groundwater levels in piezometers, 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 and neap tides, and if possible they should cover periods of seasonal peak conditions in waterway levels.

NOTE 2 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.

The possible existence of groundwater under artesian or sub-artesian pressure within pervious soil layers confined by impervious strata should be investigated by observations in piezometers installed within the pervious layers.

Most borehole or drill holes require the addition of water to facilitate progress, but it is essential that care is taken to maintain water equilibrium in the boreholes during the drilling process, in order to obtain satisfactory samples and conditions for in-situ tests, such as the standard penetration tests (SPT). If artesian conditions are present, the top of the borehole tube should be maintained at a level at least equal to the artesian head. Any water added will mask zones of inflow and changes in pressure, and so the readings taken during drilling are often not representative of in-situ conditions. Water level or pressure monitoring instruments should be installed to determine the equilibrium conditions (see BS EN ISO 22475-1).

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

NOTE 3 Information on permeability testing of soils is given in BS EN 1997-2.

7 Field tests in soil and rock

7.1 Planning

Field tests should be planned with the following in mind:

- geology/stratification of the ground;
- type of structure, the possible foundation and the anticipated work during the construction;
- type of geotechnical parameter required;
- design method to be adopted.

7.2 Normal field tests

The following field tests should be used as appropriate to obtain information about the site. They may be used individually or in combination:

- cone penetration test;
- pressuremeter and dilatometer tests;
- standard penetration test;
- dynamic probing;
- weight sounding test;
- field vane test;
- flat dilatometer test;
- plate loading test.

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

7.3 Other field tests

7.3.1 Determination of earth pressure coefficient at rest

Where earth-retaining structures are to be formed directly against the soil to be retained, in-situ measurements of the coefficient of earth pressure at rest should be obtained.

7.3.2 Detection of underground movements at depth

Where maritime works are to be constructed on sites suspected of having suffered previous instability, movements should be monitored in the ground at various depths in advance of construction, followed by similar observations during, and for as long as possible after, construction of the project. This form of monitoring also forms part of investigations of instability, which might arise as a result of earthworks associated with shoreside structures.

Trial pits and trenches may be used to determine the location of existing shear surfaces at relatively shallow depths. This method becomes increasingly more difficult with depth, and where there is reason to believe that movement is actually taking place or it is feared that movement is imminent, inclinometers installed in boreholes should be used. 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 for cases where the rate of movement is small or for installation in anticipation of movement. 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 special casing should be grouted.

7.3.3 Field tests for dredging

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. Vibrocoring, which is less expensive, 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. Very weak or mobile bed materials can result in rapid infill of the trench, and regular dredging should therefore be undertaken up to the point of the pipe launching or placement, with a consequent substantial increase in the cost.

NOTE 1 The removal of rock by dredging will almost certainly involve the mobilization of specialist equipment so that even for small quantities, costs will be relatively high.

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

NOTE 2 If fracturing is sufficiently close and open, pre-treatment might not be necessary. If pre-treatment is necessary, fractured rock can impede drilling by causing the drill to jam.

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

- *borehole logging;*
- *total core recovery (TCR);*

- *solid core recovery (SCR);*
- *rock quality designation (RQD);*
- *drillability;*
- *velocity of propagation of sound;*
- *standard penetration tests.*

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

NOTE 3 Information on the testing procedures for soil and rock is given in Annex B.

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. The cost 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 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.

It should also be considered where trench or channel formations are proposed that would cut across deposits of doubtful stability or across the paths of substantial sediment transport routes. 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.

Trial dredging should also be employed in situations where there is no satisfactory conventional soil investigation method that is capable of sampling the true ground conditions. This includes sites that contain particle sizes too large to be recovered intact by normal sampling methods. Hence trial dredging is 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, and key parameters should be pre-selected and carefully 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 and BS EN 1997-2.

8 Laboratory tests on soil and rock

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

NOTE CEN ISO/TS 17892-1 to CEN ISO/TS 17892-13 are currently in preparation, and will give methods for determining many of the properties listed below.

a) 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;*
- *tensile strength;*
- *drained/undrained shear strength;*
- *compressibility;*
- *permeability;*
- *porosity.*

b) 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;*
- *compressive strength.*

9 Geotechnical design report

NOTE Recommendations concerning the geotechnical design report are given in BS EN 1997-1.

The geotechnical design report should include the geotechnical investigation report, details of which are given in BS EN 1997-1, 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.

Section 3: Geotechnical design

10 General

10.1 Soil pressures

For the purposes of calculating soil pressures:

a) variable actions on surfaces should be determined as described in BS 6349-1:2000, Clause **44** and Clause **45** [3)];

b) water levels should be derived as described in **10.2**;

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 due to overdredging and/or scour.

10.2 Tides and water level variations

NOTE 1 Information concerning waves will be given in BS 6349-1-2, which is currently in preparation.

Maritime structures should be designed to withstand safely the effects of the extreme range of still water level from extreme low water (ELW) to extreme high water (EHW) expected 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 2 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:

a) overtopping;

b) hydrostatic pressures, including buoyancy effects;

c) soil pressures on quay walls;

d) lines of action of mooring and berthing forces, forces from other floating objects and wave forces.

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

Design for ULS conditions should be based on extreme water levels on both sides of the structure, with the astronomic tide level being considered as temporary loading and the other elements as transient loads.

Design for SLS conditions should be based on tidal variation from highest astronomic tide (HAT) to lowest astronomic tide (LAT), but taking into account tidal lag, drainage issues and the effects of fresh water flow plus any other known contributors.

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

[3)] The clauses in BS 6349-1:2000 that deal with loads and actions are expected to form part of the new BS 6349-1-2, which is currently in preparation.

10.3 Earthquakes

For each maritime structure, an assessment should be performed to establish the potential magnitude of seismic events that can be expected at this site.

NOTE 1 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.

An assessment of seismic effects should then be performed. Even in areas where low seismic activity is expected, an assessment of the importance and sensitivity of the structure in question should be made to decide whether its failure might lead to disproportionate damage in the event of a very low probability earthquake event.

NOTE 2 The following examples might fall into this category.

a) 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;

b) 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;

c) structures whose failure would impede the regional and national ability to deal with a disaster caused by a major damaging earthquake;

d) strengthening or upgrading of historic structures forming an important part of the national heritage.

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

NOTE 3 Many countries subject to earthquakes include specific seismic design considerations within their building codes, although there are considerable variations in 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. It is therefore important in seismically active areas that the designer should select a type of structure that has as little sensitivity to seismic action as can be contrived.

Fine sandy soils can be vulnerable to liquefaction, and this should be taken into account by the designer.

Specialist advice, particularly in relation to geophysical and geological aspects, should be sought where there is significant seismic activity or the danger thereof, and guidance should be obtained on the appropriate seismic actions to be used in design.

NOTE 4 Local regulations might apply. Further information is given in PD 6698.

11 Basis of geotechnical design

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

12 Geotechnical data – Selection of parameters for working design

12.1 General considerations

A detailed examination should be made 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 properties for use in design calculations should be selected in accordance with BS EN 1997-1, and with **12.2** to **12.6** and **14.1** in the present standard.

NOTE Physical characteristics for soils and rocks for use in preliminary calculations are given in Annex C.

12.2 Sands and gravels

Values of relative density and angle of soil shearing resistance, φ_r, can be obtained from empirical relationships between these parameters and the results of standard penetration tests or static cone penetration tests (see Note 1). The shearing strength of dense sands and gravels are strain-dependent, showing a marked reduction from a high peak value at small strains to a relatively low value at large strains (see Figure 3). This variation should be taken into account in relation to the large deformations that are possible with flexible structures, such as sheet pile retaining walls, or berthing structures designed to sustain large deformations as a means of absorbing the kinetic energy of moving vessels.

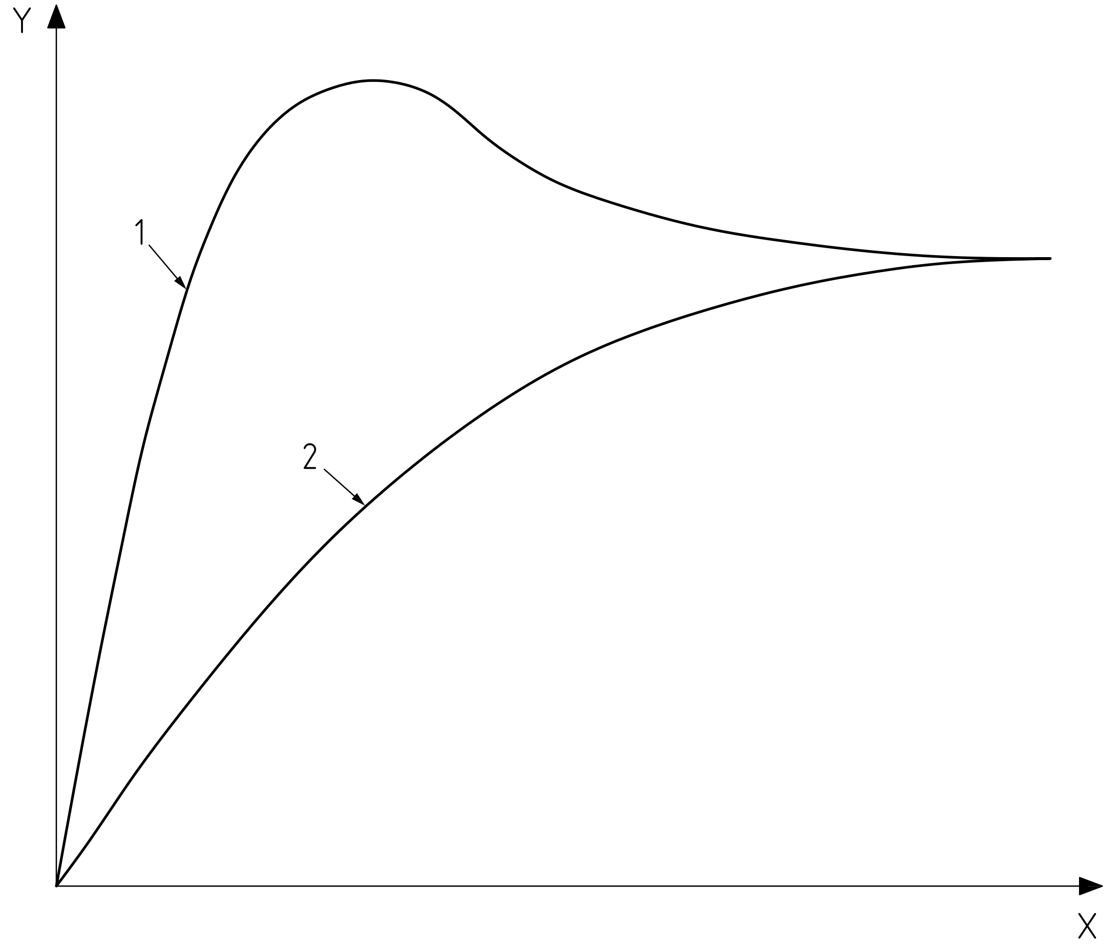
NOTE 1 Guidance on the relationship between relative density, angle of soil shearing resistance and penetration test results is given in Foundation design and construction *[4].*

The maximum angle of friction, δ_{max}, between the soil and an earth-retaining structure should be taken as $2\varphi_r/3$.

NOTE 2 Mobilization of this maximum value requires movement between the structure and the soil. Because sands and gravels are highly permeable, groundwater shows a rapid response to any fluctuation in the water levels of an adjacent waterway. On a falling tide this can result in outflow and erosion due to rapid drawdown of groundwater from unprotected slopes. Sands and gravels are also susceptible to erosion because of flowing water or wave action. Erosion of the seabed adjacent to a quay wall can cause a reduction in passive resistance to pressure from the ground retained by the wall. Similarly, erosion around bearing piles can result in weakening of their resistance to lateral actions and reduction in vertical skin friction resistance.

The deformation of the soil beneath heavily loaded foundations is likely to be a more critical design factor than considerations of failure in shear under the superimposed loading, and this should be taken into account by the designer.

Figure 3 **Plain strain shear diagram for sand**



Key

1 Dense

2 Loose

X Strain

Y Stress

NOTE 3 The deformation characteristics of sands and gravels can be obtained by correlation with the results of standard penetration or static cone penetration tests; see Foundation design and construction *[4].*

NOTE 4 The skin frictional and end bearing resistance of piles can be calculated from the angle of shearing resistance of sands and gravels as obtained by field tests or by direct correlation with field tests. Further guidance is given in Pile design and construction practice *[5].*

Both the deformation characteristics and resistance to shear exhibited by sands and gravels are affected by dynamic loading, and in particular when large numbers of cyclic repetitions of load are involved. These effects should be taken into account in relation to stresses induced by wave action, tidal fluctuations and, where appropriate, earthquakes.

Sands and gravels can be cemented in varying degrees by a saline or calcareous matrix, which can impart an appreciable cohesive strength to the soil. The possibility of complete or partial loss of the cementitious value of the matrix should also be taken into account when determining design parameters.

NOTE 5 This loss can occur in tropical or subtropical saline soils when excavation, made under water or exposed to seepages of water, allows fresh or brackish water to reach soil deposits that previously existed under fully saturated saline conditions. The saline or calcareous matrix is then dissolved with complete or partial loss of cohesion.

12.3 Silts and fine silty sands

Where these soils are predominantly cohesive, total stress shearing strengths should be obtained by means of field vane, cone penetrometer or pressuremeter tests. Undisturbed samples of silts and fine silty sands can be obtained by means of piston samplers for use in consolidated drained triaxial compression tests. The results of these tests should be used for the calculation of earth pressures and stability conditions, in terms of effective stresses.

The possibility of silts and fine silty sands becoming unstable as a result of high pore pressure generated within permeable layers of fine sand confined by more impervious silt layers should be taken into account. Pore pressure variations can be caused by tidal waters, or water from landward sources under a high piezometric head, gaining access to the permeable layers. High pore pressures can also be generated by actions applied to the ground surface or by vibrations transmitted to the soils from embedded structures.

High frequency vibrations transmitted to silts and fine silty sands can lead to liquefaction and complete loss of support to foundations or passive resistance to earth pressures. The possibility of large scale flow slides caused by pile driving vibrations or earthquakes should be taken into account in respect of shallow underwater slopes dredged in fine sands (see BS 6031).

Fine sands and silts are susceptible to erosion by flowing water and the consequent effects are as described previously for sands and gravel (see **12.2**). Account should be taken of the occurrence of piping and quicksands.

The deformation characteristics of fine sands and silts where loading is applied through foundations, and the resistance of piles embedded in these soils to axial and lateral actions, should be obtained in the same manner as described in **12.2** for sands and gravels.

NOTE Some fine sands and silts, notably loose soils, can exist in a weakly cemented and desiccated state. When water is introduced to these soils, either by diversion of surface water or by inundation in reclamation works, the structure of the soil can collapse with considerable subsidence of the ground surface.

12.4 Normally- and lightly over-consolidated clays

NOTE 1 Normally-consolidated or lightly over-consolidated clays are those that have not been subjected to heavy overburden pressures during their geological history.

Undrained shear strengths of these clays should be obtained by field vane or pressuremeter tests, or by undrained triaxial compression tests made in the laboratory on undisturbed samples from boreholes. These parameters should be used for consideration of the short-term stability of earth-retaining structures and slopes under total stress conditions.

Normally-consolidated clays are sensitive to the effects of disturbance. The degree of disturbance depends on the geological history of the clays (see Note 2). The effects of construction activities associated with maritime structures on the soil should be taken into account when assessing engineering characteristics. For example, driving bearing piles to support the deck of a wharf can result in the loss of undrained shear strength of the mass of clay surrounding the piles. Consequently there is a reduction in the passive resistance on the face of any nearby sheet piles, which might be forming an earth-retaining structure on the landward side of the wharf. Vibrations from pile driving or earthquakes can cause the collapse of slopes in normally-consolidated clays sensitive to disturbance.

NOTE 2 Guidance on the sensitivity of clay soils is given in Problems of soil mechanics and construction on soft clay *[6].*

The development of lateral pressures on earth-retaining structures and the stability conditions for these structures and for slopes should be calculated in terms of effective stresses. The appropriate shear strength parameters for normally-consolidated clays should be obtained by means of consolidated drained triaxial compression tests, made in the laboratory on undisturbed samples.

NOTE 3 Subsequent dredging for slopes and berths reduces the total stress in the soil, followed by volumetric expansion, absorption of water and softening of the clay. The rate at which the softening takes place depends upon the permeability of the soil. It can be accelerated by the presence of drainage channels in the form of layers or laminations of silt and sand within the mass of the clay.

The stress-strain relationship for normally-consolidated clays is shown in Figure 4. The selected shear strength parameters should take account of the predicted or permissible deformations in structures such as flexible sheet pile walls and berthing dolphins and the predicted deformation of slopes (see **17.2**). The effective stress parameters of normally-consolidated clays can vary according to the direction of application of the deviator stress to the test specimen. This is due to the effect of discontinuities in the specimen or to differences in the ratio of vertical to horizontal stress in the soil in situ (see Note 4). In triaxial compression tests, the effects of pore pressure changes in the test specimen should also be studied in relation to the predicted variation in piezometric levels behind under-water and above-water slopes (see **17.2**).

NOTE 4 Guidance on effective stress parameters in clays is given in Problems of soil mechanics and construction on soft clay *[6].*

12.5 Over-consolidated clays

NOTE 1 Undrained shear strength parameters can be obtained from triaxial compression tests made on undisturbed samples of over-consolidated clays. These parameters can be used to determine the stability of foundations of structures and the short-term stability of underwater slopes in terms of total stress (see ***17.2****).*

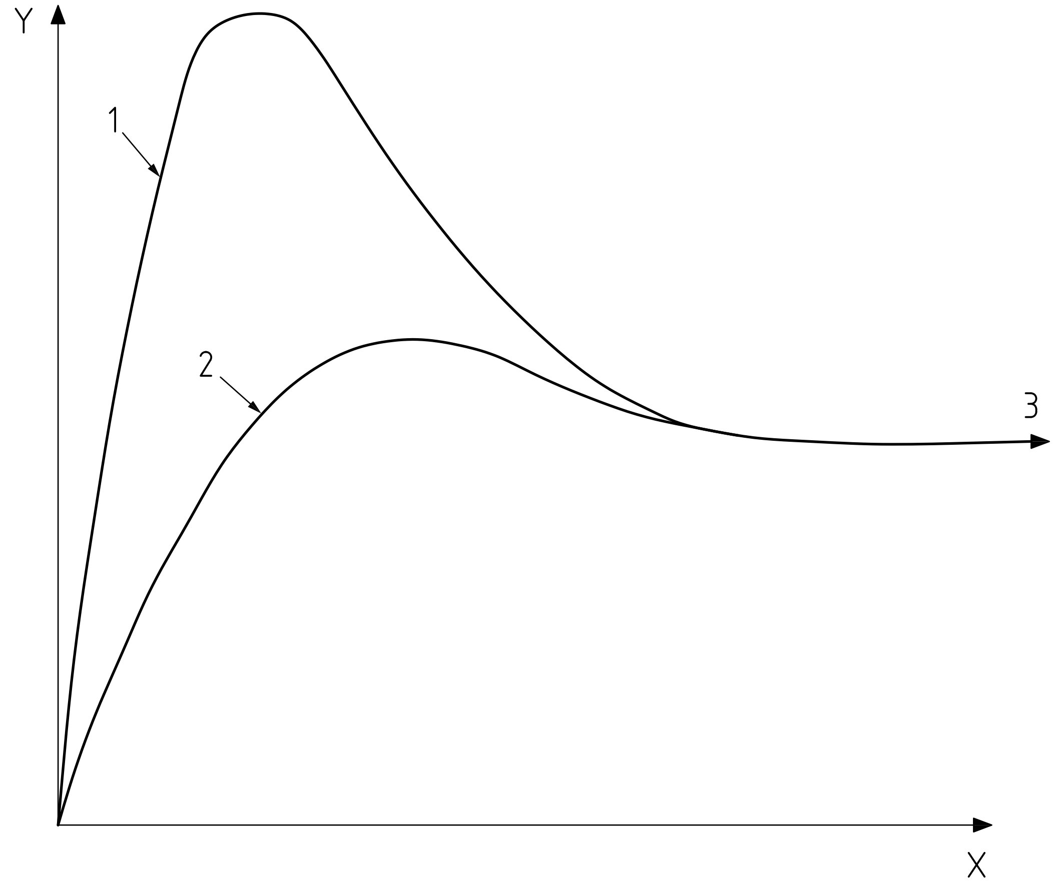
The long-term stability of slopes should be analysed in terms of effective stresses, and the appropriate parameters should be obtained from consolidated drained triaxial compression tests on undisturbed samples of the clay.

NOTE 2 Over-consolidated clays show the same strain-dependent effects as normally-consolidated clays (see Figure 4), and the influence of these on the calculation of lateral pressures and slope stability is discussed in ***17.2****.*

Over-consolidated clays are likely to have a fissured structure. The orientation of the principal fissure system within the test specimen should therefore be taken into account when assessing triaxial compression test results.

NOTE 3 The fissured structure of over-consolidated clays also affects their mass permeability. This is significant when considering the shearing resistance of a mass of clay after excavation by dredging. Relief of overburden pressure causes fissures to open allowing the ingress of water and softening of the mass. Some glacial clays can have a laminated structure, which has similar effects on mass permeability to those of a fissure system.

Figure 4 **Plain strain shear diagram for normally consolidated clay**



Key
1 Undisturbed
2 Remoulded
3 Residual value
X Strain
Y Stress

12.6 Rocks

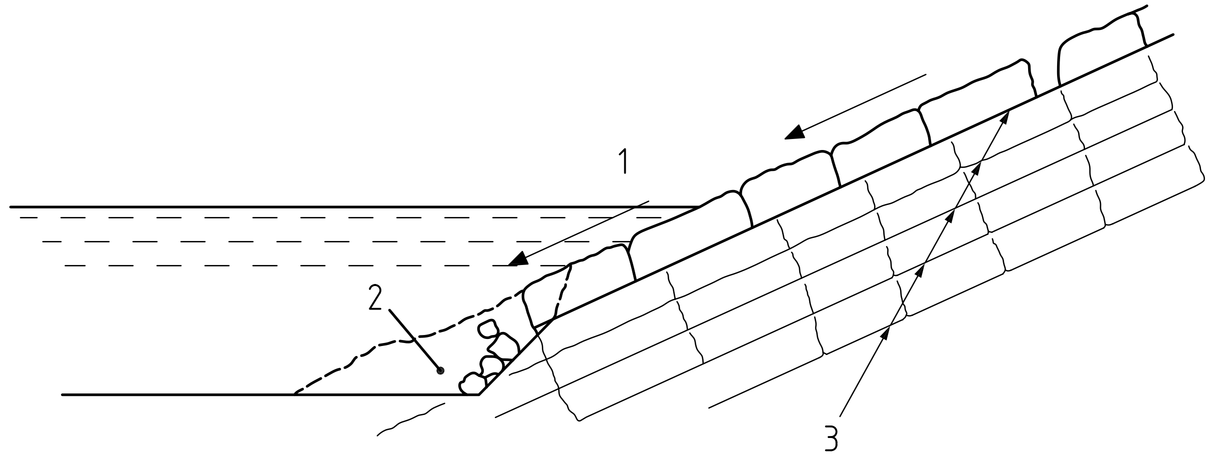
In maritime structures, the stability of rock masses should be assessed in relation to excavations for the retaining walls of quays, lock chambers and docks. Short-term stability is the relevant situation until the retaining structure is built against the rock face. Long-term stability should be assessed in the case of underwater and above-water slopes. Over the height attacked by waves, cavities and joints should be plugged and structural overhangs and re-entrant angles should be avoided.

The stability of slopes in rock formations should be assessed using the strength of the material within discontinuities.

NOTE These can be, for example, fissures, laminations, fault planes, bedding planes and layers of weak weathered rock. Behaviour of the rock is likely to be different above, within, and below the band attacked by waves. Some modes of failure are shown in Figure 5, Figure 6 and Figure 7.

Considerations of lateral pressure due to rocks are normally concerned with pressures caused by fill in the form of crushed rock, and the selection of appropriate parameters is discussed in **14.1**. However, the possibility of high lateral pressures should be taken into account where concrete retaining walls are cast against the excavated face of a rock with potential swelling characteristics.

Figure 5 **Slab slide in rock**



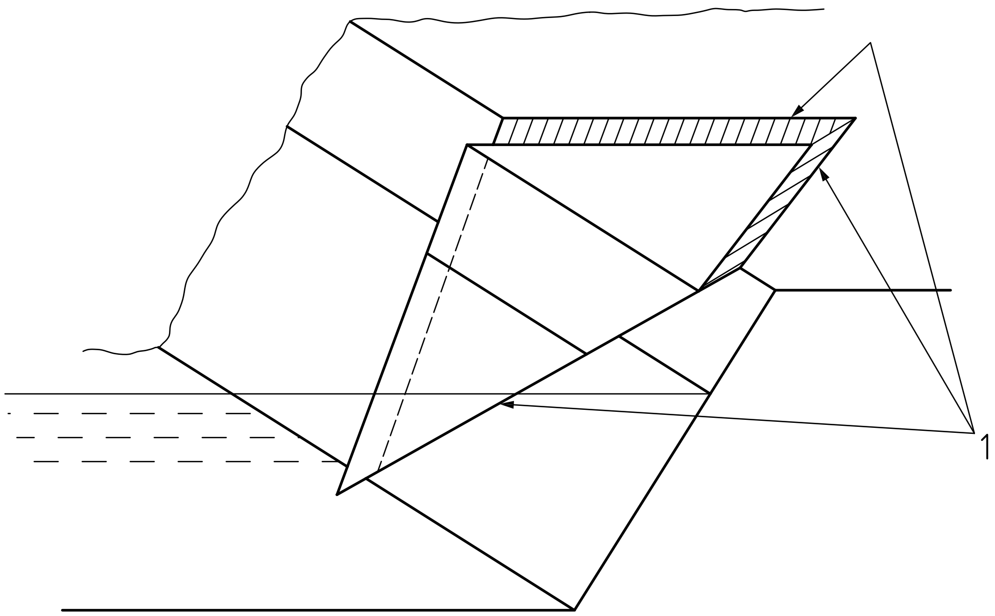
Key

1 Surface of sliding

2 Rock removed by dredging or wave action

3 Inclined bedding joints

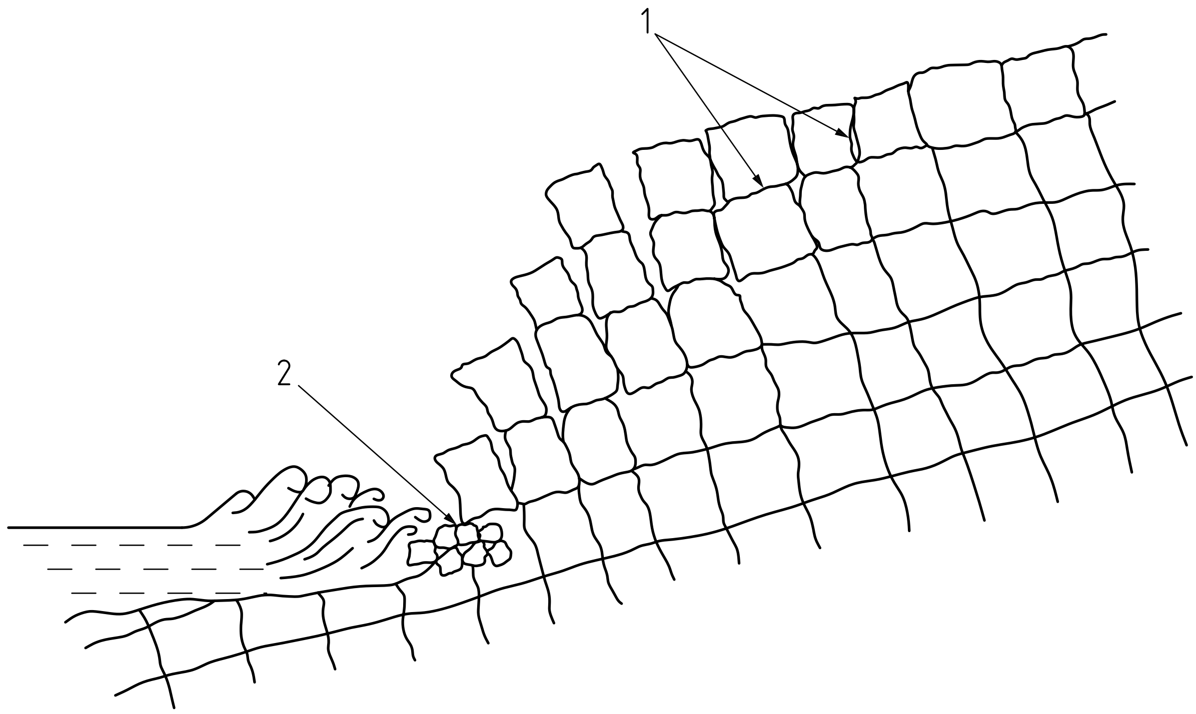
Figure 6 **Wedge failure in rock**



Key

1 Joints in rock

Figure 7 **Toppling failure in rock**



Key

1 Joints

2 Toe of cliff eroded by wave action

13 Water

13.1 Single-wall structures

13.1.1 General

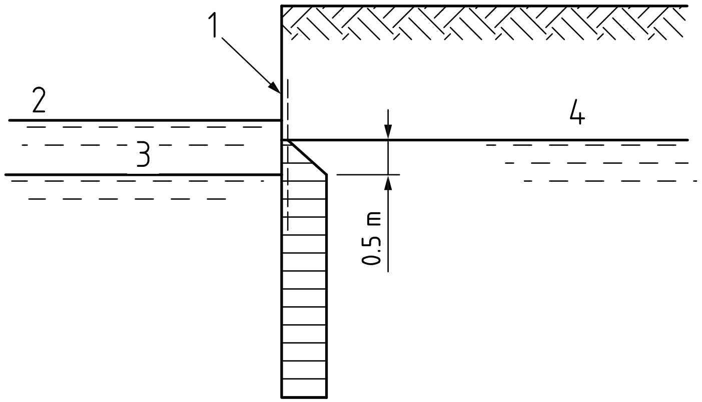
NOTE Tidal variations cause a differential hydrostatic pressure between the waterside face and the landward face of an earth-retaining structure. Fluctuations in the level of the groundwater (GWL) will normally be less than the tidal variation, depending on the type and efficiency of the drainage measures 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.

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 cases described in **13.1.2** to **13.1.6** illustrate how the differential water pressure should be derived.

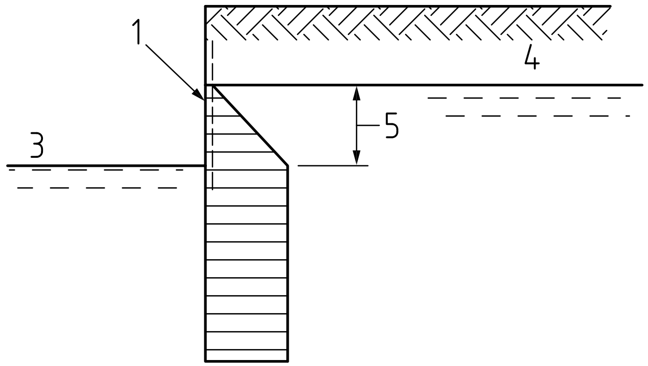
13.1.2 Minor non-tidal water level variations (drainage provided)

In non-tidal waterways where variations are due to minor seasonal fluctuations, the differential water pressure from the landward to the waterside face should be taken as 0.5 m as shown in Figure 8a), in addition to any other sources of groundwater as detailed in **10.1**.

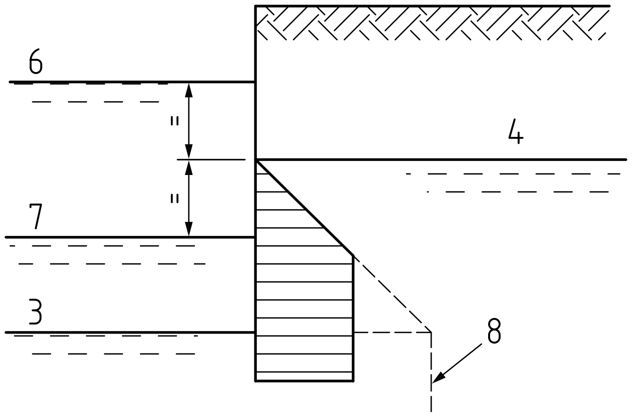
Figure 8 **Hydrostatic pressure distribution on waterfront structures where soil is retained to full height of structure**



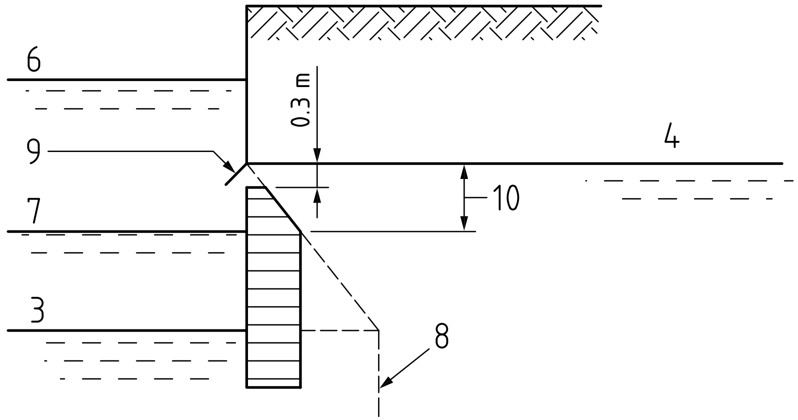
a) Minor, non-tidal



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 **10.2**)
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

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

Where a rapid fall in 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 8b)]. 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 to ensure that the most unfavourable effects of combined earth and differential water pressure are applied to the structure. Any other sources of groundwater as detailed in **10.1** should also be taken into account.

13.1.4 Large tidal variations (no drainage provided)

The differential water pressure should be taken as that due to a groundwater level at mean tide level and a low tide level in the waterway. These pressures range from mean low water (MLWS), for normal cases, to an assumed water level, down to extreme low water (ELW) (see **10.2**). For the latter case, the risks of structural failure due to the combined effects of earth pressure and water pressure conditions for extremely low tides [see Figure 8c)] should be assessed. Other sources of groundwater as detailed in **10.1** should be taken into account.

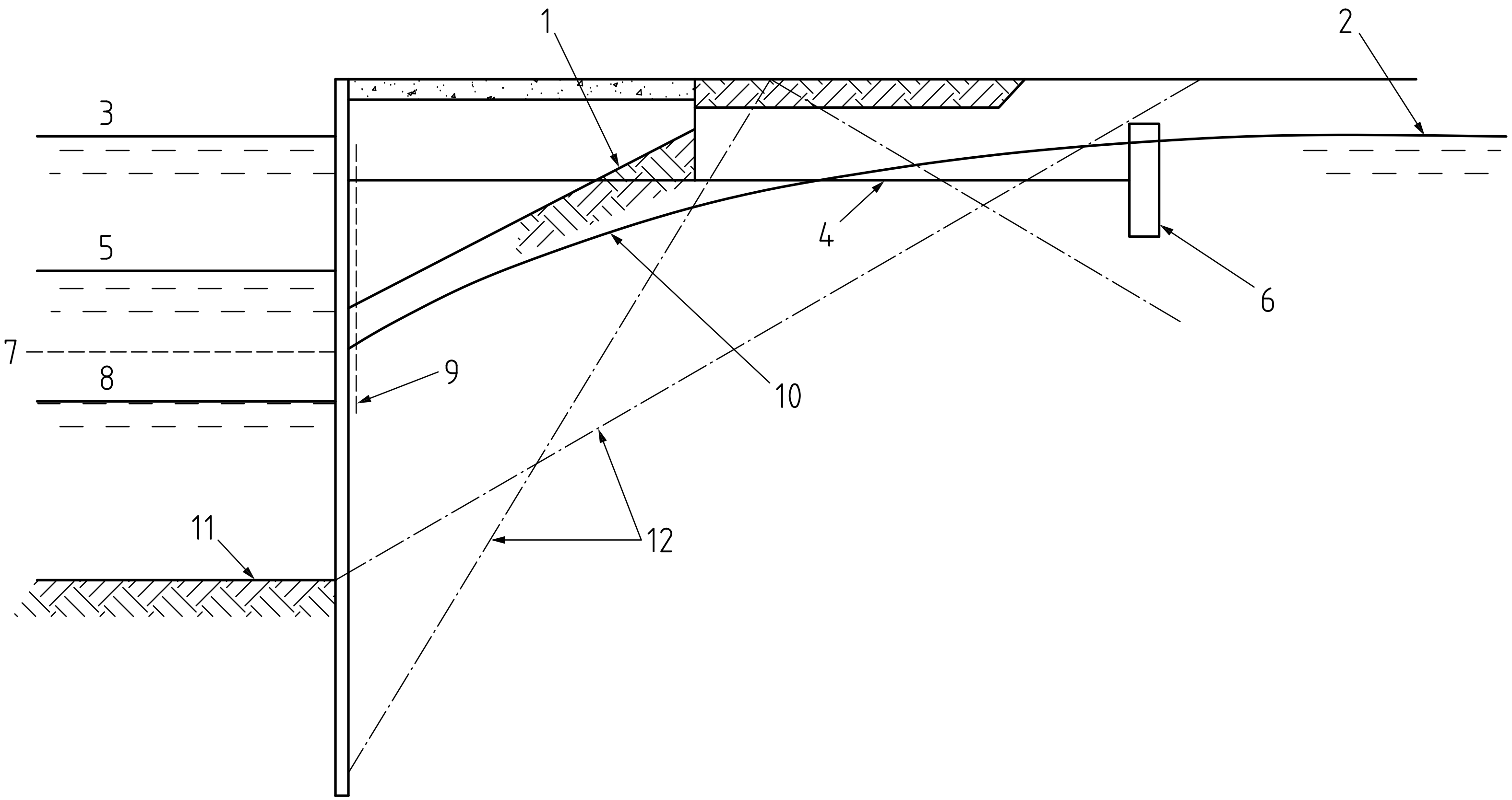
13.1.5 Large tidal variations (flap valve drainage provided)

The differential water pressure should be taken as corresponding to a groundwater level, which is 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, for normal cases, or at some intermediate level down to ELW (see **10.2**). For the latter case, the risks due to the effects of combined earth pressure and water pressures due to extremely low tides should be taken into account [see Figure 8d)], in addition to any other sources of groundwater as detailed in **10.1**.

13.1.6 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 9, should be taken into account in the design, either in relation to the effective weight of the wedge of soil behind the structure, or within the zone of any anchorages (see Clause **15**).

Figure 9 **Hydrostatic pressure distribution on waterfront structure where the soil is embanked behind the structure**



Key

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

13.1.7 Special considerations

In all the cases described in **13.1.2** to **13.1.6**, the assumed groundwater level is taken to be that due to flow in a homogeneous permeable soil. Designers should take into account hydrostatic pressures resulting from a strong subsoil water flow from a landward source. The effects of excess water pressures in permeable layers within a layered or laminated soil or rock formation should also be assessed.

Where the retaining structure is backed by clay, drying and shrinkage of the clay above groundwater level can cause a tension crack to form. Pressure resulting from surface water finding its way into such a crack should be taken into account in design calculations.

The depth of the tension crack can be calculated from:

$$d_t = 2c_u/\gamma_s$$

where:

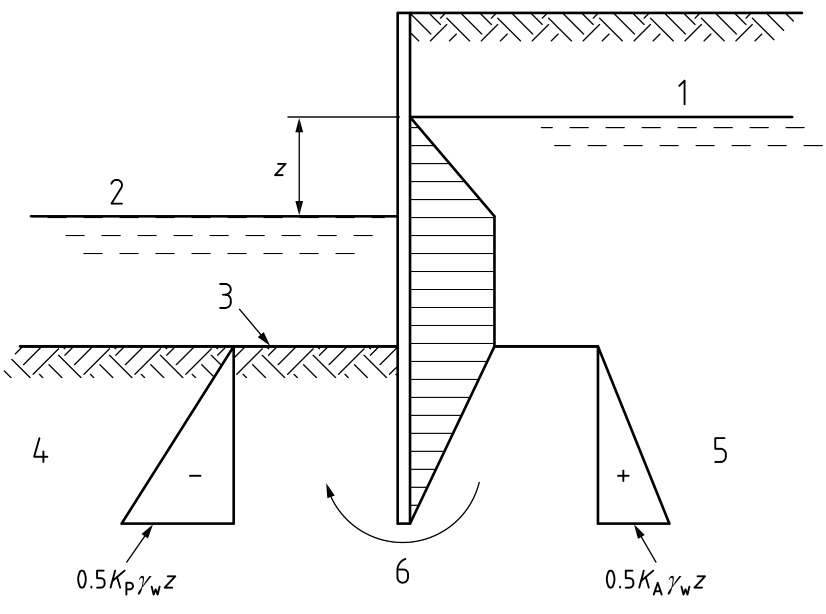
c_u is the undrained shear strength of soil;

d_t is the depth of the tension crack;

γ_s is the bulk density of the saturated clay.

In cases where permeable soil extends below the toe of the sheet piles, seepage can take place below the piles and the simplified hydrostatic pressure distribution for flow from the landward to the waterside is as shown in Figure 10. The effects of seepage forces causing an increase in active earth pressure and a decrease in passive earth resistance should be taken into account.

Figure 10 **Effects on hydrostatic and soil pressure distribution where seepage takes place beneath 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

13.2 Double-wall and cellular structures

It is essential to use free-draining material as filling within the sheet piles of double-wall or cellular structures. Drainage should be provided in the walls. The water level within the structure can then be taken as corresponding to the assumed groundwater level for **13.1.2** to **13.1.6**.

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.

14 Fill, dewatering, ground improvement and reinforcement

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

14.1 Fill materials

NOTE 1 Recommendations for the selection of material properties for and use of fill are given in BS 6031.

The design parameters for materials placed as fill behind earth-retaining structures or used in reclamation areas should be determined on the basis that they are fully remoulded soils.

It is unlikely that either normally-consolidated or over-consolidated clays would be used as fill exerting pressure on a retaining wall. In particular, over-consolidated clays can exert very high pressures due to swelling of the clay lumps after excavation and exposure to the effects of air and water, and they should not be used as backfill materials in this situation.

NOTE 2 Cohesive fills can be subject to significant long-term settlement due to self-weight settlement even after normal compaction processes. This process can induce additional stresses within tie rods and settlement beneath paving areas.

Cohesionless soils, i.e. sands and gravels, or crushed rocks should normally be used as fill behind retaining walls and in reclamation areas where stable conditions capable of carrying surface actions are required.

NOTE 4 The soil parameters to be selected for cohesionless soils depend on the manner of their deposition. Where they are discharged from a pipeline or dumped to fall through water, they are in a loose state of deposition and parameters for density and shearing resistance may be obtained from the values given for loose conditions in Annex C, Table C.1.

Where drainage can take place from cohesionless soils, placed above water level by hydraulic fill methods, or where these soils can be tipped and compacted above water level, they are in a medium dense to dense state. The density depends on the degree of compaction given to the fill and the parameters should be selected accordingly.

Consideration should be given to compacting loose materials above or below water level using special techniques of deep compaction.

NOTE 3 Guidance on deep compaction techniques is given in Piling and ground treatment *[7].*

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. This condition will apply 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 tipped through water, they are in a loose state of deposition and might undergo time-dependent consolidation as a result of crushing and degradation of points of contact between rock fragments. On an unyielding surface this can amount to up to 1% of the height of the rockfill (see Note 4). High densities with a correspondingly high angle of shearing resistance can be obtained from crushed rocks compacted above water level.

NOTE 4 Guidance on construction using crushed rock fill is given in BRE paper 15/17 [8] and in Laboratory compression tests and the deformation of rockfill structures *[9].*

The effects of weathering on rocks used as fill should be taken into account. Softening at the points of contact of rock fragments results in a reduction of angle of shearing resistance. Complete degradation results in the formation of a mass of fill of soil-like consistency, in which case the selected parameters will depend on whether the rock degrades to a cohesionless or cohesive soil or some intermediate type. This can result in settlement.

NOTE 5 The causes and effects of settlement of filled areas are discussed in BS 6031, and methods for constructing earthworks over compressible soils where settlement might be excessive are listed in BS 6031.

14.2 Ground improvement

The use of ground improvement techniques is common in reclamation projects where densification of newly placed fills is required.

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

The most common method is by the use of vibrators to densify the existing material or to reinforce with the construction of granular columns. These works should be carried out in accordance with BS EN 14731.

Where vibro-compaction is undertaken, particular care should be taken to assess the effect of both horizontal actions on existing structures generated by the vibro process and the displacement of the existing fill materials.

Other methods, such as installation of band drains plus surcharge or vacuum consolidation, should be undertaken with the benefit of specialist advice.

14.3 Reinforcement

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

15 Function and location of anchorages

*COMMENTARY ON CLAUSE **15***

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 considered.

Guidance on the design of such anchorage systems is given in BS 6349-2. Guidance on allowable stresses in tendon anchorages and on the procedure for installation and stressing is given in BS 8081. Further information on tendon anchors is given in BS EN 1997-1.

In selecting soil parameters for the design of injected tendon anchors to resist vertical uplift forces, 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.

16 Retaining structures

NOTE Recommendations concerning retaining structures are given in BS EN 1997-1.

16.1 Flexible structures

*COMMENTARY ON **16.1***

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

Guidance on the strength and durability of the various types of sheet piles will be given in BS 6349-1-4, which is currently in preparation.

Information on the design of steel sheet-piled structures is given in BS EN 1993-5 and BS 6349-2.

*Particular guidance is given in **16.1.1** for structures in maritime or riverine situations, where the method of construction and the flexibility of the wall have a direct effect on the earth pressures.*

16.1.1 Distribution of lateral earth pressure and earth resistance

16.1.1.1 Cohesionless soils

Sheet pile structures are flexible such that appreciable deflections accompanied by strains in the soil occur when they perform as earth-retaining structures. The magnitude of these strains affects the shearing resistance of cohesionless soils, as described in **12.2**, and φ_r should be taken as the value corresponding to active pressure conditions for the calculation of the pressure on the landward face of the structure.

NOTE 1 Cohesionless soils do not normally show time-dependent effects.

In the case of a cantilevered single-wall structure (Figure 11) where movement occurs by outward rotation about an apparent point of fixity near the base, it should be assumed that equal horizontal strain occurs at every point above the base. The pressure distribution in a dense cohesionless soil should therefore be taken as increasing linearly with depth after very small movements have occurred.

In the case of an anchored single-wall structure (Figure 12) where movement should be assumed to take place by outward displacement of the toe about a pivot point at anchorage level, non-uniform horizontal strains occur within the wedge of dense cohesionless soil behind the wall. Designers should assume that initially, the lower part of the wedge attains the fully active state before the upper part of the wedge, because the latter is restrained by the tension in the anchor so that forward movement is insufficient to develop active pressure conditions.

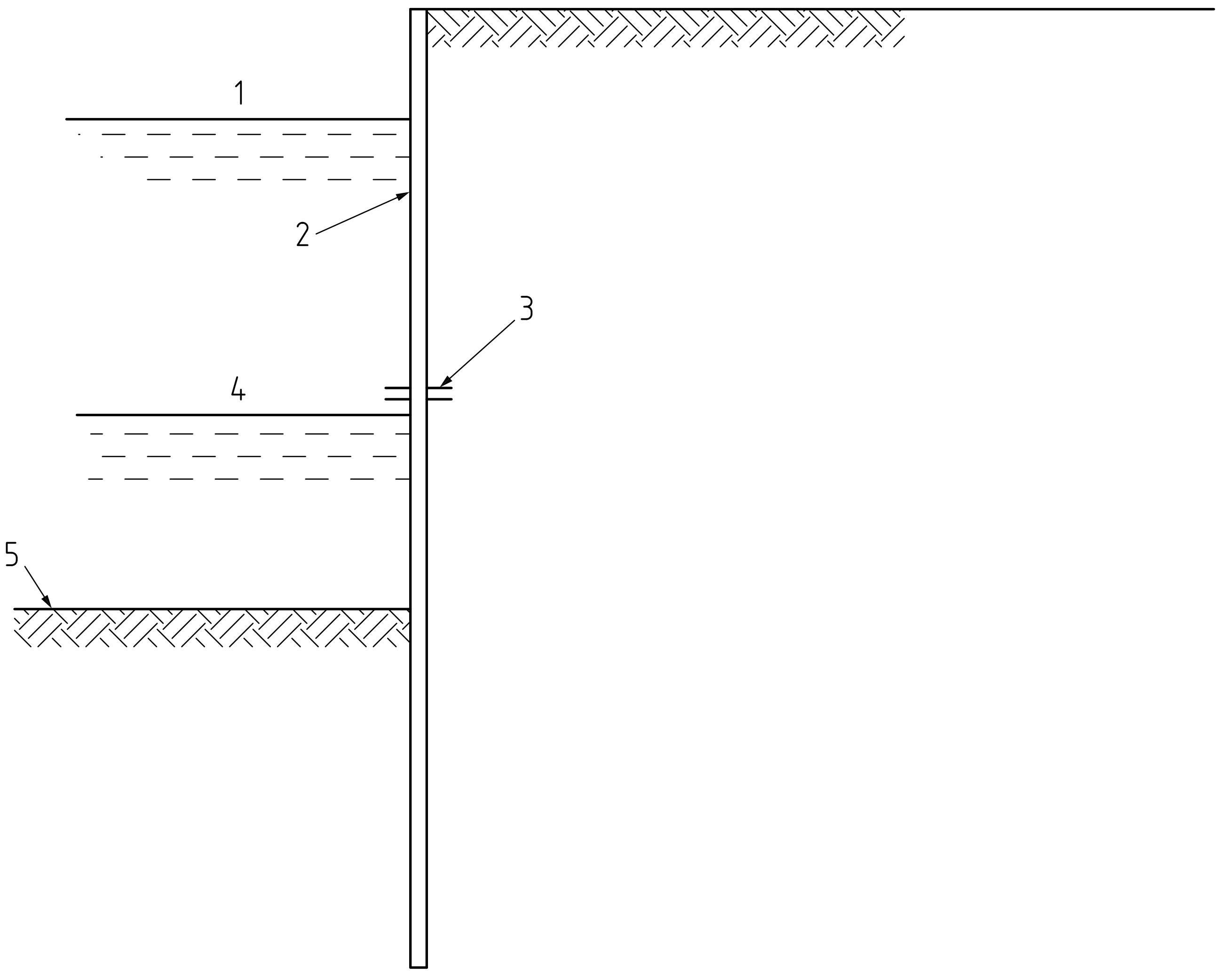
If the resultant lateral force causes yielding of the anchor then the active pressure state should be applied over the full depth of the wedge, giving a linear distribution similar to that shown in Figure 13. Where no yielding of the anchor occurs the arched pressure distribution remains non-linear, as shown in Figure 14.

The total lateral force imposed by the wedge of soil remains approximately constant during the process of initial arching, followed by yielding and redistribution of pressure. The initially high position of the resultant lateral force, though, should be taken into account when calculating the anchorage force and the stability of the wall against overturning.

Arching and the development of high lateral pressures in the upper part of the soil wedge should also 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 (see **16.1.2**).

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 11 **Cantilevered single-wall sheet pile structure**



Key

1 Mean high water springs (MHWS)

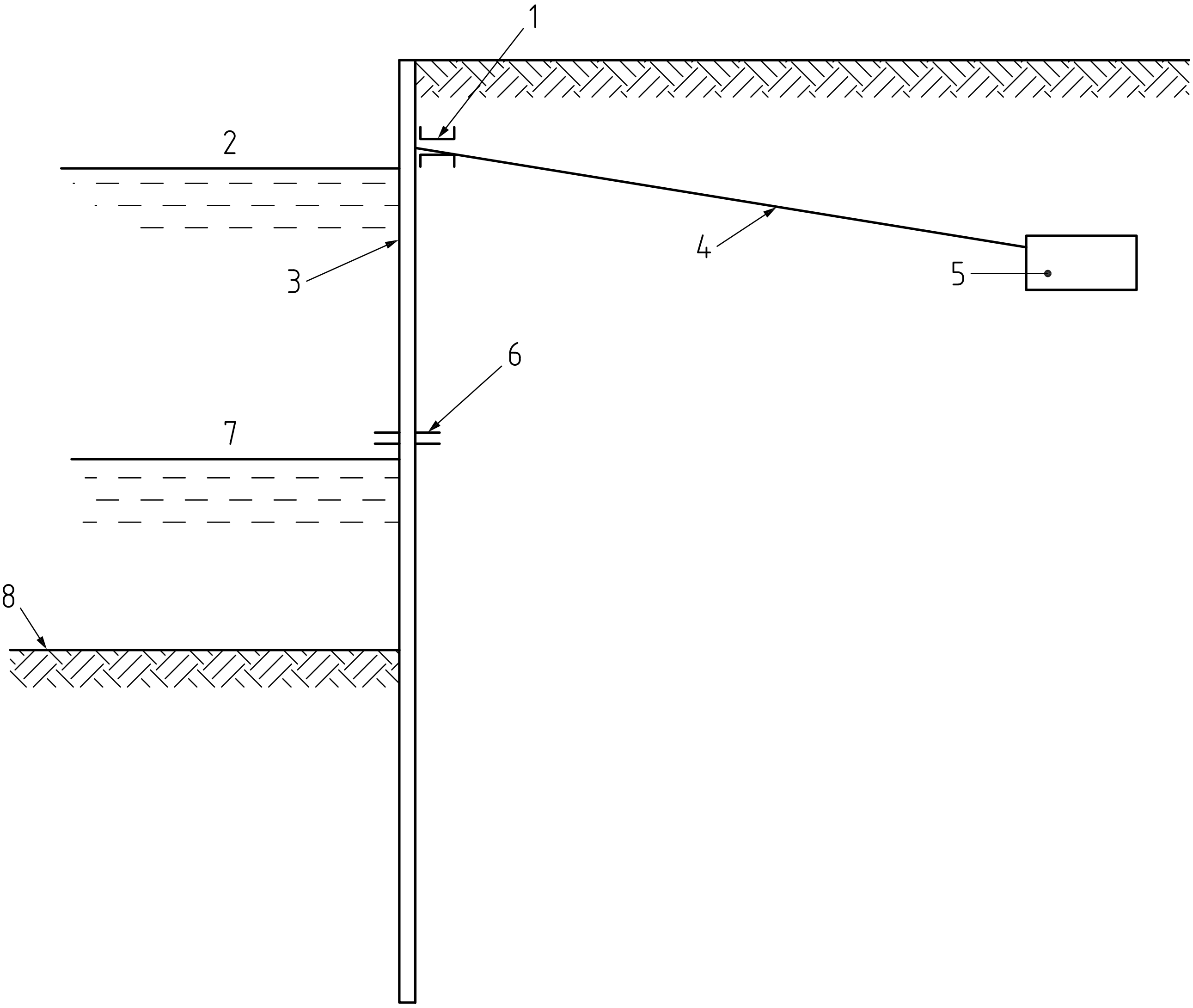
2 Sheet piling

3 Drainage

4 Mean low water springs (MLWS)

5 Dredged level

Figure 12 **Anchored single-wall sheet pile structure**



Key

1 Waling

2 Mean high water springs (MHWS)

3 Sheet piling

4 Tie rod

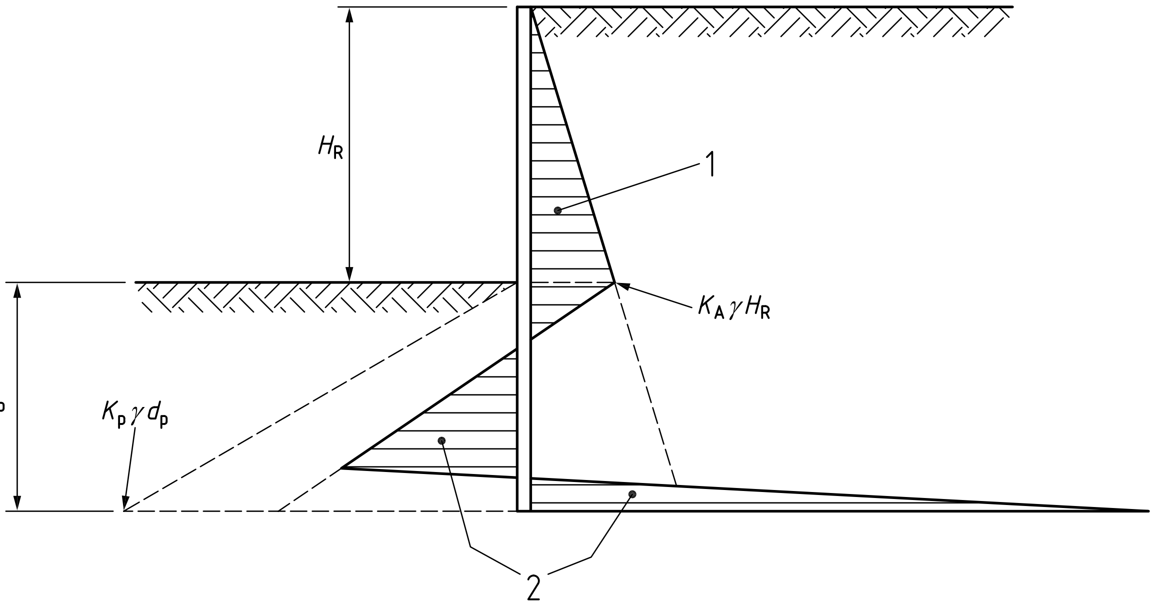
5 Anchorage

6 Drainage

7 Mean low water springs (MLWS)

8 Dredged level

Figure 13 **Distribution of earth pressure and earth resistance on cantilevered single-wall sheet pile structure**

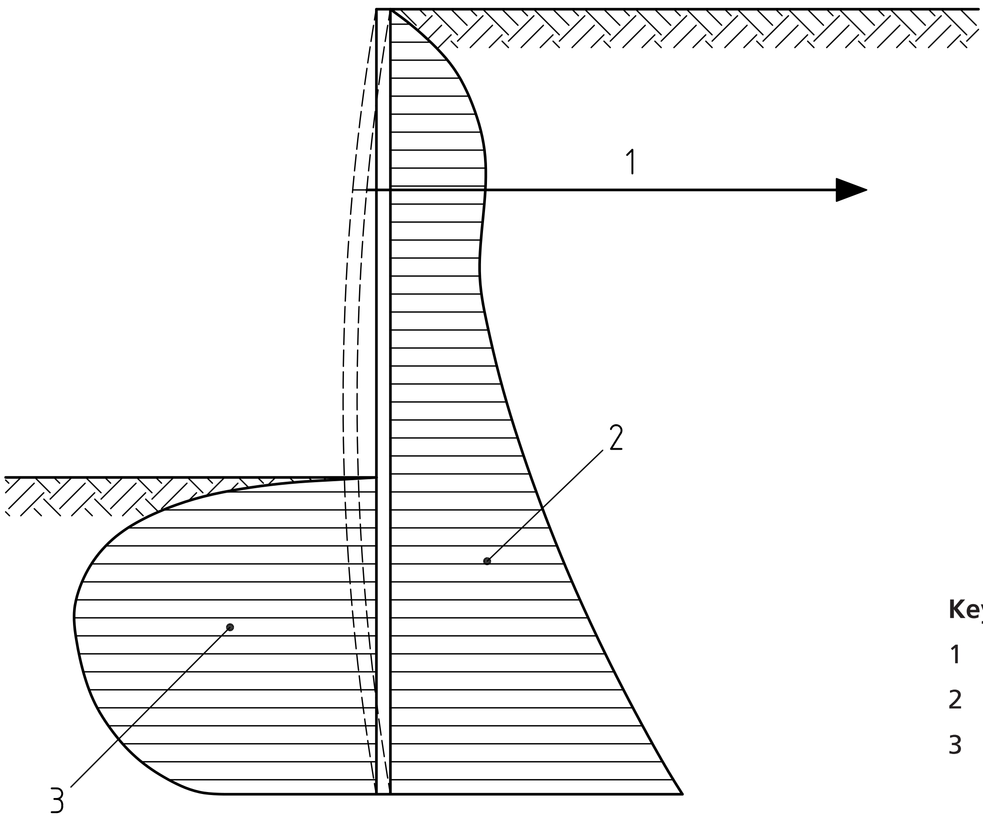


Key

1 Active pressure

2 Net passive resistance

Figure 14 **Distribution of earth pressure and earth resistance on anchored single-wall sheet pile structure**



Key

1 Tie rod

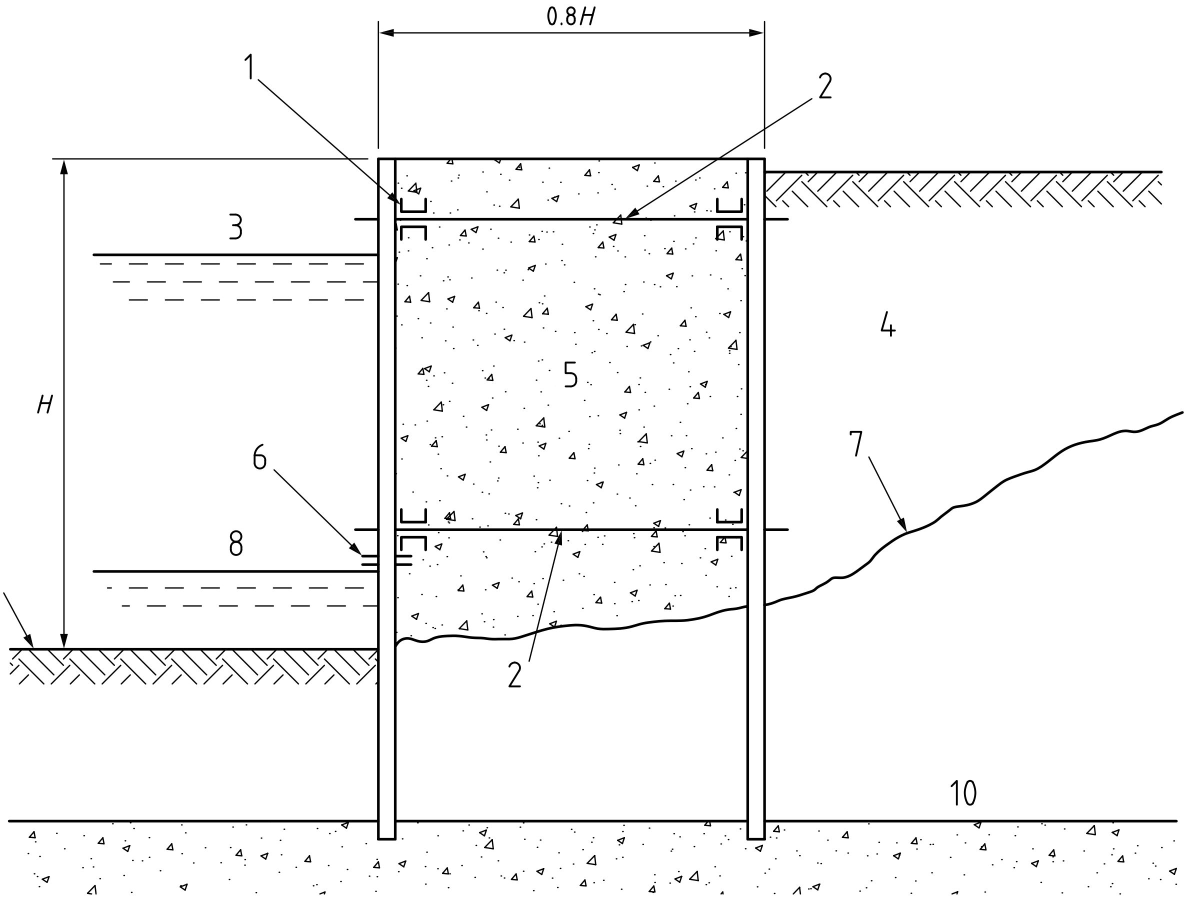
2 Active pressure

3 Passive pressure

Similar arching conditions should also be assumed in relation to the lateral pressure distribution on gravity-type, double-wall, sheet pile retaining walls (see Figure 15 and Figure 16).

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.

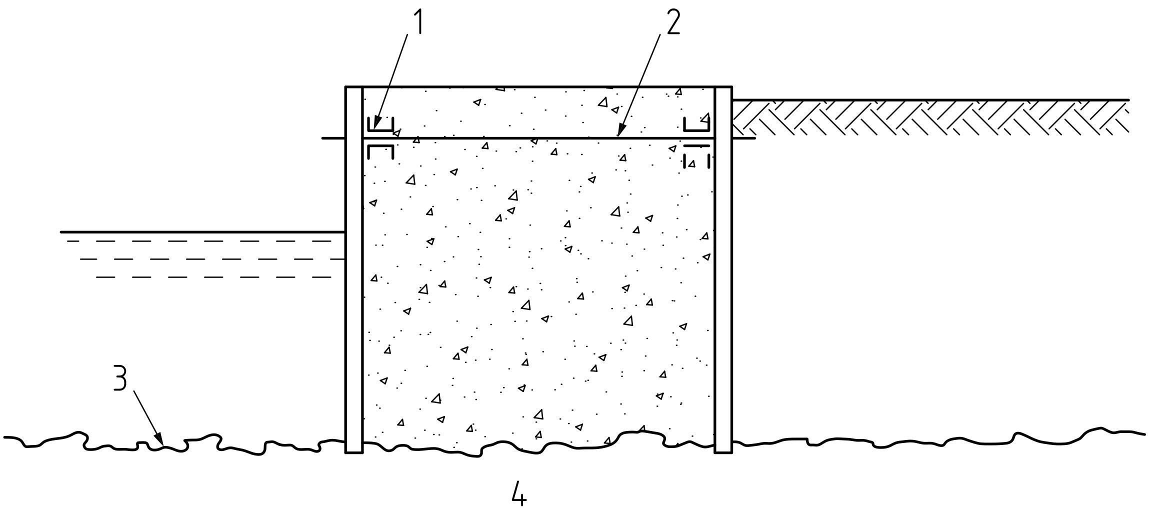
Figure 15 **Double wall sheet pile structures – Sheet piles driven into soil below seabed**



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	Fill	9	Seabed
5	Granular fill	10	Hard stratum

Figure 16 **Double wall sheet pile structures – Sheet piles terminated on rock at seabed**



Key

1 Walings

2 Tie rod

3 Seabed

4 Rock

Arching conditions should be taken into account in relation to the sequence of construction of an anchored sheet-piled retaining wall.

NOTE 4 Backfilling is carried out before the soil in front of the wall is dredged away. When this is completed, the movement of the wall due to pressure from the small retained height of soil might be insufficient to develop active pressure conditions. As dredging takes place, the wall yields and the pressure distribution changes from arched conditions at the upper level to the final assumed linear active condition. The initial and final stages are shown in Figure 17.

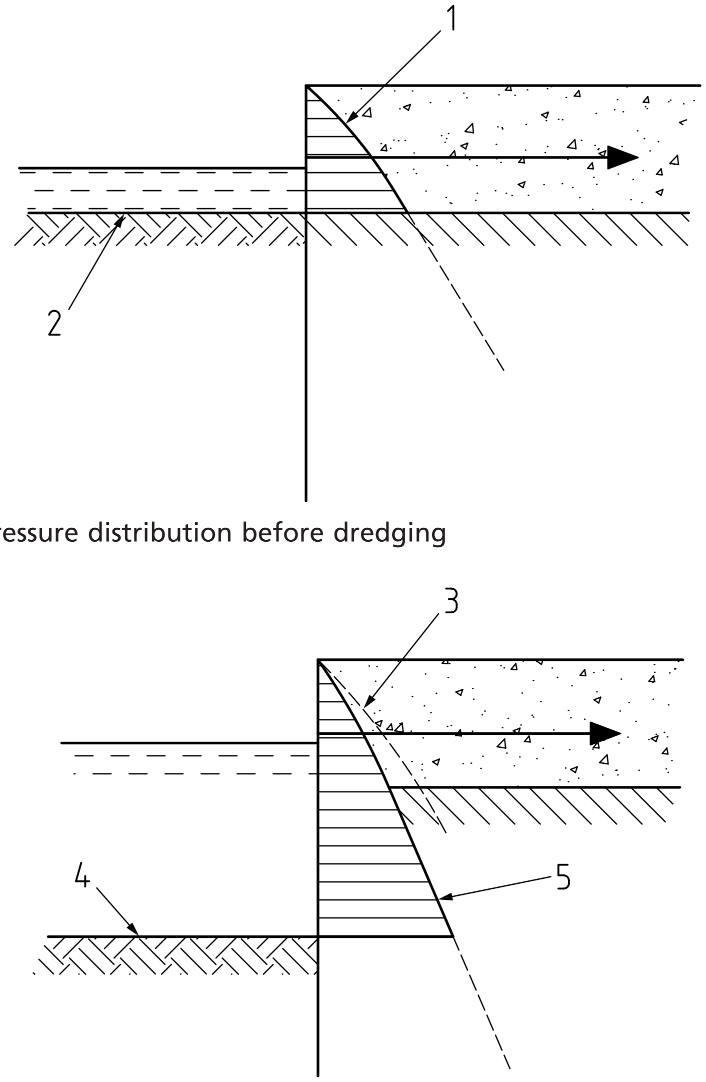
If the soil is dredged away before placing any fill behind the wall, the designer should assume that a linear active pressure distribution will develop over the depth within the existing ground after completion of dredging. When backfill is placed and the part above groundwater level is compacted, the additional yielding of the wall and anchorages might not be sufficient to develop active earth pressures from the filling at the higher levels. In this case, provision should be made for lateral pressures from the upper part of the filling at a state intermediate between the active and at-rest condition, depending on the expected forward movement of the wall as the filling is placed (see Figure 18).

NOTE 5 Where sheet piles are toed into rock, movement of the sheet piles is prevented and the distribution of active pressures is modified. At the toe the pressures correspond to at-rest conditions.

Mobilization of the full passive resistance of a cohesionless soil requires a greater forward movement than is required for the development of active earth pressure. Model tests show that an outward movement of a wall at dredged level of 5% and 0.5% of the wall height is required for mobilization of half the passive resistance in loose and dense sand respectively. The movement required to mobilize the ultimate passive resistance is 10% to 20% of the depth of embedment for loose sands and 2% to 4% of this depth for dense sands. In practice the movements of a sheet-piled structure are unlikely to be such as to develop the ultimate passive resistance, and a conservative approach to the calculation of available resistance should be allowed.

NOTE 6 The calculated passive resistance is not mobilized without movements that might be unacceptable in a permanent structure. When sheet piles are driven sufficiently deeply to achieve fixed earth support conditions, the effective depths can be diminished by the effect of reverse curvature in the piles. In the case of cantilevered walls, passive resistance can be generated on the landward side of the wall at the toe (see Figure 13).

Figure 17 **Active pressure distribution on anchored single-wall structure where filling is placed before dredging**



a) Initial pressure distribution before dredging

b) Final pressure distribution after completion of dredging

Key

1 Initial distribution for at-rest pressure

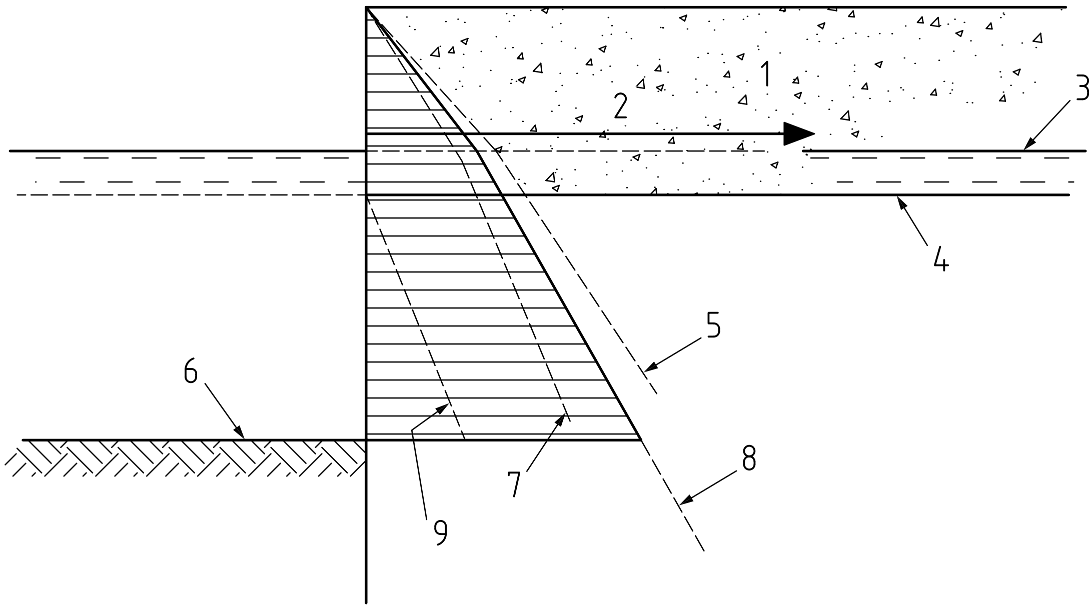
2 Existing seabed level

3 Initial distribution

4 Dredged level

5 Final distribution

Figure 18 **Active pressure distribution on anchored single-wall structure where filling is placed after dredging**



Key

1 Fill
2 Tie rod
3 Groundwater level
4 Existing ground surface
5 At-rest pressure
6 Dredging to this level before placing fill
7 Active pressure
8 Design pressure
9 Pressure distribution before placing backfill

16.1.1.2 Normally-consolidated and lightly over-consolidated clays

The forward movement of 0.5% of the wall height required to mobilize the fully active shearing resistance of a clay soil is likely to take place in a sheet pile structure. Lateral earth pressures should therefore be calculated in terms of effective stresses.

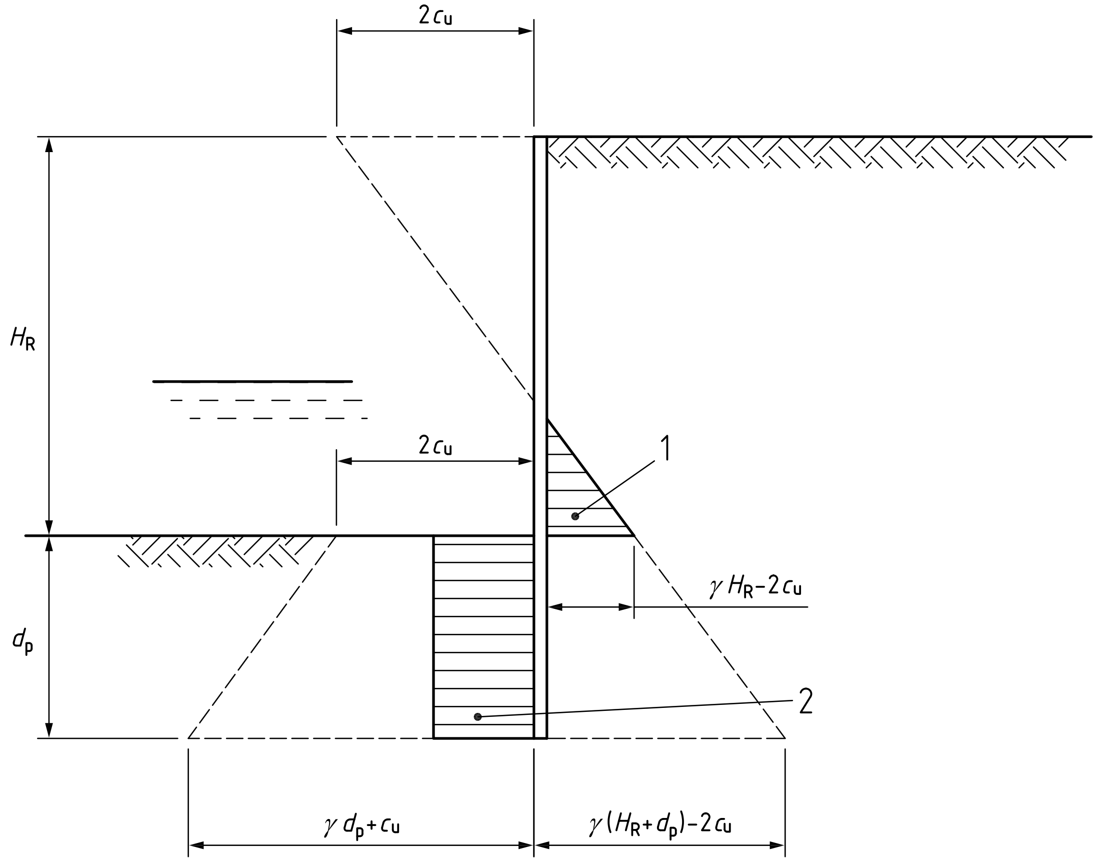
NOTE 1 This is based on the assumption of zero-effective cohesion and φ' being taken as the value representative of active conditions. The pressure distribution can be taken as linear, as shown in Figure 13.

For calculating passive earth resistance, the short-term, undrained conditions are likely to be more critical than the long-term drained state. Therefore the passive resistance should be calculated on the basis of the peak, undrained cohesion, c_u, and in terms of total stresses.

NOTE 2 The distribution of net passive resistance is uniform, as shown in Figure 19.

In the long term, the positive pore water pressures induced by the shear stresses will dissipate, leading to consolidation and strengthening of the soil, accompanied by wall movement. However, a check calculation of passive earth resistance should be made for long-term conditions in which the effective cohesion c' should be taken as zero, and the effective angle of soil shearing resistance, φ', should be the drained value for passive conditions.

Figure 19 **Distribution of active pressure and passive resistance for total stress conditions in normally and lightly over-consolidated clay**



Key

1 Active pressure

2 Net passive resistance

Alternatively, Bell's theory may be used to obtain the total active force, P_A, and total passive force, P_P, as follows (see Figure 19):

$$P_A = \frac{\gamma(H_R + d_p)^2 - 2c_u(H_R + d_p)}{2}$$

$$P_P = \frac{\gamma d_p + 2c_u d_p}{2}$$

where:

c_u is the undrained shear strength of the clay, based on the weakest 5% of samples;

d_p is the sheet pile penetration.

H_R is the retained height of the structure;

γ is the density of the soil;

A tension crack should not be taken into account except when considering hydrostatic pressure at the ground surface.

NOTE 3 A cantilevered wall usually fails before H_R reaches the theoretical maximum value of $4c_u/\gamma$, while an anchored wall also usually fails with a small increase in H_R. Causes of failure are absence of, or inadequate, passive pressure.

If the clay contains water-bearing horizontal joints or sand lenses, then uplift pressures can develop and the submerged density should be used.

16.1.1.3 Over-consolidated clays

*COMMENTARY ON **16.1.1.3***

Dredging in front of a sheet-piled wall embedded in an over-consolidated clay causes the wall to deflect forwards. The clay behind the wall swells with the development of high negative pore pressures in the soil. Initially this produces highly stable conditions in terms of effective stresses, but with time, water will be drawn into the zone of swelling, causing the clay to soften. The rate of softening depends on the mass permeability of the soil (see Figure 4). Similar swelling and softening of an over-consolidated clay occurs in front of the sheet pile wall as a result of a reduction of overburden pressure consequent on excavating down to final dredged level.

Active pressures and passive earth resistance should be calculated for long-term conditions in terms of effective stresses. The effective cohesion c' should be taken as zero and the value of φ' should be representative of fully drained conditions.

Linear pressure distribution should be assumed.

Because of the high swelling pressures, caused when an over-consolidated clay is allowed to soften, such material should not be used as filling behind a retaining structure.

16.1.1.4 Layered soils

Lateral soil pressures at any level should be calculated by factoring the overburden pressure at that level by the soil pressure coefficient relevant to the soil stratum. Thus, changes in the lateral soil pressure diagram will occur at interfaces between different strata. The pressures will also be affected by the presence of water in one or more of the strata.

16.1.2 Effects of surcharge

16.1.2.1 Single-wall structures

Uniform surcharge applied to the soil surface behind an earth-retaining structure has the same effect on earth pressures as an increased height of soil behind the wall. The lateral pressure induced by surcharge applied in the form of line and point loading should also be taken into account.

The effect of the surcharge actions can be reduced by installation of a relieving structure to the rear of the wall. Structures behind a retaining wall can be supported by piles driven below the potential surfaces of sliding. In such cases, designers should assume 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 *[10].*

16.1.2.2 Double-wall and cellular structures

Double-wall and cellular sheet pile structures can be provided with a reinforced concrete deck capable of carrying heavy, imposed loading. Where the actions on the deck structures are carried by the sheet piles, internal lateral pressures from the fill material due to the actions on the deck should not be considered as acting on the retaining structure.

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. Relative vertical movement between the structure and the retained soil has the effect of reducing the wall friction on the active pressure side. The increase in wall friction on the passive side should not be taken into consideration for design purposes.

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. Where necessary, this soil should be removed before driving the bearing piles.

Granular soil used to fill the cells should not be placed until the bearing piles have been driven. Bored and cast-in-place piles should be installed after completion of filling, but care should be taken to avoid the development of excessive pore pressures in any layered or laminated clays that might exist below the base of the structure.

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 *[11].*

Actions on the deck increase the resistance of the structure to sliding and overturning, but designers should assume zero variable actions in stability calculations.

16.2 Diaphragm walls

16.2.1 Types of structure

Diaphragm walls may be utilized for the construction of shoreside structures such as quay walls, particularly where the ground is weak or where actions are high. These walls are formed in trenches which are stabilized during excavation by the introduction of a support fluid to apply pressure to the trench walls, thereby obviating the need for support by timbering or sheet piling. The fluid may be in the form of bentonite slurry or a man-made polymer. The wall is formed by pouring concrete through a tremie pipe, displacing the slurry, which is then reused or discarded. The construction of diaphragm walls should conform to BS EN 1538.

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 is 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.

16.2.2 Lateral earth pressure and earth resistance

Earth-retaining structures constructed using support fluid techniques have a measure of rigidity, and in the case of massive counterfort or buttressed walls on an unyielding foundation, designers should adopt at-rest conditions for the calculation of earth pressure.

NOTE Cantilevered or tied-back retaining walls can be designed for active earth pressure conditions or for a state intermediate between the active and at-rest state, depending on the rigidity of the structure and the amount of yielding expected in the anchors or at the toe of the structure.

For anchored walls, account should be taken of the effects of arching of the soil on the distribution of earth pressure (see **16.1.1**).

Adequate keyed joints should be provided between adjacent panels or units, to ensure uniformity in distribution of earth pressure and earth resistance over the full length and depth of the structure.

16.2.3 Design of excavations supported by fluid

NOTE 1 The general principles that govern the stability of slurry-supported trenches are set out in BS EN 1538.

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

Provision should also be made for temporary 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 not be so great as to reduce appreciably the arching action of the soil surrounding the excavated trench. The dimensions of the panel are also governed by the capacity of the available concreting plant, bearing in mind that the concrete has to be placed in a continuous pour.

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

Tight control of support fluid quality should be maintained, particularly prior to concreting. The key control parameters are viscosity, sand content, slurry density, filter cake and filter loss. In permeable soils, the quality of the support fluid should be sufficient to ensure the formation of a suitable filter cake at the sides of the trench.

NOTE 2 This inhibits water loss from the trench, so that the full head of fluid (above groundwater level), and therefore the stability of the trench, is maintained.

The viscosity of the fluid should not restrict concrete flow at the base of the panel during concreting.

Care should be taken, particularly when constructing load-bearing walls, to avoid contamination of the concrete by thickened mud at the base of the panel.

Periodic checks should be made on the density and other properties of the fluid as required by BS EN 1538.

16.2.4 Materials

NOTE 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 *[12].*

Attention should be paid to the effect of saline water on the properties of the slurry. Where necessary, special forms of bentonite or polymer should be used to prevent undesirable flocculation of the slurry in the trench.

Measures should be taken to prevent discharge of used fluid or uncontrolled escape of fluid from the trench to adjacent waterways, watercourses or sewers where these discharges would result in pollution or blockage of drainage systems. The environmental effects of using support fluid, particularly bentonite, should be taken into account in the design.

17 Slopes

17.1 Design considerations for slopes and embankments

17.1.1 Stability analysis

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

Particular attention should be paid to the effects of variations in pore pressure within the soil mass caused by the factors described in **17.2.1**, and to the other environmental effects referred to in **17.2.2** and **17.2.3**.

In most cases, only the long-term stability of dredged slopes and embankments needs to be considered, and the stability analyses, both in cohesionless and cohesive soils, should be made in terms of effective stresses.

Stability analyses of total stresses based on the undrained shear strength of the soil should be limited to cases where only short-term stability is required, e.g. in excavations for placing box caissons or monoliths for quay walls.

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 also be controlled.

NOTE 2 This is to allow time for the pore pressures to dissipate, which results in strengthening of the soil.

The possibility of local slips or falls occurring on the face 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 forms of failure described in **17.3** should also be taken into account.

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 therefore be adopted in the selection of a profile for the permanently submerged area.

*NOTE 3 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 **17.9.3** and **17.11**, can be taken.*

17.2 Slope stability and protection – Environmental factors

17.2.1 Design principles

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

When considering the stability of slopes in the long term, the effective cohesion of all soil types should be taken as zero. Consequently, the pore pressure has a critical effect on the shearing resistance that can be mobilized on the potential plane of sliding.

Account should be taken of the effect of tidal variations on pore pressures in the soil behind the slope.

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.

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. The way that these discontinuities provide a means of drainage from soil and ingress of tidal water should be taken into account.

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 2 Guidance on the response of a porous elastic bed to water waves is given in Wave-induced pressure, stress and strain in sand beds *[13].*

NOTE 3 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.

17.2.2 Changes in slope profile

*COMMENTARY ON **17.2.2***

*Steepening can be the result of erosion of the toe of a slope by tidal or river currents, or the wash from ships (see **17.9**). 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. The effect of steepening of the upper part of the slope should also be taken into account.

17.2.3 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 examined.

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.

17.3 Modes of failure

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

17.3.1 Instrumentation to warn against instability

Where concerns exist over slope stability, instrumentation should be installed to provide early warning of any movements. This will allow remedial measures to be undertaken before failure occurs.

17.3.2 Monitoring surface and sub-surface movements

Monitoring of ground surface movement in both horizontal and vertical planes should be carried out by field survey methods. The particular methods to be used should be determined according to the accuracy required.

17.3.3 Earth pressure measurements

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

17.4 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 could result in collapse of a jetty installation or quay wall with loss of expensive equipment and revenue. Similarly, blockage of a dredged channel could 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, applying partial factors described in NA to BS EN 1997-1:2004 to actions and material properties. However, BS 6031 indicates that designers should ensure that the risk of failure and consequences of failure have been adequately considered during the design, and points out that BS EN 1990 and BS EN 1997-1 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.

17.5 Slope profile

*NOTE 1 The required slope angles are obtained by the analytical methods referred to in **17.1.1** or by empirical methods. It might be desirable to choose angles flatter than those 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 profile is likely to be governed by considerations 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 **17.1**).

Where slopes are formed in layers of soil or rock of significantly differing characteristics, the slope angles should be varied to conform to the engineering behaviour of each formation. Slope angles can also 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 and a flatter slope in the zone affected by varying tidal levels and wave action can be adopted.

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.

NOTE 2 Typical underwater slopes for various soil types are given in Table 1.

Table 1 **Typical side slopes for various soil types: underwater slopes**

Soil type	Side slope	
	Still water	Active water
Rock	Nearly vertical	Nearly vertical
Stiff clay	45°	45°
Firm clay	40°	35°
Sandy clay	25°	15°
Coarse sand	20°	10°
Fine sand	15°	5°
Mud and silt	10° to 1°	5° or less

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 can 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 [14].

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 [15].

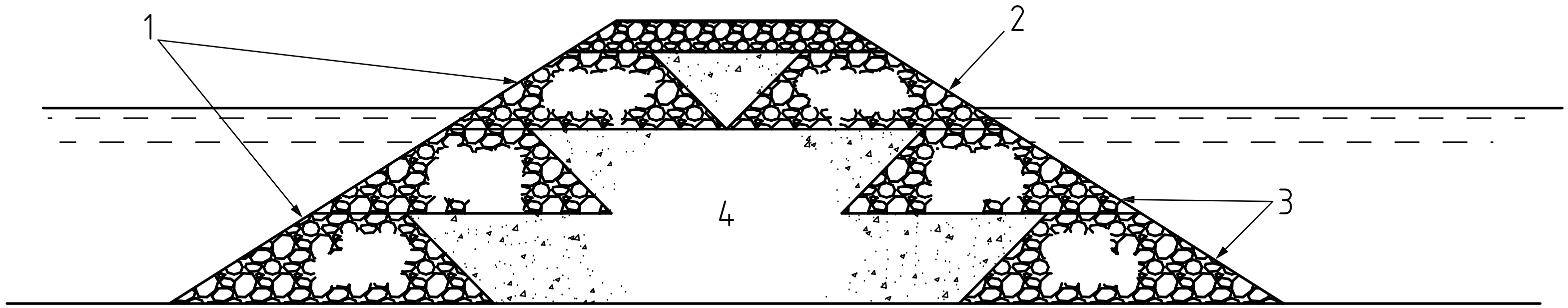
17.6 The effects of construction procedure

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, care should be taken to avoid excessive surcharge by placement of fill on or beyond the crest of the slope.

Embankments and training walls can be constructed by placement of fill on to weak soils below the seabed 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, care should be taken in the placing of the underwater fill to avoid local excessive surcharge. 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 20). Where the outer protective embankments are placed in successive stages as shown in Figure 20, 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.

Figure 20 **Embankment built in stages with core material protected by dumped stone**



Key

1 Large stone for protection against wave action and erosion

2 Stone placed by end tipping

3 Stone dumped from barge

4 Core material

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.

Suitable drainage measures should be taken to prevent accumulation of surface water or diversion of subsoil water on to 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.

17.7 Drainage

The earthworks designer should assess the requirement for pre-earthworks drainage as outlined in BS 6031.

17.7.1 Slopes in cuttings

17.7.1.1 Surface water control

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.

17.8 Monitoring stability

NOTE 1 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.

Periodic inspections should include the following observations.

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 6031:2009).

d) Seepage. Water carrying soil particles seeping from a slope is indicative of internal or seepage erosion (see BS 6031:2009).

e) Gullying. Channels eroded on a slope face indicate the need for protection against surface erosion.

Inspections should be made 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 2 Inspection of the position and inclination of pegs or beacon poles driven into a slope is a simple means of detecting gross deformations.

NOTE 3 Careful inspection of steeply cut temporary slopes for foundation excavations or trenches is required to ensure safe working conditions for operatives and to avoid damage to partly constructed works or existing structures adjacent to the excavation.

NOTE 4 Instrumentation, including apparatus for pore water pressure observation, suitable for installation in earthworks above high-water mark, 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.

17.9 Slope protection

17.9.1 General

Measures should be taken to ensure the overall stability of the slope against the modes of failure referred to in **17.3**, and to protect the surfaces of slopes against erosion by currents, waves, surface and subsoil water.

17.9.2 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.

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.

17.9.3 Above-water and partly submerged slopes

NOTE 1 Agencies causing erosion and instability of slopes within the influence of the rise and fall of tides and wave action are as follows:

*a) currents with associated vortex formation as described in **17.9.2**;*

b) scour by waves and wash from ships;

c) movement of soil particles due to the egress of water on a falling tide or retreat of waves;

d) egress of subsoil water;

e) action of winds;

f) action of surface water.

Where protection of the surfaces against any of the above influences is provided by means of a layer of rock or precast concrete blocks on the slopes, the effects of a varying water pressure on each side of the protective layer should be taken into account in the design.

NOTE 2 Protection against wave action and severe scouring conditions might require the provision of large blocks of rock or special precast concrete moulded 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.

The filter should be designed to prevent the movement of the finest particles from the soil under the influence of water flowing out of the slope on the falling tide or on retreat of waves. There are two ways of achieving this.

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

b) Alternatively, the filter may consist of other materials, including geotextile mesh or brushwood mats protected by a layer of crushed and graded stone and then the stone or precast concrete block armouring (see Figure 21).

NOTE 3 Guidance on the design of single or multi-layer filters is given in CIRIA Report C683 [15] and Soil mechanics in engineering practice *[16].*

NOTE 4 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.

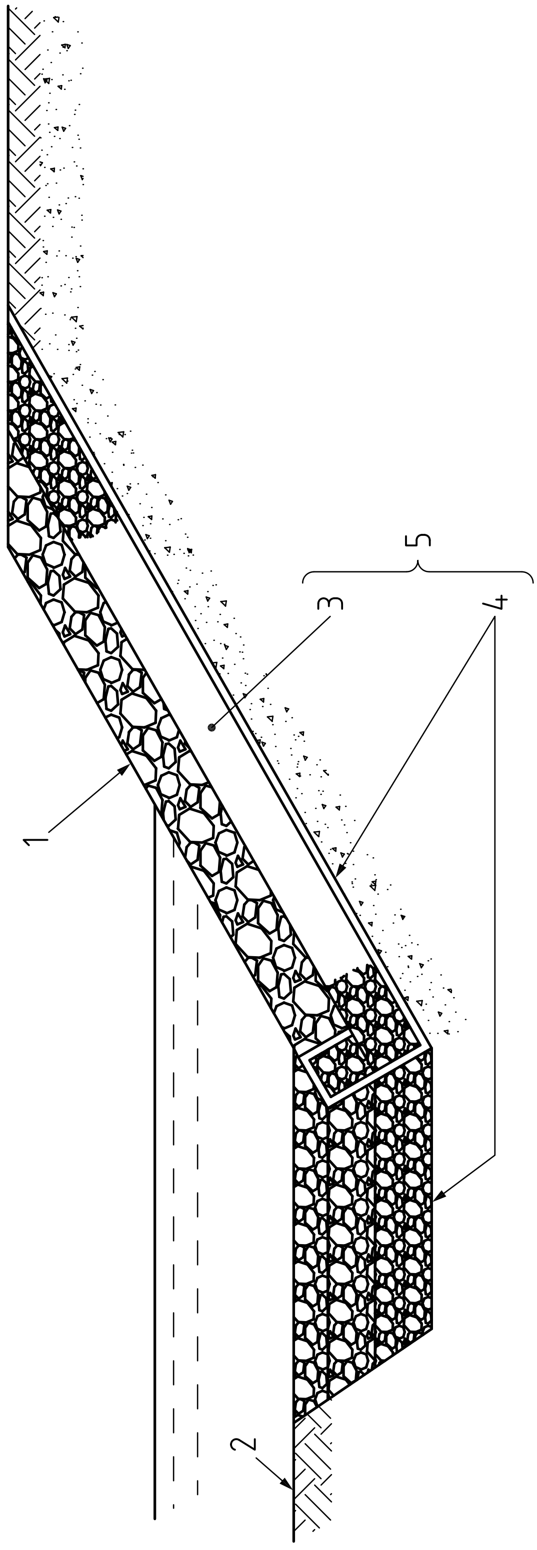
17.10 Maintenance of earthworks

*NOTE 1 Guidance on inspection and maintenance of above-water slopes is given in **17.9.3**.*

NOTE 2 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.

Maintenance of underwater slopes, other than dredging in areas of accretion, is difficult and not usually economic. However, periodic soundings should be taken in order to determine the trend of changes in seabed levels following the construction of maritime works and enable the appropriate remedial action to be taken before major instability develops.

Figure 21 **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 of gravel filter

17.11 Remedial works

NOTE 1 Guidance on treatment to remedy failure of above-water slopes is given in ***17.9.3****.*

In the case of submerged slopes, the type of remedial treatment adopted depends 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 would not be feasible if this prevented access to a berth. In such a case, the alternative of restraining the slip by anchored sheet piling would need to be considered. Toe weighting should be considered as a method for temporarily improving stability if the slide endangers important shoreside works.

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 of laying mattresses on the seabed to control further erosion around the affected area should be assessed.

NOTE 2 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.

18 Verification

Where dock basins and lock chambers are to be constructed in denatured excavations, where appropriate, trial excavations should be carried out to assess the suitability of a particular system of groundwater control, and to confirm estimates of pumping rates made on the basis of field permeability tests (see Clause **7**).

NOTE Trial dredging of soils or rocks is not always feasible or desirable, because certain types are dependent on the kind of dredger used and the manner in which it is operated. Trial dredging, though, might be desirable as a means of making full-scale experiments to determine the safe side slopes of dredged areas. The resulting excavation can also be used to study siltation rates.

Annex A (informative) Sampling and investigation procedures

A.1 General

Generally, site investigations have to be carried out by a reliable specialist. These range from government research-based organizations to specialized geotechnical contractors. Whoever undertakes the work, it is of prime importance that the work is supervised by well-qualified and experienced geotechnical engineers or engineering geologists, and that the field work in particular is undertaken by skilled drilling crews with well-maintained equipment.

Requirements for the qualification criteria of enterprises and personnel performing sampling and groundwater measurement services are given in BS EN ISO 22475-2. Requirements for the conformity assessment by third party control of enterprises, personnel and qualified operators performing specified parts of sampling and groundwater measurements in accordance with BS EN ISO 22475-1, and meeting the technical qualification criteria specified in BS 22475-2, are given in BS 22475-3.

Investigations for dredging projects normally involve work over water, and engineers and drillers who might be experienced on land-based operations do not always adapt to operations from floating craft with its many hazards.

It is advisable to obtain as much information as possible on the levels and the configuration of the deposits and on their origin. Deposits of complex structure need more detailed investigations than deposits having very regular profiles and structures. The amount of investigation also depends on the size of the area to be dredged.

A basic guide to sampling and investigation procedures is given in Table A.1.

A.2 Investigation procedures

Most of the sampling and investigation procedures have been derived from land-based techniques. Some techniques have been adapted and are under further development for underwater operation on the seabed using a diver or using remote control from the surface. Vibrocorers and gravity samplers are available as sampling methods for superficial layers of soft sediments. Recently, specific underwater samplers have been developed for dredging investigations. These include remote controlled devices either to carry out a cone penetration test or to take soil and rock samples by coring or penetration.

A.3 Self-elevating platforms

Most site investigations for dredging work are necessarily carried out from pontoons or vessels. However, the use of a self-elevating platform permits work to be carried out in a similar way to that for land-based investigation and considerably improves the quality of the information.

A.4 Test dredging

There might be some projects on which the complexity of the geology or other special circumstances warrants the use of test dredging or even makes test dredging desirable. In other cases the results of previous dredging contracts might be useful. In all cases, details of all relevant circumstances need to be provided, including all quantitative and qualitative information on the spoil and, where appropriate, a description of the dredger previously used. Great care is required by the employer in providing reliable information and by the contractor in interpreting the information.

Table A.1 **Sampling and investigation procedures** *(1 of 2)*

Rock or soil type	**Rotary drilling** A)	**Shell and auger boring**	**Underwater (sea bed) devices**	**Undisturbed sampling** B)	**Disturbed representative samples** B)	**Dynamic penetration tests** C)	**Cone penetration test**	**In-situ vane testing**
Rocks	Best method of obtaining core samples of intact rocks in in-situ conditions for examination and test	Not applicable	Useful for obtaining core samples of limited penetration	Cores represent undisturbed samples of intrinsic rock	Cutting in drill fluid may be used for identification of non-recovered layers	Used only in soft or weathered rocks and in corals	Not applicable	Not applicable
Boulders Cobbles	May be used to penetrate and obtain core samples	Chiselling required to penetrate strata	Not applicable	Cobbles retained as undisturbed samples	Not applicable	Not applicable	Not applicable	Not applicable

Table A.1 **Sampling and investigation procedures** *(2 of 2)*

Rock or soil type	Rotary drilling A)	Shell and auger boring	Underwater (sea bed) devices	Undisturbed sampling B)	Disturbed representative samples B)	Dynamic penetration tests C)	Cone penetration test	In-situ vane testing
Gravels	Not applicable	Method employed for site investigation in order to obtain representative and undisturbed samples and to carry out field (in-situ) tests	Not applicable	Not practicable to retain gravel as an undisturbed sample unless in cemented condition	Obtained from borings in tins or bags. Have to be "representative" (i.e. only from a single horizon or stratum). Essential for identification of various strata	Used with cone gives reasonable in-situ compactness estimate	Very difficult to penetrate coarse gravel	Not applicable
Sands	Useful for obtaining continuous core samples of intact soil representative of in-situ conditions and for examination and obtaining high quality samples for testing		Various devices are available to obtain representative samples, but are generally of limited penetration	Patent samplers available, difficult to sample in undisturbed condition		Useful for in-situ compactness estimate at the same time as sample is obtained	Useful method for determining in-situ properties and "hard" strata levels. In areas with very wide soil variation, might be useful to supplement borehole information	Not applicable
Silts				If cohesive in nature can use undisturbed core samplers for clay; otherwise see Sands		Can be used but interpret with care		Used for estimate of shear strength but great care needed in interpretation
Clays				Variety of undisturbed core samplers available				Very useful for shear strength evaluation in alluvial clays
Peats etc.	Not applicable			Variety of undisturbed core samplers available		Not applicable		Used for estimate of shear strength but great care needed in interpretation

A) 55 mm maximum or equivalent core size is commonly used in massive rocks, and a minimum of 70 mm is normally recommended for weak, weathered or fractured rocks. However, it is suggested that 100 mm to 150 mm will give improved results.

B) Care is needed in handling and preserving samples. Where possible, it is advisable to retain samples of rocks in conditions approximating to the in-situ state. Undisturbed and disturbed samples of soil, particularly core samples of cohesive materials, need to be protected from loss of natural moisture. Care in labelling samples is paramount.

C) Can only give an indication of in-situ density. Not to be used for identification.

Annex B (informative) In-situ and laboratory testing procedures for soil and rock

B.1 Testing procedures for soils

B.1.1 In-situ and laboratory testing

Table B.1 indicates the range of in-situ and laboratory tests that may be used to determine various soil properties or characteristics.

Table B.1 **In-situ and laboratory testing procedures for soils** *(1 of 2)*

Soil properties or characteristics	In-situ test	Laboratory test (site or central laboratory)	Reference
Particle size analysis	Not applicable	Sieving on granular soils Sedimentation on cohesive soils Combination on composite soils such as sandy clays A rough evaluation by comparison with standard soil samples by microscope or with grid counter	BS EN 1997-2 CEN ISO/TS 17892-4 A)
Particle shape	Not applicable	Comparison with standard samples and photographs	BS EN ISO 14688-1
Bulk density or in-situ density	Not applicable over water except for measurement of boulders and cobbles	The unit mass of soil as found in situ and expressed as the ratio between total mass and total volume of soil	BS EN 1997-2 CEN ISO/TS 17892-2 A)
Particle density (PD) of the solid particles	Not applicable	PD determined as the ratio between unit mass of solid particles and unit mass of particles	BS EN 1997-2 CEN ISO/TS 17892-3 A)
Compactness (in situ)	May employ several in-situ tests, e.g.: • standard penetration test (SPT) • static cone penetration test (CPT) • dynamic probing (DP)	Not applicable	BS EN 1997-2 BS EN ISO 22476
Moisture content	Radioactive meter method on land	Moisture content determination	BS EN 1997-2 CEN ISO/TS 17892-1 A)
Plasticity	Not applicable	Determination of liquid and plastic limits	BS EN 1997-2 CEN ISO/TS 17892-12 A)

Table B.1 **In-situ and laboratory testing procedures for soils** *(2 of 2)*

Soil properties or characteristics	In-situ test	Laboratory test (site or central laboratory)	Reference
Shear strength	May employ several in-situ tests, e.g.: • hand penetrometer or hand vane • vane tests • static cone penetrometer test (CPT) • dynamic probing (DP)	Laboratory vane Unconfined compression apparatus Triaxial compression	BS EN 1997-2 BS EN ISO 22476 CEN ISO/TS 17892-6 [A)] CEN ISO/TS 17892-7 [A)] CEN ISO/TS 17892-8 [A)] CEN ISO/TS 17892-9 [A)] CEN ISO/TS 17892-10 [A)]
Lime content	Not applicable	Measurement of carbonate content Visual test by applying dilute hydrochloric acid to specimen to indicate effervescence	DIN 18129
Organic content	Not applicable	Determination of organic content	BS EN 1997-2

A) In preparation.

Laboratory testing has to be undertaken on fresh samples, and tests are ideally carried out very soon after samples are obtained. However, since practical and logistical difficulties sometimes cause delay in samples being received at the laboratory, it is essential, where this might occur, that the simpler field tests (e.g. hand penetrometer or hand vane) are undertaken on site for later comparison with laboratory tests.

The in-situ compactness may be determined by using one of several tests including the standard penetration test (SPT).

An important value used in geotechnical work is the relative density of sands and gravels which has been developed by the use of this test. A commonly used scale in terms of N-values is given in Table B.2.

Table B.2 **Density of sands and gravels**

Term	SPT N-value blows per 300 mm penetration
Very loose	0 to 4
Loose	4 to 10
Medium dense	10 to 30
Dense	30 to 50
Very dense	over 50

B.1.2 Other testing

Especially in relation to environmental aspects, it might be necessary to carry out chemical tests on selected samples. The precise tests will be related to the circumstances of the project and the specific requirements.

B.2 Testing procedures for rocks

B.2.1 General

Table B.3 indicates the range of in-situ and laboratory tests that may be used to determine various rock properties or characteristics.

B.2.2 Engineering characteristics of rock

The current approach is to recommend consideration of both rock material and rock mass characteristics.

However, especially in relation to dredging works, where the rock to be excavated is underwater, it is usually necessary to describe and assess the rock by an examination of cores obtained from site investigations, which essentially indicate the nature of the rock material. Rock mass can only properly be examined by mapping natural and artificial exposures and excavations on land. Although it is valuable to examine land outcrops that are local to the dredging works, great care is needed, since isolated, natural exposures are not necessarily representative of the rock mass at the site of future excavation.

B.2.3 Rock material

B.2.3.1 General

Geological examination identifies the material and already provides valuable basic information concerning the rock type and its likely engineering characteristics. It is recommended that information about discontinuities and the weathered state of the rock is also recorded as part of the geological examination.

Information on weathering is extremely important and, rather than a classification, full details of the degree, extent and nature of weathering effects need to be included in the description of the rock material so that readers can appreciate their influence on engineering properties. Further recommendations concerning the weathered state of rocks are given in BS 5930.

B.2.3.2 Weathering of rock material

The recommended approach to description and classification of weathered rock for engineering purposes is given in BS 5930.

Table B.3 **In-situ and laboratory testing procedures for rocks** *(1 of 2)*

Name of test (see key below)	**Purpose of test**	**Remarks**	**Laboratory (L) or in-situ (S)**	**Reference**A)
Visual inspection	Assessment of rock mass	Indicates in-situ state of rock massB)	S or L	BS 5930
Thin section	Identification	Aid to mineral composition	L	—
Bulk density	Volume/mass relationship	Wet and dry test	L	BS EN 1997-2
Porosity	Measure of pores expressed as percentage ratio voids/total volume	To be calculated directly from wet and dry bulk density	L	BS EN 1997-2
Carbonate content	Useful for identification of limestone, chalks, etc.	—	L	BS EN 1997-2 ASTM D 3155
Surface hardness	Determination of hardness	Graded according to Moh's scale from 0 (talc) to 10 (diamond)	L	—
Uniaxial compression	Ultimate strength under uniaxial load	Test to be carried out on fully saturated samples. Dimensions of test piece and direction of stratification relevant to stress direction are to be stated. Recommend 1:2 length/diameter ratio for cylindrical specimens.	L	BS EN 1997-2
Brazil test	Tensile strength (derived from uniaxial testing)	As for uniaxial compression test except for length/diameter ratio recommendation	L	BS EN 1997-2
Point load test	Strength indication	Easy and fast test but needs to be matched with uniaxial compressive strength test	L	BS EN 1997-2
Triaxial compression test	Measures the strength of cylindrical rock specimens subjected to triaxial compression	In certain rock types (e.g. shales and porous limestone and chalk) and under certain conditions, the pore water pressure might influence the results. For such rock types, advanced triaxial test systems allowing for measuring pore water pressure and volumetric strains are necessary.	L	BS EN 1997-2A)
Standard penetration test (SPT)	Strength indication	Applies to corals and highly weathered rocks	S	BS EN 1997-2 BS EN ISO 22476
Seismic velocity	Indication of stratigraphy and fracturing of rock mass	Useful in extrapolating laboratory and field tests to rock mass behaviour	S	ISRM

Table B.3 **In-situ and laboratory testing procedures for rocks** *(2 of 2)*

Name of test (see key below)	**Purpose of test**	**Remarks**	**Laboratory (L) or in-situ (S)**	**Reference**[A)]
Ultrasonic velocity	Longitudinal velocity	Test on saturated core samples	L	ISRM
Static modulus of elasticity	Stress/strain rate	Gives an indication of brittleness	L	
Drillability	Assessment of the rock mass	Measurement of drilling parameters including penetration rate, torque, feed force fluid pressure, etc., and statement of drill specification and technique	S	—
Angularity	Determination of particle shape	May be by visual examination compared to standard specimens	L	BS EN 932-1

Key

Tests in **bold type** are considered to be the first priority for assessment of rock characteristics for dredging purposes; tests in *italic type* are of second priority. Tests in non-italic type can be restricted to a few representative samples for each soil type.

A) The International Society for Rock Mechanics (ISRM) has produced documents detailing standard methods for many of these tests [17].

B) Colour photography for record purposes can be very useful.

B.2.3.3 Strength of rock material

It is valuable to relate the strength of rock material obtained in the uniaxial compression test to a general scale of strength as shown in Table B.4.

Table B.4 **Strength of rock material**

Term for use in field or based on measurement	Definition for field use	Unconfined compressive strength MPa
Extremely weak	Can be indented by thumbnail. Gravel-sized lumps crush between finger and thumb.	0.6 to 1.0
Very weak	Crumbles under firm blows with point of geological hammer. Can be peeled by a pocket knife.	1 to 5
Weak	Can be peeled by a pocket knife with difficulty. Shallow indentations made by firm blow with the point of geological hammer.	5 to 25
Medium strong	Cannot be scraped with pocket knife. Can be fractured with a single firm blow of geological hammer.	25 to 50
Strong	Requires more than one blow of geological hammer to fracture.	50 to 100
Very strong	Requires many blows of geological hammer to fracture.	100 to 250
Extremely strong	Can only be chipped with geological hammer.	>250

The strength of a rock material determined in the uniaxial compression test is dependent on the moisture content of the specimen, anisotropy and the test procedure adopted.

For dredging assessment, the strength range within the descriptive term is rather large, and therefore it is very important that when giving descriptive terms the test results are also provided.

B.2.3.4 Fracture state

The state of the rock in situ is very important, and so it is essential that the drilling method and size employed are stated. In addition, in order to assess the soundness of the rock, various criteria may be used to indicate the fracture state of rock cores; these are the total core recovery (TCR), solid core recovery (SCR), fracture log and rock quality designation (RQD). All these need to be included in the log of the borehole.

The total core recovery is expressed as the length of the total amount of core recovered as a percentage of the length of core run.

The solid core recovery is defined as the length of core recovered as solid cylinders expressed as a percentage of the length of core run.

Solid core has a full diameter, uninterrupted by natural discontinuities, but not necessarily a full circumference, and is commonly measured along the core axis or other scan line.

A fracture log is a count of the number of natural fractures present over an arbitrary length, e.g. the number of natural fractures per metre of core run.

The determination of RQD is a quantitative measure of the fracture state of the rock. RQD is the sum length of all core pieces (100 mm or longer), measured along the centreline of the core, expressed as a percentage of core drilled.

Important aids, together with RQD determination, are colour photographs of the rock cores. It is also possible to use a ratio of field seismic velocity to the laboratory seismic velocity as a quantitative fracture index of the in-situ rock mass.

Annex C (informative) Properties of the ground – Physical characteristics of soil and rock

It might be desirable to make preliminary designs before full information is available concerning the geotechnical properties of the ground and any imported materials. In such cases the data in Table C.1 can be used, but for flexible structures, such as sheet-piled walls, deformation of the structure can lead to significant variations in earth pressures.

Table C.1 **Physical characteristics of soils and rocks**

General description of soil	**State of compaction or consolidation**	**Natural bulk density**		**Typical angle of soil shearing resistance in terms of effective stresses**	
		Moist	**Submerged**	**Active**	**Passive**
		kN/m^3	kN/m^3	degrees	degrees
Gravels	Loose	16.0	10.0	35	35
	Medium dense	16.0	10.0	38	37
	Dense	18.0	10.0	41	39
	Very dense	18.0	10.0	44	41
Sands, coarse or medium	Loose	16.5	10.0	30	30
	Medium dense	16.5	10.0	33	32
	Dense	18.5	11.5	36	33
	Very dense	18.5	11.5	39	34
Silts	—	16.0 to 18.0	—	24 to 27	—
Clayey silts	—	17.0	—	21	—
Silty clays	Normally-consolidated	15.0	—	15 to 18	—
	Over-consolidated	20.0	—	15 to 18	—
Glacial till	—	—	—	26 to 30	—
Peat	Unloaded	11.0	1.0	0	—
	After moderate loading	13.0	3.0	15	—
Granite	—	25.0 A)	—	—	—
Sandstone	—	22.0 A)	—	—	—
Basalts and dolerites	—	17.5 to 27.5	11.0 to 16.0	—	—
Shale	—	21.5 to 23.0	12.0 to 13.5	—	—
Stiff to hard marl	—	19.0 to 23.0	10.0 to 13.5	—	—
Limestone	—	27.0 A)	—	—	—
Chalk	—	9.5 to 20.0	3.0 to 10.0	—	—

A) Measured in the solid, i.e. not crushed or broken.

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