

BS 6349-2:2010

Incorporating Corrigendum No. 1



BSI Standards Publication

Maritime works –

Part 2: Code of practice for the design of
quay walls, jetties and dolphins

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ISBN 978 0 580 73243 0

ICS 47.020.99; 93.140

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

Committee reference CB/502

Draft for comment 09/30093195 DC

Publication history

First published January 1988

Second (present) edition, April 2010

Amendments issued since publication

Date	Text affected
November 2010	Table A.3, Serviceability limit states, the words "Favourable" and "Unfavourable" are transposed.

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Summary of pages

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Foreword

Publishing information

This part of BS 6349 is published by BSI and came into effect on 30 April 2010. 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

This part of BS 6349 supersedes BS 6349-2:1988, which is withdrawn.

Relationship with other publications

BS 6349 is published in the following parts:

- Part 1: *Code of practice for general criteria*;
- 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*.

Information about this document

The start and finish of text introduced or altered by Corrigendum No. 1 is indicated in the text by tags C1 C1.

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

- substantial reference to Eurocodes and other European standards published since the previous edition of BS 6349-2;
- some rationalization arising from the publication of BS 6349-8, which now deals with all issues associated with Ro-Ro ramps, linkspans and walkways;
- change of title from *Maritime structures* to *Maritime works* to reflect the expanded remit of the new Technical Committee that is responsible for this series. The titles of the other parts will be changed as revisions or amendments are published.

The availability of the completed Eurocode suite means that there is now guidance on the design of geotechnical structures and foundations based on limit state techniques, so there is now no need within this part of BS 6349 to draw attention to the need to interface limit state and working stress design techniques.

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 part of BS 6349 is expected to be able to justify any course of action that deviates from its recommendations.

It has been assumed in the preparation of this British Standard that the execution of its provisions will be entrusted to appropriately qualified and experienced people, for whose use it has been produced.

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.

1 Scope

This part of BS 6349 provides recommendations and guidance on the design of quay walls, jetties and dolphins.

2 Normative references

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

Standards publications

BS 4211, *Specification for permanently fixed ladders*

BS 4592 (all parts), *Industrial type flooring and stair treads*

BS 5395-1, *Stairs, ladders and walkways – Part 1: Code of practice for the design, construction and maintenance of straight stairs and winders*

BS 6031, *Code of practice for earthworks*

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

BS 6349-4, *Maritime structures – Part 4: Code of practice for design of fendering and mooring systems*

BS 6349-5:1991, *Maritime structures – Part 5: Code of practice for dredging and land reclamation*

BS 6349-8, *Maritime structures – Part 8: Code of practice for the design of Ro-Ro ramps, linkspans and walkways*

BS EN 1537, *Execution of special geotechnical work – Ground anchors*

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

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

BS EN 1991 (all parts), *Eurocode 1 – Actions on structures*

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

BS EN 1993 (all parts), *Eurocode 3 – Design of steel structures*

BS EN 1994 (all parts), *Eurocode 4 – Design of composite steel and concrete structures*

BS EN 1995 (all parts), *Eurocode 5 – Design of timber structures*

BS EN 1996 (all parts), *Eurocode 6 – Design of masonry structures*

BS EN 1997 (all parts), *Eurocode 7 – Geotechnical design*

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

BS EN 1999 (all parts), *Eurocode 9 – Design of aluminium structures*

BS EN 10025 (all parts), *Hot rolled products of structural steels*

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

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

BS EN 10248 (both parts), *Hot rolled sheet piling of non alloy steels*

BS EN 10249 (both parts), *Cold formed sheet piling of non alloy steels*
 BS EN 12063:1999, *Execution of special geotechnical work – Sheet pile walls*
 BS EN 12464-2, *Lighting of work places – Part 2: Outdoor work places*
 BS EN 12473, *General principles of cathodic protection in sea water*
 BS EN 13174, *Cathodic protection for harbour installations*
 BS EN ISO 14122 (all parts), *Safety of machinery – Permanent means of access to machinery*

Other publications

INSTITUTION OF CIVIL ENGINEERS. *ICE specification for piling and embedded retaining walls*. Second edition. London: Thomas Telford Ltd, 2007.

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this part of BS 6349, the terms and definitions given in BS 6349-1, BS EN 1990 and the following apply.

- 3.1.1 access (approach) trestle**
 bridge connecting a jetty head to the shore to provide vehicular or pedestrian access and/or support to pipes or conveyors
- 3.1.2 apron**
 area of open land adjacent to a berth
- 3.1.3 berthing beam**
 isolated piled structure with a continuous capping situated parallel to a berth and having a similar function to two or more berthing dolphins
- 3.1.4 berthing line**
 line of the face of the fenders or (where no fenders exist) the berth structure, in the undeflected position
- 3.1.5 breakwater berth**
 berth on the lee side of a breakwater
- 3.1.6 cope**
 top edge of a quay or jetty adjacent to a berth
- 3.1.7 dolphins**
- 3.1.7.1 dolphin**
 isolated structure or strongpoint used either to manoeuvre a vessel or to facilitate holding it in position at its berth
- 3.1.7.2 berthing (breasting) dolphin**
 dolphin designed to take the impact of a berthing vessel and to hold it when being pushed against a berth by wind or current

NOTE A berthing dolphin usually incorporates means for securing vessel spring lines.

- 3.1.7.3 mooring dolphin**
dolphin with bollards or hooks for securing a vessel and for preventing it from moving away from a berth due to wind or current action
NOTE A mooring dolphin is usually set back from the berthing line so that it cannot be struck by a berthing vessel and to improve the effectiveness of mooring lines.
- 3.1.7.4 warping (lead-in) dolphin**
dolphin provided at one end of a structure or at a change in direction of a berth to support vessels being manoeuvred between two adjacent positions
NOTE This is similar to a berthing dolphin.
- 3.1.8 embedded retaining wall**
retaining wall of thin cross-section that resists actions by bending
- 3.1.9 false deck**
structural slab overlaid with fill and surfacing
- 3.1.10 finger jetty**
comparatively narrow jetty that is often oriented perpendicular to the shore
- 3.1.11 gravity wall**
retaining wall of heavy cross-section that resists horizontal actions by means of dead weight and friction
NOTE The dead weight may be augmented by ground anchors.
- 3.1.12 hydraulic fill**
fill material which is transported to the reclamation area by pumping through pipelines and allowed to settle out under water
- 3.1.13 jetty**
structure providing a berth or berths at some distance from the shore
NOTE A jetty may be connected to the shore by an access trestle or causeway, or may be of the island type.
- 3.1.14 jetty head**
platform at the seaward end of a jetty or causeway
- 3.1.15 marginal berth (or quay)**
berth (or quay) parallel to the shore
- 3.1.16 quay**
berth structure backing on to the shore or reclaimed land
NOTE This is also known as a wharf.
- 3.1.17 pier**
structure projecting from the shore at which berths are provided
- 3.1.18 relieving platform**
platform built below deck level and supported on bearing piles, the principal function of which is to reduce lateral soil pressures over the portion of an embedded retaining wall below platform level.
NOTE A relieving platform is usually supported on the retaining wall.

3.1.19 Ro-Ro (roll-on/roll-off) ramp

fixed ramp or bridging structure that enables vehicles to pass between a Ro-Ro ship's vehicle decks and the shore

NOTE Adjustable bridging structures (known as linkspans) may be provided to accommodate relative level differences due to loading conditions of the ships and water levels.

3.1.20 soldier pile wall

wall consisting of vertical piles (king, master or soldier piles) driven at intervals, supporting intermediate horizontal elements (boarding, planks or lagging)

NOTE This is also known as a king pile wall.

3.1.21 suspended deck structure

quay or jetty comprising a deck supported on bearing piles

3.1.22 walkway

fixed structure providing pedestrian access to an isolated structure, such as a dolphin

3.2 Symbols

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

A_d	design value of an accidental action
A_{ED}	design value of seismic action ($A_{ED} = \gamma_I \times A_{EK}$)
A_{EK}	characteristic value of a seismic action
d_z	depth from ground level to intersection between the embedded wall and the lower surface of the zone included in the disturbing force
F_H	horizontal soil friction action
F_V	vertical soil friction action
F_1	soil action (similarly F_2 and F_3)
$G_{kj,inf}$	lower characteristic value of permanent action, j
$G_{kj,sup}$	upper characteristic value of permanent action, j
H_R	retained height of structure
l_e	effective length of anchorage
l_o	ineffective length of anchorage
M_{ST}	relevant moment for staggered points
M_S	mass of cellular structure and contained soil
P	relevant representative value of a pre-stressing action (see BS EN 1992 to BS EN 1996 and BS EN 1998 to BS EN 1999)
P_A	total active soil action per unit length of wall
P_P	passive soil action per unit length of wall
$Q_{c,k}$	characteristic value of the construction loads as defined in BS EN 1991-1-6
$Q_{k,1}$	characteristic value of the leading variable action, 1
$Q_{k,i}$	characteristic value of one of a number of accompanying variable actions, i
s	half the stagger height
$\gamma_{G,inf}$	partial factor for permanent actions, also accounting for mode; uncertainties and dimensional variations, for use with favourable loading conditions
$\gamma_{Gj,inf}$	partial factor for one (i.e. permanent action j) of a group of permanent actions that might be included in a combination, also accounting for mode; uncertainties and dimensional variations, for use with favourable loading conditions

$\gamma_{G,sup}$	partial factor for permanent actions, also accounting for mode; uncertainties and dimensional variations, for use with unfavourable loading conditions
$\gamma_{Gj,sup}$	partial factor for one (i.e. permanent action number j) of a group of permanent actions that might be included in a combination, also accounting for mode; uncertainties and dimensional variations, for use with unfavourable loading conditions
γ_Q	partial factor for variable actions, also accounting for mode; uncertainties and dimensional variations
$\gamma_{Q,1}$	partial factor for the lead variable action (i.e. variable action 1), also accounting for mode; uncertainties and dimensional variations
$\gamma_{Q,i}$	partial factor for the one additional variable action (i.e. variable action number i not being the lead variable action), also accounting for mode; uncertainties and dimensional variations
γ_P	partial factor for pre-stressing actions (see BS EN 1992 to BS EN 1996 and BS EN 1998 to BS EN 1999)
γ_I	importance factor for use in seismic assessments (see BS EN 1998)
δ	angle of wall friction
ϕ'	angle of shearing resistance in terms of effective stress (defined in BS EN 1997-1)
ψ	factor for the accompanying value of a variable action
ψ_0	factor for the combination value of a variable action
$\psi_{0,i}$	factor for the combination value of a specific variable action i
ψ_1	factor for the frequent value of a variable action
$\psi_{1,i}$	factor for the frequent value of a specific variable action i
ψ_2	factor for the quasi permanent value of a variable action
$\psi_{2,i}$	factor for the quasi permanent value of a specific variable action i

4 General design of berth structures

COMMENTARY ON CLAUSE 4

The detailed design of a particular structure cannot be considered in isolation from the overall planning, which is required to ensure that the structure meets its functional requirements. Planning matters are discussed in this clause in so far as they might influence the design.

Guidance on overall port planning is given in references [1–5].

4.1 General

4.1.1 Planning and risk assessment

Before the structure is designed, a planning process should be undertaken to determine the way in which the structure is to be used. As part of this process, the various parts of the structure should be zoned for different types of use, and the actions to be supported by each zone should be determined. A risk assessment should be performed to identify hazards and any actions that might result from these hazards, including any accidental actions.

Following the planning process and risk assessment, a design criteria document should be produced, which should define all the actions that are expected to be applied to all the zones of the structure. It should also define the strategy for operation and maintenance, including the time for which it is acceptable for the berths to be out of action while maintenance operations are taking place. This document should be agreed by the purchaser/owner/operator of the port and will form the basis of any assessment of structure performance in future.

NOTE If the risk assessment shows that a hazard can be mitigated by suitable management controls, then it might not be necessary to design the structure specifically to accommodate the actions arising from that hazard.

4.1.2 Dredging and filling

As part of the planning process, the designer should seek to minimize the environmental impact of dredging and filling operations.

NOTE One way of doing this is to make use of fill from other dredging operations being carried out in the port. Depending upon the suitability of the dredged material, the actions applied to berth structures can vary considerably. Some dredged material will be inferior in quality, and a balance needs to be struck between the environmental needs and the stability of the structure. These considerations can therefore have a significant effect on the design of the structure.

Maintenance dredging might become necessary because of the natural accretion of material or because of a build-up of material dropped between the ship and the berth during the handling of bulk cargo, such as coal. There might be benefits in providing a dredged depth greater than the minimum required, so that maintenance dredging with its disruption to operations is reduced. Account should be taken of the dredging tolerances which will be achieved during construction and during maintenance dredging (see 4.2.6).

4.1.3 Function of berth structure

A berth structure should be designed to accommodate a particular vessel or range of vessels and cargo handling operations, as defined in the design criteria document (see 4.1.1). Dolphins should be provided where necessary, e.g. at berths where the vessel length exceeds the length of quay, or at jetties with a loading platform.

NOTE 1 Dolphins can be used to resist the berthing and mooring actions, thereby permitting a lighter form of structure to be used for the quay or jetty platform itself. This is particularly applicable to bulk terminals serving oil tankers, very large crude carriers (VLCCs), liquid natural gas (LNG) carriers or other bulk vessels where maximum horizontal loads are severe and imposed vertical loads are light. Specific recommendations for dolphins are given in Clause 9 and BS 6349-4.

NOTE 2 The need for the berth to be contiguous with the shore, the apron width required, and the area of land required behind the berth are all matters which depend on the type of method of handling and storing the cargo. Bulk cargo might require extensive areas of land for storage, but this can be some distance inland with the bulk cargo transferred from berth to tank farm, silo, storage shed or open storage area by pipe or conveyor. Conversely, the area behind a general cargo or container berth is an integral part of the whole, and the movement of cargo from ship to transit shed or open storage needs to be direct and as short as possible: in general, a marginal type of berth satisfies these criteria. The need to provide an area of land behind a berth often involves reclamation and this can have a strong influence on the type of marginal structure to be adopted.

4.1.4 Access and safety

Safety should be taken into account in the design of all maritime structures, and appropriate safety measures should be put in place, including access by emergency services, escape routes from hazardous areas, and the provision of fire-fighting equipment (see 4.5.2 and 4.5.4) and life-saving equipment. The planning of such measures for new structures should take into account the safety measures already in place in the existing parts of the port, and any that are required by the risk assessment (4.1.1).

NOTE 1 Further guidance on safety aspects is given in Safety and health in ports [6] and COP 25 [7].

Depending upon the nature of the cargo to be handled, pollution prevention or mitigating equipment might be required. Where hazardous cargo is handled, such as in VLCC and oil and oil product terminals, LNG and LPG and chemical carrier terminals and similar bulk carrier berths, speed-of-approach monitoring aids and mooring line force measurement aids should be provided. In addition, where required, wave, current, wind and water level monitors should be provided. At all maritime terminals at least one tide gauge should be provided. Wave measuring equipment should be installed at exposed terminals.

Easy access to the loading platform and dolphins should be provided for normal operations (e.g. connection of loading arms, mooring line handling) as well as for inspection and maintenance of equipment.

NOTE 2 In some circumstances, it is not necessary to provide fixed access facilities to parts which require only irregular maintenance. If fixed access facilities are provided, they will also need to be maintained.

Where cold products are handled, cold splash protection should be provided where necessary for protection of the structural elements. At areas at risk of fire, passive fire protection measures might be required in addition to the active fire monitoring system.

NOTE 3 These can include vertical stanchions on the loading platform directly supporting product lines and heavy equipment such as the loading arms.

4.1.5 Berth orientation

Where there is freedom of choice as to the orientation of the berth, or where the berth is in an exposed position, the orientation should be chosen so that the predominant winds, waves and currents will have least effect on the operation of the berth, to maximize safety of berth operations. The orientation should further be selected such that the structure has least detrimental effect on the coastal or estuarial regime.

NOTE 1 A physical or mathematical model might be necessary to study these effects.

NOTE 2 The selected orientation of the berth can also be affected by an optimization between the capital costs of investment and costs of operational down-time.

Berths should not, in general, be broadside-on to strong prevailing winds and waves. In all cases the navigational and cargo handling aspects should be taken into account (see BS 6349-1).

When assessing the impact that waves will have at the berth location, the effects of period and direction should be taken into account (see BS 6349-1:2000, **31.4** and **31.5**).

The direction of the current at tidal locations will usually vary. The importance of the effects of current on navigation will depend on the strength of the current, the availability of tugs, the loading condition of the ships and whether movements are restricted by tide levels. Offsetting currents can help a ship to leave the berth but might be unacceptable if a fully laden ship has to berth against them. In cases where unacceptable currents cannot be avoided, they should be redirected by the use of training walls. The direction from which vessels will approach the berth should be taken into account. If they are always to lie alongside in the same direction (e.g. head inwards or head outwards), the configuration of mooring points and fixed or rail mounted cargo handling equipment may sometimes be refined.

The nature of the bed, the amount of solids in suspension in the water, existing current directions and velocities and the existing pattern of littoral drift all need to be examined and their sensitivity to the orientation of the structure studied. In addition to the effects on the overall regime, the local effects should be examined for unacceptable current changes, local scour or siltation of the seabed at the berth. Abrupt changes in profile might create sufficient disturbance to affect the structure and possibly navigation in the vicinity. These factors should all be taken into account when determining the orientation of the berth.

4.1.6 Berth geometry

NOTE The influence of services and fixed or rail-mounted cargo handling equipment on berth geometry is dealt with in **4.5**.

4.1.6.1 Berth length

The length of the berth is related to the length of the vessels that will use it, and should therefore take account of any changes in vessel size that might reasonably be expected during the lifetime of the berth. Where the berth is one of a number in a line, the expected mix of vessels using the line of berths should be taken into account.

Where vessels are to be berthed in line, the length allowed between vessels should take account of the method of berthing, the orientation of the berth to the prevailing wind and waves, and the degree of exposure of the berth. A minimum distance of 15 m between vessels is commonly adopted, but this should be increased as necessary according to the size of the vessels and the exposure of the berth. For fishing or pleasure craft, a berth length of 1.15 times the vessel length is desirable.

The length of berth structure that needs to be in contact with a vessel depends on the type of vessel and type of cargo to be handled. General cargo ships and many dry bulk cargo ships need a long berth face so that cargo handling equipment can have access to work any hold. Alternatively, a short berth face and fixed handling equipment may be provided, requiring the vessels to be moved along the berth. Bulk liquids and solids such as cement, which can be moved pneumatically by pipeline, are loaded and discharged from a fixed point, and the berth face need only be long enough to accommodate these.

The required length for berths for VLCCs, product and LNG/LPG carriers also depends on the manifold offsets on board, which dictate the position of the moored ship relative to the (un)loading platform.

NOTE A revision of BS 6349-1 is in preparation, and is expected to include further guidance on this issue.

4.1.6.2 Depth alongside

The depth of water required alongside the berth is related to the operating draught of the vessels that will use it, and should therefore take account of any changes in draught of the vessels that might reasonably be expected to use the berth within its lifetime, as well as the tidal range and, at exposed berths, the effects of wave action. Wash due to passing ships, wind action causing roll, and the effect upon draught of listing during loading/unloading should also be taken into account. If shipping patterns show that ships using the berth are never at full draught, a reduced depth might be acceptable. Allowances for any expected overdredge and underkeel clearance should be included in the design.

NOTE 1 To reduce the amount of dredging needed at the time of initial construction, it is common to design a berth for a greater depth of water than is initially required.

NOTE 2 A revision of BS 6349-1 is in preparation, and is expected to include further guidance on this issue.

An appropriate allowance should be made for both capital and maintenance dredging tolerances.

NOTE 3 Guidance is given on dredging tolerances in BS 6349-5.

Where appropriate, protection should be provided against scour caused by propellers and bow thrusters (4.3.5).

4.1.6.3 Cope and jetty level

The level of the cope along the berth face should normally be the same as either those of existing berths or the level of the working area behind the berth. With a virgin site, the level of the cope should be set on practical grounds.

NOTE 1 A low cope might be appropriate at berths exclusively used by small craft. For certain types of deep sea Ro-Ro berths, the cope level might need to be lower than normal. It is also possible that the existing cope level in a port has been found to be unsatisfactory for some reason.

The following factors should be taken into account when determining the optimum cope level:

- the risk of flooding and the effect and implications of such flooding;
- the effects of sea level changes due to global warming;
- the effect of the cope level on crane heights.

At cargo berths within an impounded dock, the ground surface should be at least 1.5 m above the working water level.

For a berth in an open harbour or exposed location, a statistical analysis should be prepared to determine the level and frequency of high water levels and wave heights.

NOTE 2 The application of joint probability methods for computing extreme sea levels is described in the article Estimates of extreme sea conditions – Final report [8].

The levels of berths for VLCCs and gas carriers should also be designed to optimize the effectiveness of mooring lines and to prevent any wave loading on structures supporting piping and equipment.

NOTE 3 Guidance on mooring line requirements is given in the OCIMF publication Mooring equipment guidelines [9].

4.1.6.4 Width of apron

COMMENTARY ON 4.1.6.4

No firm guidelines can be given on the width of berth aprons, which depends on the type and method of handling cargo. Whether the berth is for specialized or multi-purpose use will largely influence the apron width required. Modern general cargo berths with transit sheds on the berth require a distance between berth face and shed of 20 m to 50 m. For certain specialized cargo such as fish, apron widths may be reduced to minimize handling, depending on the handling methods. Passenger and naval berths will also have different apron width requirements. In the case of container terminals, the whole of the land area is usually open, and distances from the berth face to the rear boundary are often in the range of 300 m to 500 m.

Berths should be built with adequate back-up areas to provide storage for the volumes of cargo which are expected to be handled across the berth during its design working life.

The apron should be wide enough to allow cargo handling equipment to be operated efficiently and safely on it, and special attention should be given to the provision of areas required for turning vehicles and for access by emergency vehicles. This is of particular importance when designing container terminals.

4.1.6.5 Clearances at berth face

Careful consideration has to be given to the lines of the vessels using the berth. The flare of container ships or naval vessels can require larger clearances. The front legs of cranes, hose handling gear and other handling equipment should be set back to avoid being struck by an overhanging bow or stern or by flying bridges and to provide pedestrian access between bollards and crane structures. The range of track centres of container and bulk handling cranes is wide and, since most cranes are made to order, any convenient span can be adopted.

Dimensions for special types of bulk terminal such as those with radial loaders or unloaders should be individually assessed.

The maximum draught and degree of list of vessels berthed alongside should be taken into account when determining underwater clearances to the face of a structure.

Additional clearances should be allowed for bulbous bows where the angle between the cope line and the vessel's longitudinal axis is likely to exceed 7°.

NOTE Some information on approach angles is given in BS 6349-4.

Measures should also be taken to minimize the likelihood of ships striking structures in the spaces between individual fender units. This is particularly important where vessels might turn alongside the berth when there are currents that might delay or prolong the manoeuvre.

Maritime structures cannot normally be designed to withstand the very large vertical loads that can be imposed by a ship or other large

floating object trapped under or hung up on a projection during a rising or falling tide. Where practicable, to reduce the likelihood of these unusual and damaging vertical loads occurring, angular projections should be avoided, and any horizontal projection should be provided with a flare. Facing and deck units should be secured to the structure since, in general, gravity alone will not be sufficient to keep them in place.

4.2 Choice of structure

4.2.1 General

For any structure, a variety of different types should be compared and a choice made on the basis of the structure's functional requirements, including technical feasibility of the considered concept, capital costs, maintenance and or ease of construction. The typical designs given in this part of BS 6349 are not intended to exclude the use of alternative structural arrangements, which can include hybrids of two or more types of structure.

4.2.2 Types of structure

Maritime structures can be either solid or open-piled. An open-piled structure can be either rigid or flexible. Solid structures include all sheet and gravity walls with a solid vertical berthing face. These types of structure are most commonly used for marginal berths where fill material has to be retained, but they are also used for finger piers, jetties and dolphins.

Open structures have a suspended deck supported on piles. The structure can be either flexible with only vertical piles and without external horizontal restraint, or constructed more rigidly with raking piles or with struts to the shore. The degree of flexibility will depend on the overall configuration, framing and relative stiffness of members and their supports. A flexible type of structure might be unsuitable if it is to accommodate cranes or bulk handling equipment, especially in earthquake zones.

Flexible types of structure are also not preferred for loading platforms and trestle structures for liquid cargo berths, as the carrying piping might then become subject to significant horizontal differential deformations due to wind, wave and current loads on the substructure. The limiting values for these differential displacements depend on the piping system.

Many types of quay structure that retain the ground behind them yield slightly during or after construction, with the development of active soil pressure and passive resistance. Yielding can take the form of horizontal movement, or a horizontal movement combined with a forward rotation. These movements are in addition to natural settlements and will depend on the type of structure and the ground conditions. When selecting the type of structure to be adopted for a new quay, account should be taken of the effect of such movements on fixed superstructures that span from a support on the quay structure or from retained ground within the active wedge to another support on the ground remote from it, and also of the proximity of existing structures.

Changes to the existing maritime regime at the berth and at adjacent locations are likely to be smaller for open-piled than for solid structures, as they present less obstruction to current flow and waves. At sites where siltation is likely to occur, a solid structure can increase current velocities, thereby reducing the volume of material deposited alongside, but possibly causing adverse effects elsewhere. However, in areas of high siltation, open-piled structures can lead to a build-up of silt behind and within the structure, which can result in slumping of silt into the berth and action(s) on the structure. A solid structure can cause unacceptable disturbances for a ship at berth, due to reflected waves or deflected currents, but this can be reduced by the provision of perforations or a partly open face. Where open structures are used for marginal berths, the back-of-berth fill is usually retained behind a revetted slope. This slope can be used to absorb wave energy and reduce reflection.

The spring energy of a flexible structure can be used to absorb some or all of the energy of a berthing vessel (see BS 6349-4).

Whatever the type of structure, its design should be sufficiently tolerant for it to accommodate local variations in site conditions that might be encountered during construction.

4.2.3 Seabed conditions

The geotechnical data and bathymetric survey should be studied together to determine suitable founding levels for different types of structure and to decide whether dredging is necessary.

NOTE An increasingly important consideration is the effect of any dredging or reclamation on the local environment where the proposed maritime structure is to be built. Attention is drawn to the statutory requirement in the UK to carry out an impact assessment of all proposed maritime constructions.

4.2.4 Local construction materials

A study of the locally available natural materials for construction should be carried out. The availability of rock or general fill within a short haul distance can have a strong influence on the choice of structure.

NOTE For example, if fill is not easily available, it might be preferable to construct a berth as a suspended jetty rather than a quay wall.

4.2.5 Method of construction

The method and sequence of construction and availability of major construction plant can determine the type of structure finally selected. The maximum use of prefabricated sections may be adopted to achieve an early completion date, but the size and weight of units have to be within the handling capacity of the available plant. When the facility is part of a large project, a design involving the mobilization of specialized equipment and the establishment of a large fabrication plant for the project as a whole might have advantages. Construction from on shore by the creation of a temporary bund, a construction which works progressively forward from the shoreline end over end, or the utilization of jack-up plant, might be necessary if the weather conditions are too rough for the use of floating plant, either throughout the year or seasonally.

NOTE Where the seabed soil is weak, jack-up plant might be at risk of collapse.

The type of structure and possible influence on the construction equipment can have a large influence on the construction schedule in view of sensitivities to factors such as weather downtime and the availability of contractors. On the other hand, a large level of modularization often offers significant advantage to the project execution. This should be taken into account in the selection of the structure.

When construction requires specialized plant, the same plant might be required to carry out remedial works. The availability of the plant should be taken into account when a design includes planned maintenance.

4.2.6 Construction difficulties

The choice of structure should make allowance for the fact that the structure will generally be constructed over water from temporary working platforms. Fast currents and wave action will increase the difficulty of accurate construction.

The tolerances normal in building work on land often cannot be achieved in these conditions. In particular, a pile driven over water is seldom supported over its full length and, although it might be correctly located on the seabed, the head might deviate from the correct position. Deck units to be placed on piles should be designed to accommodate the permissible deviation in the piles.

Excavation on land can be carefully controlled, but under water it has to be executed by dredger or occasionally by diver. Dredgers will leave an undulating or stepped surface on which a layer of fine material can be deposited. Overdredging tolerances (typically 0.3 m in sand, silt and soft clay) can often be exceeded, and pockets can fill with soft material. The unevenness of cut usually obtained, and the possibility of the layers below the surface layer forming a potential failure plane under foundations, should be taken into account. A layer of gravel or rubble may be placed on the seabed prior to filling in order to prevent the formation of such a failure plane below fill placed on the surface layer. Temporary slopes on which soft material might build up should be formed with an inclination opposite to that of the potential failure planes in the soil.

When precast units are to be placed on a gravel bedding layer on the seabed, the final levelling and preparation of the bedding has to be carried out by divers, often with poor visibility. As far as possible the structure should be chosen to minimize and simplify any necessary divers' work, and to enable it to be carried out in a safe manner.

The choice of structure should take into account the need to protect the works during construction against damage caused by floating craft, waves and currents.

4.3 General site considerations

4.3.1 Settlement

The effects of long-term settlement on port operations should be taken into account. In general, only small amounts of settlement will be acceptable in quays and jetties, unless measures are taken to minimize the effect of large settlements on operations when the facility is commissioned.

At sites where highly compressible materials exist, it is important that the effects of piling, dredging, filling and future transient loads on the reclaimed ground are fully investigated during design, so that the necessary measures can be taken to ensure stability of the ground adjacent to and beneath the quay at all stages of construction and during the design life of the facility.

4.3.2 Fill material

Fill placed behind quays, in preference, should be of free-draining granular material. This will ensure that maximum natural consolidation is achieved in the submerged zone, where additional compaction might not be required. Above water level, the fill may be compacted by conventional means, and the effects of heavy compaction, which is generally required at ports, should be taken into account. Such effects can include increased lateral soil pressures on the wall resulting in additional actions on the wall or anchorage, or in additional seaward deflections. Special compaction methods can be necessary for the submerged fill if it is essential to reduce settlements.

Where hydraulic fill is placed behind retaining walls, care should be taken to ensure that pockets of soft material do not form next to the wall.

Where quay walls are built on rock, with a rock face behind, the total lateral pressure on the back of the wall may sometimes be reduced by replacing the lower part of the fill with in-situ concrete. The possibility of the wall subsequently moving away from the concrete, allowing water to enter the space between, should be taken into account.

NOTE Guidance on fill materials is given in BS 6349-5:1991, 8.3.2.

4.3.3 Drainage and filters

Drainage is often required behind quay walls to minimize tidal lag (see BS 6349-1:2000, 51.5). This has traditionally taken the form of weepholes or flap-valves. Some types of quay wall allow drainage through the wall at open joints. These should be provided with a suitable filter to prevent loss of fill material.

The use of weepholes is not encouraged, but where their use is unavoidable, they should be designed to prevent loss of fill. This can be achieved by using vertical slots whose width is less than the smallest size of stone in the adjacent filter layer. Design of the weepholes should take into account the possibility of blockage by marine growth or by migrating sand/fill material fines.

Flap-valves should be provided in preference to weepholes, to avoid blockage. The valves should be fixed just above low water level to allow maintenance. They should be connected to a series of drains constructed behind the wall to reduce tidal lag.

A method should be provided for measuring the tidal lag during the operational life of the quay.

NOTE 1 Failure to maintain a drainage system leads to a risk of accumulation of silt and detritus. Such accumulation can lead to significant variations from the design tidal lag. A suitable means of measuring tidal lag is to install a series of piezometers.

Quay superstructures containing fill material, such as relieving platforms and some types of gravity wall, should be provided with drains, such as ball valve sluices, to reduce excess hydrostatic pressures. The drains should be located above low water level to allow maintenance, and in such a way that they cannot be damaged by vessels or floating debris.

Drainage of backfill may be achieved using pipework, gravel drains, drain fabric or a rubble backfill. Rubble is often used behind mass concrete walls to reduce lateral pressures on the wall, but for other types of wall it might be unacceptable, owing to the high contact pressures which can occur. A filter should be provided between the rubble and the general fill. A gravel backfill may be used instead of rubble: this should be designed as a filter to prevent general fill leaching into it.

NOTE 2 Guidance on the design of filters is given in the CIRIA Rock manual [10].

4.3.4 Wave pressures on walls

Quay walls are usually built in sheltered waters. However, sheltered harbours can experience long period waves. In addition, passing vessels generate waves, although the period of such waves is generally too short to apply significant loads to a wall.

The height, length and angle of approach of waves should be taken into account when assessing the total hydrostatic pressure for which the wall is to be designed: drawdown in the wave trough is usually more important than pressure from the wave crest. At impermeable structures, the tidal lag should be increased to at least half the wave height to represent a wave trough where a standing wave can occur. At impermeable structures with permeable backing, the effect of wave action gradually building up water levels in the backfill should be taken into account. The effect of waves overtopping structures, such as breakwater quays, should also be taken into account in relation to a possible build-up of water levels behind the wall face.

4.3.5 Scour protection

The effects of ships' propellers and bow-thrusters, waves and currents on the stability of the seabed and any underwater slopes near structures should be taken into account. Ships with variable-pitch propellers, particularly ferries, can cause worse scour than those with constant-pitch propellers. Where scour is considered to be likely, protection, such as a rubble anti-scour apron on the seabed, should be provided in front of quay walls and particularly at berths where vessels will generally berth in the same position.

The size of rock protection for underwater slopes should be not less than that needed to resist the wash of propellers and bow-thrusters; it might be necessary to provide a filter between the rubble and the underlying material.

The effect on durability of the structure of natural bed material and small rock being disturbed by bow thrusters and thrown against the structure should be taken into account.

NOTE Further guidance on scour protection is given in the CIRIA Rock manual [10], which includes some guidance on the assessment of the effect of jet propeller wash.

4.3.6 Impounded systems

At locations subject to a large tidal range, an enclosed system with access through locks may be provided in order to reduce the water level variations at the quay. The level may either be maintained above outside high water levels, by means of pumping, or may vary, for example, allowing for opening of the gates at high tides with a gradual fall in level between high waters due to the spring/neap range, leakage and locking losses. In the latter case, an estimate should be made of the lowest probable water level within the system during operating conditions, and this should be used in earth pressure calculations for the normal design situations (see 5.2.2.2).

The water level within an enclosed system can, however, be reduced below the lowest probable normal water level due to deliberate drawdown, accidental damage to the lock gate, sabotage or possible abandonment. The lowest water level on complete drawdown after accidental damage to gates should, in the absence of special provisions for arresting the flow, be taken as mean low water springs (MLWS).

The probability of an accidental drawdown occurring will depend on the number of gates at the site, whether spare gates are available and, if so, how readily they can be brought into service, and on the water area of the system in relation to the dimensions of the gates.

NOTE Guidance on lock gates is given in BS 6349-3.

In assessing the risk that can be accepted, the consequences of drawdown should also be taken into account. These can include not only the need to repair quay walls and other port structures, but also the possibility of the entire impounded basin being made unusable for long periods of time. Pore water pressure conditions resulting from drawdown will depend on the potential rate of drawdown (based on the area of the enclosed basin relative to the possible rate of outflow) and the soil conditions.

4.3.7 Actions caused by construction operations

Most quays and jetties will be subjected to actions applied by construction plant before completion of the structure. One example of such actions is heavy plant passing along the top of filled ground behind quays: a uniformly distributed load of at least 10 kN/m² may be used to allow for this. Another common example is the use of bearing piles with temporary or incomplete decking to support plant: the nature of these actions is generally more critical and the particular items of plant to be used should be chosen accordingly.

NOTE Guidance on construction actions is given in BS EN 1991-1-6.

For the construction of typical two-stage precast elements (e.g. jetty cross-heads or precast deck slabs) the staged construction actions (fresh plastic concrete) should be assessed, and the resulting stresses/deformations should be combined with the stresses/deformations from imposed actions in the final operational situation.

All maritime structures, with the exception of those built in the dry, will be exposed during construction to actions by currents, waves and wind. The effect of these actions on piles that are not restrained or anchored during construction should be taken into account when assessing the size of the piles.

Actions on retaining walls from hydraulic fill below the level of any drains through the wall may be equivalent to actions due to a fluid having the same density as the pumped mixture of water and suspended soils, until drainage and settlement has effectively taken place. If the structure is deflected by hydraulic fill, it might not be able to yield sufficiently to allow the subsequent development of active pressures. The permanent design might then have to be based on an at-rest earth pressure.

Overall stability during construction should be assessed, as well as the strength of parts of the structure, and mitigating measures should be taken where necessary.

4.3.8 Earthquakes

Designs to resist earthquakes should be in accordance with BS EN 1998.

NOTE Detailed guidance on seismic-resistant design is given in Recommendations of the Committee for Waterfront Structures [11], Earthquake-resistant design [12] and Recommended practice for planning, designing and constructing fixed offshore platforms [13–14].

4.3.9 Paving and surface water drainage

Paving on areas of fill should be designed in accordance with the recommendations given in BS 6349-1:2000, Clause 63.

The upper surfaces of all quay aprons and jetties should be designed to allow rainwater and spray to drain away. In many cases it will be sufficient to construct the upper surface with a suitable crossfall towards the edge of the structure. The crossfall may be as high as 1:40 where differential settlement is to be anticipated, and otherwise should be typically between 1:60 and 1:100, depending on type of surface, width of apron and usage.

Where drainage at the quay face could cause pollution of the sea, or where the apron is very wide, surface water may be directed to gullies or channels provided in the apron. These may be connected to oil interceptors. Kerbed areas will usually be required to contain oil and other hazardous spillages, which may then be drained to a tank provided for this purpose.

Drainage should be provided to wheel slots in flush rail tracks, and to all ducts, trenches, galleries and connection boxes for services.

4.3.10 Actions from ice

All maritime structures should be assessed for ice events and if necessary designed to resist or accommodate the actions imposed by ice.

NOTE Guidance is given in Planning and design of ports and marine terminals [3] and Ice engineering design of marine piling and piers [15].

4.3.11 Detailing

Simplicity of construction and robustness of design is generally preferable to the reduction of quantities of materials in the structure by complicated details. If in-situ concrete is to be used within the tidal zone, members should be detailed so that the concrete can be easily placed and effectively compacted. For reinforced concrete work in the tidal zone, it is generally better to use precast units with the minimum of in-situ concrete connections.

4.4 Durability

COMMENTARY ON 4.4

Quays, jetties and dolphins are required to survive in an aggressive environment, preferably with little or no maintenance. To achieve their design life with the minimum of maintenance, it is important that all structures are designed for durability. The materials that are needed to achieve durability are described in BS 6349-1:2000, Section 7.

The design working life of a structure is chosen to represent its intended useful life. For guidance on the design working life, see BS 6349-1:2000, Clause 16.

4.4.1 Materials

COMMENTARY ON 4.4.1

In the UK, it would be unusual to have a site where the choice of materials was limited due to lack of availability or poor access. However, in overseas projects there might be a strong case for using locally available indigenous materials and using technology which is within the capability of the local plant and labour. See also BS 6349-1:2000, Section 7.

Materials should be chosen on the basis of their availability, quality and suitability for the particular project.

4.4.2 Corrosion and crack control

4.4.2.1 General

Protective measures against corrosion and crack control should be taken according to the corrosion conditions of the particular exposure zone. There are four general exposure zones:

- embedded;
- continuously immersed;
- intertidal and splash;
- atmospheric.

NOTE 1 BS EN 1993-5 gives a classification of corrosion zones for steel embedded retaining walls or steel piles.

These zones should be identified for all sections of the structure.

NOTE 2 BS 6349-1:2000, Clause 58 gives guidance on the specification of concrete in different exposure zones. BS EN ISO 12944-2 gives information on the types of painting systems available to protect against corrosion. Corrosion can also be controlled by cathodic protection. Refer to BS EN 13174 and BS EN 12473.

4.4.2.2 Steel corrosion

Steel corrosion rates should be estimated in accordance with BS EN 1993-5. However, in addition an assessment of the likelihood of microbiologically induced corrosion (MIC) should be made. If it is identified that MIC is likely to be prevalent, a cathodic protection system in accordance with BS EN 13174 and BS EN 12473 should be installed.

NOTE 1 The danger of corrosion is usually greatest in the intertidal and zones of high attack (this term is defined in BS EN 1993-5) where alternate wetting and drying of the surface occurs. Under some conditions, corrosion in the two zones of high attack are more severe than in the intertidal zone, and the three zones might need to be assessed separately. The intertidal and two zones of high attack may also be considered as

three separate zones when estimating the corrosion rates of structural steel. However, it is generally not practicable to vary the section sizes or to provide different protective systems within the zone.

NOTE 2 MIC was previously known as “accelerated low water corrosion” (ALWC) or “concentrated corrosion”. Since its cause is now known to be microbiological, it is now considered that the term “MIC” is preferable to “ALWC”.

4.4.2.3 Concrete and crack control

Concrete and crack control should be in accordance with BS 6349-1.

4.4.3 Maintenance

COMMENTARY ON 4.4.3

Because most parts of a maritime structure are underwater or underground, maintenance inspections are very difficult. Intertidal zones are exposed for short periods of time and inspection is impeded by marine growth and oily deposits on the structure. Underwater zones have to be inspected by a diver and, as well as the problem of marine growth, there might be limited visibility and problems of communication between the diver and the engineer. Even inspection of areas above the tidal zone, such as the soffit of a suspended slab, can be difficult if they can only be reached from a boat or floating platform at a particular state of the tide.

Further guidance on maintenance and inspection requirements for marine structures (in particular oil and gas jetties) is given in the Jetty maintenance and inspection guide [16].

The durability of a structure should take account of the effects of the agreed operation and maintenance strategy (see 4.1.1). Where regular maintenance is unlikely to be provided, the structure will need to be more durable.

The design should be appropriate to the maintenance capability of the operator. Where this capability is not established, it should be assumed that there will be no regular maintenance of the structure except for those areas easily accessible from land.

4.5 Services and equipment

4.5.1 Structural considerations

Arrangements for the provision of services and equipment on the berth should be finalized at the earliest possible stage. The support system for gantry cranes and other mechanical handling plant might dictate the geometry of the berth structure. Account should be taken of the differential movement of crane rail tracks. If a change in track centres is possible, caused, for example, by wall movements or lateral wheel loads, a strut/tie member should usually be provided between the track foundations.

Differential vertical movement at structural joints may be accommodated by providing a short transition support to the rail. In critical cases, provision for future adjustment of rail levels should be built into the track. The depth required for flush fixing of the rail track in the deck is also significant when choosing between different forms of construction. The recommendations given in BS 6349-1:2000, Clause 64 should be taken into account in the design of crane rail tracks. Trenches for power supplies to heavy equipment might be required.

Services can be supported above the untrafficked areas of jetty decks, but at a quay they should either be slung under the deck or accommodated within its structural depth. Because they would be exposed to a corrosive environment under the deck, it is more usual to put them within the body of the structure. Where there are a number of services, the depth required can be considerable as it is inevitable that the service lines will have to cross. These cross-over points and the outlet points can influence the structural design. A services gallery, with full height head-room for access, is often an acceptable solution.

4.5.2 Services

NOTE 1 See also 4.1.4.

Depending on the berth usage, provision should be made for some or all of the following services at the berth face:

- potable water;
- water or foam for fire-fighting (see also 4.5.3 and 4.5.4);
- chilled water;
- steam;
- bunkering oil (fuel oil, marine gas oil, marine diesel oil);
- compressed air;
- electrical power;
- communications cables;
- instrumentation;
- nitrogen;
- water treatment liquids;
- sewerage;
- specialist gases.

Fresh water supply is commonly arranged at metered hydrant pits spaced at 50 m to 100 m intervals along the cope.

Water should also be available for fire-fighting purposes. The system should be designed in accordance with the emergency response philosophy for the structure. Fresh water does least damage to cargo, but sea water can be used instead. One method is to lay a dry main system, which is left empty, to be charged with salt water in an emergency, by either a fixed or a mobile pump installation. For a more effective system, pressure-charged mains and automatic pumps for instant use may be provided. This shore-based fire emergency system may be augmented by providing fire-fighting equipment on tugs.

At some ports there is a demand for the bunkering of ships. The alternative to using a fuelling barge or setting aside a special fuel berth is to provide a fuel supply at the berth. According to the classes of vessel expected, fuel oil, marine gas oil, marine diesel oil and intermediates might be required. Fuel hydrants served by buried pipelines should be provided at convenient locations. Blending valves might be needed, and maximum and minimum supplying rates should be specified.

Pits and ducts might have to accommodate trace heating to prevent solidification of some liquids. The provision of electrical outlets for one or more systems of electrical power might be required at all berths.

Ship-to-shore telecommunications points should usually be provided at each berth, preferably near either end to be conveniently placed for the ships' superstructures. The other services listed above are required at only a few berths, such as those used for fitting-out vessels.

Allowance should be made in service runs for differential movement of sections of a structure or between the structure and the fill, by the provision of articulated or sliding joints.

NOTE 2 Guidance on earthing and bonding, and on fire-fighting equipment, at marine oil terminals is given in the International safety guide for oil tankers and terminals [17].

4.5.3 Equipment

Provision should be made for mooring devices, such as bollards, capstans, fairleads and mooring hooks, and for life-saving equipment (see also 11.9) and safety ladders (see also 11.5). Where necessary, the following facilities should also be provided:

- a) cranes and mechanical handling equipment;
- b) crane or railway tracks/wheel stops or buffers/storm anchors;
- c) area lighting;
- d) navigation lights;
- e) fire-fighting equipment (foam and/or water) (see also 4.5.2 and 4.5.4);
- f) mooring rings and/or posts for small craft;
- g) safety railing/kerbs;
- h) access stairways (see also 11.4), including hydraulic gangway towers where appropriate;
- i) inspection and access openings;
- j) cathodic protection (impressed current transformers);
- k) vessel approach aids.

NOTE Minimum equipment requirements for berth structures are given in COP 25 [7], and in Safety and health in ports [6]. For oil and gas facilities, requirements for mooring equipment are given in the ICE specification for piling and embedded retaining walls.

4.5.4 Fire-fighting

Appropriate fire-fighting provisions, including monitors and detection systems and other safety systems, should be provided. These should be remotely activated if not easily accessible, and should be in accordance with an agreed emergency response plan.

5 Design situations for quays and jetties

NOTE The terminology used in this clause is in accordance with that of the relevant Eurocodes.

5.1 General

The design of quay walls and jetties, including earth retaining structures, foundations and suspended decks, should be determined using the limit state design techniques set out in the suite of Eurocodes (i.e. BS EN 1990 to BS EN 1999), with partial factors obtained from one of two sources:

- as defined in Annex A of the present part of BS 6349; or
- derived from the rules for development of partial factors given in BS EN 1990, provided that a suitable statistical assessment has been made.

A range of design situations should be examined and the sensitivity of the design to changes in design situations checked. The selected design situations should be sufficiently severe and varied so as to encompass all conditions that can reasonably be foreseen to occur during the execution and use of the structure. The probable mode and consequences of failure of the structure should be ascertained.

NOTE In some cases of certain types of action, such as wave actions, ship berthing actions, mooring actions and ship accidental impact actions, BS EN 1990 and the national application documents do not provide guidance. Annex A has been developed to fill this gap as far as possible.

5.2 Actions

5.2.1 General

Most actions in relation to maritime structures should be determined in accordance with BS 6349-1:2000, Section 5. Actions for fendering and mooring systems should be determined in accordance with BS 6349-4.

NOTE Actions can be grouped under the general headings given in Annex B for the initial design but it might be necessary to consider the actions within a group separately when they are unusual or of particular significance.

5.2.2 Design situations for overall stability

5.2.2.1 General

The actions described in 5.2.2.2 to 5.2.2.4 are not exhaustive, and any other critical actions which might occur should also be analysed. Although some combinations of actions are mutually exclusive, the probability of two or more large actions being applied to the structure simultaneously should be assessed. Depending on the consequences of failure, in most cases, it will not be practicable to design for the simultaneous application of all possible extreme actions. However, where there is a very low probability that two large actions will occur simultaneously, suitable partial load factors and combination factors should be determined in accordance with Annex A, which will allow for the low probability of such combinations occurring.

5.2.2.2 Persistent design situations

The persistent design situation refers to any combination of actions that might reasonably be expected to occur during the design life of the structure (referred to in the Eurocodes as the “reference period”), and which is associated with normal operating conditions. This should include any foreseeable modifications to the structure, earthworks, paving, storage patterns, handling equipment or dredged depth.

The maximum persistent value of each type of action described in Annex B should be considered in combination, using the partial factors in Annex A.

NOTE Examples of persistent design situations are:

- a) *overdredging of seabed within specified tolerances;*
- b) *deepening of the seabed due to scour;*
- c) *increase in hydrostatic head due to drawdown in an impounded basin occurring during planned inspections at intervals not exceeding 1 year;*
- d) *water levels in the tide range mean high water springs (MHWS) to mean low water springs (MLWS);*
- e) *environmental actions that generally have a return period of 1 year or that impose restrictions on port operations, but excluding earthquakes and tsunamis;*
- f) *normal berthing operations as described in BS 6349-4;*
- g) *actions due to containers, using diversification factors for stacks more than one container high;*
- h) *actions caused by normal port traffic use.*

5.2.2.3 Transient design situations

Transient design situations refer to any combination of actions that might be expected to occur during a much shorter period than the design working life of the structure, associated with the most severe credible actions that could physically be applied, excluding accidental actions, such as that due to an uncontrolled berthing.

The likelihood of more than one transient action occurring at any time should be assessed. The secondary effects of such an action, e.g. the effect on soil properties of earthquakes or flooding, should be taken into account, along with the possibility of long-term changes in conditions, such as deterioration of drainage systems, which could apply during application of the most credible actions.

NOTE Examples of transient design situations are:

- a) *an increase in the hydrostatic head due to drawdown in an impounded basin occurring as a result of occasional inspections or accidental damage to gates;*
- b) *astronomical water levels outside the tide range for persistent design situations;*
- c) *environmental actions with a return period equal to the reference period for the structure;*
- d) *abnormal berthing operations, as described in BS 6349-4 (not to be confused with an uncontrolled berthing operation);*
- e) *loads due to containers without the use of diversification factors;*
- f) *earthquakes and tsunamis (see also 4.3.8);*
- g) *temporary actions during construction.*

5.2.2.4 Accidental design situations

Design situations involving accidental actions should be assessed in accordance with BS EN 1990, also taking into account the requirements of BS EN 1991-1-7.

5.2.2.5 Water pressures

It is usual for water pressures and the water levels associated with them, to vary as a result of tidal and associated meteorological conditions. When using BS EN 1997-1 for the design of embedded retaining walls, gravity retaining walls or revetment slopes associated with quay walls or jetties, it might be necessary to separately identify the permanent and variable components of water pressures so that appropriate partial factors can be applied. In general, the tidal lag component will represent the variable part of the load, the remainder being the permanent part.

BS EN 1997-1:2004, 2.4.6.1(8) also gives the option of applying a safety margin to the characteristic water level, which should take account of the anticipated tidal lag and effectiveness of any drainage system over the design life of the structure.

When considering the effects of water pressures on the structural system, they can be regarded as coming from a single source [BS EN 1997-1:2004, Note to 2.4.29(P)] and the appropriate partial factor selected accordingly.

5.3 Deflection control and deformations

5.3.1 General

Deflections in maritime structures during the design life of the facility can be very large, particularly with large quay walls. Provided that the design has been carried out in accordance with the appropriate Eurocode (BS EN 1990 to BS EN 1999), this can be accepted. However, a check on the long-term movements should be made to ensure that the geometry of the quay wall remains suitable for the berthing of ships and conforms to other operational requirements. The final alignment of crane rails should also remain within acceptable operational tolerances. This might require crane rails to be tied to the quay wall frontage so that the relationship of each rail remains constant, or it might require the introduction of adjustment measures to allow the crane rails to be restored to alignment, if the quay wall movements become too great to maintain the tolerances.

5.3.2 Jetties, platforms and crane rails

For jetties and platforms carrying normal highway vehicles or normal cargo handling operations, the deflections should be controlled in accordance with the following standards:

- BS EN 1992-1-1:2004, 7.4 for deflections of concrete structures, in conjunction with Table 1;
- BS EN 1993-1-1:2005, 7.2 for deflections of steel structures (limits are given in the National Annex). For marine structures, the limits shown in Table 1 should be used;

- BS EN 1993-2:2006, C.1.2.2 for deck plates and longitudinal girders;
- BS EN 1993-6:2007, 7.3 for deflections for crane rails;

NOTE 1 These deflections are based on the assumption that the crane rails are initially installed in accordance with the tolerances set out in BS ISO 12488-1 for a Class 2 crane.

- BS EN 1992-1-1, BS EN 1993-1-1, BS EN 1993-2 and BS EN 1993-6 for the type of serviceability limit state to be used in each case.

NOTE 2 The Eurocodes BS EN 1990 to BS EN 1999 respectively do not always include prescriptive requirements for deflections, which is why Table 1 is provided.

Table 1 Operational deflection limits for maritime structures

Deflection type	Structure type	Deflection limit	Additional comment
Vertical deflections	Cantilevers	Length/180	For petrochemical or similar pipelines, use Length/200.
	Spanning beams	Span/200	Unless the beam supports special claddings or coatings requiring a more restricted deflection control. For petrochemical or similar pipelines, use Span/400.
Horizontal deflection	Tops of piled bents or columns	Height/300 with a maximum of 100 mm	—
Dynamic effects (for motions in vertical or horizontal direction)	Generally	Refer to BS EN 1990:2002 +A1:2005, Annex A2	—

5.3.3 Trestle jetties carrying petrochemical or similar pipelines

For trestle jetties carrying sensitive items such as petrochemical or similar pipelines, there is usually a need to consider whether there are other risks that should be taken into account when deciding on the limits of acceptable deflections. Deflections of structures should be limited as shown in Table 1. The fixing arrangements should take into account temperature and settlement actions, and the pipeline should be articulated both along its length and between trestle and shoreline such that unnecessary effects do not result. In addition, differential deflections of crossheads supporting such pipes should be limited so that any additional stresses induced in the pipeline arising from the deflections, when combined with the effects of other actions, are limited to values that the pipeline material can support without rupture. These deflections can arise from elastic movements of the trestles due to loading changes, lateral wind loads and temperature distortions.

The structure and piping for petrochemical or similar pipelines may be precambered, to limit the deformations and to optimize the ability for liquid product to drain from the process lines when it is required to clear the pipes for cleaning or repair.

6 Embedded retaining walls in quay and jetty construction

COMMENTARY ON CLAUSE 6

Embedded retaining wall structures can be divided into three groups:

- a) *anchored single-wall (including propped single-wall structures);*
- b) *cantilevered single-wall;*
- c) *single-wall with relieving platform.*

The distribution of earth pressure and resistance of these structures depends on the type and flexibility of the structure as well as on the nature of the soil. This interdependence also applies to surcharges. The sequence of construction of embedded retaining walls is a major design consideration (see also 6.16). The following structural forms may be used in embedded retaining walls:

- *sheet piles;*
- *in-situ concrete piled walls;*
- *diaphragm walls;*
- *soldier piles and sheeting;*
- *combi walls;*
- *barettas (see also 8.7.3.3).*

6.1 General

Embedded retaining walls should be designed in accordance with BS EN 1990, BS EN 1991, BS EN 1992, BS EN 1993, BS EN 1995, BS EN 1997, and the present clause.

NOTE 1 Cellular sheet-piled and double-wall structures are not embedded retaining walls as defined in 3.1.8, and are covered in Clause 7.

NOTE 2 Additional advice on the construction of embedded retaining walls, which might have an effect on the design of the wall, is given in BS EN 1536, BS EN 1537, BS EN 1538 and BS EN 12063.

NOTE 3 Types of embedded retaining wall are described in 6.6.

6.2 Suitability of embedded retaining wall structures

Embedded retaining wall structures are most suitable when the ground below dredged level is of medium or dense granular soil, or firm to stiff cohesive soil. Ideally, the ground above dredged level at the time of installation of the wall should also be one of these types.

Embedded retaining walls may be used in weak soil, but it might be preferable, where weak soil lies below dredged level, to replace the soil with sand before wall construction to avoid the large penetration required for stability. If the depth of soft clay is very great, an embedded retaining wall might be unsuitable and another form of construction, such as a suspended deck structure (see Clause 8), should be used instead.

Where the seabed is mainly of rock, pre-treatment of the bed or pile toe or provision of rock sockets or dowels might be necessary to allow sheet piles to be used.

Embedded retaining walls are suitable for use in quays in impounded basins where a cut-off is required. They can also be used to form a temporary cut-off or cofferdam for the construction of an in-situ concrete gravity wall. The embedded retaining wall may be incorporated into the permanent structure.

Embedded retaining walls may be used where it is necessary to increase the depth of water in front of an existing structure. This form of construction is especially suitable where the seaward displacement of the berth face is to be kept to a minimum.

6.3 Materials for embedded retaining walls

6.3.1 General

Materials used in the construction of embedded retaining walls should conform to BS 6349-1.

NOTE For steel materials, reference should also be made to BS EN 10025, BS EN 10210, BS EN 10219, BS EN 10248 and BS EN 10249 as appropriate.

6.3.2 Sheet-piled walls

6.3.2.1 Steel

Steel sheet piles are the most widely used embedded retaining wall elements in quays and jetties. They are relatively light and easy to handle, they can be supplied in long lengths and can be extended and cut without undue difficulty.

NOTE 1 Guidance on the installation of steel sheet piles is given in BS EN 12063, and this includes guidance on welded joints and many other issues.

This type of pile can be driven to considerable depth with low displacement in a wide range of ground conditions and into weathered rock. Where hard driving is encountered, the pile section required might exceed the section needed to resist bending. This should be taken into account in assessing the reduction of moments. With various forms of pre-treatment, steel sheet piles may also be installed in solid rock, in a trench backfilled with concrete, by pre-splitting the rock or by the use of toe dowels as detailed in BS EN 12063.

The interlocks are generally tighter than for timber or concrete piles. The principal disadvantage of steel sheet piles is corrosion, which should be allowed for in design.

Some types of floating fenders can cause abrasion of steel sheet piles or their protective coatings, and this should be taken into account in selecting the form of fendering to be adopted.

The most commonly used interlocking pile sections for steel embedded retaining walls are the U- and Z- types. Several combined wall sections are also available, usually comprising H-piles, box piles or tubular piles with either interlocking sheet pile elements or separate interlocks. Additional bending resistance can be achieved, where the piles are to be driven to a predetermined level, by welding reinforcing plates to the pile flanges in zones of maximum bending moment.

NOTE 2 Examples of interlocking steel pile sections and combinations of elements suitable for use in embedded retaining walls are given in BS EN 1993-5.

Construction tolerances should generally be in accordance with **8.13** and BS EN 12063.

NOTE 3 The ICE specification for piling and embedded retaining walls gives further guidance on tolerances.

6.3.2.2 Concrete

Concrete sheet piles may be used for the construction of walls of moderate height and where driving is not too hard. The penetration required might have to be achieved by pre-boring or jetting. In rock, the piles may be installed in a trench backfilled with concrete.

The main advantage of properly designed concrete sheet piles is their durability, although the concrete mix might have to be modified if sand abrasion is expected. Sheet piles can also be used to advantage when heavy vertical actions are to be carried, since they will reduce the tension due to bending of the piles. However, the weight of the sheet piles, the care required during handling, the difficulty of forming extensions and the usually poor interlock at joints are all factors that in many cases will dictate against their use. If there is a danger of loss of material, the joints may be sealed by providing a filter behind the wall or grouting after driving. Pre-stressed piles should generally be used in preference to reinforced concrete, to save weight. They are more durable, stronger in bending and better able to resist tension set up during handling, driving or in service.

Where abrasion due to floating fenders is likely, the concrete quality should be selected accordingly. The construction tolerances given in **8.13** should be applied to concrete sheet piles.

6.3.2.3 Timber

Timber sheet piles can provide an economical wall for moderate heights of retained material and where driving conditions are not too severe. Examples of suitable applications are bulkhead walls behind suspended decks and quays for small craft. Most timbers require protective treatment against rot and marine borers: see BS 6349-1:2000, **60.2**. Rubbing strips should be provided where abrasion is expected.

The joints between timber sheet piles are not generally tight, so the joint should be chosen with care if driving in existing ground where retained material could escape.

NOTE Typical joint arrangements are shown in BS EN 12063:1999, Figure F.1.

6.3.3 In-situ concrete piled walls

Embedded retaining walls of in-situ concrete bored piles are usually built on existing ground or on an artificial embankment, using either a contiguous or a secant pile system. This type of wall is generally most suitable for use in cohesive soils and weak rock and where heavy vertical loads are to be accommodated. They may also be built in granular soils when a casing or support fluid is used during excavation, although environmental issues connected with the use of support fluids or pumping of concrete in water bearing soils need to be taken into account.

NOTE Typical arrangements are shown in BS EN 1536:2000, Figure 6.

In contiguous pile walls, care is needed to ensure that material is unable to escape through gaps between the piles. In secant pile walls, the piles are usually 1.0 m to 1.2 m in diameter, installed at centres equal to approximately 0.9 times the diameter: to ensure overlap, the verticality tolerance in the direction of the wall should generally not exceed 1 in 200. Construction tolerances for in-situ concrete piles should generally be in accordance with 8.13 and the *ICE specification for piling and embedded retaining walls*.

Concrete for in-situ piled quay walls is usually placed by tremie. The surface finish of this type of wall is generally irregular, depending on the soil. It should, if necessary, be improved above water level by adding an in-situ concrete facing, or by extending the capping down to this level.

6.3.4 Diaphragm walls

Embedded retaining walls in the form of in-situ concrete diaphragm walling are often used where there is a sufficient width of existing ground or artificial embankment available for use as a working area. They can be built in most types of soil and may be toed or doweled into solid rock, but are generally unsuitable where flowing or artesian water is present during construction. Diaphragm walls are used for high walls or where heavy vertical loads are imposed on the wall. They are particularly suitable at sites where the toe levels can be predetermined to within ± 0.5 m, to assist the advance fabrication of reinforcement cages.

NOTE 1 Examples of typical panel arrangements and design methodology are given in BS EN 1538.

The ground level should be at least 1.0 m to 1.5 m above maximum ground water level to ensure stability of the trench when support fluid is used, but environmental issues connected with the use of support fluids or pumping of concrete in water bearing soils need to be taken into account. Slurry trench construction should conform to BS 6349-1:2000, Clause 55.

Diaphragm walls are usually constructed using straight panels, 0.5 m to 1.5 m thick. The resistance may be increased by using T-panels, either with abutting flanges or with straight panels between them. L-panels are suitable at corners. Panel sizes are generally limited by the weight of reinforcement cages to be handled and by the volume of concrete, since it has to be poured in a single operation. Panel lengths are typically 4.5 m to 5.0 m, the practical limits being 2 m and 10 m.

Diaphragm walls are usually of reinforced concrete, but pre-stressed walls can be built using tendons anchored at the top of the wall and looped at the base. Care is necessary in detailing the reinforcement to provide adequate space between bars. A concrete cover of at least 75 mm should be adopted. The recommendations in 6.3.3 on surface finish also apply to diaphragm walls.

NOTE 2 Further guidance is given in the ICE specification for piling and embedded retaining walls.

Where recesses are necessary, they should be designed to interfere as little as possible with concreting. The tolerance which is necessary in the positioning of recesses should be taken into account in the design.

Tolerances should generally be in accordance with BS EN 1538.

NOTE 3 The ICE specification for piling and embedded retaining walls gives further guidance on tolerances and joints between panels.

6.3.5 Soldier piles and sheeting

Embedded retaining walls constructed using soldier piles and horizontal sheeting are not widely used in maritime works, owing to the high cost of the diving work involved and the tight constructional tolerances required to avoid loss of backfill. However, this type of wall might be a suitable way to provide a deeper retaining face outside an existing wall, where minimum interference with port operations is essential.

The piles may be of steel or concrete, either precast or in situ: pre-stressing is normally advisable to reduce weight and improve durability. Preformed piles may be installed, by driving or placing in a preformed hole, in any seabed material, but might require an excessive penetration in soft clays and silts. In-situ concrete piles are subject to the usual limitations of bored piles.

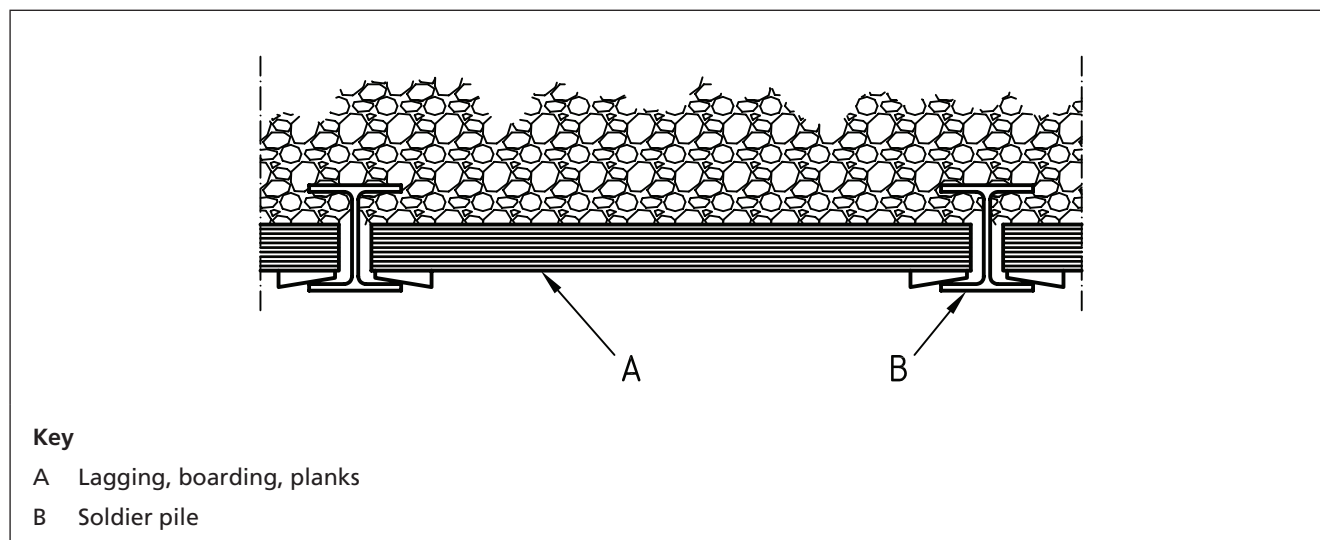
The penetration of the piles into the seabed should be calculated to take into account the consequences of failure of the ground support. The heads of all piles should be temporarily supported until the capping is cast and the anchors installed.

The sheeting in quay walls of this type is usually of precast concrete planks. The sheeting should be secured to the soldier piles to prevent differential movement and consequent loss of material. Figure 1 shows an example of such a construction. Care should be taken to provide appropriately sealed joints to prevent loss of material.

Construction tolerances for soldier piles should be in accordance with 8.13.

NOTE The ICE specification for piling and embedded retaining walls gives further guidance on tolerances.

Figure 1 Soldier-piled wall: typical cross-section



6.3.6 Combi-walls

Combi-walls are retaining walls composed of primary and secondary elements. The primary elements are normally steel tubular piles, I-sections or built-up boxes, spaced uniformly along the length of the wall. The secondary elements are generally steel sheet piles of various types installed in the spaces between the primary elements and connected to them by interlocks (see BS EN 1993-5:2007, Figure 1.5).

Combi-walls should be designed in accordance with the requirements for combined walls given in BS EN 1993-5.

6.4 Section modulus of wall

6.4.1 Steel sheet piles

The section modulus of steel sheet-piled walls in vertical bending due to soil actions should be calculated in accordance with BS EN 1993-5.

Where the section modulus of a steel sheet-piled wall is increased by fixing reinforcing plates to the piles or by introducing box sections, these additional elements need only be provided over the height where they are required for bending resistance.

Account should be taken of deflections which might arise whilst backfilling until the interlocks develop sufficient friction to enable the full combined modulus to be developed.

6.4.2 Combined steel sheet piles

In combined steel sheet-piled walls in which the intermediate sheeting is required to contribute to the bending resistance, welding of the interlocks will in many cases be required. If the modulus of the H-section, box or tube far exceeds that of the intermediate sheeting, it may be assumed that all horizontal actions are transmitted to the large modulus elements and no welding of interlocks is then required. The intermediate sheeting, in the latter case, may be of lighter section than would be necessary if it were designed to span vertically. Soldier-piled walls are also designed assuming that all horizontal actions are transmitted to the soldier piles.

6.4.3 Diaphragm walls

The section modulus for a diaphragm wall should be calculated in accordance with BS EN 1538.

NOTE Further guidance is given in the ICE specification for piling and embedded retaining walls.

6.5 Actions and effects of actions on embedded retaining walls

NOTE 1 The actions on a retaining wall are summarized in BS EN 1997-1:2004, 2.4.2 and some or all of these will be appropriate to the design of a specific structure.

The stresses obtained by analysis should be in accordance with BS EN 1992, BS EN 1993, BS EN 1995 or BS EN 1997 as appropriate to the material under consideration. Crack widths (see 4.4.2.3) should be calculated for the serviceability limit state.

These stresses in steel and timber elements should be based on the wall section at the end of the design life.

The maximum bending moment in embedded retaining walls does not usually coincide with the most severe zone of corrosion, and this should be taken into account in the analysis of steel sheet-piled walls.

The geotechnical parameters should be established in accordance with the requirements of BS EN 1997-1 through an appropriate geotechnical investigation. The partial factors to be applied to actions, and the method to be used for combining actions, should be in accordance with Clause 5 of the present standard. The partial factors applied to the geotechnical parameters should be taken from BS EN 1997-1.

The driving of steel sheet and timber piles should be in accordance with BS EN 12063.

Where support fluid is used in the excavation of in-situ concrete embedded retaining walls, the permissible bond stress of deformed steel reinforcement might have to be reduced.

NOTE 2 Further guidance is given in the ICE specification for piling and embedded retaining walls.

Walings should be designed in accordance with BS EN 1993-5.

6.6 Design of embedded retaining wall structures

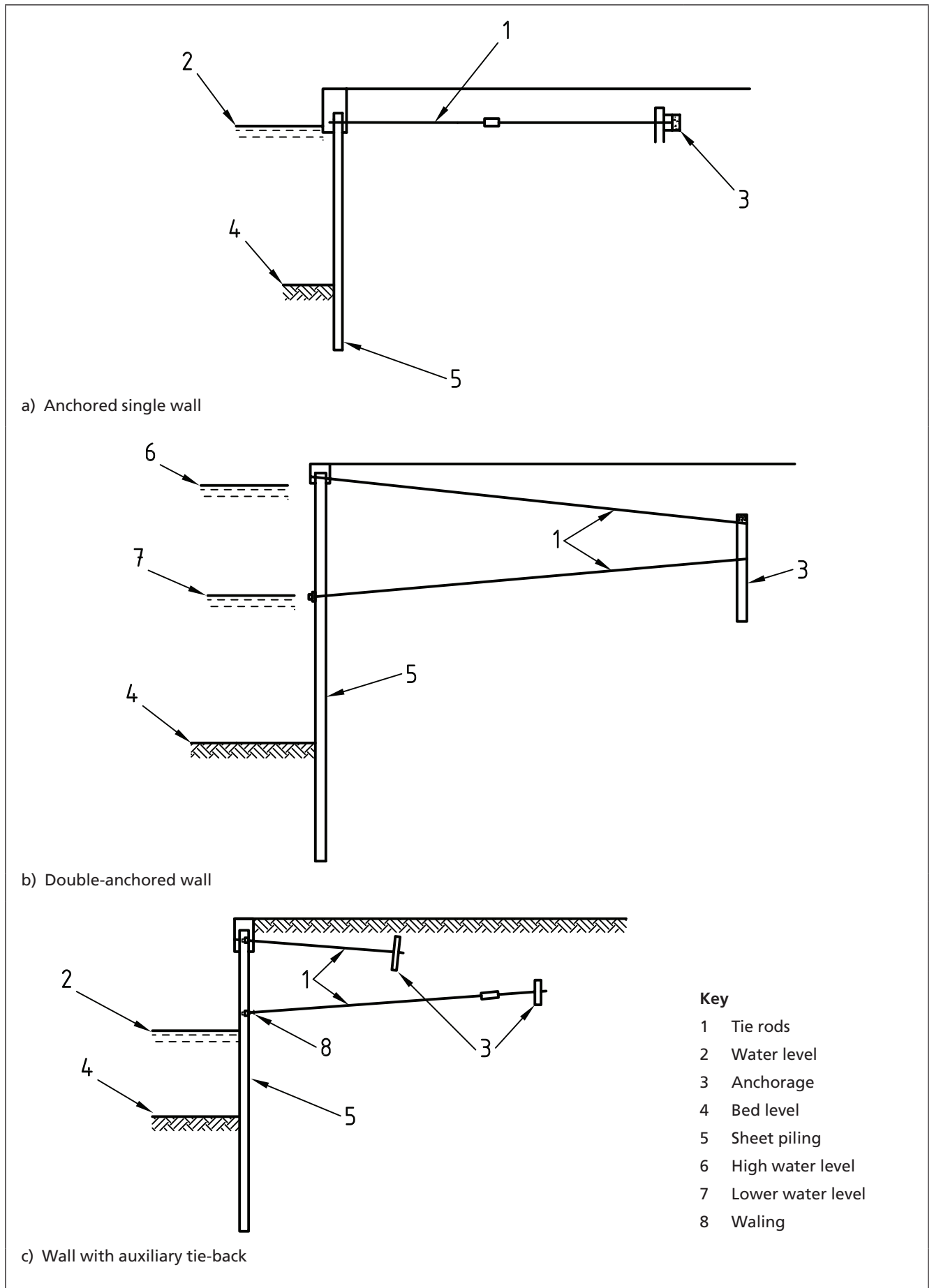
6.6.1 Anchored or propped single-wall structures

COMMENTARY ON 6.6.1

Resistance to active pressure from the retained soil is provided by the bending strength of the embedded retaining wall, which spans vertically between anchors (or props) at one or more levels, and the distributed lateral resisting pressure on the lower embedded part of the embedded retaining wall, where passive soil resistance is available. Figure 2a) and Figure 2b) show examples of walls with one and two tie levels, respectively.

Walls with a substantial cantilever above tie level might require auxiliary tie-backs to the top. These may be connected to the main anchorage or to a separate anchorage [see Figure 2c)].

Figure 2 Anchored embedded retaining wall structures

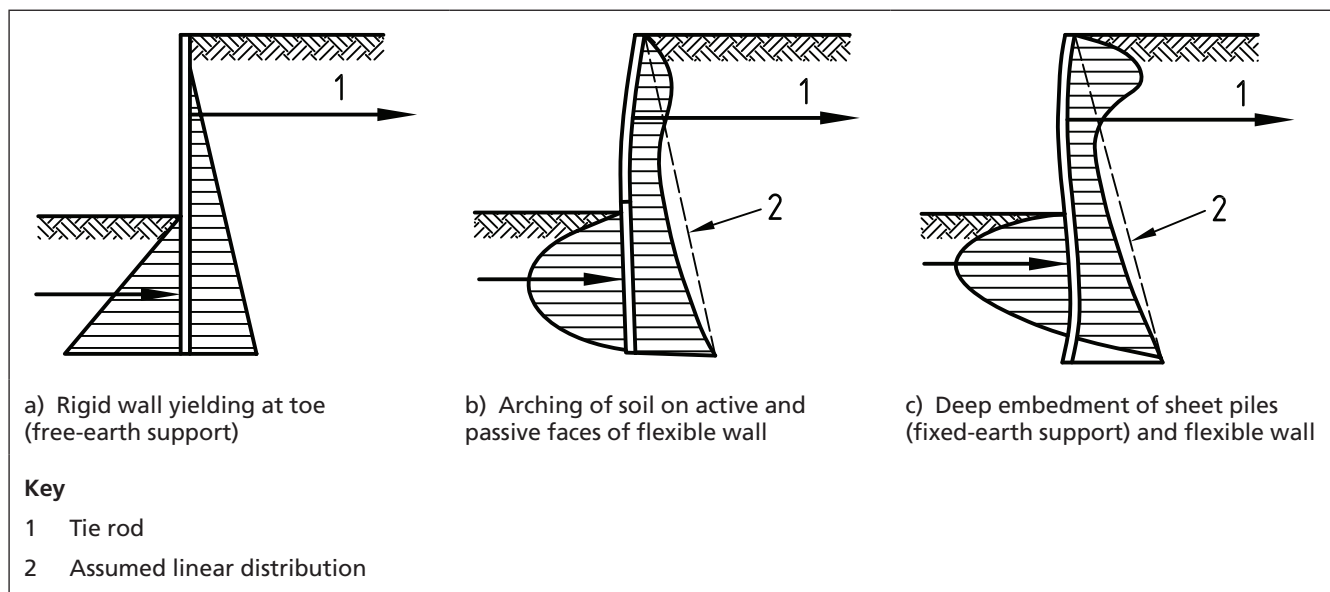


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6.6.1.1 Single-anchored or propped walls

The design of single-anchored or propped walls should be determined in accordance with BS EN 1992, BS EN 1993, BS EN 1995 or BS EN 1997 as appropriate. The pressure distribution shown in Figure 3a) represents a condition of free-earth support in which the rigid anchored wall yields by forward translation. The effects of flexibility in causing arching of the soil are shown in Figure 3b). Where the sheet piles are driven to greater depth [see Figure 3c)], fixity occurs in the lower portions of the piles, giving a reversal of curvature in the piling.

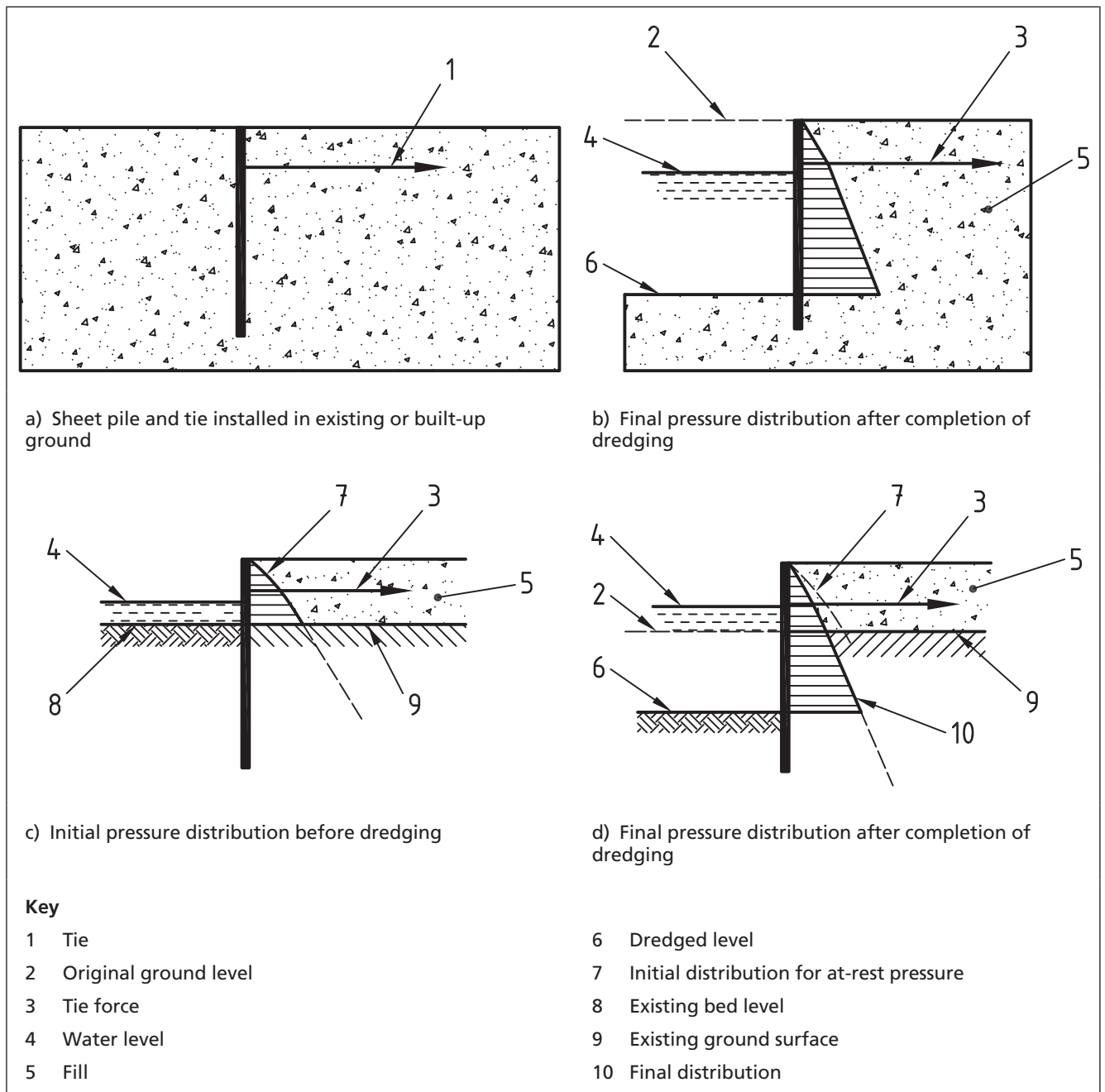
Figure 3 Effects of flexibility of single-anchored or propped embedded retaining wall structure on earth pressure and resistance



A fixed-earth condition will generally result in a more economical design for flexible walls than a free-earth condition, since bending moments in the wall and anchorage forces tend to be lower. A deeper wall is required to achieve fixity in the fixed-earth condition, but in a free-earth condition the shallower penetration required for stability alone has to be increased to provide adequate resistance to forward movement. It is not usually advisable to assume a fixed-earth condition for walls in cohesive soils for permanent structures, owing to the long-term change in properties of these soils.

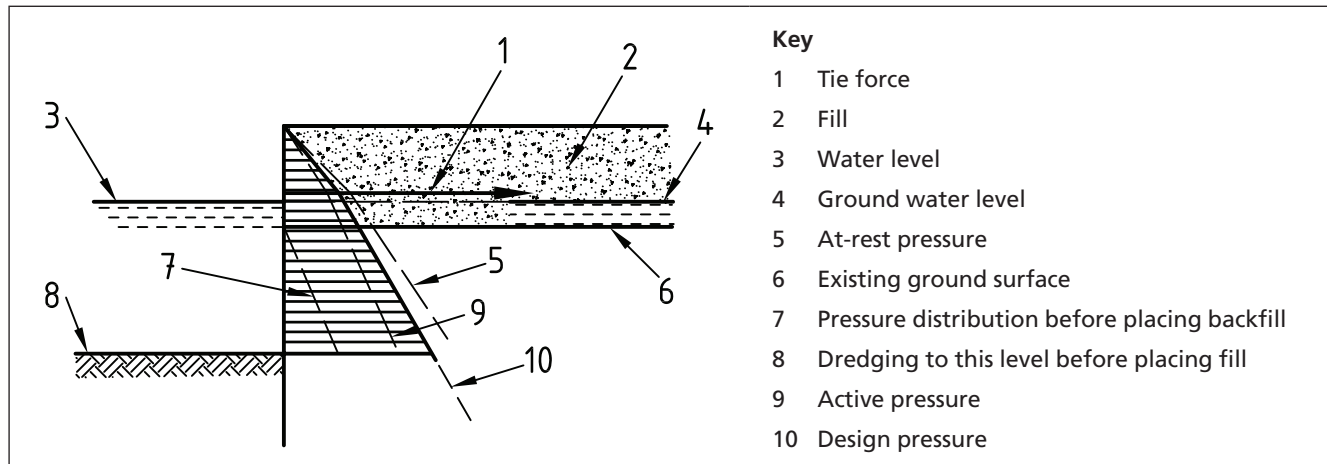
Arching conditions should be taken into account in relation to the sequence of construction of an anchored sheet-piled retaining wall. Figure 4 illustrates two scenarios where dredging takes place after the installation of the embedded wall and tie rods. Figures 4a) and 4b) indicate the situation where the embedded wall is installed where the existing ground level is at the top of the wall or where the ground has been filled to the finished level prior to installing the embedded wall. Figures 4c) and 4d) indicate the situation where the embedded wall is installed where the existing ground level is lower than the final ground level behind the wall, and where backfilling is completed before the soil in front of the wall is dredged away. In the latter case, 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 will yield and the pressure distribution will change from arched conditions at the upper level to the final assumed linear active condition.

Figure 4 Active pressure distribution on single-anchored or propped embedded retaining wall structure where fill is placed before dredging



If, however, the soil is dredged away before placing any fill behind the wall, 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 ground water level is compacted, the additional yielding of the wall and anchorages might not be sufficient to develop active earth pressures from the upper fill. In this case, provision should be made for lateral pressures from the upper part of the fill at a state intermediate between the active and at-rest condition, depending on the expected forward movement of the wall as the fill is placed (see Figure 5).

Figure 5 Active pressure distribution on single-anchored or propped embedded retaining wall structure where fill is placed after dredging



However, for the purpose of determining the stability of the wall against overturning and for calculating anchorage forces and bending moments in the embedded retaining wall, the effect of flexibility of the structure should be taken into account. With a flexible wall, the anchorage forces are higher and the bending moments between anchorage and seabed are lower than those calculated assuming a linear stress distribution. The wall flexibility is reduced where the wall is built into a stiff concrete capping beam. The effects of flexibility, which are to increase the anchorage forces and to raise the level of the resultant passive resistance, should be taken into account. Water pressures (see BS 6349-1:2000, 51.5) should be calculated in accordance with BS EN 1997-1.

Where concrete is cast against an embedded retaining wall to relieve pressure, it should generally be assumed that full hydrostatic pressure will be developed between the wall and the concrete. The wall should be designed to resist this action.

The design of non-vertical embedded retaining walls should take into account the batter of the wall, in accordance with BS EN 1997-1.

The effects of actions in the structure, allowing for flexibility, should be determined using BS EN 1992, BS EN 1993, BS EN 1995 or BS EN 1997 as appropriate. These standards apply to both the free-earth and the fixed-earth support methods.

The overall stability against sliding of the structure and the mass of earth retained by and beneath the structure should take into account the requirements of BS EN 1997-1. Sliding may be rotational, planar, or on a composite failure plane.

The geotechnical parameters for stability considerations should be determined in accordance with the requirements of BS EN 1997-1. For cohesive soils, the long-term considerations are likely to be critical except where dredging is carried out in front of the wall, when undrained conditions might govern, depending upon the speed of construction. In cohesive soils, the effects of soil shrinkage causing tension cracks within and landward of the anchorage zone should be taken into account. The effects of scour and dredging below the nominal design depth should also be taken into account, in relation to the depth of embedment required to mobilize passive resistance. Allowance should be made for over-dredging in accordance with

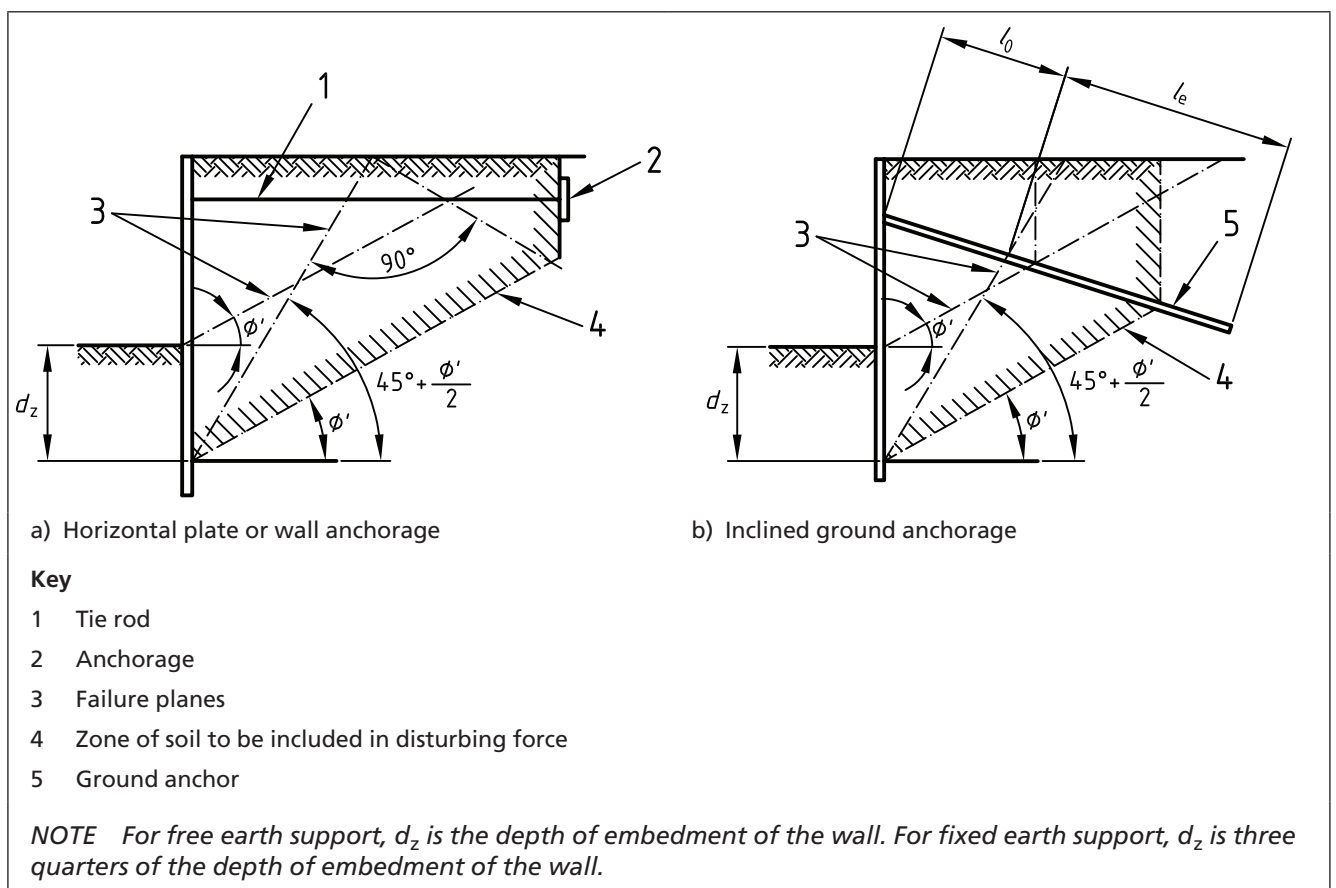
BS EN 1997-1:2004, 9.3.2.2. In cohesive soils, removal of overburden by dredging will cause swelling and softening of an over-consolidated clay, which should be taken into account in the selection of soil parameters.

When considering the stability against sliding of the structure and earth masses, the mooring actions and surcharge loads (see BS 6349-1:2000, 51.4) on the structure should be included, where these act as unfavourable actions. The effects of earthquakes on stability in accordance with BS EN 1998 should be taken into account.

In general, anchorages should be designed in accordance with BS EN 1997-1, but steel anchors and walings should be designed in accordance with BS EN 1993-5, and concrete elements should be designed in accordance with BS EN 1992-1. The weight of the soil that lies above the lower failure plane should be included in the disturbing forces in relation to the length and inclination of the anchors (see Figure 6). Where cantilever anchorages are used, the lower failure plane may be assumed to intersect the anchorage at the lower point of zero shear. It is generally recommended that computer software that is able to perform the calculations required for BS EN 1997-1 is used to determine the effects of actions in an embedded retaining wall, but this is strongly recommended for stiff wall sections, where introduction of the horizontal modulus of subgrade reaction of the soil will allow a more realistic earth pressure distribution to be determined.

Anchorages may be limited to a single level at or above MHWS because of the difficulty in installing anchors and walings below MLWS or at some intertidal level. They should be located at sufficient depth to allow underground services to run above them and to minimize the effect of local surcharge loads from cranes or stored materials.

Figure 6 Location of anchorages in relation to failure planes in retained fill



6.6.1.2 Design of double-anchored walls

Double-anchored walls should generally be designed in accordance with the recommendations for single-anchored or propped walls (see 6.6.1.1). However, double-anchored walls are considerably more complex than single-anchored walls, and for this reason, computer software that is able to perform the calculations required for BS EN 1997-1 should generally be used to determine the effects of all actions for double-anchored walls.

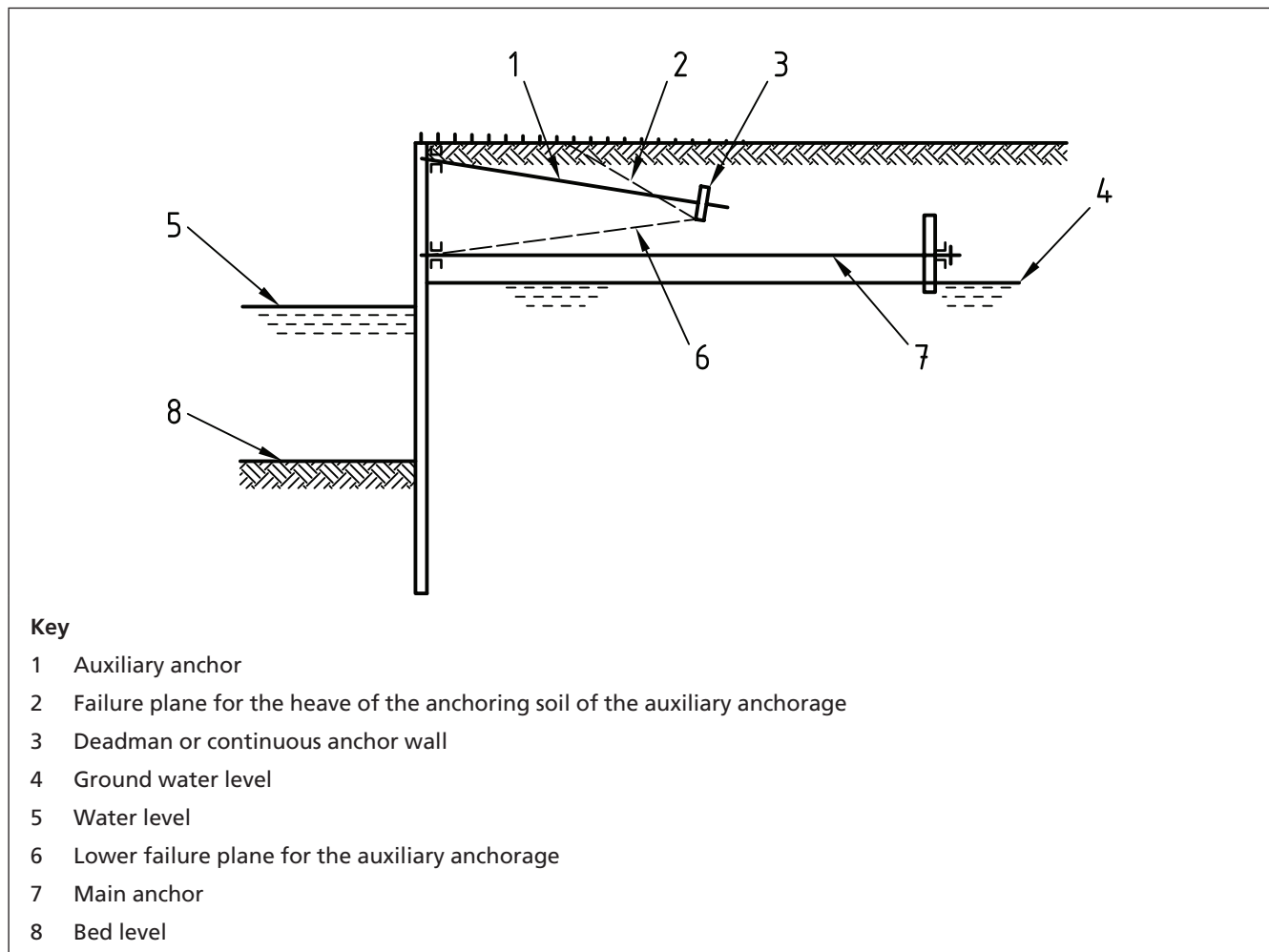
NOTE Manual calculation methods are described in the *Pile Buck publication* Sheet pile design [18] and Recommendations of the Committee for Waterfront Structures [11].

6.6.1.3 Auxiliary tie-backs

Where auxiliary tie-backs are provided only to resist heavy local horizontal actions applied to the top of the wall, the main tie-backs should be designed ignoring the auxiliary ties. The auxiliary ties, which should be attached close to the top of the wall, may be designed by assuming that the wall behaves as a propped cantilever, fixed at the level of the main ties. In practice, the auxiliary waling and ties provided are usually the same as the main ties.

The resistance of the auxiliary anchorage should be checked for the failure planes shown in Figure 7, and as described in 6.6.1.1.

Figure 7 Embedded retaining wall with auxiliary anchorage: failure planes

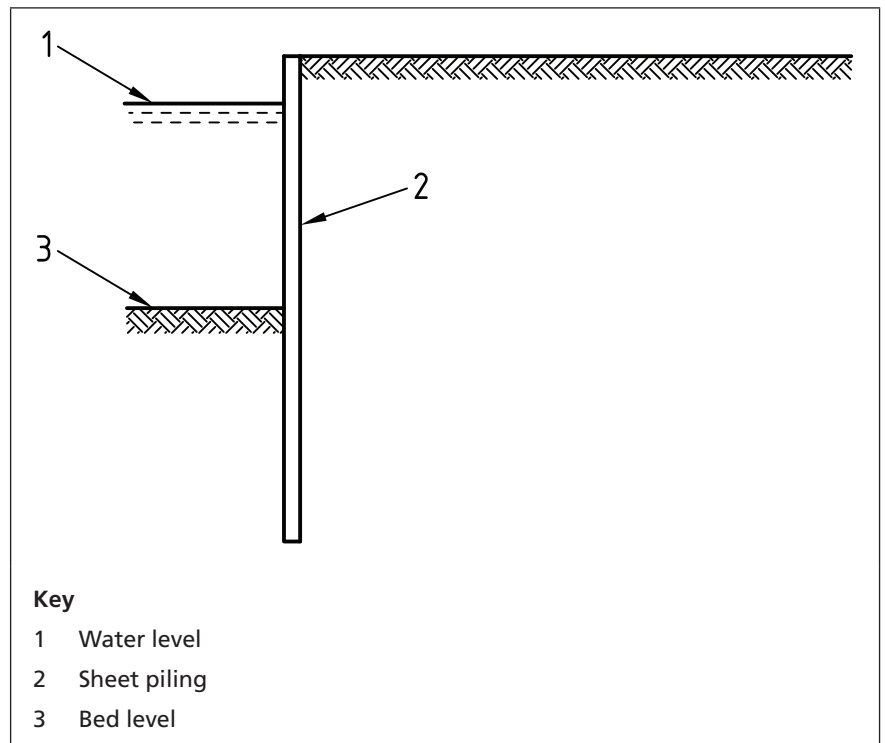


6.6.2 Cantilevered single-wall structures

COMMENTARY ON 6.6.2

Cantilevered single-wall construction is limited to low retaining walls, generally with a retained height of soil of less than 5 m, since resistance to active pressure of the retained soil is provided only by the passive resistance of the embedded portion of the piles (see Figure 8). The soil yields in mobilizing this resistance, with potentially large deflection of the tops of the piles.

Figure 8 Cantilevered embedded retaining wall structure

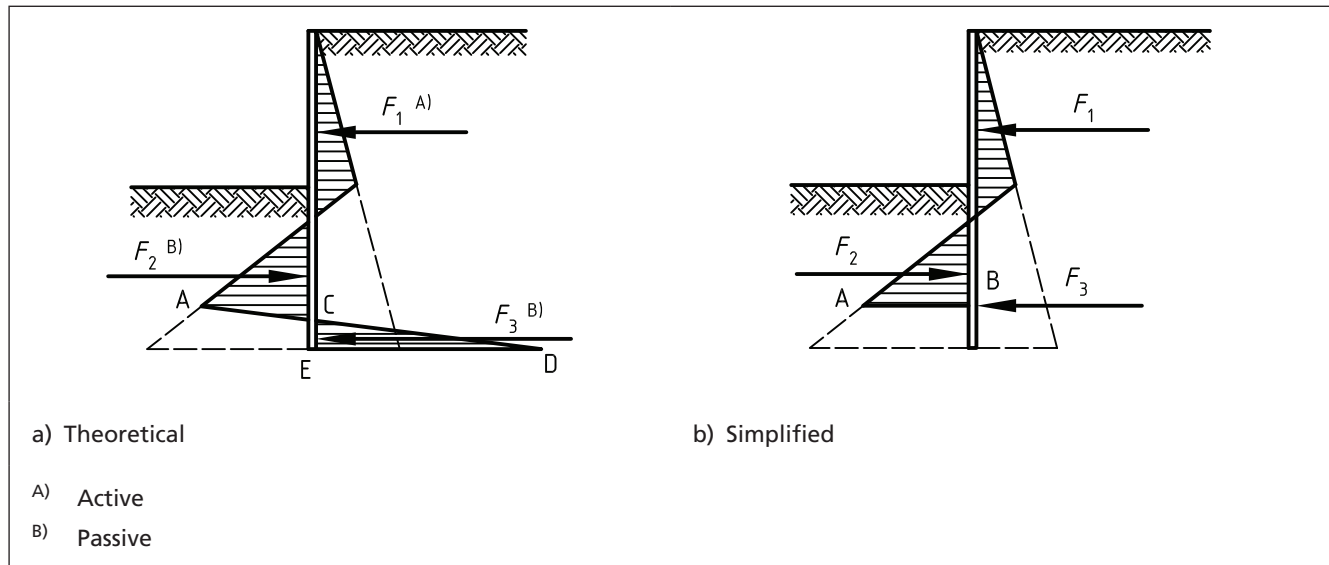


The distribution of active earth pressure and passive earth resistance on a cantilevered embedded retaining wall is shown in Figure 9a). The wall should be embedded sufficiently to obtain fixed-earth support conditions at the toe.

For the purpose of this calculation the wall should be assumed to be rigid. The pressure distribution can be simplified as shown in Figure 9b), when the action represented by the area CDE of Figure 9a) can be replaced by a single horizontal fixed direct action F_3 acting at the point A, and the line AB is assumed to be horizontal. The value of the action F_3 is such that the algebraic sum of the actions F_1 , F_2 and F_3 is zero.

The overall stability against rotational failure of the wall and the surrounding earth masses should be taken into account as described in 6.6.1.1.

Figure 9 Simplified distribution of active earth pressure and passive earth resistance on cantilevered embedded retaining wall structure



6.6.3 Embedded retaining wall with relieving platform

COMMENTARY ON 6.6.3

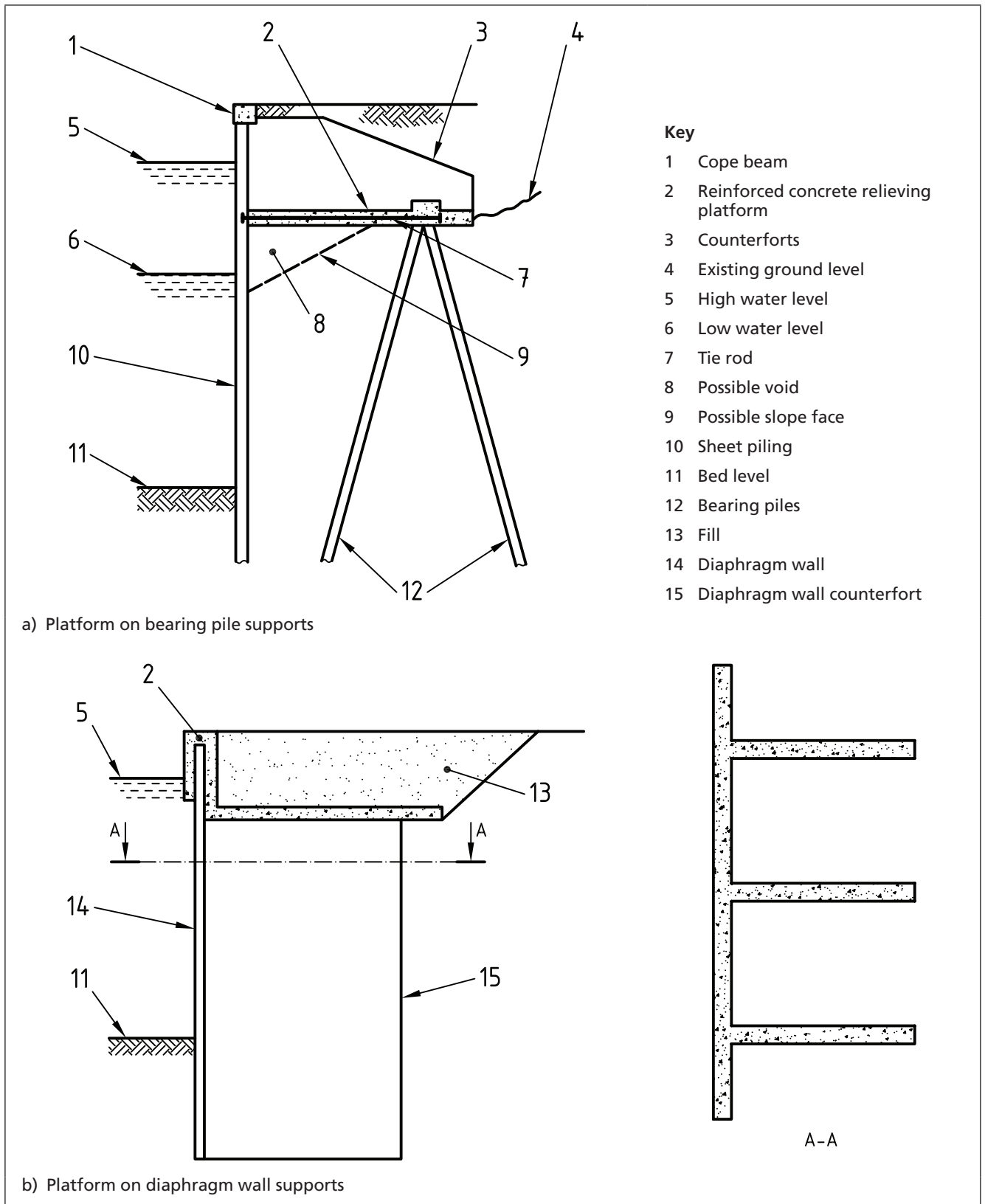
An embedded retaining wall with relieving platform is applicable where soils are weak and/or a greater height of wall is required. There are two basic types of relieved wall.

- Figure 10a) shows the most common form of relieved wall, in which the relieving platform is constructed behind the wall on existing or filled ground.
- Figure 10b) shows a variant in which diaphragm walling is used both for the embedded retaining wall and as an alternative to the bearing piles. The transverse diaphragm walling may be continuous or in the form of barrettes (see also 8.7.3.3).

The vertical actions are supported by the embedded retaining wall and the bearing piles. The raking piles or diaphragm wall elements provide a rigid anchorage to the wall, uplift actions being reduced by the actions from the platform. Where it is desired to reduce the vertical soil actions on the wall, the platform may be designed as a partially hollow reinforced concrete section.

The lateral soil pressures acting on embedded retaining walls built in front of a relieving platform conforming to the geometrical arrangement shown in Figure 10a) may be assumed to be relieved by the platform. To achieve maximum relief of lateral pressures on the wall, the width of the platform should be such that its rear edge intersects the failure plane of the soil, and the platform should be built as low as practicable. Figure 11a) shows the assumed theoretical pressure distribution for the case of a rigid wall with free-earth support. The pressure distribution will, in practice, be modified according to the flexibility of the wall, the closeness of bearing piles under the platform, the toe level of the bearing piles, the degree of fixity at the wall/platform junction, and the rigidity (which may generally be assumed to be infinite) of the raking pile anchorage. Relatively rigid walls are unlikely to yield sufficiently to cause active pressures to occur on the rear face, and at-rest conditions might then apply.

Figure 10 Embedded retaining walls with relieving platform



The landward bearing pile in each row may be assumed to provide additional relief of the embedded retaining wall by resisting the lateral load which would otherwise have been applied to the back of the wall between the platform soffit and the intersection of the lower failure plane and the pile over a length of wall equal to twice

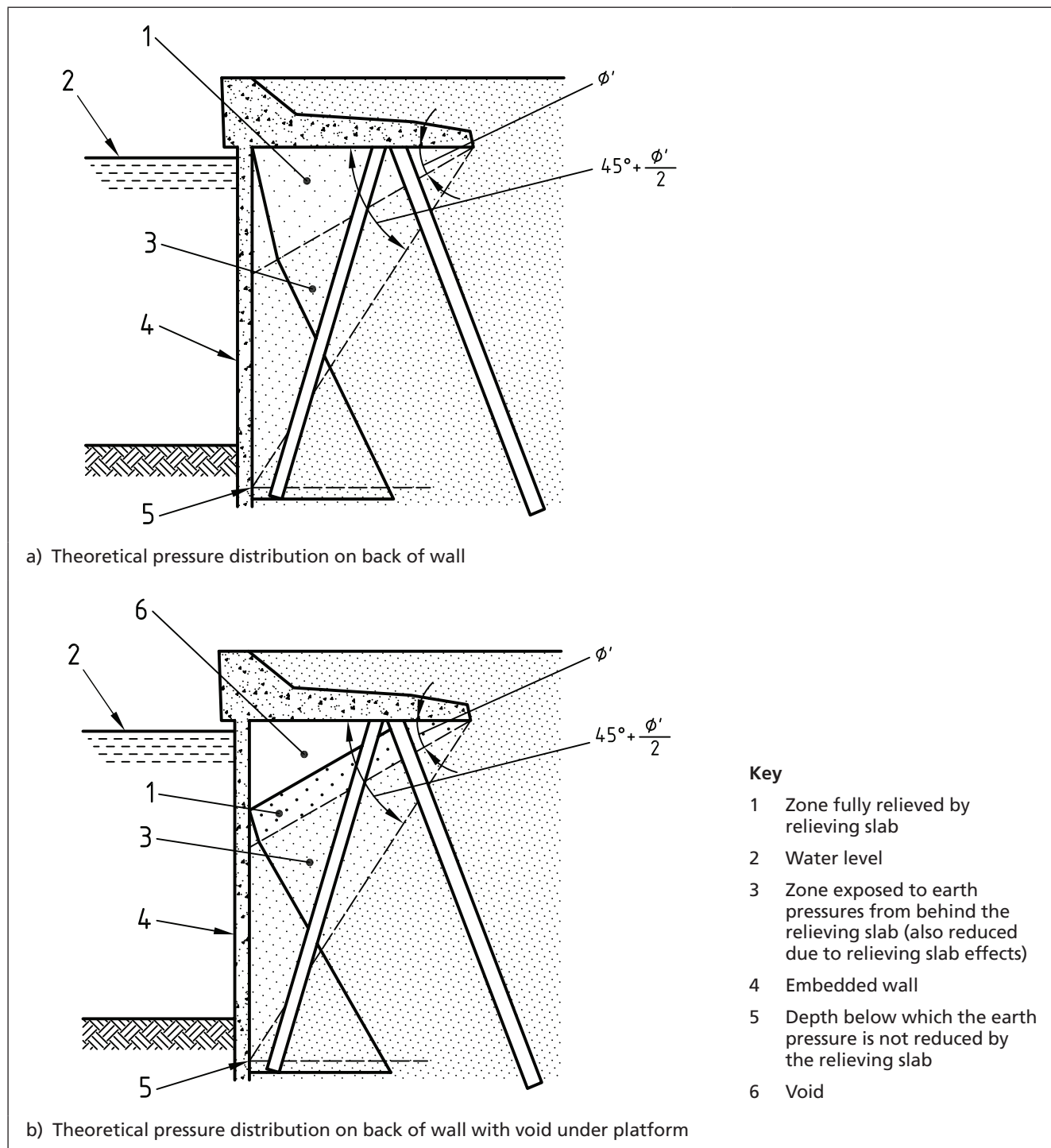
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the overall pile width. Where the toes of bearing piles whose toes are raked seaward lie within the failure wedge, these should be assumed to apply a concentrated load on the wall horizontally at 1 m above the bearing pile toe level.

Active pressures acting on the back of the wall should be calculated to take into account the effects of the bearing piles.

Where a void exists at the underside of the relieving platform, the theoretical pressure distribution on the back of the embedded retaining wall may be assumed to be as shown in Figure 11b)].

Figure 11 Pressure distribution on embedded retaining wall in front of relieving platform



The overall stability against sliding of the wall and the surrounding earth masses should be taken into account as described in 6.6.1.1. The total pressure acting on the back of the relieving platform structure should be calculated neglecting any relief afforded by the platform or the piles. The effect of the relieving platform bearing piles on overall stability should be taken into account. The platform and piles should be designed in accordance with Clause 8. Separation between underside of relieving slab and top surface of material beneath is beneficial to assist in preventing pressure from the slab being exerted on the material below.

The overall stability of the diaphragm wall variant shown in Figure 10b) should be assessed as described in Clause 7.

In earthquake regions, the benefits of shielding the embedded retaining wall by the platform should be balanced against the total mass of the superstructure which will be affected by horizontal seismic actions. The shielding effect will in any event be less under earthquake conditions owing to the flatter dispersion through the soil of soil and surcharge loads. Longitudinal earthquake actions may be resisted by all the bearing piles if they are fully embedded: otherwise, longitudinal raking piles should be provided.

Ground anchors should be used where the overturning resistance is insufficient. Lateral pressure on the embedded wall may be further reduced by sloping the ground under the platform or by taking advantage of an existing slope [see Figure 10a)]. The embedded retaining wall may either extend to cope level, or be stopped off at the platform level with a reinforced concrete retaining wall built up from the platform slab to cope level.

6.7 Toe level

6.7.1 Minimum embedment

The minimum embedment required for overall stability of the wall should be determined in accordance with BS EN 1997-1 and the relevant material design codes. The toe elevation of elements resisting vertical actions will be influenced by the depth at which soil resistance to these actions is achieved. Additional factors to be taken into account in assessing embedment are given in 4.1.6.2.

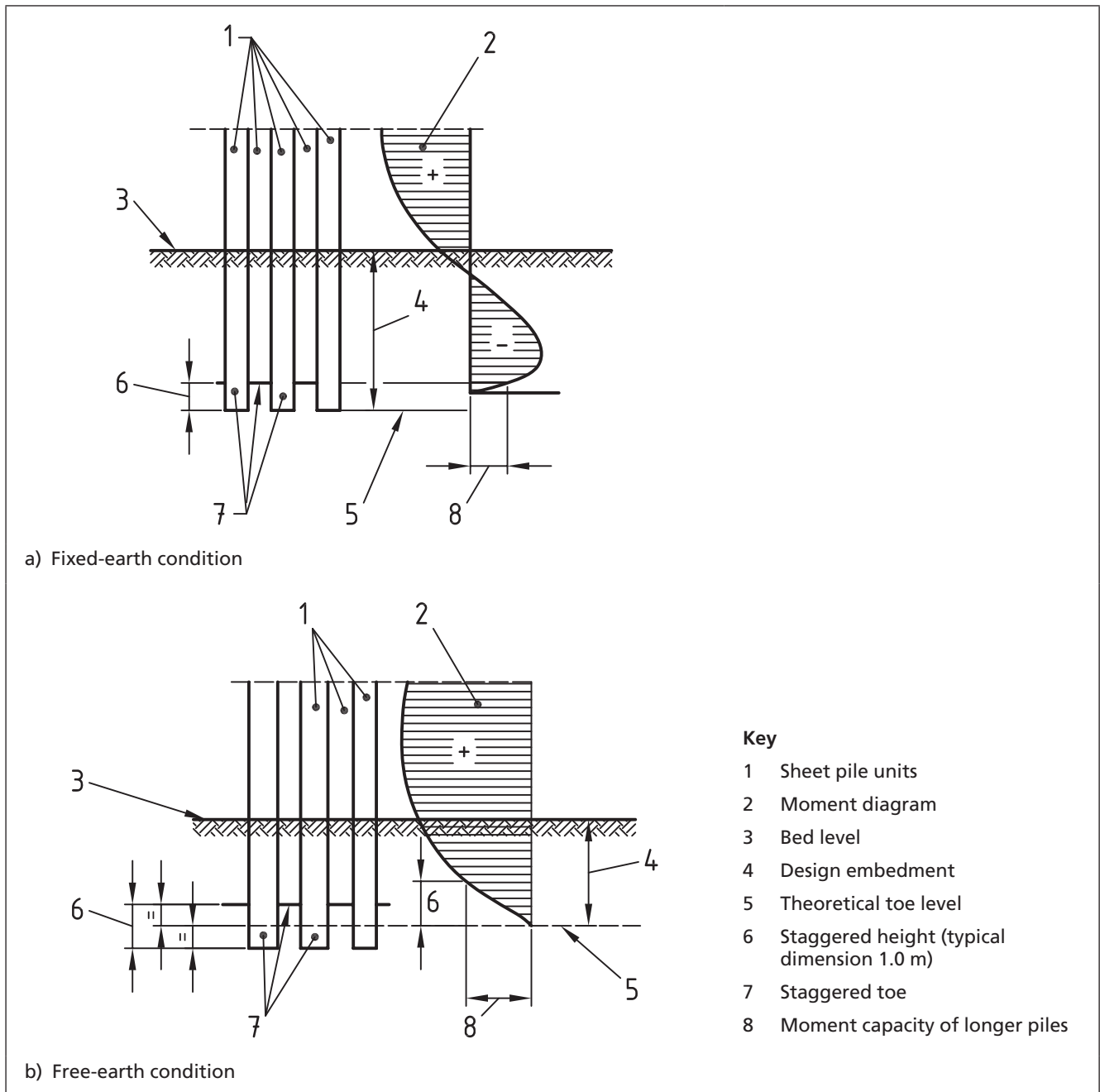
6.7.2 Staggered toe elevations

Sheet-piled walls may be designed such that the toes of alternate piles or pile pairs are staggered, according to the number of piles installed at a time. When staggered toes are used, the toe lengths of the longer piles should be capable of resisting the total bending, shear and vertical forces which would occur in a wall with uniform toe levels. If the toes are staggered by not more than 1.0 m, additional analysis of the piles subjected to these additional forces might not be necessary.

In fixed-earth conditions, the longer piles should extend to the embedment required for a wall with uniform toe level. In free-earth conditions, the longer piles should be driven deeper than for a wall with uniform toe levels, and the shorter piles shallower, by a distance equal to half the stagger height. In the free-earth case, therefore, the result is an increase in stability rather than a saving in materials.

Figure 12 illustrates staggered toe arrangements.

Figure 12 Staggered toe elevations for sheet-piled walls



6.8 In-situ capping

Embedded retaining walls are usually provided with an in-situ concrete capping. This serves as a cope and may be used as a fender support. The capping may also be used to distribute vertical and horizontal imposed loads, to support crane rails or to function as a waling.

The capping should generally be at least 0.5 m high, the actual height depending on the wall cut-off level in relation to cope level and on fendering arrangements. The seaward face of the capping should be dimensioned in relation to the face of the embedded retaining wall to give the necessary clearances (see 4.1.6.5). Reinforcement in accordance with BS EN 1992 should be provided to resist shrinkage

and thermal stresses in the beam resulting from the restraint imposed by the wall and the soil. The capping should also be provided with lateral reinforcement above the sheet piles, where troughed steel piles are used and where vertical actions are to be resisted: the transfer of vertical actions to the sheet piles should be checked. Links should be provided to each part of the capping section: this might require passing bars through holes in the piles or welding them to the piles.

If longitudinal horizontal actions are to be resisted, the junction of the capping and the wall should be designed accordingly.

Movement joints should generally be provided at 15 m to 30 m intervals. Cracking in the capping near the joints can be minimized by debonding the capping and the wall for a short distance each side of the joint. The capping should be provided with suitable means to transmit actions across movement joints where this is required.

For cappings built on sheet-piled walls, a minimum pile embedment of 75 mm is recommended. Where the capping or superstructure is stiff in relation to the wall, the bending stresses that would be set up in the wall when a rigid connection is used can be eliminated by means of a hinged joint between wall and capping. This should be designed to ensure that all horizontal and vertical actions are effectively transmitted and that soil is unable to escape through the joint. The capping may be of composite construction in which a precast concrete facing unit is provided on the seaward face and the remainder is cast in situ. This method has the advantages of ensuring a good quality, durable finish on the exposed face and of avoiding the need for formwork over water. The precast units have to be designed to allow for irregularities in the wall, especially in the case of troughed steel sheet piles.

Where separate bearing piles are provided to resist vertical actions, these should support the capping. Careful detailing is required if no vertical actions are to be transmitted from the capping to the wall, especially if the capping is to function as a waling.

6.9 Vertical actions applied to a wall

6.9.1 Types of action

Vertical actions that can be imposed on embedded retaining walls are as follows:

- a) permanent actions due to superstructure and relieving platforms;
- b) imposed loads due to cargo, ship ramps and handling equipment;
- c) vertical components of berthing and mooring actions;
- d) vertical components of anchor loads;
- e) vertical components of soil pressures resulting from wall friction, which can cause downdrag.

6.9.2 Distribution of concentrated actions

If an in-situ capping is provided, it may be assumed to distribute concentrated imposed loads at 45° to the vertical, from the limits of the contact area to the effective centre of the bottom reinforcement in the capping or to the top of the wall, whichever is higher.

6.9.3 Resistance to vertical actions

Vertical actions imposed at the cope should be resisted with minimal settlement of the structure. Load tests are desirable where isolated vertical actions are high, and the wall should be designed taking account of effects occurring during testing.

Resistance should be achieved by one of the following methods.

- a) *By the wall itself.* The toe of the wall over the length of the dispersed load should have an adequate bearing capacity, which might require that the wall be taken down to rock. In the case of troughed sheet-piled walls, the gross width of the wall toe is generally effective in medium dense granular soils and in firm clays, on the assumption that a plug will form at the toe. If plug formation is uncertain, vertical plates may be welded across the trough faces at the pile toes.

In the case of sheet-piled walls with staggered toes, it will be necessary to check the vertical resistance of both the deeper and the shallower piles to establish the net capacity of the wall.

In soldier-piled walls, only the piles should be assumed to resist the vertical actions. The effective column length of the load-bearing wall may be taken as being equal to the vertical distance between the points of zero bending moment due to horizontal actions. Composite action of the wall elements in resisting large concentrated actions should be ensured in steel sheet-piled walls by welding the upper portion of the interlocks, and in in-situ concrete walls by grouting the joints as required.

- b) *By the provision of a combined wall with load-bearing elements.* Where the embedded retaining wall has insufficient vertical resistance, a combined section may be used. In sheet-piled walls the vertical actions may be carried by H-section, box or tube piles (see also 8.12.4), and in diaphragm walls, T-shaped panels may be used. Where a combined section is required to resist horizontal actions, the same load-bearing elements may be used to carry the vertical actions.
- c) *By the provision of separate bearing piles.* Separate bearing piles may be installed to seaward or landward of the embedded retaining wall to carry all or part of the vertical imposed loads. Bearing piles are also used to support relieving platforms.

The piles should be installed at sufficient depth to ensure that there is no danger of partial transfer of the actions to the embedded retaining wall (see also 6.6.3). Bearing piles close to embedded retaining walls should be driven below the potential surfaces of sliding.

6.9.4 Uplift

Net uplift of embedded retaining walls is generally not acceptable because of the reduction in passive resistance which would result.

6.9.5 Design of load-bearing elements

The analysis of wall elements that resist vertical actions should take into account the maximum lateral actions which might simultaneously be imposed through the soil.

6.10 Berthing and mooring actions normal to the wall

6.10.1 Berthing actions

Berthing actions may be transmitted by fenders to the capping or to the embedded retaining wall. The capping may be analysed as a horizontal beam on an elastic support, due account being taken of the contribution of the wall in resisting the action. The retaining wall above relieving platforms should be designed to resist berthing actions in bending or by thrust in the counterforts, if provided.

Embedded retaining walls may be designed to resist large concentrated berthing actions applied directly at a low level, but the benefits of soil arching and wall friction should, in this case, be completely ignored in assessing the active soil pressures. Where direct berthing actions are to be accommodated, a fendering system should be selected which will distribute the actions as much as possible and avoid causing abrasion of the wall.

6.10.2 Mooring actions

The components of bollard pulls acting normal to the wall should be resisted in one of the following ways.

- a) *By the in-situ capping and embedded wall.* The action is assumed to disperse through the capping and is resisted by beam action of the capping and cantilever action of the wall above anchor level. The action is transmitted to the anchorage. This method should be used only for small bollard pulls.
- b) *By the provision of additional tie-backs.* Additional tie-backs may be provided at bollards in the form of auxiliary tie-backs to a separate anchorage (see 6.6.1.3) or additional tie-backs to the wall anchorage. Distribution of the action along the anchorage may be achieved using diagonal tie-backs arranged in opposing pairs. The tie-backs should be attached to the top of the embedded wall.
- c) *By the retaining wall above relieving platforms.* The retaining wall provided above relieving platforms is designed to transmit the action to the platform by cantilever action or by means of counterforts. The action is resisted by the raking piles under the platform.

6.11 Longitudinal imposed actions

Horizontal actions applied to the wall in a longitudinal direction by ships (via fenders and mooring lines) and cranes are resisted in bending by the wall elements in a plane at right angles to the direction of the wall. The actions should, in general, be distributed along the wall by the waling or capping.

Composite action of the wall elements is achieved by continuity of the capping with the top of the wall, by fixing the walings to the embedded retaining wall and, in the case of steel sheet piles, by welding the upper lengths of the interlocks. In diaphragm walls and in contiguous and soldier-piled walls, composite action of the wall elements is unlikely to occur without special connectors: unless such connectors are provided, these types of wall should be designed assuming that each panel or pile acts independently.

When sheet piles are used, the maximum biaxial bending effects in the wall may exceed the allowable values for single bending by up to 10%.

The capping and walings should be designed to resist the direct tensile and shear stresses caused by the longitudinal loads. The actions are transmitted to the ground by friction between the rear face of the wall and the soil and by passive resistance at projecting faces, such as the webs of troughed sheet piles, combined sheet piles and T-panel diaphragm walls. The passive resistance at any level cannot exceed the frictional resistance within the soil between the projections.

The vectorial sum of the value of wall friction used in the resistance of longitudinal horizontal actions and active pressures on the rear face should not exceed the maximum permissible value. The value used in each part of the calculation should therefore be less than the maximum.

Longitudinal actions may be transmitted to the anchorage system by means of diagonal tie-backs or a relieving platform, where appropriate.

6.12 Walings and tie-backs

6.12.1 General

Failures in embedded retaining walls have generally been due to failure of the anchorage system rather than of the wall itself. Design of walings and tie-backs and their connections should, therefore, take into account all actions which might tend to increase load effects.

6.12.2 Walings

NOTE Guidance on the design of walings is given in BS EN 1992 and BS EN 1993-5.

Walings are required to transfer horizontal actions from the wall to the tie-backs. Walings should be designed as continuous beams spanning between the tie-backs. Where the tie-backs are inclined, the walings and fixings should be designed to carry the vertical action component. Other vertical actions to be resisted are the dead weight of the waling, the weight of any soil and surcharge loads directly above the waling, and the vertical component of active soil pressure acting on the vertical plane passing through the rear edge of the waling. Walings may be used to distribute longitudinal imposed actions along the wall (see 6.11). The wall/waling connections should be capable of transferring the actions to the wall. Walings should be designed to resist the direct effects resulting, and connections to resist the additional shear effects.

Walings may be placed in front of or behind the embedded retaining wall but, in quays and jetties built in sea water, they should normally be placed behind to prevent mechanical damage and reduce corrosion. Tie-bolts or other means are required to transmit the action from the wall to the waling. Where the capping or deck projects well in front of the wall, the waling may be placed outside and the tie-bolts can be omitted. If stress reversals due to horizontal actions can occur, the waling fixings should be designed accordingly.

In sheet-piled anchor walls, walings should be placed behind the wall and tie-bolts are not then required. Steel walings comprising two channel-sections should be drilled through the web of the upper channel at intervals of approximately 3 m to permit drainage of water collecting on top, except where the walings are permanently immersed.

Where the in-situ concrete capping is to function as a waling, the connection between it and the wall should be sufficient to transmit

all internal effects. If inclined anchor piles are employed, the capping should be designed to resist any torsion resulting from vertical bending due to long-term soil actions on the anchor piles: alternatively, pinned connections may be provided.

Embedded retaining walls adjacent to relieving platforms or suspended deck quays do not require a separate waling if these structures are built in advance of the wall, thereby providing resistance to the horizontal soil actions as soon as the junction is formed.

6.12.3 Tie-backs

In maritime practice, tie-backs for embedded retaining walls are usually in the form of steel tie rods, but bar or cable tendons can also be used, especially where ground anchorages are required.

NOTE Guidance on the design of tie rods, including the importance of corrosion protection, is given in BS EN 1993-5. Guidance on the design of tendons is given in BS EN 1537.

In relieving platform structures, the platform transmits the horizontal actions from the wall to the piles. Tie-backs are commonly attached to the embedded retaining wall at a single level. This level should generally be as far below the top of the wall as practicable, usually just above low water level, in order to reduce the vertical span, and thus the bending moments in the wall. The cantilever moment in the top part of the wall further helps to reduce the span and toe moments.

Auxiliary tie-backs may be provided to the top of the wall or full tie-backs may be provided at two levels (see 6.6.1.3). Where diagonal tie-backs are provided, they should be arranged in opposing pairs.

Where ties are attached directly to the wall and no walings are used, the wall element should be designed to resist the high local compressive action which occurs.

The tie-backs should be designed to accommodate the settlement of the ground behind the wall resulting from imposed loads in addition to the effects of actions caused by berthing and mooring.

Measures which may be taken to avoid settlement effects in tie rods include the provision of hinged couplings and fixings in the rods, laying the rods to an upward curve, laying the rods on the invert of large diameter ducts, and supporting the rods or the enclosing ducts on piles. Provision should be made in ducts for the rotation and compression which can occur at the ends. Care should be taken in the design of hinged connections if stress fluctuations in the tie rods can occur.

Where the tie rod is not perpendicular to the wall, bending of the tie rod should be minimized by the use of spherical or tapered washers under the nuts or a bridle.

It is usual to apply a small pre-stress to tie rods to limit wall movement. The amount of pre-stress should be not less than the maximum amount of stress relief which could occur during the life of the structure, in order to avoid the sudden imposition of an action in variable or cyclic action conditions.

Tie rods may be provided with upset ends or rolled threads as a means of maintaining the full cross-sectional area throughout. If large stress fluctuations are likely to occur, upset ends should be avoided for steel other than low carbon steel.

Tie rods may be of low carbon or high yield steel. Low carbon steel is preferable where deflection of the wall is to be minimized and where long-term settlement could cause bending of the tie rods. Tie rods should be designed in accordance with BS EN 1993-5. In sea water conditions it is recommended that the diameter of a tie rod be not less than 50 mm.

Where tendons are to be used, particular attention should be given to the effects on the soil of cyclic actions, and of long-term settlement of the soil causing vertical actions on the tendon. Corrosion protection should be designed to accommodate the deformation of the tendon if settlement is expected.

6.13 Anchorages for quay walls

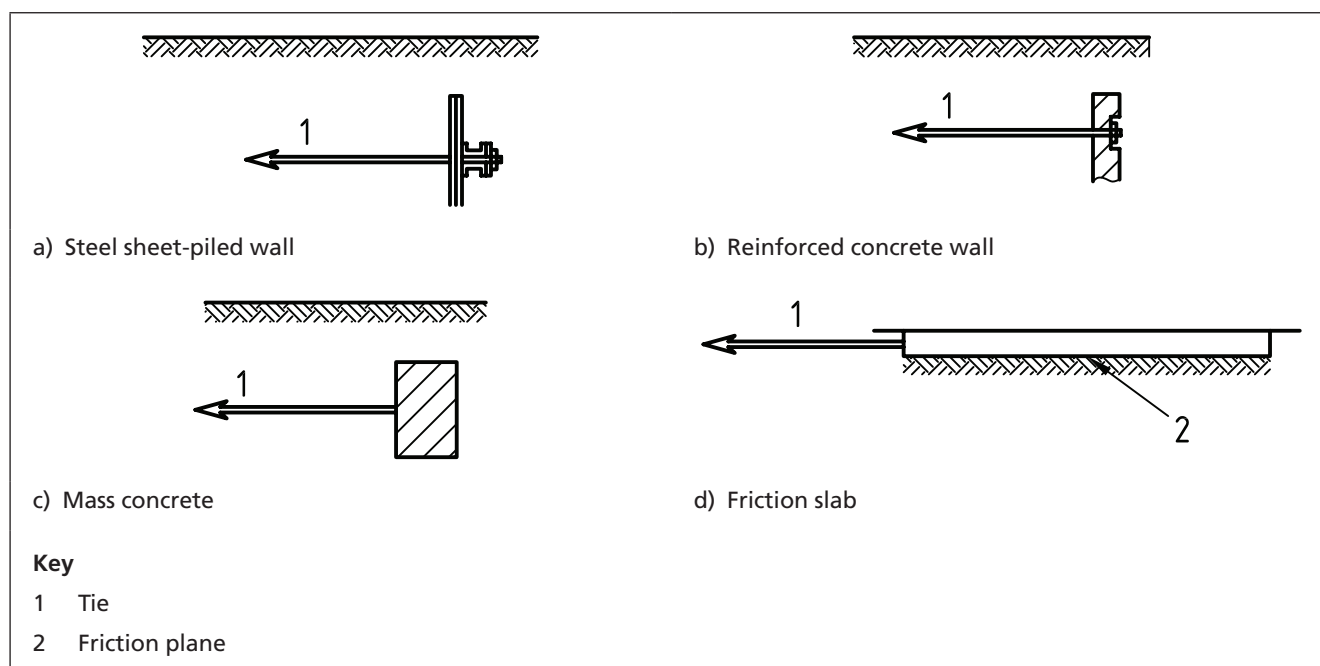
6.13.1 General

The design of anchorages for embedded retaining walls should generally be in accordance with BS EN 1997-1:2004, Section 8.

6.13.2 Deadman anchorages

Deadman anchorages include steel sheet-piled walls, concrete walls, concrete blocks and concrete slabs (see Figure 13).

Figure 13 Examples of deadman anchorages



Walls may be designed as balanced anchorages where the ground in front of the wall provides adequate resistance. Steel sheet-piled walls require a waling to distribute the tie actions. Concrete walls may be cast in situ, or precast. The type to be adopted will depend partly on existing ground and sea water levels in the vicinity.

Concrete blocks and friction slabs, which may be buried or at ground level, resist tie-back actions partly by friction at their base and partly by

passive resistance of the soil in front. They are generally less efficient than walls in achieving the required horizontal resistance and will usually be limited to walls where tie effects are not very large. This applies particularly where the ground water table can rise sufficiently to reduce frictional resistance. If a suitable gravity structure exists behind the wall, this may be used as an anchorage. Hollow anchor blocks may be used where the tie-backs are pre-stressed tendons. Stressing of the tendons can then take place in the dry compartment in the block. This arrangement can be advantageous where ground water levels are high, in which case small monoliths can be used to form the blocks.

Restraint to forward movement of deadman anchorages is provided by the passive resistance of the soil against the face of the member. Account should be taken of the location of the anchorage in relation to potential failure planes within the mass of the soil retained by the structure. An anchorage is fully effective only if placed behind zones of potential failure.

Deadman anchorages may be either continuous walls or a series of separate units. Continuous walls should be provided with suitable drainage measures to prevent differential hydrostatic pressures acting on them.

Where poor ground exists in front of and/or behind a deadman anchorage, it should, where possible, be replaced with compacted granular material in order to improve the passive resistance and reduce the active pressure on the back. Where backfill is placed against the vertical faces of buried block or wall anchorages, care should be taken to compact the fill in such a manner that the required passive resistance to forward movement of the anchorage will be obtained in a uniform manner.

Sheet-piled wall anchors may be used to resist vertical actions, such as those imposed by crane tracks, by providing intermittent deep piles and a capping of appropriate design. The vertical actions acting on sheet-piled walls are largely the same as those for embedded retaining walls (see 6.9).

Staggered toes may be provided on sheet-piled anchor walls, as described in 6.7. A stagger of 0.5 m may be adopted in anchor walls at least 2.5 m high without additional calculation.

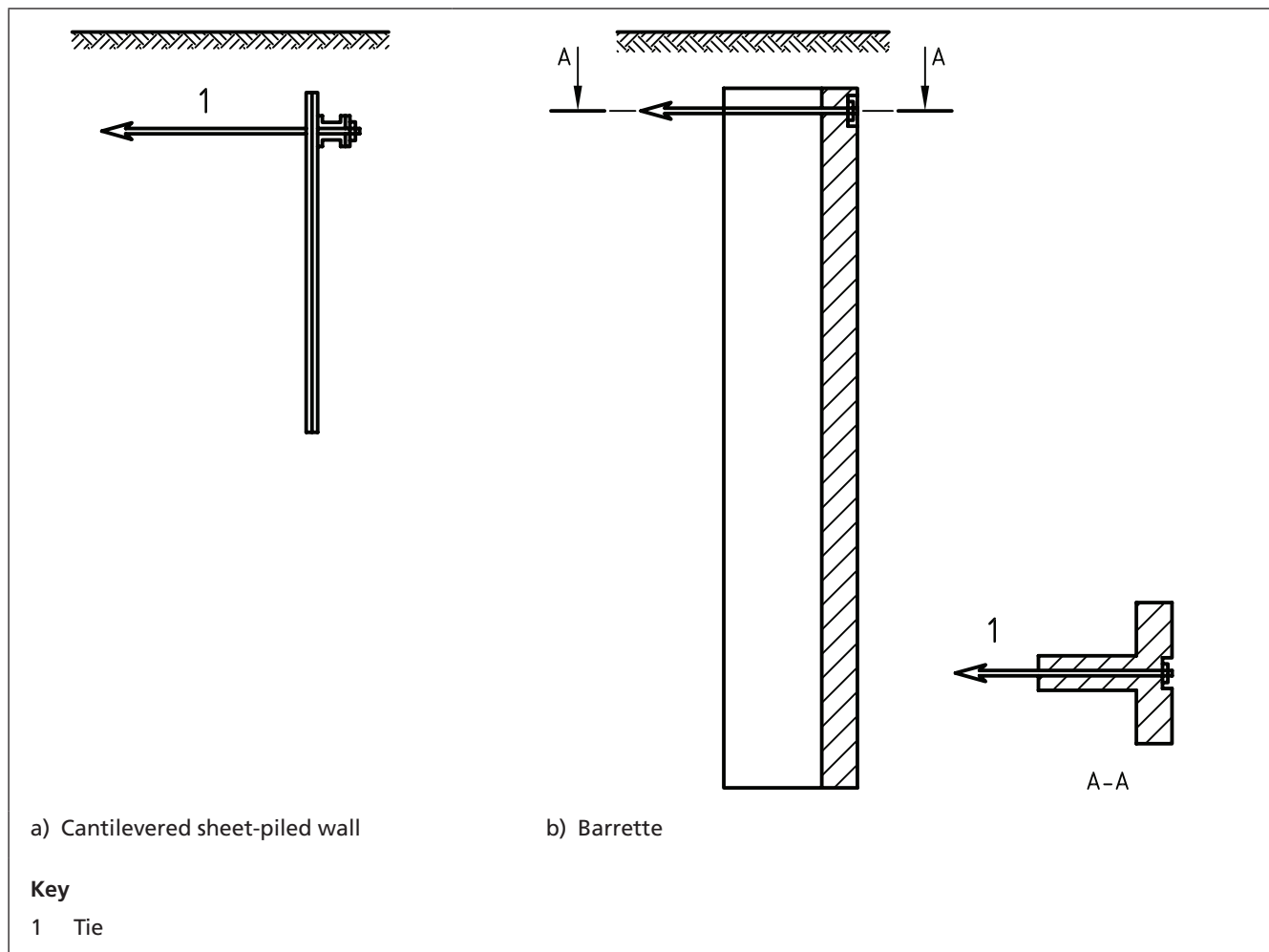
6.13.3 Cantilever anchorages

Where poor soil exists above a better quality soil, or where the ground water level is too high to suit construction of a balanced anchorage, a deeper anchor wall may be installed by driving sheet piles or forming barrettes using slurry trench construction: see Figure 14 for examples. The anchor wall is then designed as a cantilever: large bending moments and deflections can occur in this type. If the anchor wall has to be driven to a great depth, a double-wall structure (see 7.8) might be more appropriate than an anchored single-wall structure.

As for deadman anchorages, continuous anchor walls should be provided with appropriate drainage measures.

It is often convenient to design cantilever anchor walls to support crane track.

Figure 14 Examples of cantilever anchorages



6.13.4 Raking piles

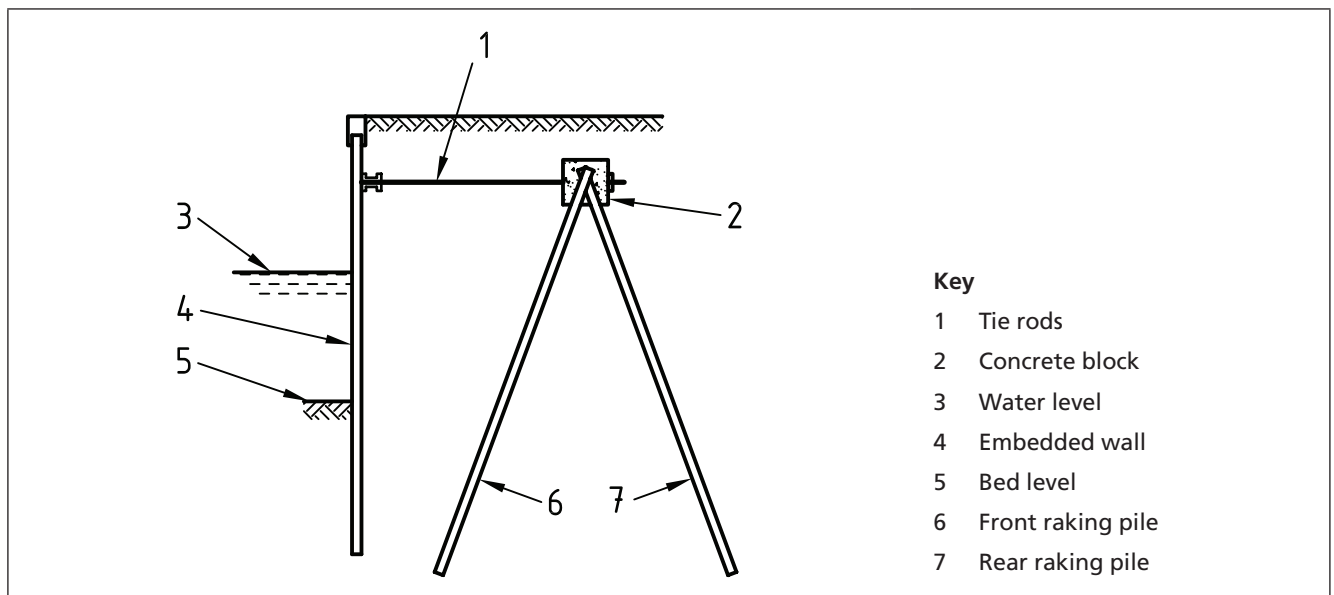
Raking piles may be used to form an A-frame tied to the wall by a rod or tendon, or as bearing piles to a relieving platform [see Figure 15 and Figure 10a)].

A-frames are often used instead of deadman anchorages where the width of ground behind the wall is limited, where poor ground exists or where the existing ground level is low. They have the advantage over cantilever anchorages in that the actions are resisted by axial effects in the piles rather than in bending, and only very small forward movement of the anchor is experienced.

The piles should, if possible, be positioned behind the active soil wedge to allow frictional resistance to be developed along the full length of the pile and to avoid effects in the piles due to the active soil wedge.

The piles in an A-frame are usually connected by a concrete cap. The dead weight of the cap and overlying soil is used to reduce uplift in the tension piles.

Figure 15 Example of a raking pile anchorage



6.13.5 Ground anchors

Ground anchors include grouted, stressed and unstressed tendons, and piles installed at a flat rake (see Figure 16). This type of anchor may be connected directly to the embedded retaining wall or waling.

Ground anchors may be used where other types would not provide the necessary resistance, where there is insufficient space for tie rods or where the ground contains services.

Tendons grouted into soil or rock should be designed in accordance with BS EN 1537. They are usually installed at an angle to the horizontal to achieve the required resistance with the minimum length of tendon. The tendons may be installed in a single row or in multiple rows.

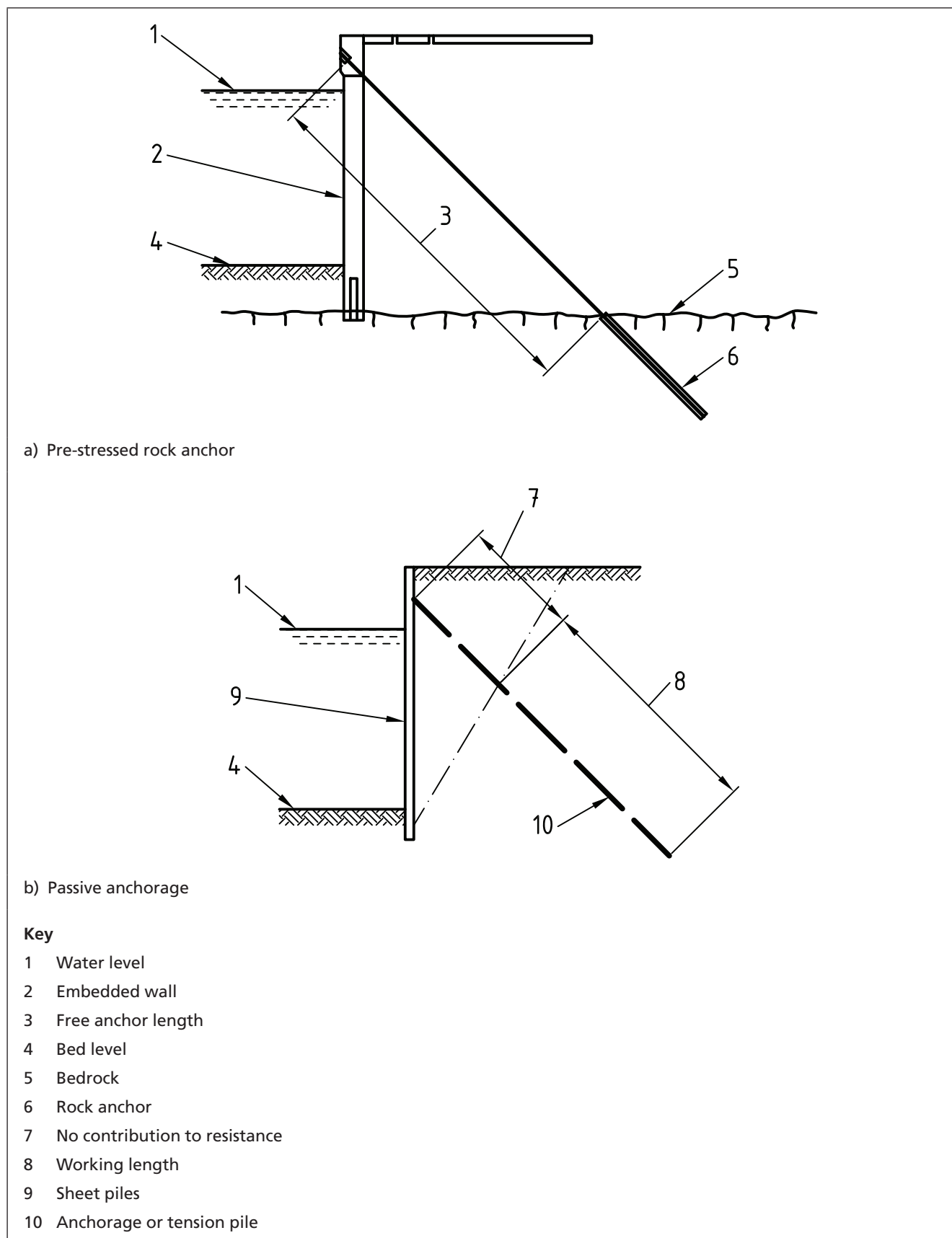
Piles most commonly used as anchors in conjunction with steel sheet-piled walls are steel H-piles, since these provide good friction resistance, can be driven to good penetration and will usually resist the effects which occur during driving at a flat rake. Driving may be carried out before or after installation of the sheet piles. Jetting should not be used for flat raked piles.

When inclined H-piles are used, the connection with the embedded retaining wall should be designed to take into account the tolerance in position which can be achieved by driving. A pinned connection may be provided to avoid setting up bending effects at this point.

Other types of pile which may be used for this purpose are grouted anchor piles and bored anchor piles. The latter type can be provided with an enlarged base.

Inclined anchor piles should be designed such that soil resistance is obtained only outside the active wedge, in the same way as grouted anchors. The piles should be designed to resist compressive actions due to berthing. Where the ends of the piles are not buried, the effective length should be used while performing the design calculations.

Figure 16 Examples of ground anchors

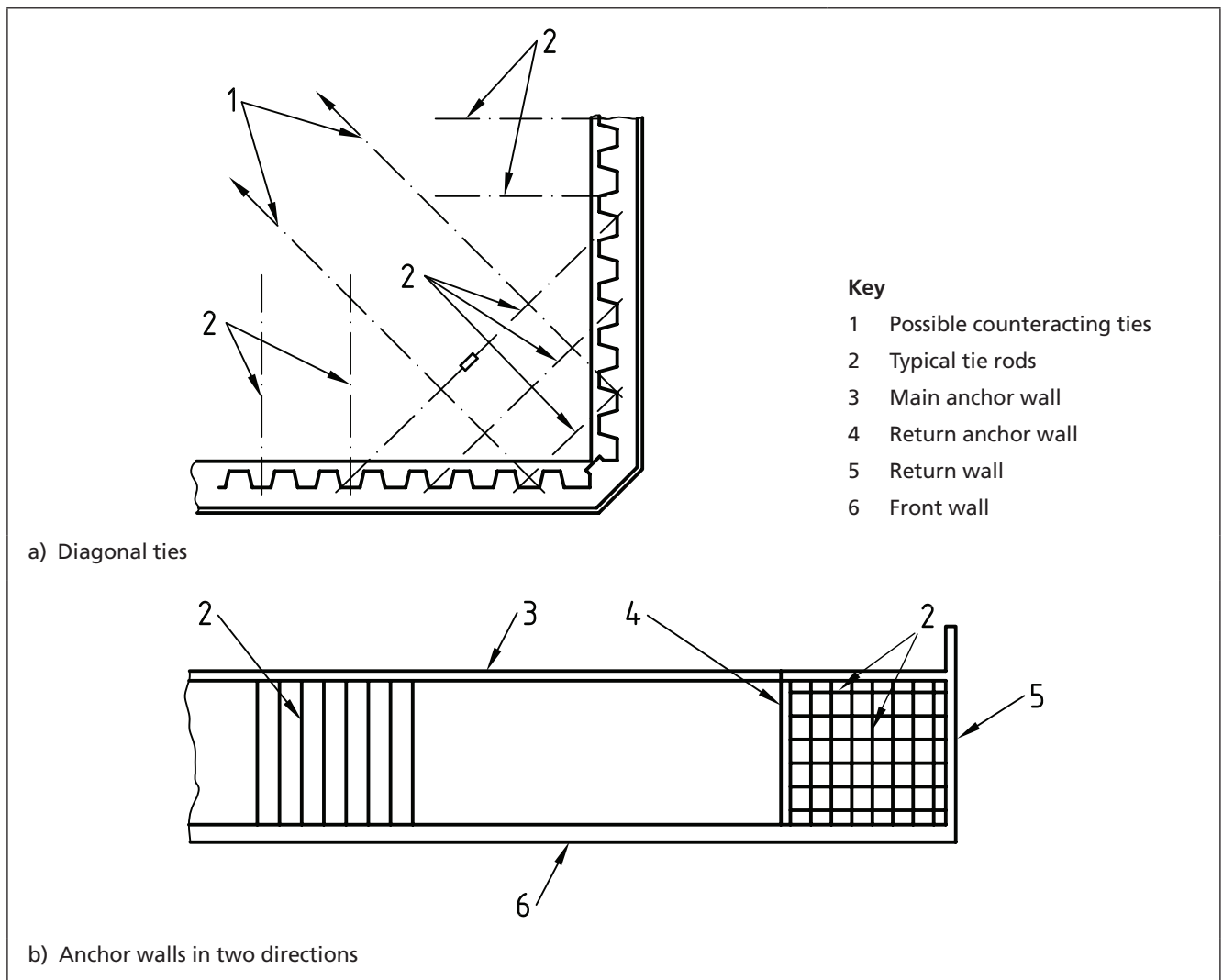


6.14 Corners

6.14.1 General

Special anchoring provisions should be made at corners to achieve sufficient resistance to horizontal soil and imposed loads and to hydrostatic actions. These provisions may take the form of diagonal ties between the two walls or anchor walls in two directions. Figure 17 shows typical examples of each type. Corners may be rounded to reduce damage both to vessels and the wall, and to improve hydraulic conditions nearby.

Figure 17 Anchoring arrangements at corners



6.14.2 Diagonal ties

Corners tied diagonally across the corner require the horizontal actions applied at or near corners to be transmitted to the soil by wall friction (see also 6.11). The effects in the diagonal ties cause additional tensile effects to be set up in the walings. These additional effects may be reduced by providing diagonal ties back to the anchor walls. The effects of the diagonal tie rods on the forces in the walings can be counteracted by the addition of a diagonal tie(s) perpendicular to the corner diagonal ties.

Alternatively, the capping may be made continuous over the end portion of the wall to distribute the actions over as great a length of wall as possible. A considerable length of wall is needed to achieve the necessary resistance. Return walls should generally be built with the same toe level as the main wall to achieve maximum resistance.

The resistance of wall panels to overturning resulting from pull-out should be checked.

6.14.3 Anchor walls in two directions

Corners comprising anchor walls in two directions, parallel to the retaining walls, should be designed in the same way as straight anchored walls. The two anchor walls should be independent of each other to achieve maximum passive resistance and avoid overstressing the walings. Tie rods and walings should be arranged at different levels for each direction.

Where the corner is in a vulnerable position, the anchor wall toes should be stepped down to the same level as the retaining wall toe to minimize loss of material in the event of damage to the corner.

Corner stability may be increased by constructing a strongpoint, in which the retaining and anchor walls form a cell. The design is similar to that of a double-wall gravity structure.

6.14.4 Relieving platform

Walls built with relieving platforms will require additional raking piles at corners to resist the actions applied in each direction.

A special relieving platform at the corner might be suitable for walls anchored by inclined piles. The platform is used as a capping block for raking piles installed to resist soil, berthing and mooring actions applied at the corner. Care should be taken to avoid interference of bearing and sheet piles. The inclined anchor piles are discontinued at the platform.

Diaphragm wall structures with inclined rock anchor tie-backs can also be strengthened at corners by providing a relieving platform supported on diaphragm wall elements to avoid excessive anchor lengths. The strongpoint thus formed may be designed as a gravity structure.

6.15 Earthworks

6.15.1 Fill

Fill placed behind embedded retaining walls and in front of and behind deadman anchorages should be granular material capable of free drainage. Loss of material through joints in the wall should, where necessary, be prevented by providing a suitable filter behind the wall (see also 4.3.3). Material of similar quality should be used to replace weak soils, such as soft clay and silt and fine-grained sand, which could liquefy in earthquakes, from behind or in front of the wall and anchorages, in order to minimize vertical and horizontal movements.

Soil replacement below seabed level should take place before piling. Material above this level may be placed before piling, in which case it will need to be dredged out.

At in-situ concrete walls, all soil replacement at the wall should be done in advance of wall construction.

6.15.2 Settlement

An appropriate allowance for or reduction of soil settlement should be made (see also 4.3.1). Account should be taken of the potential for self-weight settlements within the fill materials.

6.15.3 Drainage

Most forms of embedded retaining wall are fairly impermeable, and provisions for drainage of the fill will generally be required to reduce tidal lag (see also BS 6349-1:2000, 4.3.3 and 51.5). Drainage of the soil above relieving platform slabs should be provided where it is considered that inundation of the surfacing could occur.

6.15.4 Scour protection

Scour protection might be required in order to prevent scour of the seabed in front of embedded retaining walls below the level for which the wall is designed (see also 4.3.5).

6.16 Construction sequence

6.16.1 Effect on design

The construction sequence is a major consideration in the design of embedded retaining walls, because the distribution of stress on the wall is affected by each stage of dredging or excavation, filling, compaction and anchoring (see 6.6.1.1).

6.16.2 Wall construction

Sheet piles may be installed by impact, vibration or pressing. Driving should be carried out in such a way that additional effects due to deformation of the pile clutches are kept to a minimum.

The piles may be installed in existing ground, through fill or in water. Floating or land-based plant may be used to install the piles. Temporary fill and anchoring may be used to support land-based plant, the fill remaining in front of the wall being removed after piling. Construction may also be carried out using a piling rig running on completed sheet piles and A-frame anchorages.

Piles installed with a large unsupported length require temporary support to maintain line and to avoid buckling. Where the line of installed piles is irregular, the walings may be used to assist in straightening the top of the piles, but care should be taken to avoid overstressing the piles and walings resulting in locked-in effects. U- and Z-section steel sheet piles are normally installed in pairs to achieve a stiffer and more balanced section during driving.

NOTE 1 Guidance on the installation of steel sheet piles is given in BS EN 12063.

The construction of in-situ concrete walls is usually carried out in existing or filled ground. A considerable width might be required for the working area alongside the wall.

NOTE 2 Guidance on the construction of diaphragm walls and the use of support fluids in excavation is given in BS 6349-1:2000, Clause 55, and BS EN 1538.

6.16.3 Anchorage

It is generally convenient to install tie rods so that their centre section rests directly on the ground. This might make it necessary for backfilling to be done in two stages at walls where a considerable height of fill is required, while at walls installed in high ground, trenches will be necessary.

Filling should be programmed so that the actions imposed on the wall can be resisted at each stage by the anchorage available at that stage. The filling in front of deadman anchors should be completed before the anchorage is required to resist an action.

Care should be taken, in handling tie rods, not to damage them by impact or bending. Pre-stressing should generally be carried out at turnbuckles, since threads can be damaged if stress is applied by tightening the end nuts. Turnbuckles should be located where they can be readily accessed without leaving them unduly exposed.

6.16.4 In-situ capping

The capping can be useful in masking an irregular wall line, and its width should be designed taking the driving tolerances into account. Where straightening of the wall is required, this may be achieved by temporary walings if the capping is to function as a permanent waling. The temporary support should be removed only after the concrete has gained enough strength to stand unsupported.

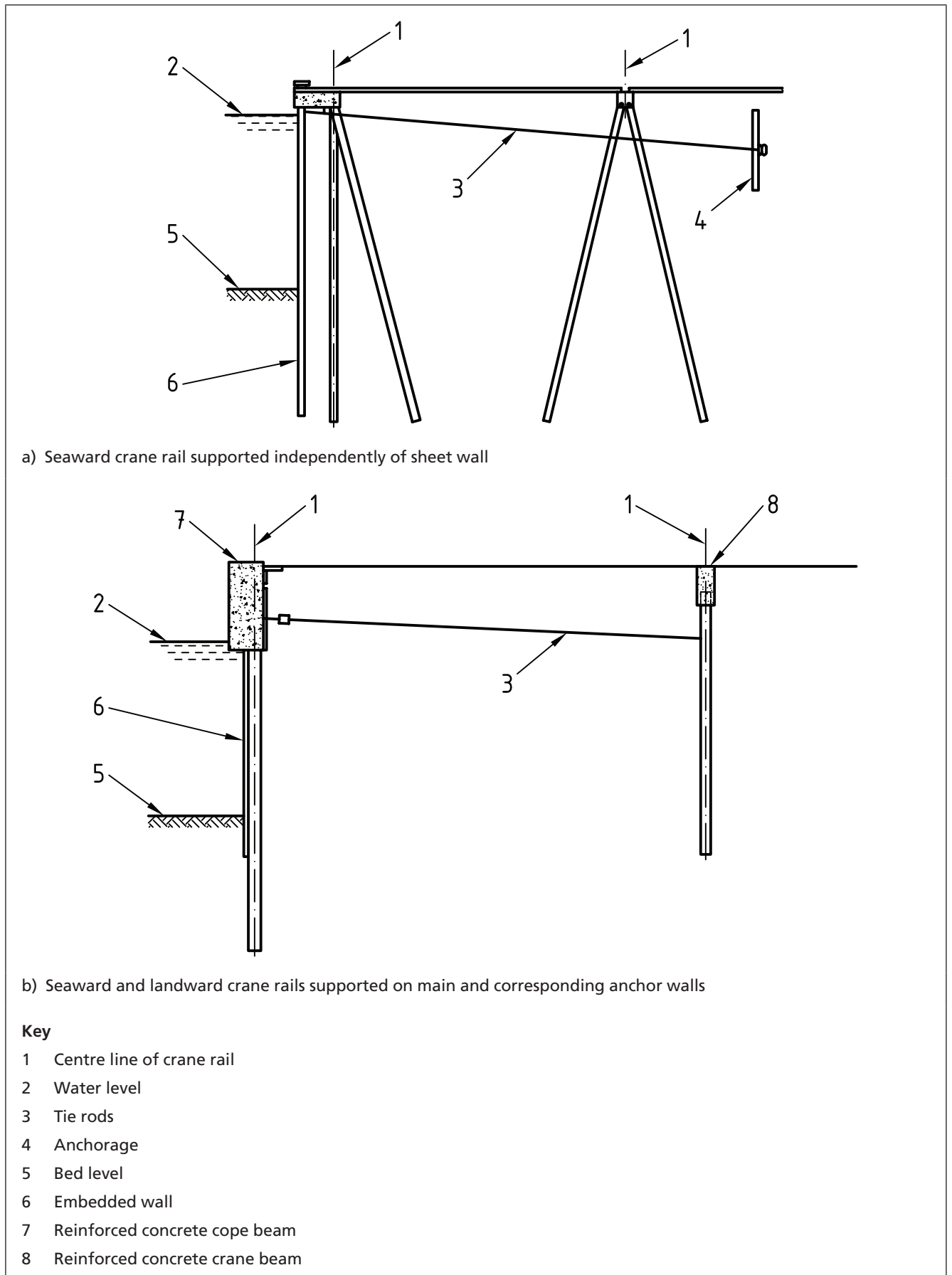
6.17 Services and crane tracks

Services at embedded retaining wall quays may be buried in the ground or in the fill above relieving platforms, with access points as required, or laid in duct. Concrete ducts may be built behind the wall or as an integral part of the capping. Care should be taken to ensure that services do not interfere with the tie-backs.

When designing service connections between wall and backfill, differential movement should be allowed for. Ducts for rail-mounted crane power supplies should be constructed integrally with the crane rail beam where there is any possibility of differential movement between beam and fill.

The seaward rail of quayside cranes and bulk handling equipment may be mounted on the capping directly above the embedded retaining wall, or on a beam supported on separate bearing piles. The beam may be part of the capping or independent of it. The landward rail may be mounted on a ground beam, on a beam supported on piles, on an anchor wall or on a relieving platform. Maintenance of rail centres is achieved by means of tie beams connecting the rail beams. Rails should not be fixed until the tie beams have been constructed. Figure 18 shows examples of typical crane rail foundation arrangements.

Figure 18 Examples of crane rail foundation arrangements



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7 Gravity walls in quay and jetty construction

7.1 General

Gravity structures are usually used where the seabed is of good quality. They may be used where the foundation near dredged level is of rock, dense sand or stiff clay. Some types may be founded on weaker soils if the resulting movements are acceptable, or if the soil is dredged and replaced with a granular material or rubble.

Gravity walls are built behind a cofferdam in the dry and are usually constructed in situ, but most walls are constructed in water by a method used only in maritime works, in which large precast units are lifted or floated into position and installed on a prepared bed under water. This form of construction is dealt with in detail in 7.4.

Gravity walls used in maritime works are generally required to retain reclaimed ground, the quality of which can be selected. It is usual to use rubble or a free-draining granular fill immediately behind a quay wall so that the effects of tidal lag are minimized and earth pressures are reduced (see also 4.3).

7.2 Types of wall

Gravity quay walls can be divided into three categories:

- category 1: concrete or masonry blockwork walls, precast reinforced concrete walls, concrete caissons, cellular sheet-piled structures, double-wall sheet-piled structures;
- category 2: in-situ mass concrete walls, in-situ reinforced concrete walls;
- category 3: diaphragm wall construction, monoliths.

Walls in category 1 would normally be constructed in water using floating plant, those in category 2 in the dry, often behind cofferdams, and those in category 3 from ground above water level, the soil in front of the wall being subsequently removed. Subclauses 7.4 to 7.12 describe some common types of gravity wall in these categories. The selection of the most appropriate type of gravity wall will be influenced by the site conditions and method of construction.

7.3 Design of gravity walls

7.3.1 Principal modes of failure

7.3.1.1 General

The principal modes of failure of a gravity structure are:

- deep slip;
- overturning;
- sliding;
- foundation failure.

Earth pressure calculations and general design principles should be in accordance with BS EN 1997-1.

7.3.1.2 Deep slip

The stability of the earth mass in which the wall is constructed should be assessed in accordance with BS EN 1997-1.

7.3.1.3 Overturning

The global stability of the structures should be verified in accordance with BS EN 1997-1 using the partial factors given in Annex A of the present part of BS 6349. The submerged weight of underwater portions of the wall should be used in calculating the resisting forces.

7.3.1.4 Sliding

Base resistance to sliding should be calculated in accordance with BS EN 1997-1.

Where the walls have been sunk or driven on to a firm stratum, the analysis of sliding resistance is similar to that for walls constructed on land. However, for many forms of maritime structure, the wall is founded on a granular levelling bed laid on the existing or dredged seabed. In this case it is also necessary to check for shear failure through the granular bed and at the seabed interface (see 4.2.6). Where the construction sequence involves laying the bed well in advance of the wall, it might be necessary to remove any fresh silt layer by suction cleaning immediately prior to placing the wall units. A similar problem can arise where there is a delay between placing successive layers of concrete blocks. Walls can have a sloping or a serrated base to improve the resistance to sliding. The depth of the serrations should be comparable to the average stone size of the rubble bed; an angle of friction $\delta = \phi'$ may then be assumed instead of $\delta = 2/3\phi'$. For economy, walls should wherever possible be designed to yield, in which case active pressures due to retained soil may be used. Where there is doubt as to the degree of movement which will occur, earth pressure at rest should be used in the investigation of sliding, with allowance for water pressure difference, hydraulic uplift and all disturbing forces such as mooring loads.

The most critical case might be the temporary condition during construction, especially where hydraulic fill is being placed behind a wall. Passive resistance in front of the toe of the wall should be taken into account only when there is no possibility of the material being removed by scour or dredging.

7.3.1.5 Foundation failure

Foundations for gravity wall structures should be designed in accordance with BS EN 1997.

7.3.1.6 Control of settlements

The effects of settlement of the foundations, if any, on the quay wall should be taken into account.

NOTE Preloading of the structure may be used to reduce settlement of the structure in operational use. Preloading is usually carried out by placing kentledge onto the structure or surcharging to replicate the most onerous combination(s) of variable actions used in the design. Preloading is usually left in place until the rate of settlement has met a chosen design criterion.

7.3.2 Distribution of concentrated actions

Typical concentrated actions are horizontal actions due to berthing and mooring operations and vertical loads from ship ramps, mobile crane outrigger pads or rail-mounted cranes. The area of contact for the berthing action will depend on the profile of the ship's hull, the angle of approach and the fender system (see BS 6349-4). In general it is necessary to take into account the effects of a concentrated action over the whole length between movement joints of the structure to which it is applied; actions applied near a construction or movement joint or the end of the wall usually cause greater difficulties. When calculating bending moments in the capping, considered as a beam, it is normal to assume a 45° dispersion through the in-situ concrete from the limits of the contact area to the effective centre of any tensile reinforcement.

7.3.3 Materials and stresses

The materials used in the construction of gravity walls should be in accordance with BS 6349-1.

The stresses obtained by analysis should be in accordance with BS EN 1992, BS EN 1993, BS EN 1995 or BS EN 1997 as appropriate to the material under consideration.

7.3.4 Detailing

7.3.4.1 Concrete capping

Most forms of gravity wall should incorporate an in-situ capping of plain or reinforced concrete, to provide a true and level cope. This capping can be used to overcome any irregularity due to misalignment, settlement or construction tolerances, and to distribute concentrated actions into the structure. It should be placed after initial settlement has taken place and keyed into the wall below. Appropriate means should be provided to transfer horizontal actions across movement joints and to link the individual units of the wall.

7.3.4.2 Services and crane rail tracks

Ducts for services and foundations for crane rail tracks may be accommodated in the concrete capping.

7.3.4.3 Front face

A batter may be provided on the front face of a quay wall to compensate for forward tilting due to foundation settlement after backfilling.

Care should be taken to ensure that the necessary berthing clearances are achieved (see 4.1.6.5). This generally requires the face of the in-situ capping to project to seaward of the wall face. Precast facing units may be used to avoid shuttering over water.

7.3.4.4 Prevention of loss of material

Careful detailing is required to prevent loss of material from behind or in front of the wall. Filters should be used to prevent loss of fine material through joints in walls (see 4.3.3). Where a watertight structure is required, the joints should be sealed. Scour and wave action can remove material in front of or underneath the structure. This should be prevented by the use of a protective anti-scour apron

in front of the wall. This should extend outward at least 1.5 times the beam of the biggest design vessel.

Material can also be lost from underneath the structure if a large hydrostatic head occurs behind the wall or artesian pressure exists beneath it. To prevent this, suitable filters should be provided.

7.4 Concrete blockwork walls

7.4.1 General

Heavy precast mass concrete blocks provide a robust maintenance-free structure.

The size of blocks used can be chosen to suit the availability of plant. Blocks of about 15 t are likely to be the smallest that will be used. Larger blocks will generally prove more appropriate on large projects or where heavy cranes are available.

7.4.2 Types of construction

7.4.2.1 Bonded blockwork

Typical examples of bonded blockwork are shown in Figure 19a). This form of construction is less tolerant of differential settlement than sliced or column blockwork, although the effects may be reduced by completing each course of construction before placing the course above.

Bonded blocks are often keyed to provide better interlock and to assist in placing under water. Instead of returning the wall at the ends to retain the filling, a scar (or stepped) end may be formed to protect the slope to the edge of the fill behind. This facilitates the construction of an extension without causing additional settlement in the completed wall.

7.4.2.2 Column block work

In column blockwork, the blocks are stacked in vertical columns: an example is shown in Figure 19b). The main advantage of this form of construction is that it can accommodate differential settlement.

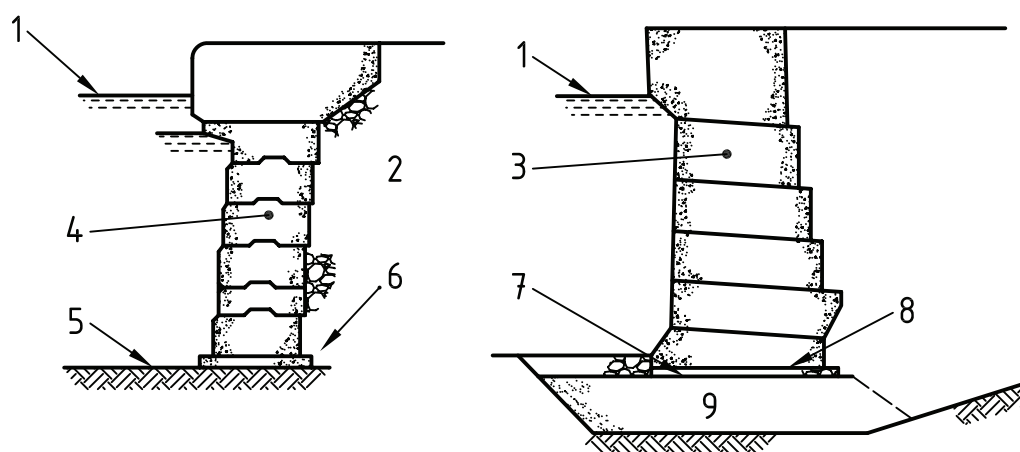
If ground conditions are suitable, it is possible to preload the columns to minimize long-term settlements and then construct a keyed joint between them. If long-term differential settlement is expected, keys between adjacent columns are omitted.

A scar (or stepped) end may be formed in this type of wall to protect the fill.

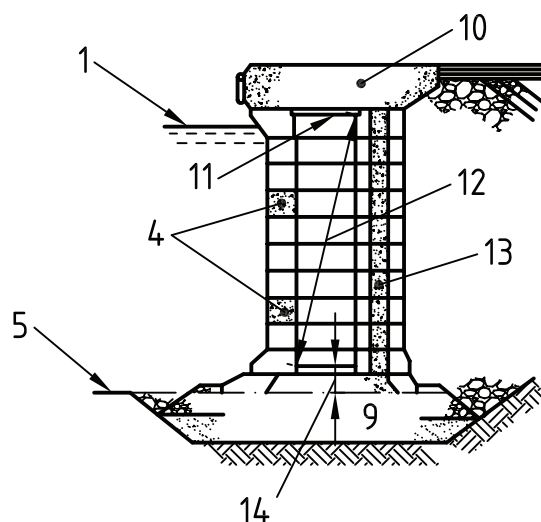
7.4.2.3 Sliced blockwork

Blocks in sliced blockwork construction are laid at an inclination of 10° to 25° from the vertical. The short scar (or stepped) end in this form of construction permits the use of a smaller crane than other types of blockwork. Specially shaped base blocks and abutments are required, and wedge blocks might have to be provided to correct the inclination from that specified. As a consequence, a large number of different block types are required. Blocks are provided with a tongue-and-groove interlock. Blocks are placed with the tongue uppermost so that the groove of succeeding blocks can be engaged and the block slid into place. The effects of differential settlement may be partially accommodated along the inclined slip planes.

Figure 19 Concrete blockwork walls



a) Examples of bonded blockwork on rock and prepared bed foundation



b) Example of column blockwork wall where lifting capacity is limited and foundation conditions are relatively poor

Key

1	Water level	8	Serrated base
2	Rock fill	9	Granular base
3	Precast concrete blockwork wall	10	In-situ concrete capping block
4	Precast concrete blocks	11	Precast slab
5	Bed level	12	Void
6	In-situ underwater concrete base	13	In-situ concrete plug
7	Gravel bedding layer	14	In-situ concrete in base block cavity

7.4.2.4 Open blockwork

Blockwork may be of partly open construction to provide partial absorption of incident waves.

7.4.2.5 Arch and buttress blockwork

This type of construction is suitable only where a rock foundation exists, since differential settlement cannot easily be accommodated by this form of gravity wall. Less concrete is required than for the other

types of blockwork, and sliding resistance can be critical. Construction can take place in the dry or over water by supporting lifting equipment on the previously constructed buttresses. Two basic types of block are used in arch and buttress blockwork, one for the buttresses and one for the arches between. Blocks are placed vertically above each other.

7.4.3 Shape of blocks

Blocks should be robustly proportioned and arrises should be chamfered to minimize spalling and damage during construction, due to high pressures in service. Horizontal shear keys should be provided with adequate clearances to permit placing to the required tolerances.

Blocks are most commonly solid, but hollow blocks may be used to reduce handling weights in column and in arch and buttress walls. The voids, which may be formed by internal openings or by the spaces left between I-shaped units, can be filled with mass concrete or granular material to increase the wall mass. Where concrete filling is used, the joints should be sealed to prevent loss of grout. In bonded construction, interlocking dumb-bell-shaped blocks may be used.

7.4.4 Foundation

A rubble base foundation at least 1.0 m thick should normally be provided on top of the founding stratum: this thickness may be increased if the bed material is too weak to resist the applied pressures. If the seabed is sandy, a filter layer of graded gravel should be placed between the rubble and subsoil to minimize settlement into the seabed.

The base should be topped with a bedding layer capable of being screeded to level by divers. The base and bedding are sometimes laid at a slope to increase resistance to sliding. If settlement is expected, this may be allowed for by constructing the bedding higher than the final required level, or at a greater slope. The importance of keeping the surface clean is discussed in 7.3.1.4. The tolerance in level of the bedding layer surface should not exceed ± 10 mm in sheltered waters, but depends upon the material used.

The base should extend at least 1.0 m on each side of the wall base to accommodate any construction tolerance and to spread the actions. On the seaward face, where the bedding might be disturbed by currents or propeller action, it should be protected by an anti-scour apron. The foundation in front of the rubble base can also be liable to scour, requiring extension of the anti-scour apron beyond the 1.5 \times beam recommended in 7.3.4.4.

If the seabed is rock, the rubble base may be omitted and replaced with an in-situ concrete blinding layer. The thickness of this layer should typically be 0.3 m, unless construction is undertaken in the dry, in which case it can be reduced to a minimum of 0.15 m.

7.4.5 Joints between blocks

Blocks are laid with dry joints. Nominal widths for vertical joints of 75 mm in column blockwork and 40 mm in bonded blockwork are typical.

Where practicable, blocks should be placed to within ± 25 mm of the specified position. Provision should be made to prevent loss of fine material through the joints.

Where a watertight structure is required, the joints can be sealed using a grout-filled canvas tube placed in a preformed recess on the front face. In some cases, both faces are sealed in this way to allow grouting of the whole of the joint between blocks.

7.4.6 Prevention of voids

Where the back face of the wall projects into the fill, care should be taken to prevent the formation of voids behind the wall.

Fill materials should conform to **4.3.2**.

7.4.7 In-situ capping

Movement joints in in-situ concrete cappings are normally spaced at a minimum of three block-lengths, but in general at a maximum of 15 m, to accommodate thermal effects and any movement due to settlement. The spacing will also depend upon the cross-sectional dimensions and structural function of the capping.

7.4.8 Design of blockwork wall

7.4.8.1 General

The cross-section of the wall and the size of individual units should be selected so that the stability criteria are met, both at foundation level and at all horizontal joint levels. Individual blocks or combinations of blocks should be stable at all stages of construction and backfilling.

7.4.8.2 Ground pressure

A more even ground pressure can be achieved by projecting the toe of the wall beyond the front face. To achieve the necessary clearances, the maximum toe projection may be located within the thickness of an anti-scour apron. A more even pressure can generally be achieved by extending some of the blocks on the landward side to counterbalance the disturbing earth actions. These blocks should generally be shaped to prevent formation of voids in the filling, although this might not be necessary where rubble backfill is used and where water movements or earthquakes are not expected.

7.4.8.3 Virtual back

Where the rear edge of any block projects significantly behind the back of the wall above, overall stability at the level of the underside of the projecting block should be calculated assuming the existence of a virtual back extending vertically upward from the rear of the block.

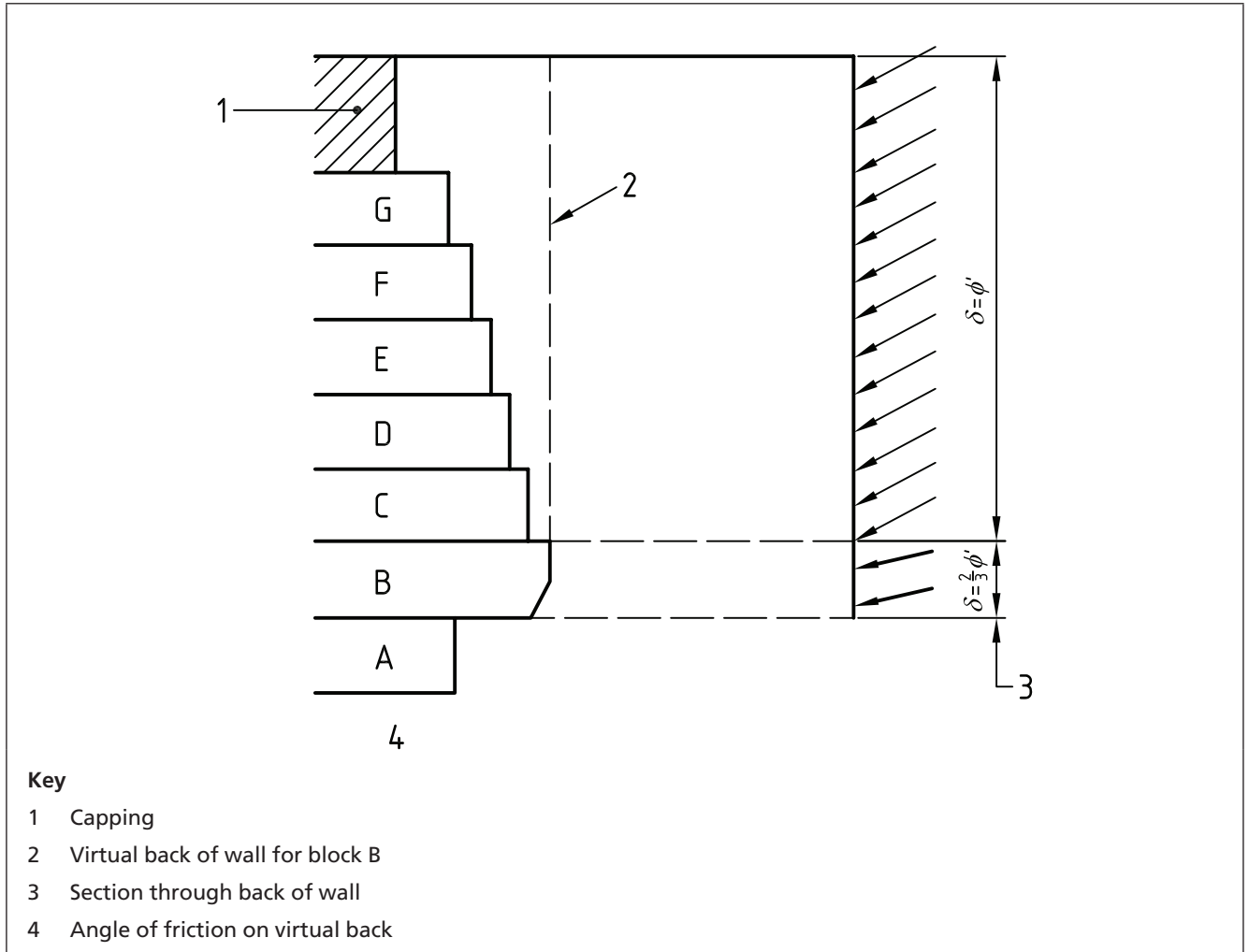
The lateral pressure at failure by sliding or tilting forward should be taken as acting on concrete elements which abut or intersect the virtual back at an angle of $2/3\phi'$ to the horizontal (see Figure 20).

7.4.8.4 Relieving effect of overhanging elements

Where the rear face of a capping block or precast block projects behind the rear face of the block below, partial relief of the lateral pressure over the upper part of the lower block may be assumed. This partial relief is similar to that which occurs under a relieving platform [see Figure 11a)], and may be assumed to apply in overall stability

calculations where the rear face of the lower element lies on or behind the back or virtual back of the wall. Where a large overhang occurs, as in the case of block B in Figure 20, a more thorough analysis will be needed.

Figure 20 Angle of friction on virtual back of blockwork wall



7.4.8.5 Block interfaces

At the horizontal block interfaces, the eccentricity of the resultant of the vertical and horizontal actions should be limited such that the concentration of the local effects in the blockwork and any bedding material does not cause failure. Except under accidental actions, the eccentricity should not exceed one-third of the width of the common interface.

The sliding resistance at horizontal joints with no shear keys depends on the type of surface finish on the top and bottom of the blocks. Limiting coefficients of friction between two plain surfaces of precast concrete units lie typically within the range 0.4 to 0.6.

7.4.8.6 Hydrostatic uplift

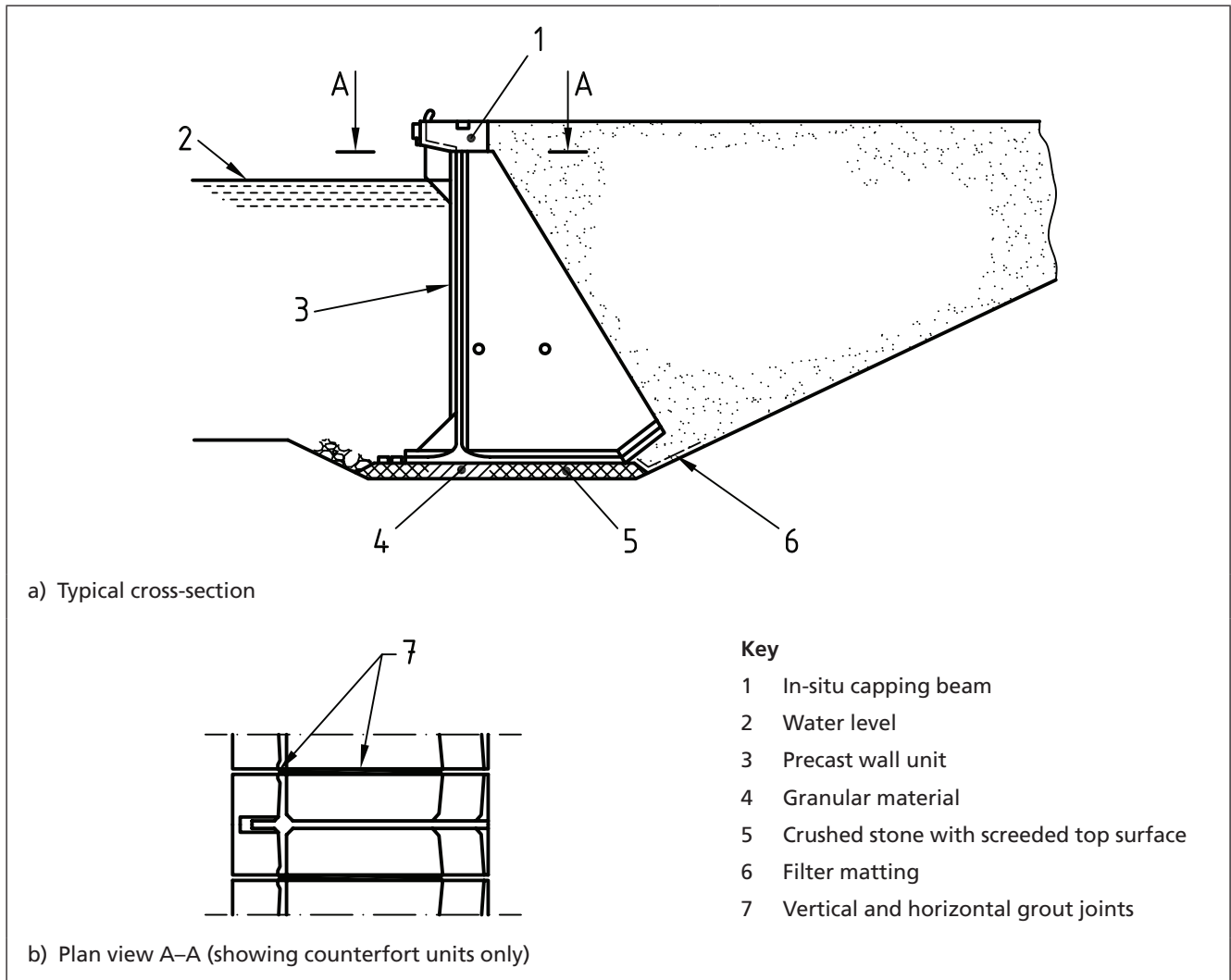
Full hydrostatic uplift should be assumed in horizontal joints. Where sealed joints are provided, the possibility of uplift should be taken into account.

7.5 Precast reinforced concrete walls

7.5.1 General

Precast reinforced concrete walls are used in maritime works, both as quay walls and as bulkhead walls adjacent to suspended deck quays. Plain cantilever walls are not generally used for heights in excess of 8 m, and for greater heights a counterfort wall may be used. Units more than 18 m high have been used (see Figure 21).

Figure 21 Precast reinforced concrete wall



An in-situ concrete capping is cast above the precast units after filling. Less concrete is required than for a blockwork wall, but a considerable weight of reinforcement has to be provided and attention has to be paid in design to minimizing the possibility of corrosion. Precasting of the wall units is more difficult than for blocks and might require slipforming.

The viability of using this form of construction for deepwater berths depends on the availability of suitable lifting equipment, either floating or on a jack-up pontoon, and on there being a sufficient length of wall to justify mobilizing such plant and formwork. Tight construction tolerances are essential, and this type of wall is unlikely

to be suitable at sites subject to appreciable wave or current action because of the difficulty of placing the precast units.

7.5.2 Foundation

The recommendations given for the foundations for concrete blockwork walls (see 7.4.4) and concrete caissons (see 7.6.3) apply equally to those for precast reinforced concrete walls. For high walls, the tolerance on the level of the top of the rubble foundation should be determined in relation to the design of the joints between units.

7.5.3 Precast units

The toe of the precast units should be arranged so that it does not project into the berthing area (see 4.1.6.5).

Crack widths in the walls of the precast units should be determined for all maximum bending conditions, including during each stage of construction. Crack widths are discussed in 4.4.2.3.

Methods of lifting and handling the units should be devised during design. Lifting points should be specified. Generous splays should be provided at internal corners to minimize local cracking. Arrises should be chamfered to minimize mechanical damage.

7.5.4 Joints between units

The vertical joints between the wall units should be sealed before backfilling is started. A rigid joint, such as a grout-filled canvas or plastics tube placed in a preformed recess, may be used. If settlement or horizontal movement is expected, a flexible joint will be required.

The joints should be designed to take account of tolerances in casting and placing and differential settlement. The possibility of impact damage in the area of joints during placing of the units should be taken into account.

7.5.5 Fill and drainage

Fill material placed behind the wall units may be fine granular material or rubble. The maximum size of stone might have to be restricted on account of the high local actions which could be imposed on the wall. If the tidal range is significant, drain holes should be provided to reduce tidal lag. A suitable filter should be installed around drainage holes to prevent loss of material.

7.5.6 In-situ capping

The capping should not be cast until the adjacent filling has been completed in order to reduce the effect of movement of the precast units. Movement joints should generally be provided in the reinforced concrete capping at centres not exceeding 30 m. The capping should be effectively anchored to the top of the wall and to the counterforts.

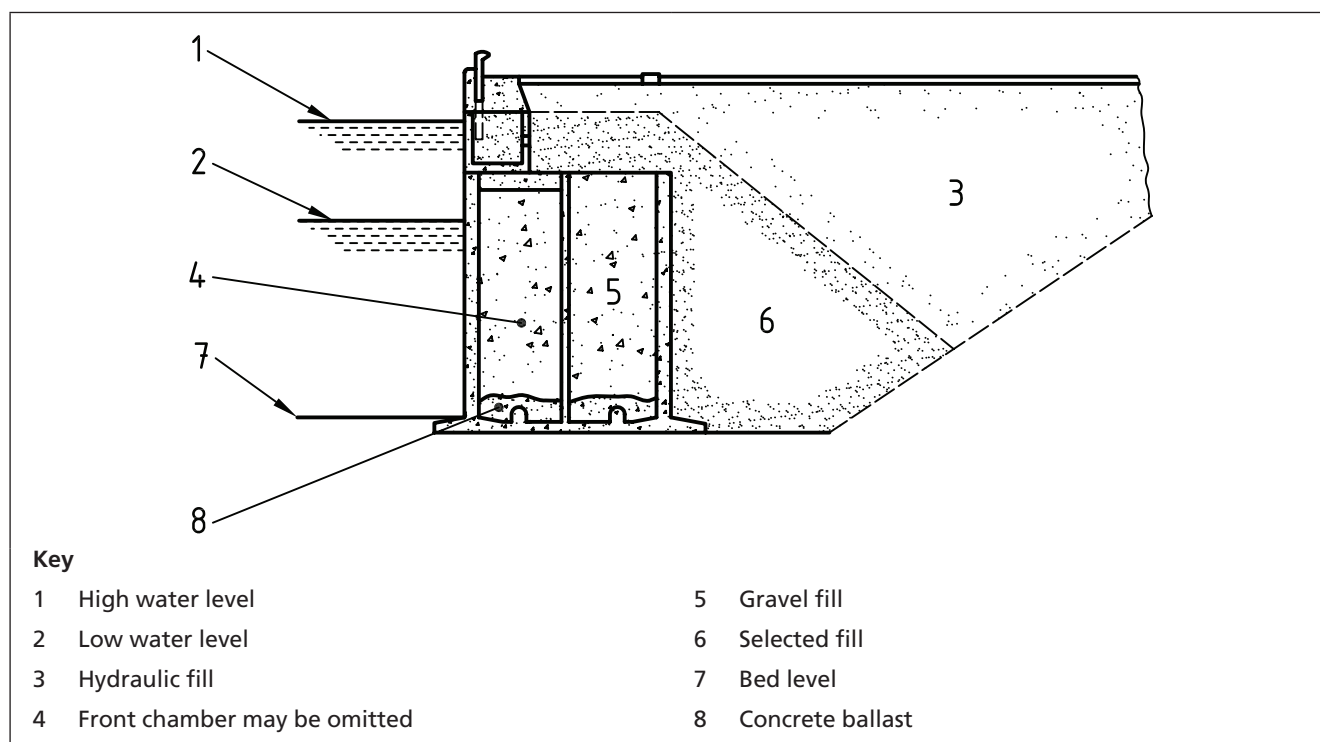
For cantilever walls, the capping beam is supported both vertically and horizontally. For counterfort walls, the capping beam is supported vertically by the vertical wall, but horizontal actions are resisted only at the counterforts. The capping and fenders should extend sufficiently to seaward of the wall to avoid vessel impact on the wall units.

7.6 Concrete caissons

7.6.1 General

Concrete caissons consist of open-topped cells prefabricated in the dry, which are usually floated to their final location and then sunk into position on the seabed. Caissons are usually designed so that, after sinking, the top is just above low water level with due allowance for waves. The cells are filled, usually with sand and sometimes with concrete or gravel. The superstructure may consist of a solid in-situ concrete capping, or a reinforced concrete edge retaining wall which is backfilled and the top surfaced with concrete paving. One possible capping arrangement is shown in Figure 22.

Figure 22 Concrete caisson



The positioning tolerance for caissons can usually be greater than that for blockwork or precast wall units. Caissons can therefore often be used where wave disturbance is appreciable. Caissons, after filling, form self-stable structures which can be used to support heavy construction equipment. They can be used for both quay walls and jetties.

This method of construction is more appropriate when there is a considerable length of quay to construct, and where there is sufficient water depth to allow the manoeuvring of the floating caissons between the casting area and the final location.

7.6.2 Shape

Concrete caissons may be built in a wide variety of shapes on plan. The most common shape is rectangular, while circular and cloverleaf shapes are also used.

Caissons are usually limited to about 30 m in greatest plan dimension to avoid high longitudinal effects, but units more than 100 m long have been used.

Large caissons will generally need to be strengthened with internal walls. These also permit economies to be made in the base and wall thickness. The walls are usually cast in situ, but precast elements have also been used in conjunction with post-tensioning of the finished structure. The compartments of cells may also be used as ballast tanks while the caisson is floating.

An outer projection is often provided on the base slab to increase stability. This should be protected or designed so that it is not liable to accidental damage by vessels.

Caissons without compartments may be built using precast concrete units as a shutter for placing thick mass concrete walls. After the caisson has been positioned, tremie concrete is used to form the internal base and to fill the wall voids. A caisson centre may be left unfilled to reduce bearing pressures and virtual mass during earthquakes. Caissons should be provided with generous splays on internal corners to avoid local cracking and with chamfered arrises to minimize mechanical damage.

7.6.3 Foundation

Caissons should be placed on a granular foundation, the nominal size of which depends on the wave action expected during construction and in service. It might be possible to level off an existing sandy bed to serve as the foundation, provided that caissons are placed in calm weather, that a protective, anti-scour apron is laid, and that the necessary sliding resistance can be achieved.

Where existing bed material is inadequate or irregular, it should be replaced to the required depth with suitable material to ensure even seating. Alternatively, an even bedding can be obtained by grouting under the caisson after placing. This also increases the sliding resistance and reduces the disturbance of the foundation material by wave action.

If the bearing capacity of the soils beneath the caisson is inadequate, then this can be addressed by a number of methods such as grouting, vibro replacement, soil mixing or piling.

7.6.4 Fabrication

Caissons may be fabricated in the dry by the following methods:

- a) on land adjacent to water, then launch either by sliding the caissons down a prepared slope, lowering the caissons using a crane or lifting dock, or by controlled dredging away of the ground in front of and beneath the caissons;
- b) behind a bund, then launch by removing the bund and floating the caissons at high water;
- c) in a dry dock;
- d) in a floating dock.

It is also possible to construct caissons on land or on a pontoon, by casting the minimum height of caisson required to allow it to float, and then launching it. Fabrication is completed by casting the remaining height of the walls while afloat, the draught increasing progressively.

Casting caissons afloat requires calm water conditions.

7.6.5 Floating conditions

The stability of a caisson should be checked for all conditions such as casting (if over water), launching, towing and sinking. The effect of waves, especially those of long period, should be taken into account. In the static and sinking conditions, the trim of a caisson may be adjusted by ballasting. If it has to be towed more than a short distance, the additional navigational and stability requirements should be taken into account.

An accurate knowledge of tidal conditions is essential to the success of the positioning operation. To assist in the control of sinking the caisson, stopcocks may be fitted in the outer walls.

7.6.6 Fill

Caissons are usually filled with sand, which may be pumped or tipped. Compaction by vibration can be carried out to achieve a secure formation to the superstructure, but can also increase internal soil pressures in the upper parts of the fill.

The seaward compartments can be filled with lean-mix concrete if conditions are such that the front wall could seriously deteriorate, or in order to provide increased resistance to vessel impacts. Otherwise the front cells may be left either empty or partially filled to adjust the overall centre of gravity and reduce bearing pressures.

7.6.7 Joints between caissons

Joints between caissons should be sealed when they are used to retain material and when waves are to be prevented from passing through the gaps. The joints should be designed to take account of placing tolerances and uneven settlement. A placing tolerance of ± 150 mm should be achieved in sheltered waters.

In-situ concrete keyed joints are normally used between rectangular caissons, placed within vertical recesses formed in the outer walls of each caisson. Tongue-and-groove joints and flexible seals are suitable only where differential settlement is likely to be small. For circular caissons, keyed joints may be used, but where this would result in casting difficulties, an in-situ concrete seal may be cast against in-situ formwork on the shore side of the junction.

Examples of joints are shown in Figure 23.

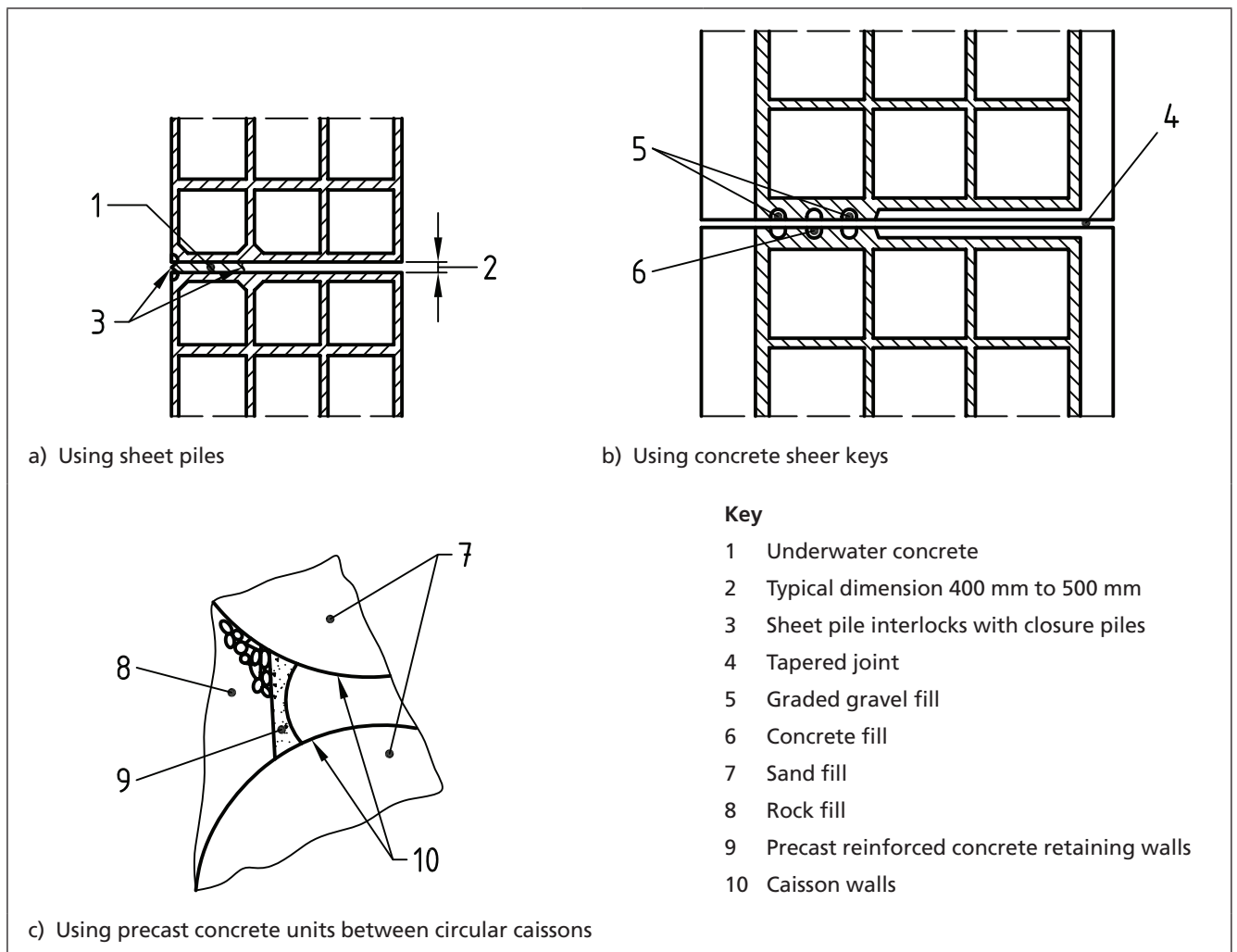
7.6.8 Drainage and filters

Drainage through the wall to relieve differential hydrostatic pressures will usually be best achieved by providing holes or slots in the lower part of the superstructure retaining wall. A suitable filter should be provided to prevent loss of material. Alternatively, a gap may be left between caissons and a rubble filter placed behind the wall.

7.6.9 In-situ capping

The in-situ capping should be cast after the caissons have been filled and the joints completed. The capping may be partially supported on fill placed inside the caissons if the fill has been compacted for this purpose.

Figure 23 Examples of joints between caissons



7.6.10 Design of caisson wall

7.6.10.1 General

Concrete caissons should be designed for overall stability at every stage of construction and service. Overall dimensions should be determined from the geometrical and loading requirements of the caissons in their final position.

7.6.10.2 Ground pressure

The ground pressures at foundation level should be determined for all actions with the assumption that the base slab of the caisson is rigid.

7.6.10.3 Reinforced concrete

Caisson members should be designed for each stage of construction and service. Particular care should be taken to design the walls for all conditions of lateral pressure due to unbalanced levels of water and granular or wet concrete fill, taking into account any compaction applied to the sand fill. The structure may be designed in accordance with silo design practice, where appropriate.

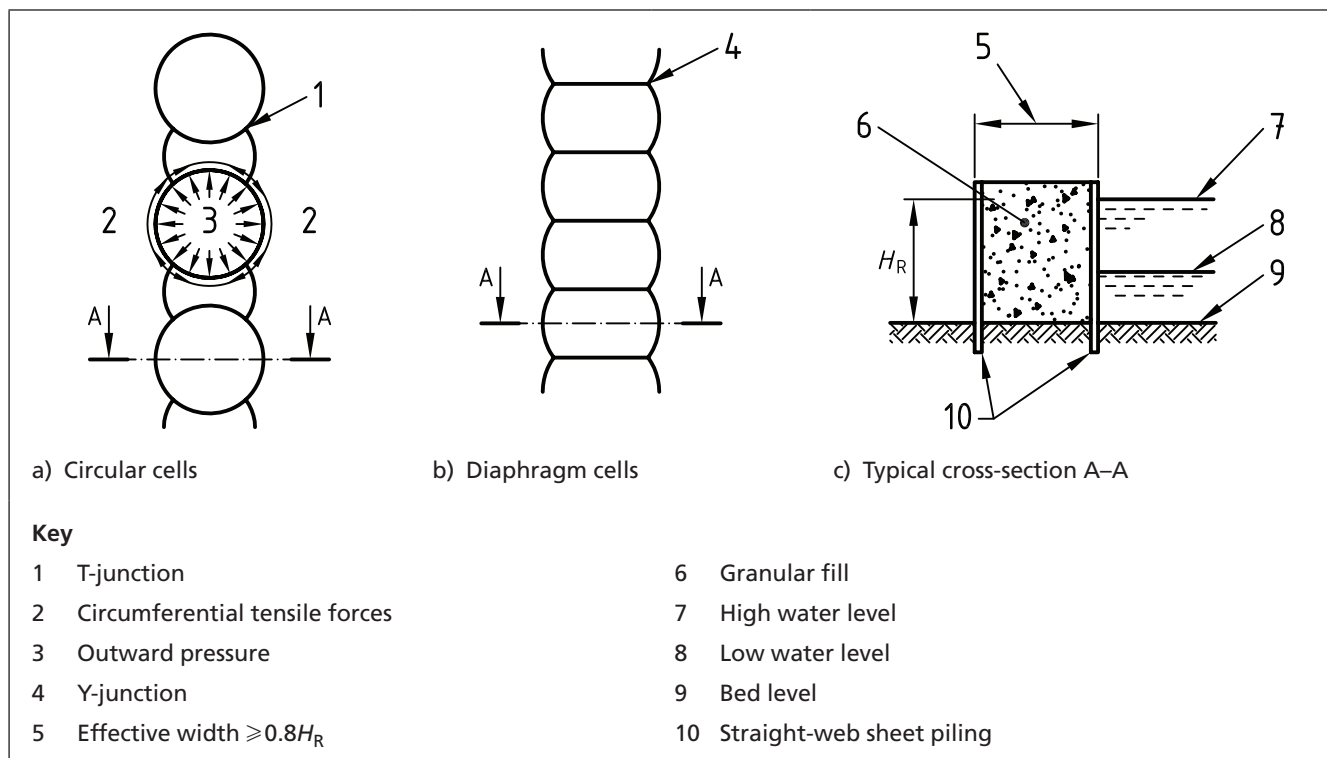
A partial loss of ground support should be assumed in the design of long caissons, which should be analysed as beams in the longitudinal direction. Design of base slabs should take into account the possibility of loss of ground support under filled cells and of higher than normal ground pressures under empty cells, where these occur. The slabs are assumed to be supported by the compartment walls.

7.7 Cellular sheet-piled structures

7.7.1 General

Cellular sheet-piled structures consist of cells formed by interlocking straight-web steel sheet piles, driven or placed with their tops above water level. The cells are filled with granular material. The superstructure may be a solid in-situ concrete capping, or a reinforced concrete edge retaining wall which is backfilled and the top surfaced. This type of structure is illustrated in Figure 24.

Figure 24 Cellular sheet-piled structures



Cellular sheet-piled walls are gravity structures that function partly as cantilever walls where pile embedment can be achieved.

Sheet-piled cells may be founded on soft rock, granular material or very stiff clay. Where soft clays exist, they should be removed before filling the cells. This soft material should be dredged prior to cell construction at sites where the soft material layer is very thick, to avoid causing compression of the cells. Cellular structures may be used for both quay walls and jetties. They use less steel than double-wall sheet wall structures because the steel resists the lateral soil actions in hoop tension rather than bending, and walings, tie rods and bulkheads are generally not required.

This type of structure is particularly liable to damage by wave action during construction without the provision of adequate robust temporary works. This form of structure possesses little resistance to horizontal actions before the internal filling is completed.

7.7.2 Types of construction

Cellular walls are usually of either the circular or the diaphragm type. Figure 24 shows a typical plan of each. As a general rule, the effective width of the wall should be not less than 0.8 times the retained height.

7.7.3 Materials and stresses

Sheet piles for cellular walls are of the straight-web type to transmit horizontal tension. Fabricated junction sections comprising T or Y shapes are generally available. Extruded sections are also manufactured for use where high tensile forces are to be resisted.

7.7.4 Construction considerations

Straight-web sheet piles are not able to resist high driving stresses, therefore, where a substantial thickness of soil lies above the required toe level, it might be preferable to remove some of the soil. Typical maximum driving depths are 3.0 m in dense sand and 1.5 m in hard clay. Hard driving can damage the ends of the piles and cause splitting of the interlocks.

The piles should not be driven hard into rock and, where rock level is highly irregular, it might be necessary to level the surface to minimize differential stresses in the steel.

Circular lengths of wall should be driven using two gates, one near water level and another near the seabed, to ensure that the required shape is achieved. The gates should be secured to prevent movement and damage to the piles by waves and currents.

Filling of cells in diaphragm-type structures should be done in such a way that the height of fill between adjacent cells does not differ by more than 1.5 m at any time unless otherwise allowed for in the detailed design. Inter-connecting arcs in circular-type structures should be driven before the cells are filled, to avoid driving difficulties caused by distortion of the cells. Where bearing piles are required inside the cells, they should be driven before the cells are filled to avoid increasing internal pressures.

7.7.5 Fill

Fill for the cells should be of granular material having a high bulk density to aid stability, and the fill should have a high angle of internal friction to provide internal shear strength and sliding resistance at the base. The fill should also be free-draining to avoid the development of excessive internal hydrostatic pressures, and should be well graded to prevent loss of material by scour or leakage.

Backfill behind cellular walls may be coarse sand, gravel or rubble.

7.7.6 Drainage

The cells should be provided with appropriate drainage measures.

7.7.7 In-situ capping

The capping should be either thick enough to disperse heavy vertical actions throughout the cell fill or designed to transfer them to bearing piles, if provided. Load-bearing cappings should not be supported directly on the sheet piles, and a gap should be provided between the capping and the top of the sheet piles to allow for settlement of the fill. Where a retaining wall is provided at the cope, it should also be designed as a deep beam spanning between cells.

The capping should be designed to prevent the ingress of water to the cells.

The cope line should be located far enough to seaward of the cell walls to prevent accidental damage by vessels. Ideally fenders should be supported on the concrete capping, not on the sheet piles of the cells.

7.7.8 Design of cellular wall

7.7.8.1 General

The overall stability of the structure should be checked at all stages of construction and in service, taking into account the cell filling and backfilling sequence and possible differential hydrostatic pressures.

7.7.8.2 Lateral pressures

Mobilization of the internal shear effects within the cell fill is required to resist lateral soil pressures. The pressure on the landward side of cellular quay walls will be in the active state as a result of the deformation which occurs in the structure.

The fill in the cells is confined by the hoop tension effects acting on the sheet pile interlocks, and the lateral pressure within the cells should therefore be based on at-rest values. The maximum pressure may generally be assumed to occur at one-quarter of the wall height above the toe of the sheet piles.

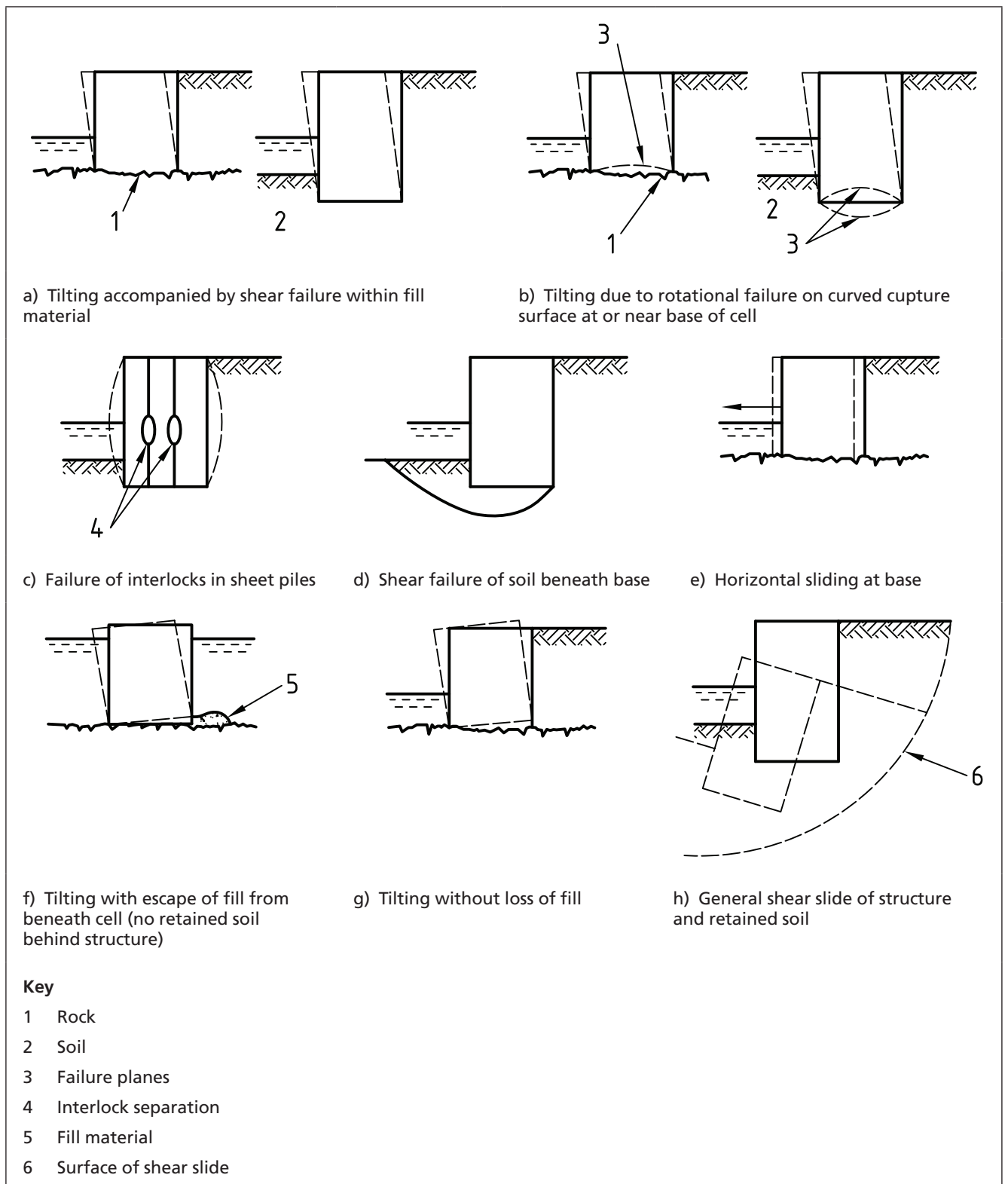
7.7.8.3 Modes of failure

Calculations of the stability of cellular sheet-piled structures should take into account the modes of failure shown in Figure 25.

Where the sheet piles are driven into soft clay, the possibility of heave within or beneath the cells should be investigated.

NOTE Further guidance is given in Stability of strutted excavations in clay [19].

Figure 25 Modes of failure of cellular sheet-piled structure

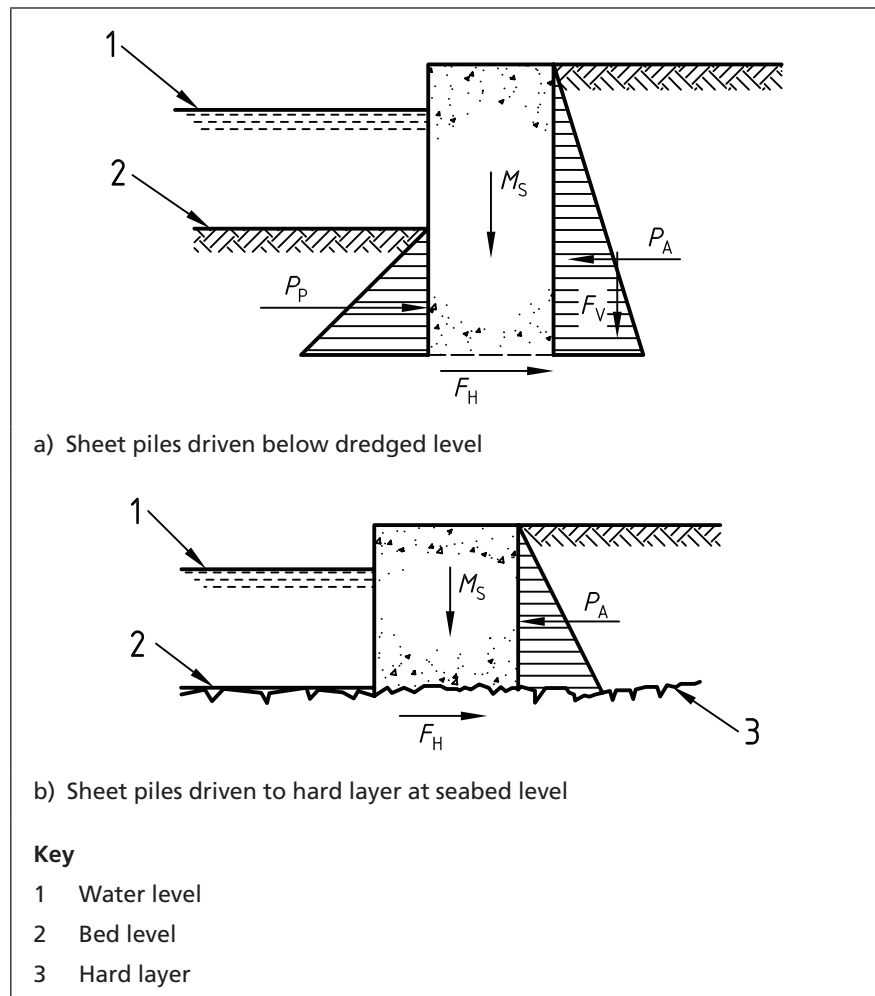


7.7.8.4 Modes of resistance

Where the sheet piles are driven into soil below dredged level [see Figure 26a)], the resistance to sliding or overturning of the structure will be provided by passive resistance of the soil below dredged level, pull-out resistance of the sheet piles on the landward side, friction on the base of the filling material, and weight of the structure and infill material.

Where the sheet piles are driven on to hard rock at, or a short distance below, dredged level [see Figure 26b)], the resistance to overturning and sliding will be provided by base friction and gravitational actions only.

Figure 26 Distribution of active earth pressure and passive earth resistance on a cellular sheet-piled structure



7.7.8.5 Stress in junction piles

In addition to calculating interlock tension in the main cells, the tension at the junctions of arcs and circular cells should be estimated since the largest circumferential effects can occur at this joint. In large diameter cell walls, the stress at junctions might be high enough to warrant the use of special junction piles (see 7.7.3).

Provided that permitted driving stresses and interlock tension do not cause damage, a lighter section of lower grade steel may be used for the sheet piles on the landward side of a quay wall. The toes of these

sheet piles may be installed at a higher elevation than those on the seaward side provided that loss of material would not result and that the total frictional resistance between the piles and the fill remains adequate. The tops of the landward sheet piles of quay walls may also be stepped down to reduce the quantity of steel.

7.7.8.6 Passive resistance of berm

Where passive resistance is provided by a berm along the front of the wall, the reduced resistance due to the slope of the berm should be taken into account. It should be determined whether the wall will move sufficiently to mobilize full passive resistance within the depth of the berm.

7.7.8.7 Wall movement

Cellular quay walls deform considerably when backfilling takes place. The effects of such movement and of any accompanying settlement should be taken into account in the design, particularly where bearing piles are installed within the cells.

7.8 Double-wall sheet-piled structures

7.8.1 General

Double-wall sheet-piled structures comprise two parallel sheet-piled walls connected near the top by tie-rods and walings, with fill placed between. The superstructure usually comprises a concrete capping to the seaward sheet-piled wall(s).

Double-wall sheet-piled structures function as gravity structures when the toes of the two rows of sheet piles are prevented from spreading apart. Otherwise, the structure should be designed as a single sheet wall structure tied back to a cantilever anchor wall. The sheet piles resist the internal soil pressures in bending.

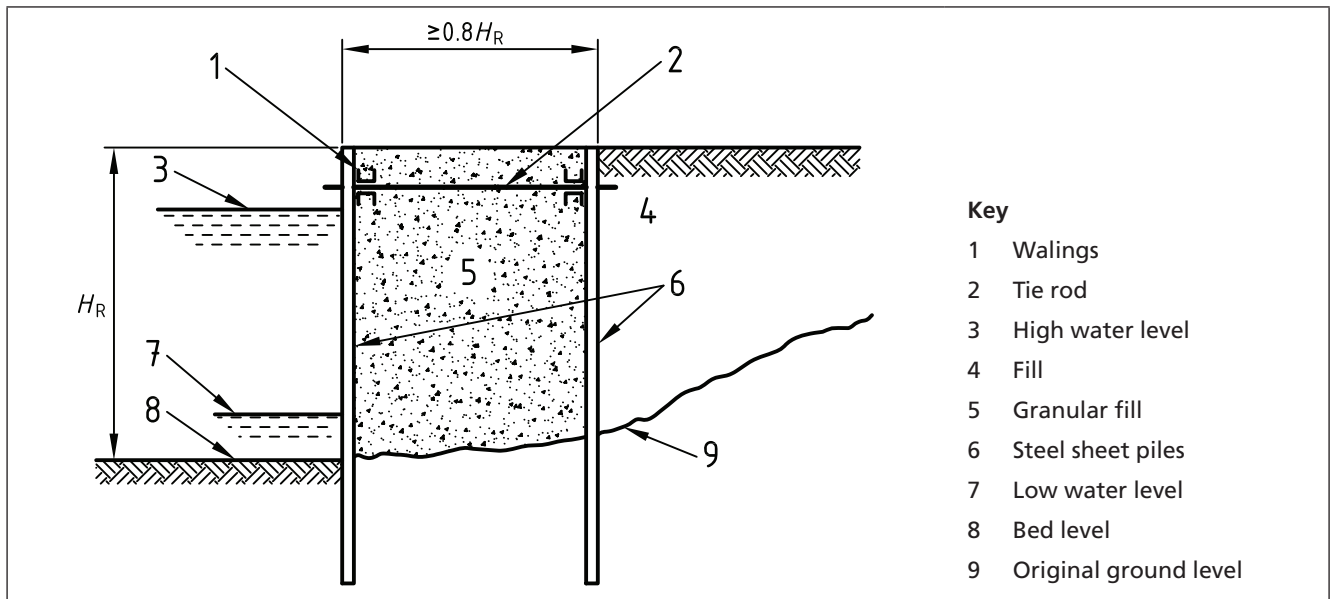
Double-wall structures are most suitable when founded in medium or dense granular soil, firm to stiff clay, or chalk. Where the seabed is mainly of rock, pre-treatment might be necessary to ensure penetration of the sheet piling. Where the double-wall structure is installed in a soft clay, the soft clay should be removed from between the walls before placing the fill. Where the thickness of the soft layer is large, this might need to be done by dredging prior to wall construction. The possibility of heave of the soil below excavation level should be investigated.

NOTE Further guidance is given in Stability of strutted excavations in clay [19].

Double-wall structures may be used for both quay walls and jetties. They can provide an alternative to a cellular sheet-piled structure where a heavier pile section is required for driving, where some degree of stability is required during construction or where straight-web sheet piles are not available. However, they are not in general as efficient as circular cellular structures, owing to their rectangular shape.

Figure 27 shows an example of a double-wall sheet-piled structure.

Figure 27 Double-wall sheet-piled structure



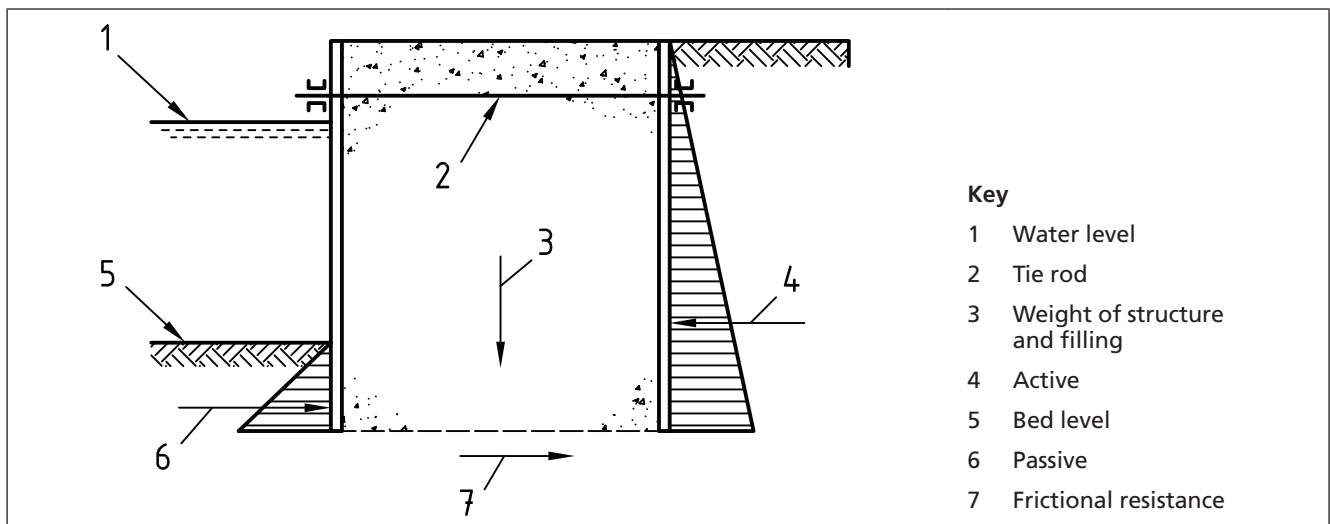
7.8.2 Type of construction

Combined action of the two walls is achieved either by tying the tops of the walls together and providing adequate penetration of the sheet piling (see Figure 28) or, where this is not practicable, for instance in hard rock, loose sands or soft clays, by connecting the walls at a low level using tie-rods. The latter alternative will generally require underwater working.

Transverse bulkheads should be provided to form strongpoints at the ends and at intermediate positions, in order to assist construction and to confine any damage which might occur due to unfavourable sea states. The strongpoints may comprise a square or rectangular cell tied in both directions.

The distance between the parallel walls should be not less than 0.8 times the total wall height above hard stratum or toe level, whichever is higher.

Figure 28 Distribution of active earth pressure and passive earth resistance on a double-wall sheet-piled structure



7.8.3 Materials and stresses

Materials and stresses for sheet piles, walings and tie-rods in double-wall structures are similar to those for single sheet walls (see Clause 6), and steel U- and Z- sections will usually be suitable. The section modulus of the piling should be calculated in accordance with 6.4.1.

7.8.4 Fill

Fill placed between the double walls should have the qualities described in 7.7.5. Backfill behind the structure may be coarse sand, gravel or rubble.

7.8.5 Drainage

Appropriate drainage measures should be provided.

7.8.6 In-situ capping

The capping on the wall at the seaward face(s) should be designed in accordance with 6.8.

7.8.7 Design of double-wall structures

7.8.7.1 General

The overall stability of the structure should be checked for all stages of construction and service, taking into account the filling and backfilling sequence and possible differential hydrostatic pressure.

7.8.7.2 Lateral pressures

Where the double-wall structure is founded on soil such that lateral earth pressures are resisted partly by passive resistance of the soil below dredged level, the structure will deflect towards the water side. For these conditions, the lateral earth pressure on the landward side of the wall may be taken as active (see Figure 28).

Where the two walls are installed on a hard rock surface at, or a short distance below, dredged level, the lateral earth pressure on the landward side of the wall will be in a condition intermediate between the active and at-rest states, depending on the amount of horizontal deflection of the structure.

7.8.7.3 Internal stability

The internal stability of the structure should be checked assuming a lower failure plane within the fill, in the same way as described in 6.6.1.1 for a single sheet wall with cantilever anchorage.

7.8.7.4 Foundations

The bearing capacity of the soil beneath double-wall structures should be adequate to support the weight of the structure and infill material, any superimposed loading on the walls, and the vertical actions transmitted to the landward side wall due to wall friction and the impact actions of berthing vessels, where applicable. Foundations for such structures should be designed in accordance with BS EN 1997.

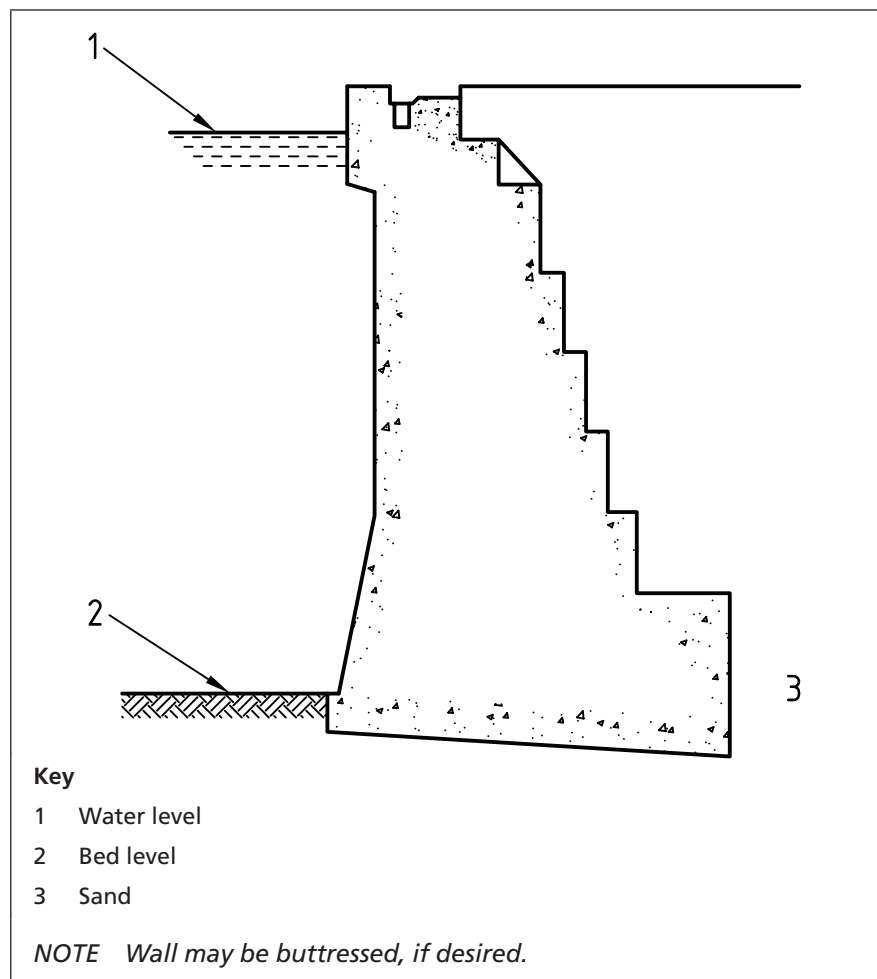
7.8.7.5 Analysis of structural members

The bending moments and shear effects in the sheet piling, and the effects in the connecting tie-rods, should be calculated in the same way as for anchored single sheet walls (see 6.6.1.1).

7.9 In-situ mass concrete walls constructed under water

In-situ mass concrete walls may be of solid, voided or buttressed-arch construction (see Figure 29). The buttressed-arch form of construction will generally require ground anchors to provide the stability required, and these should be designed in accordance with BS EN 1537.

Figure 29 In-situ mass concrete wall



The tensile resistance of concrete is not normally utilized in design, but where tension does occur, stresses should be in accordance with 7.3.3. No tension should be assumed at horizontal construction joints unless they are carefully detailed to transfer the tension and water bars are provided to prevent the ingress of water and development of hydrostatic uplift at the joint.

Cracking of the surface of mass concrete sections may, if required, be controlled by the addition of nominal reinforcement calculated for the outer 0.25 m thick surface zone. A minimum concrete cover of 75 mm on the reinforcement is recommended.

Where a sheet wall is used as a permanent shutter on the face of a mass concrete wall, ties are normally provided to anchor the

sheet wall to the concrete. It should generally be assumed that full hydrostatic pressure will be developed between the sheet wall and the concrete. The ties used to anchor the sheet wall to the concrete should be designed to resist the full hydrostatic pressure.

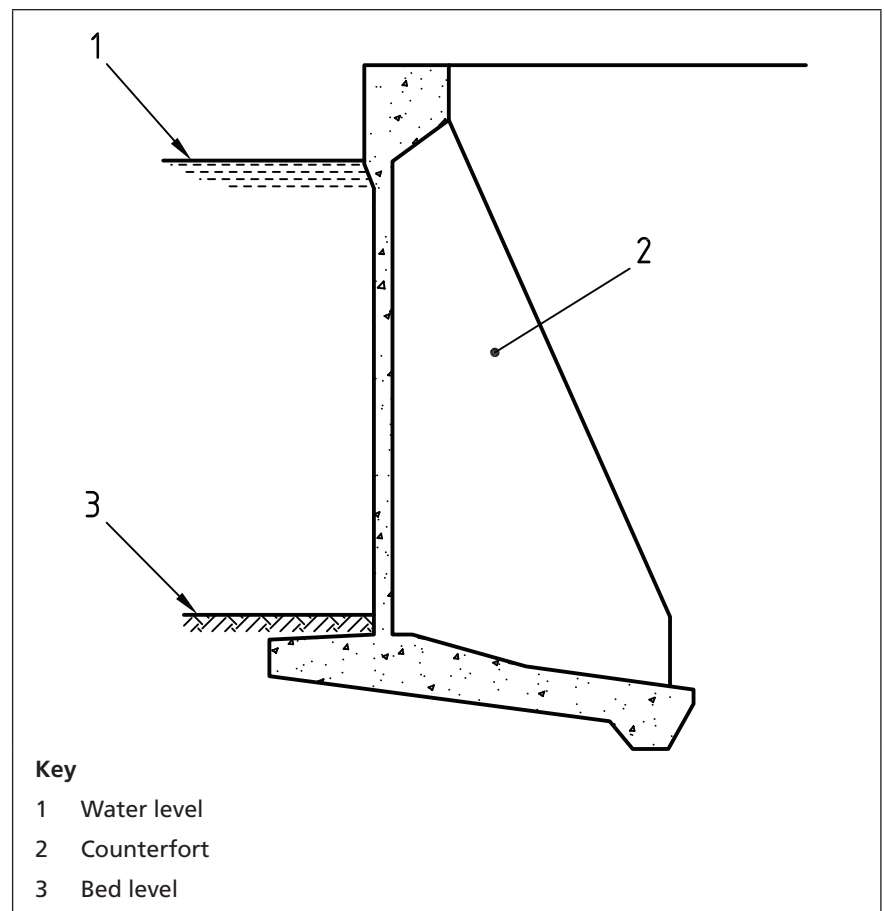
7.10 In-situ reinforced concrete walls

In-situ reinforced concrete walls (other than diaphragm walls, which are discussed in 7.11) should be constructed in the dry, either behind a cofferdam or within an excavation in existing ground.

For walls up to about 8 m high, a plain cantilever design may be used. For higher walls, a counterfort or buttress design will be required (see Figure 30). Ground anchors can be used to resist both overturning and sliding. Ground anchors should be designed in accordance with BS EN 1997 and BS EN 1537.

The recommendations given in 7.5 are generally applicable to this form of construction, except where specifically related to the use of precast concrete.

Figure 30 In-situ reinforced concrete wall

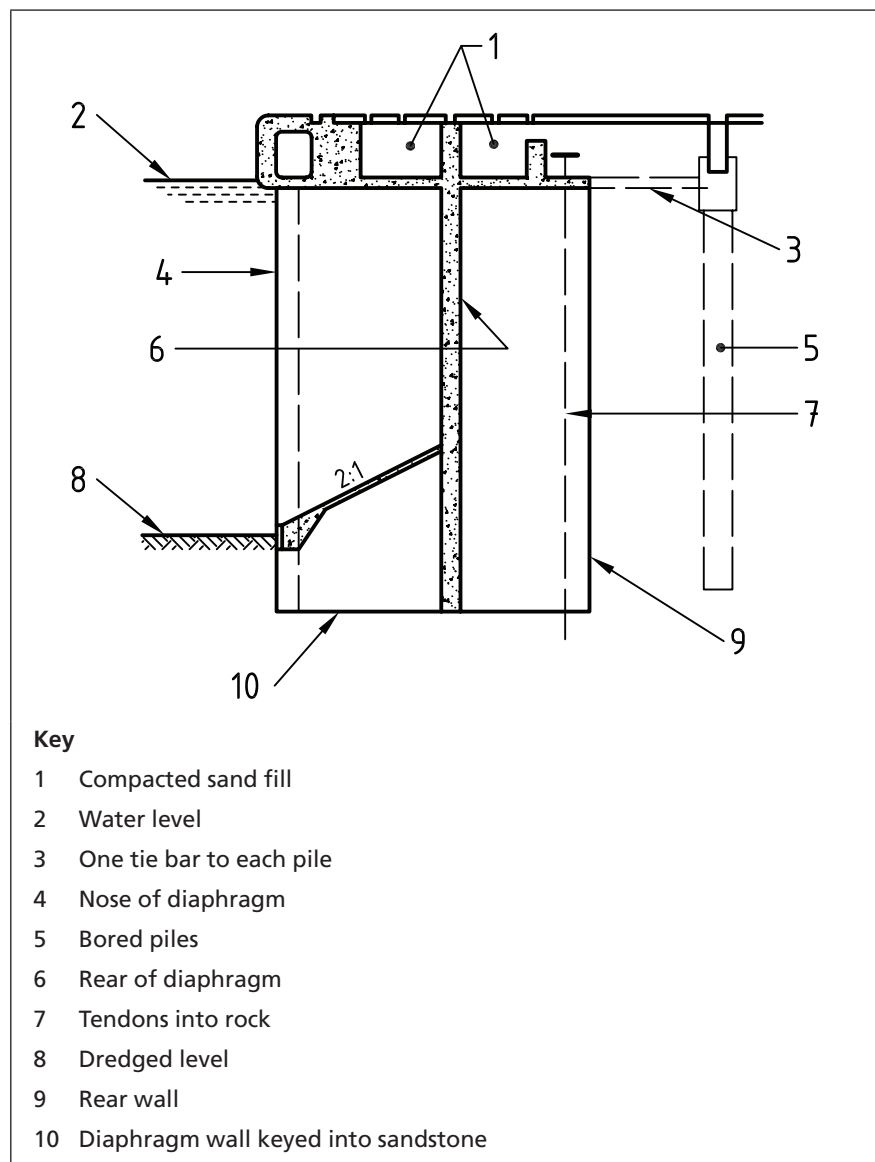


7.11 Diaphragm walls

Diaphragm walling is often a suitable form of construction for gravity quay walls. The walls are generally constructed in cellular or tee formations to just above water level, and an in-situ concrete capping built on top. Figure 31 shows an example of a gravity diaphragm quay wall.

The most common application for this form of construction is in sheet walls, which are covered in Clause 6.

Figure 31 Gravity diaphragm wall



Gravity diaphragm walls may be built in existing ground or in an embankment, and are suitable where large actions are to be resisted or where insufficient space for tie-backs is available. They are generally required to be formed on rock and are also suitable for use in impounded basins, to provide a cut-off to water seepage.

The structures are usually designed as horizontal arches incorporating buttresses or counterforts. The anchoring effects can be provided by the resistance to rotation through the ground of the transverse elements. This resistance is a function of the pressure exerted on the sides of the trench by the head of wet concrete, which has to be poured in one operation.

In most soils, forward movement of the toe will be prevented by surface friction with only modest penetration of the panels below the bed, but slight tilting of the wall can occur in soft ground if slurry

is left at the bottom of the trench. This movement is generally just sufficient to mobilize the frictional actions. It is desirable to make a full-scale (axial) jacking test on site, particularly if the nature of the soil varies with depth: the test may also be used to check the lubrication effect of any bentonite which might remain at the wall/soil interface.

Depending on the nature of the soil and the plan form of the transverse elements, a degree of arching can be developed, which will modify the resultant active pressure. It is appropriate to provide a concrete deck with this type of structure, which, if designed as a wide beam, will afford integration of the elements and a degree of longitudinal stiffness. If set at a lower elevation, it can also act as a relieving platform with the surcharge providing a restoring moment to the whole structure.

When walls of this type are built on an unyielding foundation, at-rest earth pressures are likely to apply. Additional resistance to overturning or sliding may be obtained using vertical rock anchors, in accordance with BS EN 1997 and BS EN 1537.

Vertical slip at panel joints may be prevented by using a suitable shear connector or by means of a stiff deck structure which may also be used to support crane tracks. Suitable connectors may also be provided to resist tension in counterforts where these comprise more than one panel. Reinforcement may be omitted from panels that are subjected to compressive actions only, providing they are individually stable against buckling.

Drainage may be provided through the top of the diaphragm wall or the lower portion of the in-situ capping, with suitable filters as required.

7.12 Monoliths

7.12.1 General

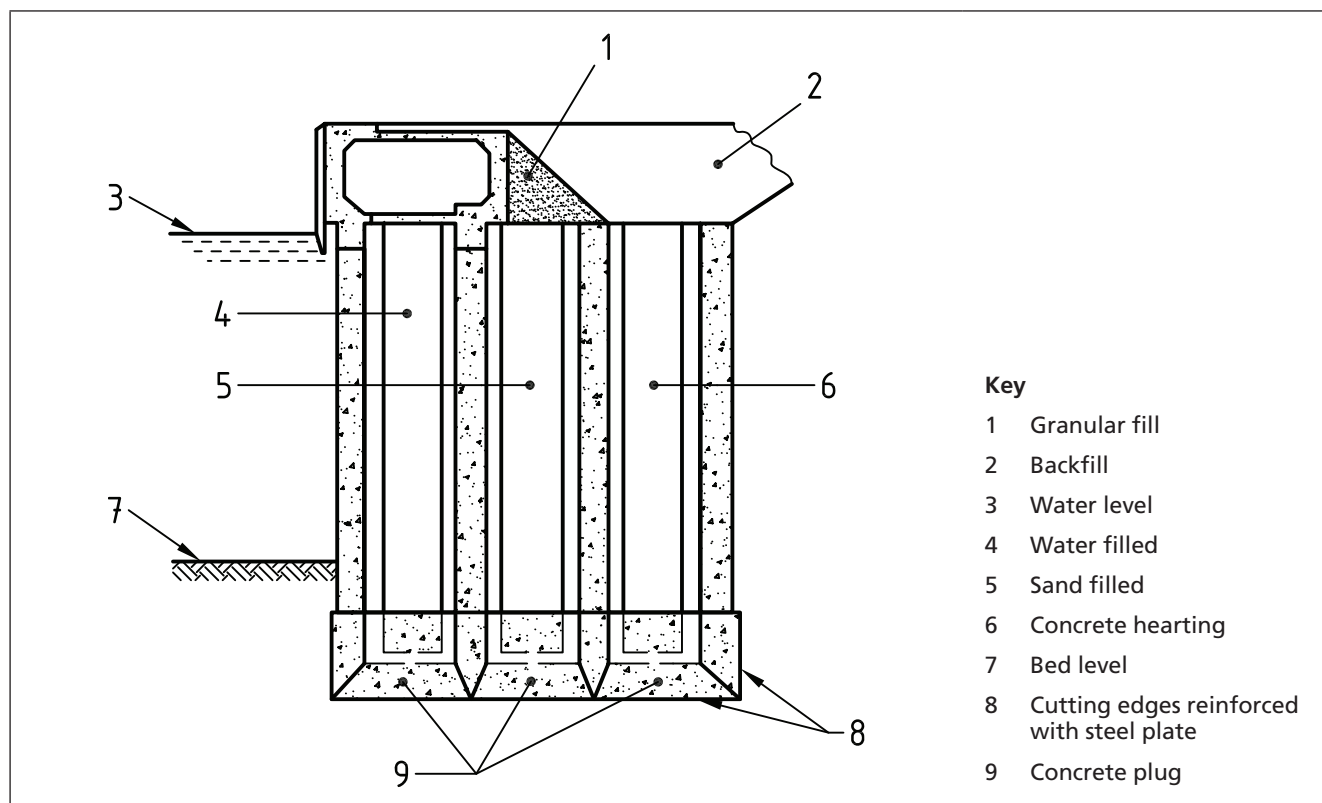
Monoliths, also known as open caissons, are built directly above their final position by alternately raising the walls and sinking the structure into the ground. This process is usually aided by excavation of spoil inside the monolith to speed sinking. Monoliths are generally constructed using mass concrete or lightly reinforced concrete.

Monolith construction commences with fabrication of cutting edges. The walls are progressively extended in the dry, keeping step with the sinking of the monolith so as to maintain the top of the walls above water level at all times. Construction can take place to any depth provided that the lateral soil pressures can be accommodated. Once the monolith walls have been completed, an in-situ concrete plug is poured in the base and the monolith filled with lean-mix concrete as required. The joints between monoliths are then sealed and an in-situ capping built. An example of a monolith is shown in Figure 32.

Monoliths are suitable for the construction of a deep water quay in poor ground or through fill. Where obstacles are likely to be encountered in the ground, sinking might be difficult. If the monoliths are to be sunk on to a sloping rock surface, special measures will be required to achieve an even bearing. Owing to the nature of the construction, precise positioning and verticality of monoliths cannot be guaranteed.

In ground containing flowing ground water, monoliths might be more suitable than diaphragm wall construction.

Figure 32 Monolith



7.12.2 Shape

Monoliths are constructed of similar shapes on plan to those used for caissons. However, being generally of mass or lightly reinforced concrete, the walls are thicker and more robust. The toe of the outer walls contains a cutting edge of concrete or steel. The lower inside area of these walls should be shaped or tapered to assist sinking and excavation and to provide a key for the concrete plug at the base. The internal walls should terminate at least 0.5 m above the cutting edge.

7.12.3 Sinking

The interior of the monolith is usually excavated by grabbing, with help from divers if there are obstructions under the cutting edges. Sinking may be assisted by bentonite lubrication of the outside face, by jetting (in granular soils), and/or by kentledge on the monolith walls. In cohesive soils, the effective weight of the monolith may be increased by dewatering. In granular soils, dewatering should not be done because of the danger of causing a blow-out.

The sinking operation should be carefully controlled to keep the monolith plumb. Typical tolerances are ± 0.3 m for position on plan and 1:100 for verticality. A row of monoliths should be constructed by sinking alternate units initially followed by the infill units.

7.12.4 Plugging

The base of the excavation should be cleaned and levelled in the dry or by diver. The concrete plug should be sufficient to form a complete seal, and may be placed in two stages, the first underwater and the second in the dry.

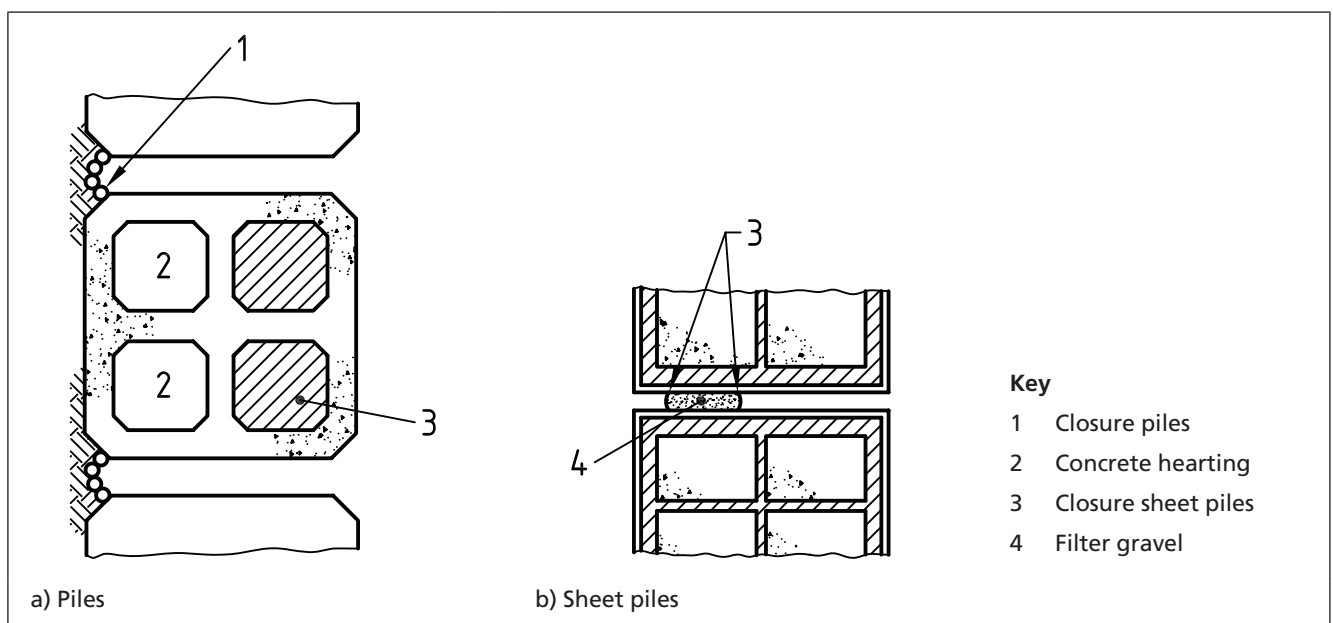
7.12.5 Fill

Stability calculations should be made to determine whether the cells should be filled or left empty.

7.12.6 Joints between monoliths

The spaces between monoliths, which are typically between 0.5 m and 3 m wide, may be sealed by means of piles (see Figure 33). The space may be excavated and then filled with concrete or, in weak ground, grout intrusion of preplaced aggregate might be more suitable. Instead of a continuous concrete seal, a series of precast concrete piles may be used to infill the space. If movement of the monoliths in service is expected, a flexible seal of gravel may be used instead of concrete.

Figure 33 Examples of joints between monoliths

**7.12.7 In-situ capping**

The in-situ capping should be cast after the monoliths have been filled and the joints completed.

7.12.8 Design of monolith wall**7.12.8.1 General**

Monoliths should be designed for overall stability at every stage of construction and service. The possible reduction in adhesion after bentonite has been used in sinking operations should be taken into account. Overall dimensions should be determined from the geometrical and loading requirements of the monoliths in their final position, over the range of foundation levels adopted.

Overall stability can, if necessary, be improved by the addition of tie-backs (see Clause 6). The distribution of ground pressure under the base should be checked and the founding level should be designed to ensure that material cannot be removed from underneath the base by scour.

7.12.8.2 Concrete plug

The concrete plug should be designed to resist the maximum uplift pressure occurring at foundation level.

7.12.8.3 Concrete walls

The walls should be designed for all conditions of lateral pressure as stated in 7.6.10.3. The design of the monolith should also take into account the uneven bearing of the cutting edges during sinking. In its final position, the presence of the concrete plug will ensure an even bearing of the base. Design of the mass concrete walls should take into account the provisions of 7.3.3. Anti-crack steel reinforcement may be provided in accordance with 7.9.

7.12.8.4 Movement

The loosening of the adjacent soil during excavation can result in settlement and tilting of the monoliths. These movements should be allowed for in the design. In earthquake areas, loose soil should be compacted prior to construction of the monolith.

8 Suspended deck structures

NOTE For convenience, throughout this clause the term "piles" is used to represent the supporting elements, e.g. individual bearing piles, barrettes, small caissons, whatever their cross-section and method of installation.

8.1 General

Suspended deck structures may be of steel, concrete or timber, or of a combination of more than one of these materials.

Vertical actions are resisted by the piles, which derive their support from the soil by end-bearing or shaft friction, or a combination of the two. Horizontal actions may be resisted in several different ways, some of which impose uplift. The deck is generally used to distribute locally applied horizontal actions within the structure.

Structures may be classified as rigid or flexible (see BS 6349-1:2000, 47.1) as regards their response to horizontally applied actions. The degree of flexibility depends upon the materials used, the ground conditions and the framing of the structure.

8.2 Suitability

Suspended deck structures will usually be the most suitable type in the following circumstances:

- a) ground consisting of weak upper materials overlying a stronger stratum;
- b) ground immediately below seabed consisting of suitable material for bearing piles;
- c) non-availability of suitable backfill for use in a retaining wall type of quay;
- d) necessity to minimize interference with hydraulic regime;
- e) great depth of water.

If the seabed is rocky, piles may be installed with some form of pre-treatment of the ground.

8.3 Applications

Suspended deck structures are commonly used for marginal quays and finger jetties.

When used for marginal quays, the structure will usually be built above a revetted slope, which rises from dredged level near the berthing face to ground level behind the structure. Finger jetties and access trestles are generally constructed above natural seabed, although dredging might be required at the berth itself.

8.4 Types of structure

The type of suspended deck structure is related to the methods used to provide resistance to the horizontally applied actions and to distribute vertical actions to the piles.

The most common pile systems, with the method of resisting horizontal actions, are described in Table 2, and illustrated in Figure 34. Raking piles should not normally be employed in compressible ground, owing to the progressively increasing vertical actions due to settlement and downdrag causing bending of these piles, unless the bending can be effectively resisted. Horizontal tie-backs (see Clause 6) may be provided where the upper strata are poor or where large transverse actions, such as those due to soil or earthquake, are to be resisted.

Table 2 Pile systems for suspended deck structures

Type of structure	Application	Flexible/rigid (see Commentary on Clause 6)	Principle method of resisting horizontal actions	Figure reference
All vertical piles	Jetty heads	Flexible	Bending of piles	Figure 34a)
All vertical piles	Marginal quays	Partly flexible and partly rigid	Bending of piles, plus passive resistance of backfill	Figure 34b)
All vertical piles plus horizontal tie-backs	Marginal quays	Partly flexible and partly rigid	Bending of piles, horizontal tie-back (seaward loads), passive resistance of backfill (landward loads)	—
All vertical piles plus strut/tie members to shore	Marginal quays	Rigid	Strut/tie	Figure 34c)
Vertical and raking piles	All	Rigid	Raking piles	Figure 34d)
All raking piles	Access trestles and jetty heads	Rigid	Raking piles (transverse loads), longitudinal raking piles or shore anchorage (longitudinal) loads	—

NOTE Although the principal means of resisting horizontal actions are given above, there are usually associated vertical actions that arise from the structural response to the actions. These depend on the stiffness and articulation of the structure, for instance if vertical slip is permitted between jetty head and pile, vertical actions in piles can be significantly reduced, though not completely eliminated.

Figure 34 Examples of suspended deck structures

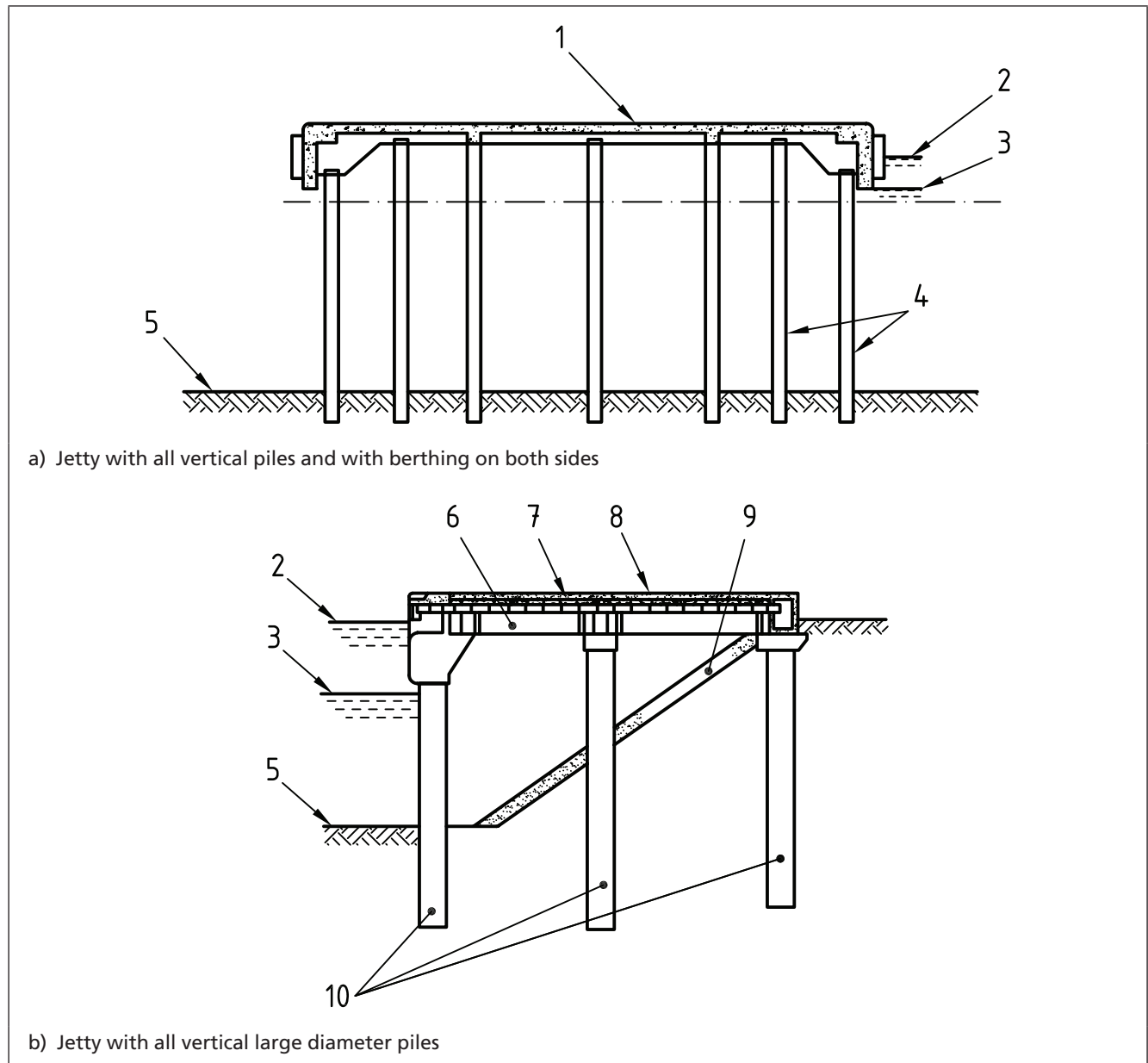
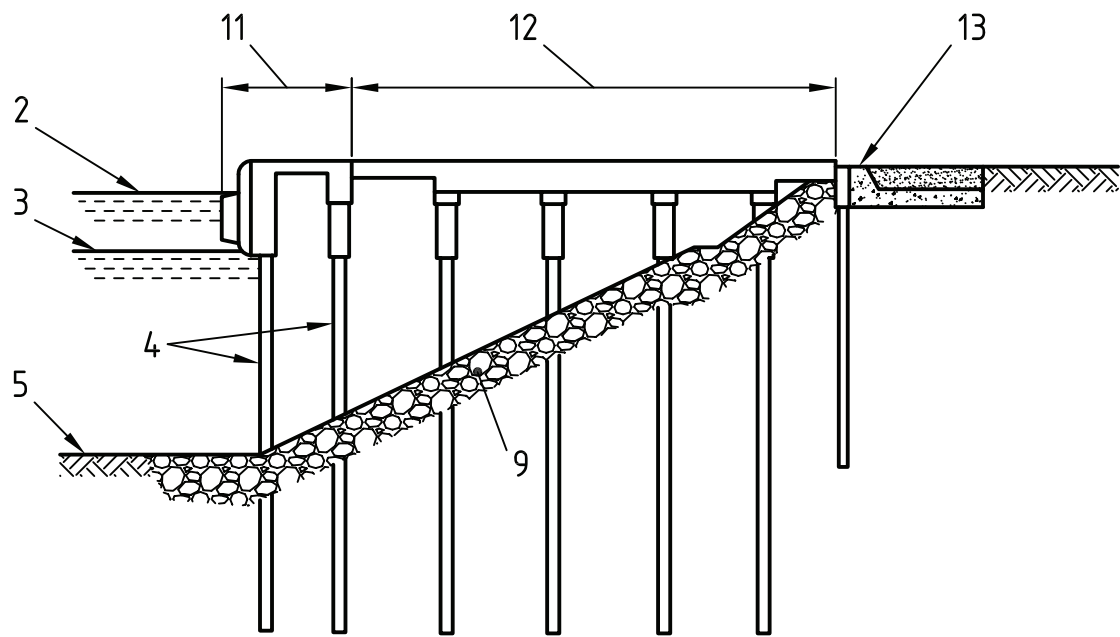
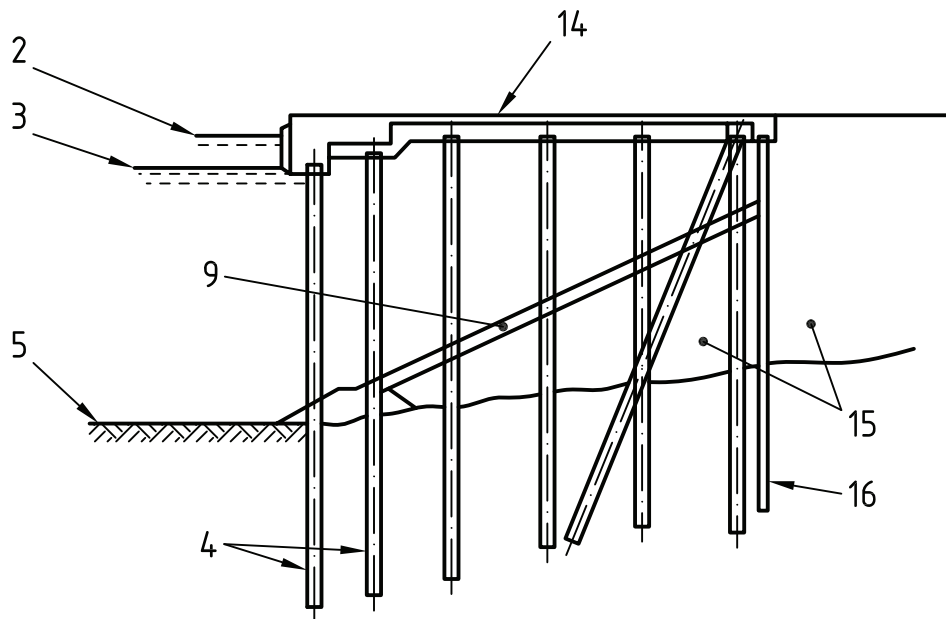


Figure 34 Examples of suspended deck structures (continued)



c) Jetty with all vertical piles plus struts to shore



d) Jetty with vertical and raking piles

Key

- | | |
|--------------------------------|--------------------------------------|
| 1 Reinforced concrete deck | 9 Rock armour |
| 2 High water level | 10 Large diameter plates |
| 3 Low water level | 11 Quay |
| 4 Piles | 12 Access bridge acts as strut |
| 5 Bed level | 13 Shore abutment |
| 6 Pre-cast concrete beams | 14 Reinforced concrete beam and slab |
| 7 Pre-cast concrete slab units | 15 Granular fill |
| 8 In-situ concrete decking | 16 Retaining wall |

Strut/tie systems in marginal quays most commonly take the form of a series of access bridges connecting the quay to strongpoints on the shore.

Loading platforms for large oil ships and gas carriers are generally constructed as suspended decks. The trestle and the (un)loading platform provide support for piping and equipment susceptible to (differential) lateral deformations and/or movements. Pipe stresses can increase beyond their limiting values if the supporting structure is too flexible. This is often the governing factor for the required lateral stiffness of the foundation. A pile concept consisting of raking piles will provide more stiffness than a concept based on vertical piles only. Vertical piles, however, are easier to install. For jetties designed for very large crude carriers (VLCCs) and large gas carriers, the required draft – and thus the unsupported length of the pile – is generally too large to achieve the required stiffness using vertical piles only. A mixed concept of raking piles and vertical piles is generally most effective.

If raking piles are installed close to the berthing line, account should be taken of the risk of collision between the hull of a ship and the front side of the raker piles.

The designer should take into account (local) construction practices and the availability of equipment in selecting the structural concepts of the jetty facilities.

An alternative concept for jetty structures at exposed sites consists of the installation of jacket structures anchored by steel piles. Such a concept is particularly appropriate in the following situations:

- if the required height of the structure is significant due to a strong variability of the water depth in the jetty area;
- in rocky subsoil conditions, which lead to difficult pile installation methods;
- if the structure's lateral stiffness requirements are high, e.g. resulting from criteria imposed by the piping design;
- if there is limited time for carrying out site works, e.g. in circumstances where the weather is only sufficiently calm for a short period.

Under normal circumstances, jacket structures are generally not preferred. When they are used, it is often in conjunction with large span trestle units.

8.5 Overall stability

8.5.1 General

The proposed structure should be checked for overall stability by reference to BS EN 1997-1 and in accordance with **8.5.2** and **8.5.3**.

The use of bracing to reduce effective length of piles or to distribute berthing actions is not generally preferred due to the increased difficulty of construction and maintenance.

8.5.2 Slope failure

The stability of the slope under a marginal quay should be analysed for all critical design situations. These will include the placing of fill

to raise the ground level behind the quay, pile driving, the imposition of heavy loads behind the quay, and, at some locations, earthquake actions. The analysis should take account of any variations which could occur in water levels and pore water pressures, including the possibility of liquefaction. Where long-term lateral soil actions could be applied to the piles, ground improvement or other suitable measures should be taken in advance of pile driving to prevent this.

8.5.3 Settlement or uplift of structure

Settlement of a suspended deck structure can occur as a result of excessive imposed loads, negative skin friction on the piles or consolidation of the cohesive soil layer in which the piles receive frictional resistance. Uplift is usually the result of excessive horizontal actions but might exceptionally be due to hydrostatic pressure.

8.6 Earthworks at marginal quays

8.6.1 General

Earthworks associated with suspended deck structures will be required only for marginal quays, except in the case of finger jetties requiring dredging. Where earthworks are needed, they should conform to BS 6031 and BS 6349-5.

Anchorage and sheet bulkhead walls for earthworks should conform to the same recommendations as for embedded retaining walls (see Clause 6).

8.6.2 Slope stability and protection

In ground which will stand at an acceptable slope under water, permanent protection will normally be needed in the form of filter and armour layers. Where some risk of rotational failure of the slope exists, a toe trench filled with rock or sand might be necessary to enhance the slope stability.

Where the temporary underwater slope achieved by dredging is too flat, a steeper slope may be achieved by building one or more underwater rubble embankments to retain hydraulic fill: rubble should be of suitable grading to act as a filter and protect the slope from erosion. Where the seabed consists of soft material, the soil beneath the embankment may be replaced with granular material, or a gravel blanket may be placed on the seabed before constructing the embankment.

After driving the piles, armouring may be placed over the seaward slope, provided that care is taken to avoid damage to the piles by bending or abrasion. Figure 34 shows typical examples of slope protection under suspended deck quays.

The range of levels over which a filter is required on the face of the slope should be determined. A filter might also be required at the base of a rubble embankment or slope. The possibility of loss of fill from the top of the slope under the rear edge of the superstructure due to ground water movement, after settlement or to wave action, should be taken into account. Careful detailing is required to provide protection from wave action.

8.6.3 Drainage

If the fill behind a suspended deck marginal quay is not a free-draining material, suitably designed drainage measures should be provided to collect water from along the rear face of the superstructure and to allow it to flow to suitable free draining areas (usually the rock slope) below: the drain may be in the form of a rock filter or a proprietary drainage strip. This will minimize local instability of the fill behind the quay, and thereby maintain the lateral soil pressure for resisting berthing actions on the rear face of the superstructure.

Drainage of the soil above relieving platform slabs should be provided, where necessary, to prevent inundation of the surfacing (see 4.3.3).

8.7 Types of bearing pile

NOTE Detailed provisions for the types of bearing pile described in 8.7.1 to 8.7.5 are given in BS 6349-1:2000, 61.2. Further guidance is given in A review of bearing pile types [20].

8.7.1 Timber piles

Timber piles are easy to handle and cut to length; they require only simple driving equipment, and their flexibility can be useful in energy-absorbing structures.

The length of timber piles is generally restricted varying from 12 m to 18 m. Lengths up to 24 m are sometimes available for specially ordered hardwood timbers such as Greenheart. Additional pile penetration in weak ground can sometimes be achieved by jointing two lengths of timber.

Timber should generally be treated (see BS 6349-1:2000, 60.2) or cased with concrete muffs to prevent attack by marine borers.

8.7.2 Precast concrete piles

8.7.2.1 Reinforced concrete

Precast reinforced concrete piles may be driven in many types of ground, and may also be placed in pre-bored holes in the seabed.

If the piles are to be cast at a yard remote from the site, their maximum length might be restricted to about 20 m. If necessary, provision may be made for extending the piles using either a proprietary interlocking joint or a dowelled system in conjunction with grout or epoxy resin. Any such joint should have the necessary bending moment capacity and be protected against corrosion.

8.7.2.2 Pre-stressed concrete

Pre-stressed piles differ from reinforced precast piles (8.7.2.1) in that they are better able to resist tensile stresses during handling, driving and in service than reinforced piles. A smaller cross-section may therefore be used, which can be advantageous if downdrag is a problem. A pre-stressed pile will have a greater design resistance than a reinforced pile of the same section.

8.7.3 Cast in-situ piles

COMMENTARY ON 8.7.3

Cast in-situ piles are not recommended for maritime structures due to the difficulties of guaranteeing the quality of underwater concrete and the technical construction difficulties. However, it might be necessary to use them in exceptional circumstances, in which case the following guidance will apply.

8.7.3.1 Driven casings

Driven casings for cast in-situ piles may be used in preference to precast concrete piles to save handling, to permit a larger section to be used, to reduce driving requirements, or at sites where the pile lengths are irregular. The piles should normally be used to resist axial actions only, since control of the concrete placed through reinforcement cannot be assured. Reinforcement should be provided in compression piles from the head to a short distance below seabed level, and over the entire length in tension piles.

The casing (usually of steel) should normally be left in place permanently. The casing should be of sufficient strength to withstand handling and driving stresses, although the latter can be low if the casing is bottom-driven. The casing is normally disregarded for design of the concrete pile, although it may be taken into account for the protection it provides. The concrete should be placed in the dry or by tremie and should be at least grade 25 when the casing provides protection for the entire design life, and grade 40 when the casing is designed to be sacrificial.

This type of displacement pile may be used in sizes of up to about 35 m in length and 700 mm in diameter, and at a maximum rake of 1:2.5.

8.7.3.2 Bored piles

Bored cast in-situ piles are particularly suitable for use as non-displacement friction piles in stiff clay or weak rock, or to achieve large load-bearing capacities by means of the large shaft diameters available or by under-reaming the toe.

NOTE The ICE specification for piling and embedded retaining walls gives guidance on typical dimensions and installation conditions for bored piles.

At sites where flowing or artesian water exists, or where the ground is mainly of granular materials, this type of pile is unlikely to be suitable.

8.7.3.3 Barrettes

Barrettes, or short lengths of diaphragm wall, may be used in place of piles. The high section modulus of barrettes, together with their comparatively large bearing area, makes their use particularly suitable where large horizontal and vertical actions are to be resisted, for example in supporting relieving platforms. Care should be taken to remove the slurry from the toes of the barrettes.

NOTE Further guidance on diaphragm wall construction technique is given in BS 6349-1:2000, Clause 55.

8.7.4 Steel piles

Steel piles are relatively light and easy to handle, and can be driven in most types of ground, including many types of rock: H-piles are particularly suitable for this. H-piles are also useful as tension piles

because of the large penetrations which can be achieved. Circular and box piles are more suitable where large axial and bending effects are to be resisted (in corrosive conditions they are less vulnerable because they have less exposed surface). The advantages of each type may be employed in a composite pile comprising a bottom H-section, driven to achieve uplift resistance and welded to the inside of a tubular upper section which is used to resist vertical and horizontal actions.

The fracture toughness of welds should conform to BS EN 1993-5. For special steels, the fracture toughness should conform to BS EN 1993-1-10. Steel piles have good energy-absorption properties and may therefore be used, when installed vertically, to resist horizontally applied actions. If a concrete encasement is added for corrosion protection, the necessary limitations on maximum strain might require the permissible energy absorption to be reduced.

Steel piles can be manufactured in a series of cans (i.e. lengths of steel plate, bent to form a tubular section and welded down the seam), or from spirally welded strips. The manufacture of such piles should be in accordance with BS EN 10210 for longitudinally welded piles and BS EN 10219 for spirally welded piles.

Steel piles may be readily extended by butt-welding an extension piece, and they are therefore useful at sites where the pile lengths cannot be accurately predicted.

NOTE 1 Guidance on butt welds in steel bearing piles is given in BS EN 12699.

Hollow steel piles might be required to be filled with concrete at the toe, when end-bearing capacity is critical, or at the head, when a full bending moment connection to the deck is required.

For structures subject to dynamic actions, suitable impact values for the steel should be specified to prevent brittle fracture.

External corrosion may be minimized by means of protective coatings or cathodic protection or may be allowed for in design (see 4.4.2).

NOTE 2 Further guidance on steel bearing piles is given in Steel bearing piles guide [21].

8.7.5 Screw piles

Particular care should be taken where screw piles are used in soils having a low remoulded strength. It is also important to ensure that the pile penetrations reach the intended bearing strata.

8.8 Installation of piles

COMMENTARY ON 8.8

This subclause should be read in conjunction with 4.2. The construction plant used for the installation of the piles will depend mainly on the type of pile, site exposure, depth of water (if any) during piling and plant available. The design, installation and testing of piles is covered in BS EN 1997-2.

8.8.1 General

The methods of support for a particular site should wherever possible be determined at the design stage, since this will influence the construction sequence, any temporary actions on the piles and the construction tolerances which can reasonably be expected.

Raking piles can be difficult to drive from floating plant in tidal waters, and the design should take account of any temporary works that will be required, as far as can be reasonably determined by the designer. The effect on piling tolerances of driving through rubble slopes should also be taken into account (see also **8.13**).

8.8.2 Temporary bracing

Temporary bracing might be required to prevent oscillation of the head due to waves or currents. The bracing should be sufficient to prevent movement in all horizontal directions, and may be fixed to the previously constructed section of the structure when this itself is secure.

It should be ensured that assumptions made in the design regarding bracing at each stage are followed in the construction method adopted.

8.8.3 Measures to improve performance of piles

8.8.3.1 Slip layer

The use of slip layers is not recommended as a means of reducing downdrag on piles in settling fill. Instead, allowances should be made for settlements, so that pile structures on settling fill are allowed to move to a sufficient extent to relieve the downdrag. Such structures should then be designed to accommodate any settlements that might result.

8.8.3.2 Vertical anchors

In cases where adequate uplift-resistance in tension piles cannot be reasonably provided by means of embedment, a rock anchor may be installed axially through the pile shaft to achieve the required resistance (see BS 6349-1:2000, **53.2.3**). Design of the rock anchor should be in accordance with BS EN 1537. Anchors may be of a pre-stressed or passive type.

8.8.4 Construction sequence

The most likely or suitable sequence of the principal construction operations should be determined at the preliminary design stage. Where necessary, restrictions should be imposed on the actual sequence to be adopted. Examples of interrelated operations which might affect design or construction of piles are:

- a) formation of the seabed, underwater slopes or toe trenches and pile driving or boring;
- b) placing of rubble protection and pile driving;
- c) placing of fill or surcharges and pile driving;
- d) pile driving and pile load tests;
- e) extension or re-driving of piles and construction of superstructure;
- f) placing of rubble protection and construction of superstructure;
- g) pile driving and installation or testing of rock anchors;
- h) driving of pilot piles and precasting remaining piles.

Particular factors which should be taken into account are:

- 1) increase in pore water pressures resulting from filling or pile driving;
- 2) settlement or lateral movement of fill and the effect on driven piles;
- 3) disturbance of pile and rock anchor load tests by piling operations;
- 4) difficulty of access to redrive piles surrounded by other piles;
- 5) delay in construction of superstructure caused by in-situ pile extensions;
- 6) difficulty of driving piles through rubble, or placing rubble after piles are installed;
- 7) delay caused by late driving of pilot piles or pile load tests.

8.8.5 Connections

The pile/beam connection is usually designed to transfer uplift actions and bending moments due to horizontal loads from the beam to the pile head. Where full continuity is not required, the pile head should still extend at least 75 mm above the soffit of the concrete beam.

8.9 Types of superstructure

8.9.1 Concrete deck

8.9.1.1 Flat slab

Flat slabs may be used where a high water level would make the construction of beams difficult and where it is easy to install the necessary falsework.

8.9.1.2 Cross-head beams and slab

Cross-head beams may be of either in-situ or precast concrete, or composite section, using a precast trough filled with in-situ concrete. Precast beams may be supported on a pile cap of in-situ or precast concrete: alternatively, steel brackets may be fixed to the head of the pile to support the beams until continuity of the cross-head has been achieved by construction of in-situ concrete joints.

8.9.1.3 Two-way beams and slab

Beams may be provided in the longitudinal direction as well as transversely for the following purposes:

- a) at the front and rear edges of the deck, to support the edge of the deck, to provide additional stiffness in resisting horizontal bending, to resist berthing actions, or, in marginal quays, to retain the top of the fill and transfer berthing actions into the fill;
- b) to transfer lateral and uplift actions to raking piles;
- c) to support heavy loads (e.g. rail tracks or shed walls).

Precast units may be used in most structures of this type.

Account should be taken of the plant required for the maintenance of mechanical handling equipment when calculating loads on the jetty head.

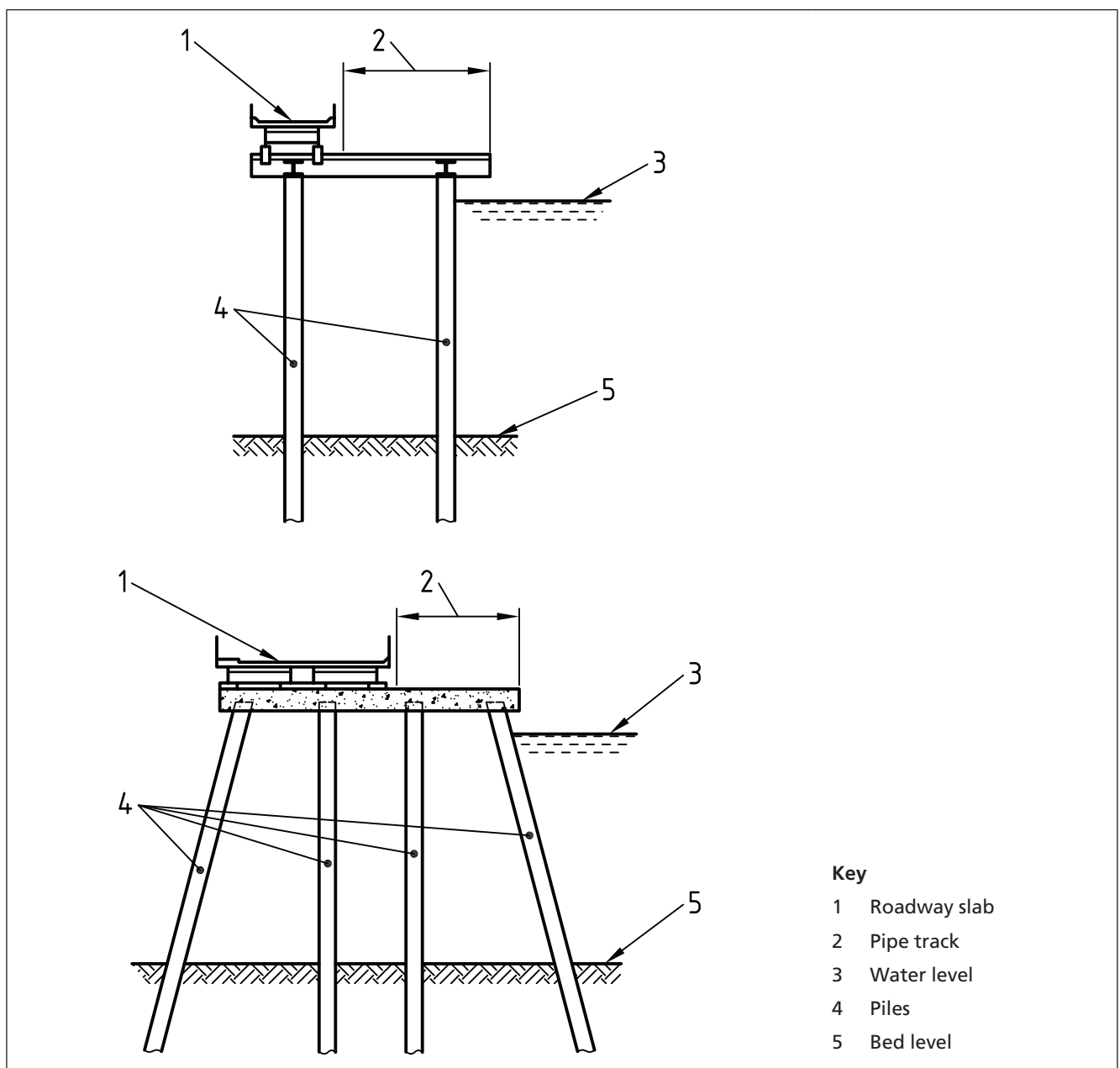
8.9.1.4 Access trestles

The most suitable concrete deck construction for access trestles will normally be cross-heads supporting a deck comprising either an in-situ or a composite bridge-type structure. Open-deck construction may be used where appropriate, for example at jetties used for handling bulk cargoes where vehicular or pedestrian access is not required. Longitudinal beams might be required to provide lateral stiffness to the deck, as well as support to the slab itself.

For long trestles, speed of construction is usually important and should be a major consideration in design. To this end, precast concrete cross-heads, flat soffits on the deck and simple shapes should be adopted.

Examples of access trestles are shown in Figure 35.

Figure 35 Access trestles



8.9.1.5 Relieving platforms and false decks

Relieving platforms and false decks may be constructed using one of the types described in 8.9.1.1 to 8.9.1.3. Where the soffit is near water level, beams may be constructed as upstands. If counterforts are required, they can also serve as cross-head beams.

The retaining wall along the outer edge of the deck, which may also function as an edge beam or berthing face, will usually be of in-situ reinforced concrete or precast units.

Fill above relieving platforms and false decks should be compacted granular material. A surfacing constructed with cross-falls should be provided to minimize the quantity of rainwater entering the fill: as a precaution, drainage holes should be incorporated in the concrete at the underside of the fill.

8.9.2 Steel and concrete deck

Steel beams may be used in preference to concrete, where loadings do not normally exceed highway loading or where rapid construction is required. The steelwork should be protected against corrosion (see BS 6349-1:2000, Clause 66). Steel beams are not usually used in quay superstructures subjected to loads in excess of highway loading because of the high need to provide a frame of sufficient rigidity in the vertical and horizontal directions. Arrangements in common use are:

- a) a grid of steel beams (for open-deck jetty heads): the deck, where required, may be of concrete or open-mesh steel;
- b) a steel cross-head supporting concrete deck with edge beams (for access trestles);
- c) a steel cross-head supporting longitudinal steel beams and a concrete slab (for access trestles).

8.9.3 Timber deck

Lightly loaded structures may be built entirely of timber (i.e. piles, beams and deck together with bracing), or a timber deck may be provided on steel or concrete beams. Timber decks are likely to be suitable only in structures for use by pleasure or fishing vessels.

8.10 Design methods

8.10.1 Vertical actions from superstructure

For the purpose of initial design, quay and jetty structures may be designed so that vertical actions are resisted:

- a) by the vertical piles alone, with raking piles (where provided) resisting all the horizontally applied actions; or
- b) by the vertical piles and opposing pairs of raking piles.

All actions can then be assumed to be resisted by axial effects in the piles (i.e. no bending). The validity of this assumption should be checked using the relative elastic deformations of the piles. The elastic deformation of differentially loaded piles will alter the effects in the superstructure. The deformation depends on the type of ground into which the pile transfers the action, and on the effective length, cross-section and material of the pile.

The possibility of uplift resulting from waves passing under the deck, particularly in the case of exposed jetties, should be taken into account. Where necessary, vent holes should be provided in the deck. The effect of hydrostatic uplift on the low decks, such as relieving platforms, should be checked.

8.10.2 Geotechnical actions

8.10.2.1 Retaining walls

The horizontal actions due to soil on the rear face of retaining walls supported by the deck are resisted in rigid structures by axial effects in raking piles or tie-backs.

8.10.2.2 Negative skin friction

The effects of negative skin friction should be taken into account, which will usually be mitigated by allowing settlement. See **8.8.3.1**.

8.10.2.3 Other soil actions

Where there is danger of slope instability causing lateral actions on the embedded lengths of piles, the lateral resistance of the piles should be calculated to be sufficient to withstand the actions.

8.10.3 Berthing and mooring actions

8.10.3.1 General

Berthing and mooring actions should be determined in accordance with BS 6349-4.

8.10.3.2 Transverse actions

Movements and resulting effects caused by berthing and mooring actions applied at right angles to the berthing face are calculated by analysis of the entire structure as a three-dimensional model. Elastic extensions of tie-backs, and extensions or shortening of raking piles, will result in small horizontal deflections at the heads of any vertical piles, but effects induced in vertical piles by these deflections may usually be ignored.

Berthing and mooring actions applied to structures built on vertical piles only are resisted in bending by the vertical piles in the ratio of their stiffnesses. Thus the rear row of piles in a marginal quay, which is usually almost entirely embedded in the ground, will resist a high proportion of the action, and might require piles of greater bending and shear resistance than elsewhere. The effects can be reduced by sleeving the upper portion of the piles to permit freedom of movement.

In marginal quays built on vertical piles only, where the rear edge of the deck abuts the fill or a bulkhead wall, berthing actions may be assumed to be resisted by the pressure of the soil acting on the rear face. If passive resistance is assumed in design, the movement required to mobilize the passive resistance should be estimated. The proportion of action resisted by the piles should be ascertained to assure both their bending capacity and the reduction in their vertical resistance.

The length of structure over which the transverse actions are resisted should be calculated by elastic analysis. When carrying out bending moment calculations, concentrated actions may be assumed to disperse at 45° to the cope line over the full width of the concrete deck. The dispersed length along the opposite edge of the deck is the length over which the actions are assumed to act as a uniform effect. The effect of two simultaneously applied actions giving overlapping dispersed lengths, such as mooring actions on two adjacent bollards, or mooring actions imposed on one side of a jetty with berthing actions on the other, should be taken into account. Similar actions applied near the end of a structure should be distributed with care to ensure that the resultants of the applied and distributed actions coincide.

8.10.3.3 Longitudinal actions

Longitudinal berthing and mooring actions that are applied to structures built on vertical piles only may generally be assumed to be resisted in bending, in this direction, by all piles supporting that part of the deck which is continuous with the action application point. Thus, only small longitudinal deflections will normally occur because of the large number of piles resisting the action. Where deflections are not within the acceptable range, longitudinal raking piles should be provided. In structures containing raking piles, these piles should be designed to resist the components of actions in their line of action with no contribution by bending action from the vertical piles.

8.10.3.4 Energy absorption

Flexible structures can be designed to achieve the maximum energy absorption of berthing actions by combined use of the structure and the fendering, to reduce fendering requirements. However, it is important to ensure that calculations are performed to model the situation at different states of the tide, since it is usual for the berthing impacts at low tide to be applied at a lower level and therefore at a point where the structure has less capability of absorbing energy.

The contribution of the structure will be greater where piles of high yield steel in conjunction with varying wall thickness are used. Piles generally have a constant diameter over their full height in view of the costs involved in constructing a tapered section, including potential stiffeners to take loads during pile driving. To prevent premature buckling of the steel section, the requirements of BS EN 1993-1 should be followed.

The small proportion of berthing energy which can be absorbed by rigid structures may generally be ignored in energy calculations.

8.10.4 Other actions

8.10.4.1 Temperature

Changes in overall temperature of structures are usually accommodated by allowing expansion and contraction of the deck to take place unimpeded. The resulting longitudinal movement, assumed to take place to both sides of the midpoint or of a strongpoint, causes deflection of pile heads, which should be calculated. Long quays may be subdivided by transverse movement joints to limit pile deflections.

The thermal movement of the structure should be taken into account when locating raking piles to resist longitudinal actions, together with

the anchoring effect of fully embedded piles such as at the rear of marginal quays.

Jetty access trestles are normally anchored at the shore end and left free to move at the seaward end, an expansion joint being provided next to the jetty head. Long access trestles should be subdivided to limit pile deflections, each section having an anchorage of raking piles.

Trestles and anchorages for pipelines should be strong enough to withstand the friction effects arising from temperature actions.

8.10.4.2 Waves, currents and ice

Actions due to waves, currents and ice acting on the piles are transmitted to the deck and the ground. The deck reactions are resisted by bending of the piles in flexible structures, and by raking piles in rigid structures. The expected increase in pile section resulting from marine growth during the design life should be taken into account. The horizontal deflection of the deck of flexible structures due to these actions should be estimated.

A piled or suspended screen, either solid or perforated, may be provided in front of the structure to reduce effects in the piles or on vessels moored alongside. The structure should then be designed to resist actions transmitted from the screen. Waves (especially those of long periods) and currents cause motion in vessels berthed alongside, giving rise to bollard and fender actions (see BS 6349-1:2000, Clause 42, and also BS 6349-4), which can be resisted as described in 8.10.3.

8.10.4.3 Wind

In the design of access trestles, the effect of wind on overall stability should be checked.

8.10.4.4 Earthquakes

NOTE See also 4.3.8.

Actions induced by slope slump should be resisted by members having support systems deriving their resistance from below the level of expected induced instability. The possibility of slump of any underwater slopes should be investigated. Longitudinal accelerations sometimes require special pile/deck connections to achieve an effective moment transfer.

The large mass of relieving platform superstructures should be taken into account in calculating effects.

8.10.5 Fatigue

Fatigue effects in suspended deck structures are not usually significant, but might need to be taken into account for structures subjected to repeated waves or vortex shedding in strong currents (see BS 6349-1:2000, Clause 47). Where fatigue effects are important, connections should be designed taking these into account.

Fatigue loads can play a role during construction of a piled structure. In particular, fatigue effects in free-standing piles subjected to wave loads can be significant, even during construction, depending on the local wave and current conditions.

Fatigue effects can also be important if the final piled structure has a large free-standing length without being supported by raker sets.

8.11 Loading considerations

8.11.1 Self-weight and imposed loads

8.11.1.1 Self-weight

The distribution of self-weight loads in the superstructure depends on the sequence of construction. The self-weight loads to be resisted by each load-bearing member at each stage of construction should be calculated and the support condition of the member appraised.

For structures with relatively high vertical loads and very small horizontal loads (e.g. trestles in sheltered water), especially in deep water, a fictitious horizontal load in accordance with BS EN 1993-1-1:2005, 5.3.7 should be included.

8.11.1.2 Fixed actions

Fixed actions from footings of fixed superstructures may be assumed to disperse at 45° to the vertical from the edge of the contact area down to the neutral axis of the deck member.

8.11.1.3 Imposed loads on deck

Imposed loads (including horizontal and secondary actions) should be determined in accordance with BS 6349-1:2000, Clause 44 and Clause 45, and included in the design of the superstructure in the appropriate loading case. Actions imposed by Ro-Ro ship quarter ramps should be taken into account where appropriate.

NOTE Further guidance is given in Recommendations of the Committee for Waterfront Structures [11].

8.11.1.4 Erection actions

Actions occurring during each stage of construction should be estimated, where possible, and the structure checked accordingly. The effect of these actions can exceed the effects of actions on the completed structure. If it is not possible to estimate or specify such actions with any accuracy when the design is prepared, they should be checked before construction commences.

8.11.2 Horizontal actions

8.11.2.1 Berthing

Berthing actions should be calculated for each class of vessel expected to use the berth during its design life: the energy should be calculated in accordance with BS 6349-4. The effects of vertical and longitudinal effects associated with friction between the fenders and the vessel should be included.

Because of potentially disproportionate damage arising from collisions with loading platforms for petrochemical or other hazardous liquids, berthing facilities for VLCCs, and LPG/LNG carriers (other than small size), should preferably be separated from the loading platform to avoid impact loads on the platform. Impact loads should be taken by breasting dolphins, for which the most suitable design is likely to comprise a rubber fender(s) mounted on either a flexible or a fixed dolphin.

Suspended decks can be particularly sensitive to berthing actions, especially where there is a large tidal range or variety of vessels to be accommodated. The type of fendering to be used should be decided at an early stage of design. The implications of high reactions due to abnormal or accidental berthing should be assessed.

Figure 36 shows typical structural arrangements used to resist berthing actions.

8.11.2.2 Mooring

Mooring actions from vessels' lines are applied via quay fittings such as bollards, hooks, fairleads or rings fixed to the top of the deck, to the front face or on a raised platform on the deck. Except where small actions are involved, the quay fitting should be fixed to the seaward end of a cross-head beam, which should then be designed to resist the total actions that can be applied over a range of horizontal and vertical directions. The actions that need to be resisted are described in BS 6349-1:2000, Clause 42.

8.11.3 Environmental actions

8.11.3.1 Wind

Wind actions acting directly on the maritime structures should be calculated in accordance with BS EN 1991-1-4. Wind actions on ships that are transferred to maritime structures via mooring lines should be calculated in accordance with BS 6349-1. The actions are resisted by axial and bending effects set up in the deck members.

8.11.3.2 Temperature

Overall changes in temperature are discussed in 8.10.4.1. Where structures, or parts of them, are restrained from movement, the resulting effects, and the means of resisting them, should be in accordance with BS EN 1993 and BS EN 1992 for steel and concrete respectively.

8.11.3.3 Earthquake

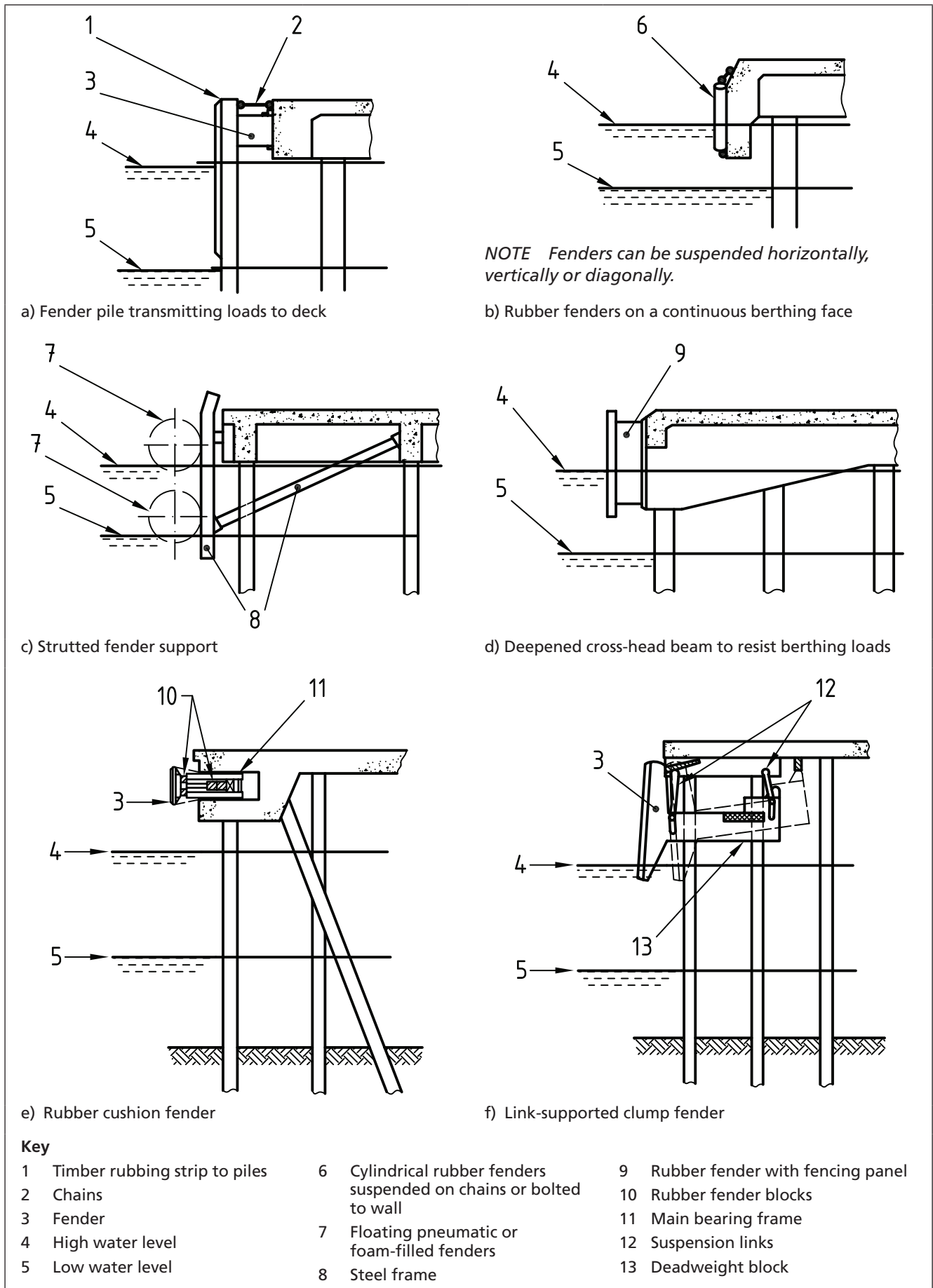
Flexible structures are designed to resist earthquake actions by bending of the piles. The use of raked piles for structures that have a high probability of being subject to earthquake actions is not recommended, because experience has shown that the rigidity of such structures generates very high local stresses, which often lead to failure.

8.11.3.4 Ice

An assessment should be made to decide whether ice actions will be applied to the structure. Actions caused by ice on the pile at the water line should be assessed and taken into account. The pile design calculations might need to be supported by (probabilistic) ice modelling and physical testing. The geometric pile design can be optimized to reduce the loads. In this respect, special cone structures have been used in the past to ensure flexural breaking of the ice.

The design against local buckling might require additional wall thickness or reinforcement.

Figure 36 Fender support arrangements



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8.11.4 Design situations

8.11.4.1 General

The superstructure should be designed to resist all combinations of actions that might realistically be assumed to act on the structure simultaneously. Actions can act directly on the superstructure or indirectly via the piles.

The individual actions that the superstructure should be designed to resist in each combination are described as the permanent actions *G*, variable actions *Q*, and accidental actions *A* as defined in BS EN 1990:2002+A1:2005.

8.11.4.2 Limit state design

The methods of limit state design, and the way partial factors are used to define representative actions in design situations, should be in accordance with BS EN 1990. The values of partial factors, combination factors and rules for combinations not included in BS EN 1990 should be taken from Annex A in the present part of BS 6349.

8.12 Design of piles

8.12.1 General

Installation actions can be a controlling factor in the choice of pile size. The structural design of concrete piles should be analysed by limit state methods in accordance with BS EN 1992-2, while that of steel piles should be analysed by limit state methods in accordance with BS EN 1993-2. Timber piles should be analysed with reference to BS EN 1995-2.

The effect of pile joints in precast and timber piles should be taken into account.

For all piles, design should be in accordance with BS EN 1997-1.

8.12.2 Column buckling

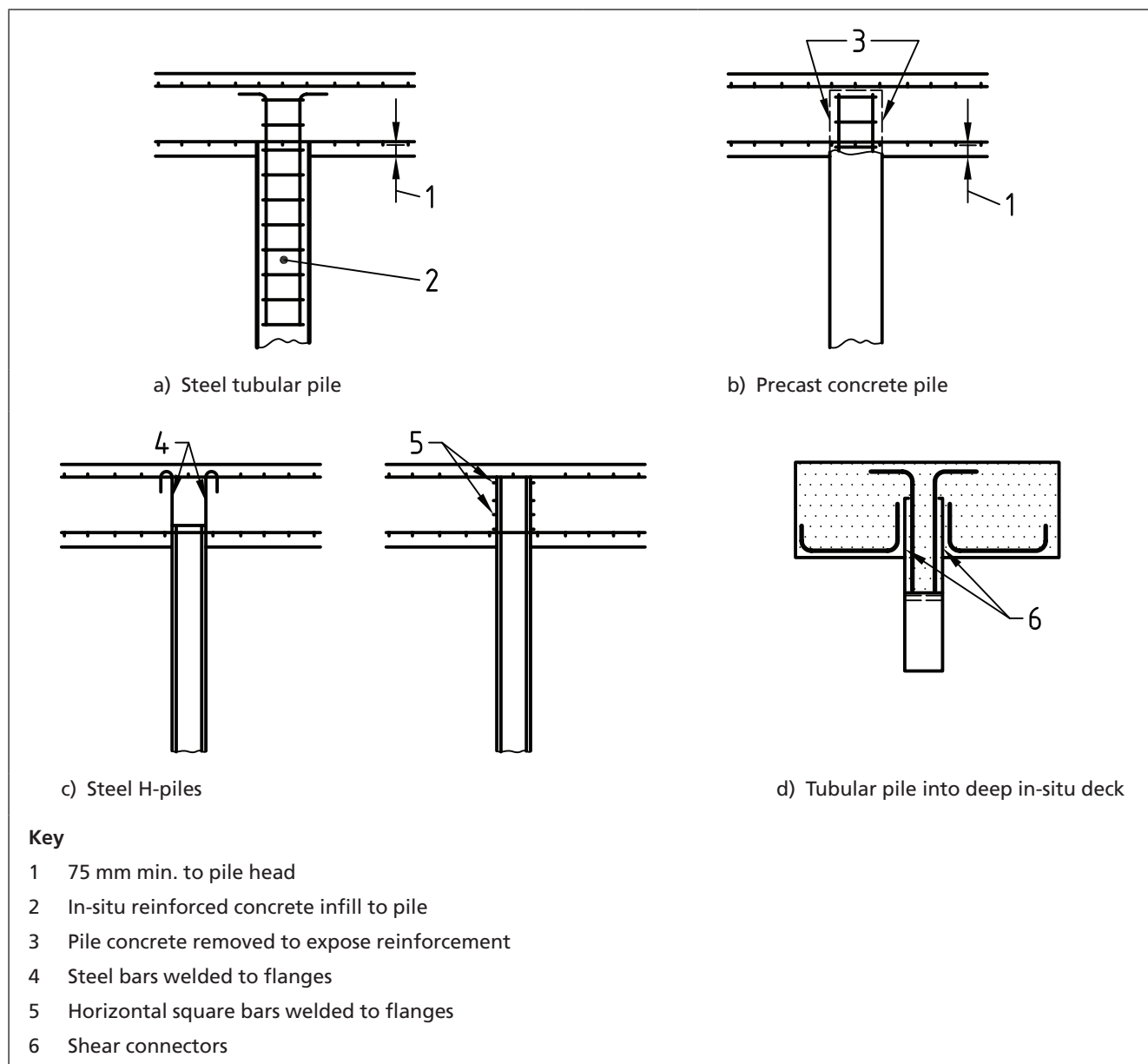
Piles extending through water and/or soft soils should be designed to avoid failure by column buckling.

The effective column length of the piles should be determined assuming that the top of the pile is either at the level of the underside of the deck or at the level of any bracing in the plane being considered, whichever is the lower, and that the bottom of the pile is its apparent point of fixity, taking into account any future deepening, scour, overdredging or soil liquefaction. The point of fixity of piles in slopes under marginal quays should be determined taking into account the angle of the slope and the likely degree of restraint due to any rubble protection provided.

The heads of piles are usually built into the superstructure, thereby achieving the minimum effective length and maximum load-bearing capacity. Figure 37 gives examples of pile/deck connections.

Unless temporary bracing is used, the effective length of piles during construction and testing is usually greater than in the final condition. Allowable effects due to actions applied prior to completion of the pile/deck connection can be limited by the temporary effective length.

Figure 37 Examples of pile/deck connections



8.12.3 Local buckling of steel piles

Steel piles should be checked for local buckling of flanges, webs or other elements of the member. Elements of the pile should, where appropriate, be checked for local buckling arising from shear actions. Unstiffened tubular steel piles should be checked for local buckling due to axial compression and bending.

NOTE Guidance is given in BS EN 1993-1 and BS EN 1993-5.

Where local buckling is a problem, precautions should be taken by the use of a local internal concrete plug or other means of stiffening.

8.12.4 Load transfer to soil

Open-ended tubes and hollow sections frequently plug and become displacement piles, particularly in cohesive soils. H-piles can behave similarly. Designers should use their experience of soil/pile interaction and apply appropriate factors to determine the effective perimeter and corresponding end-bearing area of the pile section to use in design calculations.

Where a plug of soil forms, the end resistance is calculated using the gross cross-sectional area, but for hollow sections, steps should be taken to ensure that the internal water level is at least as high as the outside water level, to prevent subsequent plug wash-out leading to potential loss of toe resistance. If plug formation is doubtful, the net pile area should be used. When lateral deflections associated with cyclic loads at or near the seabed are relatively large, consideration should be given to reducing or neglecting skin friction in this zone.

8.12.5 Vertical actions

8.12.5.1 Vertical piles

The action to be resisted by the soil is the sum of the pile head action and the pile self-weight permanent action. If there will be downdrag during its working period due to negative skin friction and it is decided that this will need to be resisted because additional settlement cannot be tolerated, then the maximum downdrag should also be added to the actions calculated. Providing that piles are installed within the tolerances recommended in 8.13, incidental eccentricities can be ignored. However, where lateral deflections are caused by applied actions, the effect of induced eccentricities should be taken into account.

The net uplift action is calculated as the maximum uplift effect due to imposed actions minus the permanent self-weight action. Uplift to be resisted in friction by the soil should be calculated ignoring downdrag. The group effect of piles should be taken into account.

8.12.5.2 Raking piles

Raking piles designed to resist vertical actions should be arranged in opposing pairs of equal angle of rake unless they are required simultaneously to resist a lateral action. The vertical action at the head is assumed to be divided equally between each of the two piles.

The dead weight of raking piles and downdrag acting on them are resisted in bending for the component acting transversely to the pile section, and in compression for the longitudinal component.

8.12.6 Horizontal actions

8.12.6.1 Flexible structures

Vertical piles resisting horizontal actions in double bending should be designed to resist the maximum vertical loads which can occur simultaneously, due account being taken of the effect on pile capacity of induced horizontal deflections.

Where large horizontal actions cause high bending moments in the stiffest piles (such as the rear piles of a marginal quay), piles of greater bending resistance might be required. For piles with long embedments, this may be achieved more effectively by increasing the bending resistance only where required.

Piles resisting lateral actions should be embedded to obtain the required resistance from the soil taking account of possible socketing effects in the soil resulting from reversals of loading. This will usually be possible without excessive penetrations if the seabed is of medium to dense granular soil, firm to stiff clay, or weak rock.

NOTE Guidance on the soil resistance of laterally loaded piles is given in Pile design and construction practice [22] and Design of laterally loaded piles [23].

The restraint offered by rubble protection layers on slopes under marginal quays should be taken into account in assessing the relative stiffnesses of each row of piles. The piles should be designed to resist actions originating from any possible direction, and non-symmetrical sections should therefore be checked for biaxial bending. Where large actions are applied predominantly from one direction (usually at right angles to the centreline of the structure), the major axis of the pile should be orientated at right angles to it.

The effect of concrete infill and muffs on pile stiffness should be taken into account.

8.12.6.2 Rigid structures

Actions applied horizontally to rigid structures are resisted by raking piles in direct compression or tension. Raking piles should, therefore, normally be provided parallel to both axes of the structure. For greatest efficiency, raking piles should be installed with as large a rake as possible, except where this would cause interference with other structures or with vessels, or where downdrag is likely to have an important effect.

Where means other than raking piles are adopted to provide rigidity, the effects of horizontal actions applied over the entire range of possible angles should be taken into account.

8.12.7 Wave and current actions

8.12.7.1 During construction

Actions due to waves and currents acting on unbraced piles during construction should be taken into account. In particular, the possibility of current-induced oscillations should be investigated (see BS 6349-1:2000, 38.3) and, if necessary, suitable measures should be taken: these may include altering the pile stiffness, fitting spoilers to the piles or bracing them.

8.12.7.2 Permanent condition

Wave and current actions on individual piles should be calculated as described in BS 6349-1:2000, Clause 38 and Clause 39. The piles resist the applied actions by beam action, the end conditions corresponding to the degrees of fixity at the seabed and deck.

Guidance on the analysis of the bow-string oscillation of fixed-ended piles due to current actions is given in BS 6349-1:2000, 38.3. This analysis should be carried out where fast currents occur.

8.12.8 Tension anchors

Tension anchors for jetty structures should preferably be designed as passive anchors. Design should be in accordance with BS EN 1997 and BS EN 1537.

Piles anchored to the ground to obtain the required pull-out resistance should be capable of resisting the compressive actions imposed by the anchor, including any test actions.

8.12.9 Testing

The actions applied during testing of piles, for vertical, pull-out or lateral actions, should be taken into account in pile design, using the effective length of the pile at the testing stage. Similarly, the testing of pile tension anchors should be allowed for in pile design.

8.12.10 Pile movements

8.12.10.1 Vertical

Vertical movements of pile heads, resulting from the deformation of the pile and the ground under maximum instantaneous and sustained actions, should be calculated for the permanent and variable actions together with any downdrag. Pile size and spacing should be adjusted, if necessary, to limit movement, in particular, differential settlement of the deck and the resulting effects occurring at the pile head.

8.12.10.2 Horizontal

Horizontal movement of pile heads should be assessed for each combination of maximum horizontal actions. The possibility of two adjacent structures deflecting in opposite directions should be taken into account.

8.12.11 Effects in piles

8.12.11.1 Concrete piles

Reinforced and pre-stressed concrete piles should be designed as columns subjected to axial and bending effects, using partial factors in accordance with Annex A and with sufficient reinforcement to resist driving stresses.

Tension piles should be designed ignoring the tensile resistance of the concrete.

8.12.11.2 Steel piles

Steel piles should be designed in accordance with BS EN 1993-2 and BS EN 1993-5. Effects should be calculated taking into account the driving stresses and corrosion during the design working life of the structure.

The partial factors and combination factors listed in Annex A should be used.

8.12.11.3 Timber piles

Timber piles should be designed in accordance with BS EN 1995-2, using the combinations of loads and actions defined in Annex A of the present part of BS 6349.

8.12.11.4 Soil resistance

Soil resistances should be calculated in accordance with BS EN 1997.

8.13 Construction tolerances

8.13.1 Control of tolerance

Methods of construction should be designed to avoid multiple tolerance requirements at a single location. The tolerances recommended in **8.13.2** and **8.13.3** may be reduced if special construction methods, such as appropriately designed guide frames, are used.

8.13.2 Pile head

The centre of piles at the junction with the superstructure should be within 75 mm of the nominal position for structures built on land or in sheltered waters. For piles driven through rubble slopes, this tolerance may be increased to 100 mm. For access trestles and jetty heads, a tolerance of 75 mm to 150 mm should be adopted, depending on the exposure of the site. The design of piles and deck members should take into account the specified tolerances.

8.13.3 Pile centreline

The inclination of vertical piles should not exceed 1:100 from the vertical in sheltered waters and 1:75 at exposed sites. The deviation of raking piles should not exceed 1:30 from the specified rake in sheltered waters and 1:25 at exposed sites.

9 Dolphins

NOTE Guidance on dolphin layouts and fendering and mooring systems is given in BS 6349-4.

9.1 Types of structure

9.1.1 Function

Berthing (or breasting) dolphins are usually provided at jetties, such as those at bulk handling terminals, where the jetty head is required only to support cargo handling equipment. They might also be required at Ro-Ro terminals where there is no quay for vessels to berth alongside. Dolphins for manoeuvring vessels may be located at dock or lock entrances or at berths having fixed cargo handling equipment, where vessels might have to be winched along the berth.

Mooring dolphins are usually provided at jetties (including bulk terminals) whose length is less than that of the berthing vessels, and may also be used as fixed moorings at locations remote from a berth.

9.1.2 Structural stiffness

NOTE Dolphins may be rigid or flexible as defined in BS 6349-1:2000, 47.1.

9.1.2.1 Flexible structures

Flexible dolphin structures usually comprise a group of vertical piles built into a heavy concrete cap or a braced frame and deck. Multiple rows of cantilever piles can also be used. Flexible dolphins may also be built using one or more large diameter vertical piles. For light loads, timber piles may be used. Figure 38 shows examples of flexible dolphins.

9.1.2.2 Rigid structures

Rigid dolphin structures may be of open-piled or solid construction. Examples are:

- a) a group of raking piles (with or without vertical piles) with concrete cap;
- b) a group of braced piles supporting a platform;
- c) a group of vertical piles with concrete cap and strut/tie to shore;
- d) a concrete caisson;
- e) a steel sheet-piled cell.

Figure 39 shows typical examples of rigid dolphins.

NOTE Steel jackets are sometimes used as dolphins at deep water terminals.

Figure 38 Examples of flexible dolphins

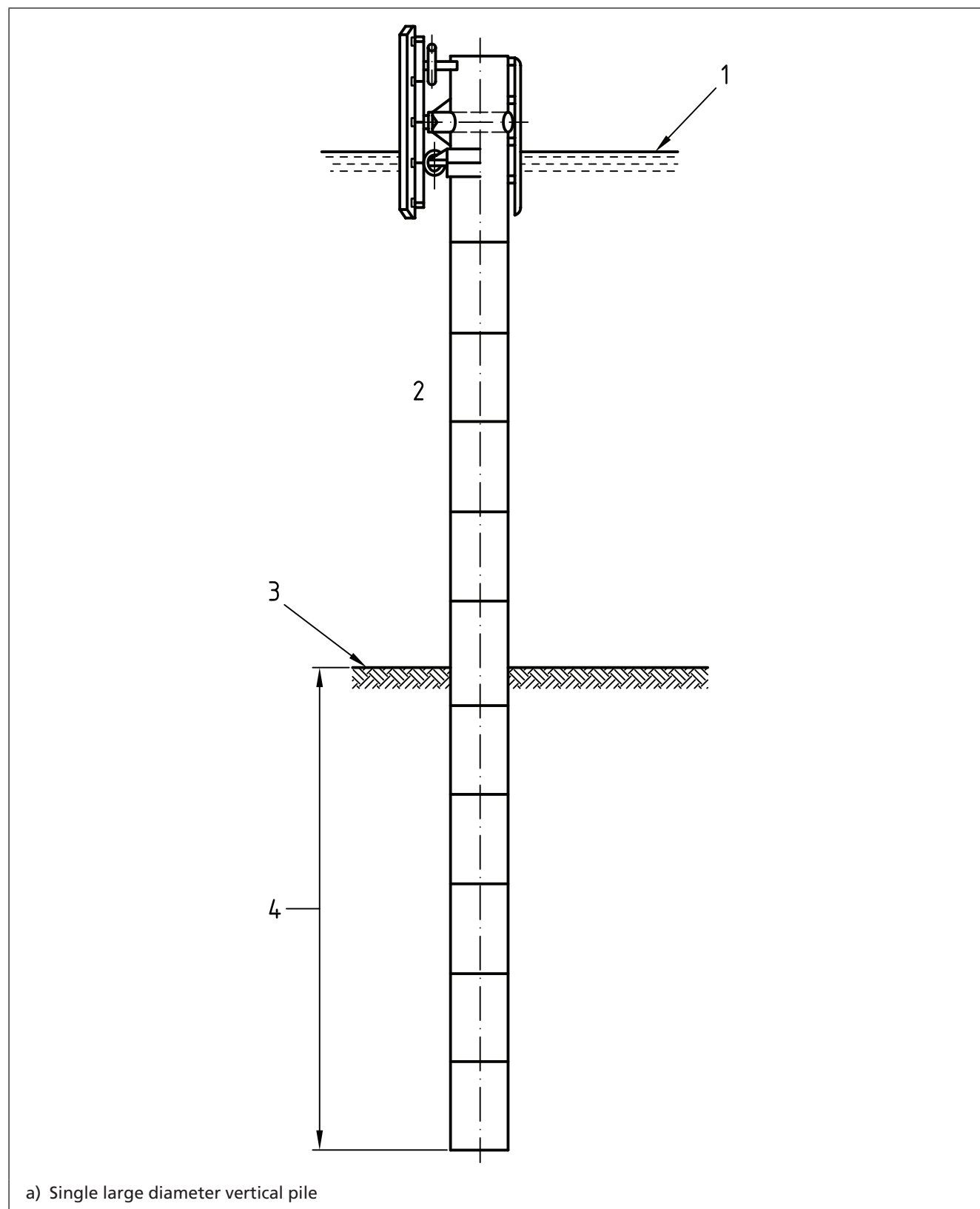
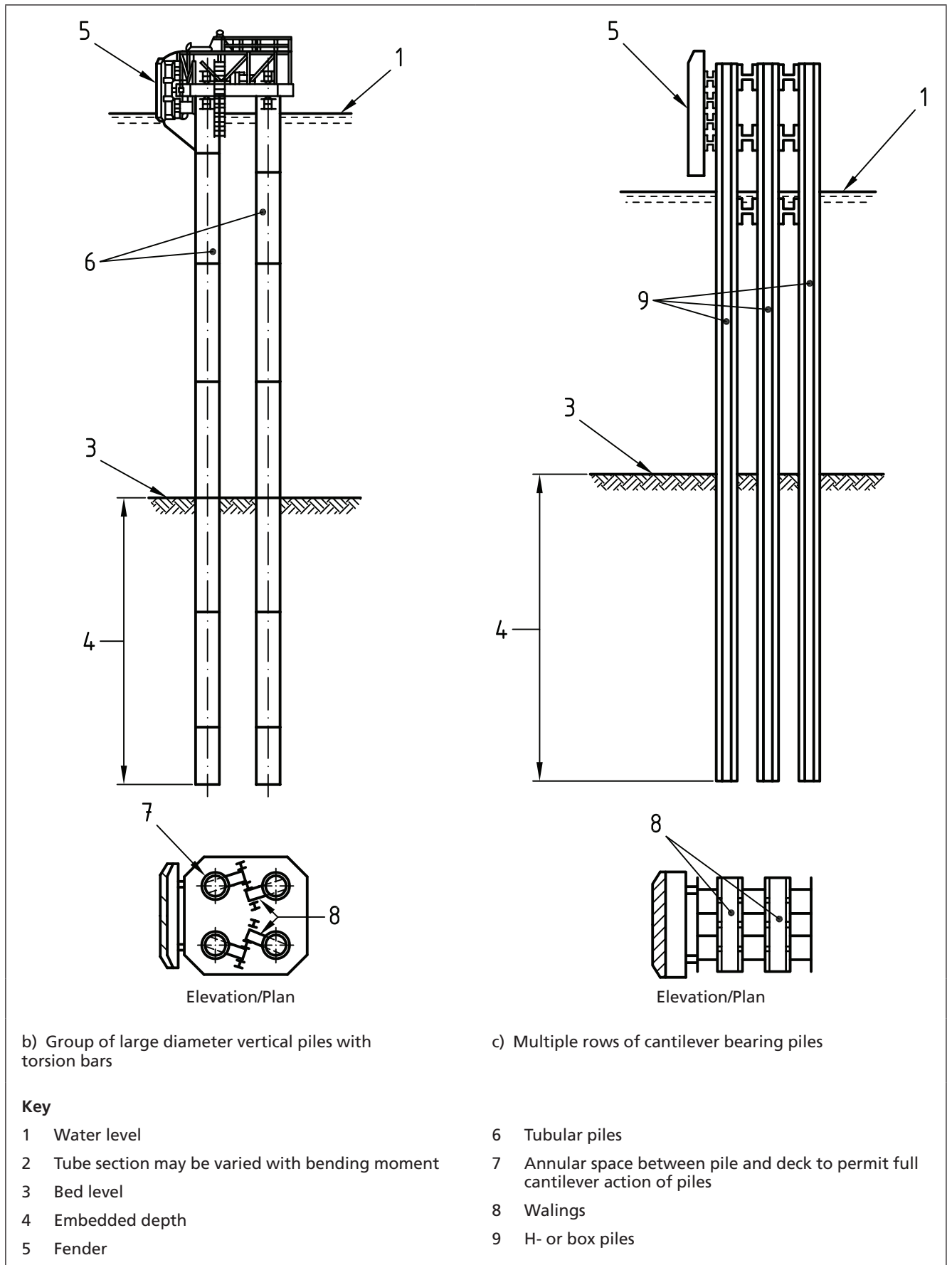
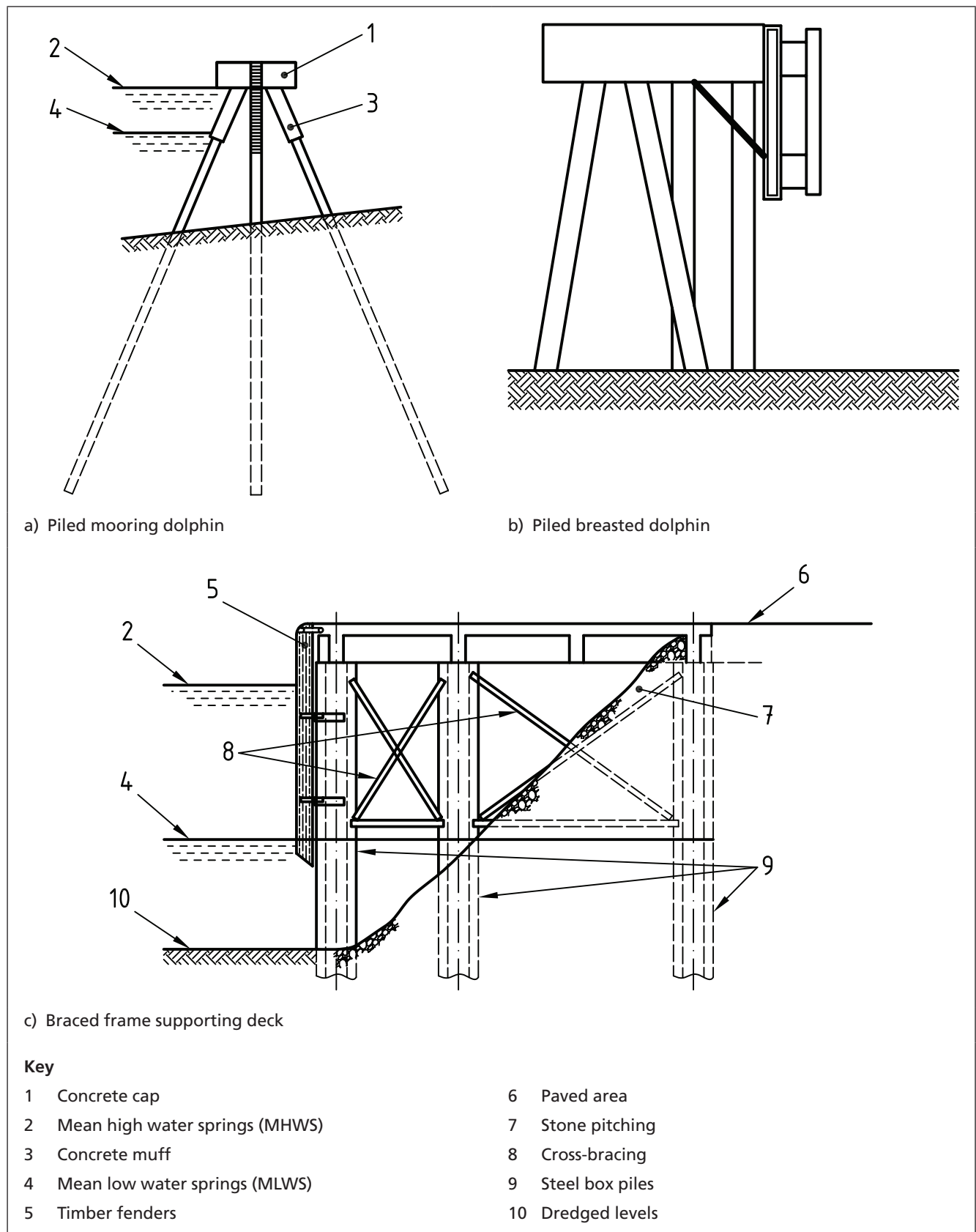


Figure 38 Examples of flexible dolphins (continued)



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Figure 39 Examples of rigid dolphins



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9.1.3 Choice of structure

The type of structure to be adopted depends largely on the factors discussed in 4.3. With groups of piles, the resistance to uplift caused by horizontal actions should be carefully assessed. The advantages and disadvantages of individual structural types are discussed in Clauses 6, 7 and 8. Onshore mooring points may be provided, depending on the proximity of dry land, shallow water or a suitable existing structure.

The flexibility of the structure should be taken into account in relation to its function and the type of operations to be carried out at the berth. Whereas berthing or lead-in dolphins may be flexible, mooring dolphins should be relatively rigid so that the tension in vessels' lines is maintained. At container terminals, it might be necessary to restrict the flexibility of berthing dolphins in order to maintain suitable conditions for handling operations.

9.2 Actions

9.2.1 Berthing actions

Berthing actions should be calculated in accordance with BS 6349-4. The design of berthing and lead-in dolphins and berthing beams should take into account all normal variations in the elevation, direction and magnitude of applied actions.

Account should also be taken of the possible consequences of accidental vessel impact on the berth structure, both to the structure itself and to structures alongside. Where appropriate, the structure may incorporate elements designed to break off or collapse under accidental actions, or the whole structure may be designed to collapse in a way which provides additional energy absorption. The proportion of energy absorption to be provided by the fendering will depend on the flexibility of the structure. In rigid structures, the small amount of energy absorbed by the structure should be ignored. The choice of type of fendering to apply actions at the most suitable points on the structure can lead to more effective design concepts.

9.2.2 Mooring actions

Mooring actions should be calculated in accordance with BS 6349-4. They include actions from breast, head and stern lines, and from spring lines. In addition to the variations in inclination, direction and degree of applied actions, combinations of actions on a structure should be taken into account. These may include actions applied by two vessels simultaneously.

The structural form of the dolphin should be appropriate to the position and elevation of the mooring points. At exposed sites, physical or mathematical model tests might be required to determine the optimum mooring system and the actions resulting, taking into account the elasticities of the mooring lines and fenders and the response of the dolphins (see BS 6349-1:2000, 31.3).

9.2.3 Environmental actions

NOTE Guidance on environmental actions is given in BS 6349-1:2000, Clauses 34 to 40, and on movements and vibrations in BS 6349-1:2000, Clause 47.

The design of dolphins at exposed sites should take into account cyclic actions due to waves, while vortex shedding from circular piles should be analysed for sites where currents might be sufficient to cause this effect.

The resonance of flexible structures during earthquakes should be checked.

9.3 Design of flexible dolphins

Flexible dolphins are designed to absorb the kinetic energy of a berthing vessel by horizontal displacement of the pile heads. Berthing actions applied at the corner of a dolphin will result in torsion in the structure; this may be reduced by judicious positioning of the fenders, or made use of by providing torsional strength to increase energy absorption.

Where large diameter steel tubes are used, the design may take into account the torsional resistance of the piles in order to increase the total resistance of the structure. The piles should be sufficiently embedded to obtain the required lateral and torsional resistance from the soil (see also 8.12.6.1 and *Pile foundation analysis and design* [24]). The combined deflection of the structure and fenders under maximum load should generally not exceed 1.5 m, to avoid problems with ship handling. Large deflections can adversely affect safety in rope handling and other operations. Large deflections should also be taken into account for connections of walkways and other structures.

Hulls should not be able to strike piles when the head of the structure is deflected.

Piles for flexible dolphins usually comprise tubes of high yield steel, which are suitable on account of their high strength and energy-absorbing properties. Tubes of variable thickness are often used for economy in structures constructed in deep water. Steel H-piles can be used but are more vulnerable to corrosion and are weak about their minor axis. Pre-stressed concrete cylinders may also be used. For berthing beams, steel H-piles are most suitable.

9.4 Design of rigid dolphins

9.4.1 Raking pile structures

9.4.1.1 Design methods

Raking pile structures are conventionally designed assuming that only axial actions are transmitted to the piles as a result of actions applied to the structure by vessels. Environmental actions such as wave and current actions will cause lateral effects in the piles.

Raking piles should not project into locations where they could be struck by vessels.

Where ground conditions are such that considerable downdrag might occur, the resulting effects set up in raking piles might be excessive. This type of structure might not then be suitable.

Pile axes should preferably be arranged so that they intersect in pairs or groups at a minimum of two locations on plan. The resultant applied action should, where possible, pass between these locations, which should be as widely spaced as possible, especially in cases where the applied action has a considerable turning effect about the elastic centre of the pile group. In many instances, particularly in the case of mooring dolphins, it will not be practicable to provide more than one pile group. By arranging the piles so that the elastic centre of the pile group lies on or near the resultant applied action, the likelihood of rotation is reduced with consequent lessening of the pile effects. Where the position of the resultant varies, the elastic centre should be located as close as possible to the intersection of the resultant actions. The effect of head and toe fixity may usually be neglected during trial designs, but in the final design should be checked fully.

Bending moments and other internal effects are best determined by computer analysis of a three-dimensional model of the structure.

9.4.1.2 Resistance to uplift

The horizontal actions applied to rigid piled dolphins are resisted by axial effects in the raking piles, and the pile head connection should be designed accordingly. Some piles might have to resist uplift. Where it is not possible to achieve the necessary resistance, the uplift may be resisted by a rock anchor installed through the pile after driving.

Net uplift on the structure may be reduced by providing a massive concrete pile cap, which also serves to disperse concentrated actions and to ensure rigidity. Such caps are commonly 1.5 m to 2.5 m thick.

9.4.1.3 Overall stability

The overall stability of structures should be verified in accordance with BS EN 1990, using the partial factors listed in Annex A of the present part of BS 6349 for the ultimate limit state EQU (as defined in BS EN 1990:2002+A1:2005). Where there are overload conditions such as accidental impact or certain types of extreme line pull, these should usually be regarded as accidental actions.

9.4.2 Other types of structure

Rigid dolphins other than those on raking piles may be solid structures or a group of vertical piles with a concrete cap and a strut/tie to shore.

Solid structures should be designed in accordance with Clause 7. Common types of structure for this purpose are concrete caissons, steel sheet-piled cells (circular or rectangular) and concrete blocks with or without vertical rock anchors.

Structures which incorporate a strut/tie to a strongpoint on the shore resist horizontal actions by axial effects in the strut/tie members, which should be designed to resist failure by buckling: for buried struts, the bracing effect of the ground may be taken into account. Longitudinal berthing and spring line actions will cause one of the members to be in compression and the other in tension. Uplift due to mooring actions applied to strut/tie dolphins is resisted by the vertical piles of the dolphin.

Stability of the structure is determined in the same way as for a quay structure of the same type. Tie rods for strut/tie members should be designed as described in Clause 6. In overload conditions the design

situation should be regarded as an accidental design situation, and the load combination formulae for accidental design situations listed in Annex A should be applied.

9.5 Mooring equipment

The mooring equipment to be provided on dolphins will depend on the sizes and types of vessel using the berth, the equipment provided on board the vessels, the environmental conditions and, in many cases, the experience of the port operator and labour force.

Where large line forces are expected, mooring line action measuring equipment, linked to a display in the terminal operations room, can be provided. This may be used to adjust the mooring lines during cargo handling operations, thereby allowing adjustment of line actions.

Mooring devices may be proprietary designs with safe working loads guaranteed by the manufacturer or else structurally designed using the appropriate Eurocodes to provide resistance to the ultimate limit states. In either case, a safe working load for the device can be derived, and all such mooring devices should have a safe working action of not less than the maximum breaking load of the largest line. Guidance on line pulls is given in BS 6349-4. The factors of safety against yield of the material and against breakage of the device should be specified in relation to the safe working load of the mooring device. Factors of safety of 1.25 and 2.5 respectively are commonly adopted. Overloading of the dolphin structure may be prevented by a failure device such as break-off foundation bolts.

Provision should be made, if necessary, in the pile cap for cable ducts to each item of motorized equipment. Cables to the dolphin should be run in ducts fixed to the walkways where these exist, or otherwise laid on the seabed. If there is a risk of damage by anchors or material falling through the water, the cables should be laid in a trench.

9.6 Access and working space

9.6.1 Access

Access to dolphins for operating equipment and handling lines generally takes the form of a series of walkways from the jetty head. The walkways should be positioned so as to minimize the potential of damage by vessels. A typical layout is shown in BS 6349-4. All isolated dolphins and berthing beams should be equipped with ladders. For safety reasons and ease of line handling operations, the ladders should be installed on a face which is not used for berthing ships.

Where dolphins are built adjacent to land, pedestrian access should be provided. Walkways and ladders should be provided in accordance with Clause 11.

9.6.2 Working space

Dolphins should be provided with a platform having adequate working space. Where an adequate area is not derived from structural considerations, a minimum clear space of 1.5 m should be provided around the working side of quick-release hooks, capstans and winches, and 1.0 m around bollards and fairleads.

A clear space should be provided at the ends of walkways and heads of ladders.

All working areas should be adequately drained and illuminated. Illumination should conform to BS EN 12464-2.

NOTE Guidance on illumination levels is given in CIBSE lighting guides LG1 [25] and LG6 [26], and CIE publications 129/1998 [27] and 015/E 2005 [28].

9.6.3 Handrails and toe kerbs

Handrails and or toe kerbs should be provided on appropriate faces of all dolphins.

10 Roll-on/roll-off terminal ramps

Ro-Ro facilities should be designed and constructed in accordance with BS 6349-8.

11 Pedestrian access

11.1 General

Walkways, stairways, ladders and handrails should conform to BS 4211, BS 4592-0 and BS 5395-1 in respect of general design, durability and safety wherever practicable. Permanent means of access to machinery should conform to BS EN ISO 14122.

NOTE Further guidance is given in Safety and health in ports [6] and COP 25 [7].

Access for inspections and maintenance work under suspended decks may be facilitated by the provision of occasional permanent fixings on piles and deck soffits to enable small boats to be moored and staging to be supported. For major trestle structures, a permanent access structure for maintenance work might be required.

11.2 pontoons

Pontoons are a preferred means of access from small passenger vessels, fishing boats and leisure boats to the shore.

11.3 Walkways

Walkways are usually required for access by pedestrians to isolated structures such as dolphins and pontoons, between sections of open-decked structures or to gain access within structures. They should be situated so as to minimize the risk of damage by vessels and cargo handling equipment.

Walkways may be constructed in concrete, steel, timber or aluminium, and should be designed in accordance with BS EN 1992, BS EN 1993, BS EN 1995 or BS EN 1999 respectively, and normally with the loading requirements given in BS EN 1991-2:2003, Section 5, and in BS 4592-0. Open type metal flooring conforming to BS 4592 should be used to minimize the risk of standing water and ice formation where the general public has no access.

The clear width of walkways should be not less than 1.0 m. Where two-way wheelchair access is required, the minimum width should be increased to 2.0 m.

The maximum gradient of ramps should be not greater than 8%, where wheelchair use is anticipated.

NOTE 1 These recommendations are not fully in accordance with BS 8300 because of the specialized nature of the environment; nevertheless the use of BS 8300 to assess further disabled access issues is encouraged.

Where walkways are supported on flexible structures, due allowance should be made for all possible horizontal and rotational movement. Failsafe mechanisms should be incorporated in the design such that the failure of any element does not lead to the collapse of the walkway. All supports designed to allow movement to take place should be adequately protected and, where relevant, lubricated in sealed units. The design should take into account the need to maintain and replace bearings.

Walkways should be adequately illuminated. Illumination should conform to BS EN 12464-2.

NOTE 2 Guidance on illumination levels is given in CIBSE lighting guides LG1 [25] and LG6 [26], and CIE publications 129/1998 [27] and 015/E 2005 [28].

11.4 Stairways

Stairways may be provided at berthing faces for access to and from small vessels such as ferries and harbour craft, particularly where the tidal range exceeds 1.0 m. At deep-water quays, they should be located where they will not be blocked by berthed ships, preferably on return walls and otherwise at ends or changes in direction of quays. A typical arrangement is shown in Figure 40.

The type of construction will depend upon that used for the berth structure. In most cases, stairways will be of concrete, but steel might be more appropriate at skeleton-type jetty decks. Open type metal treads should conform to BS 4592.

NOTE 1 BS 8300 discourages the use of open type treads, but in the maritime environment it is acknowledged that such treads are needed to reduce hydraulic or wind forces.

Guidance on dimensions of steps is given in BS 5395-1, BS 8300 and BS EN ISO 14122-3. For maritime structures, a minimum width of 1.0 m is recommended where only port and vessel personnel will use the stairs, and 1.5 m where the general public has access.

Where the tidal range is 3 m or more, landings should ideally be provided at about 0.75 m above mean high water level, mean water level and mean low water level, and at height intervals not exceeding 3.0 m. For smaller tidal ranges, the number of landings may be reduced. Landing length should be at least the width of the stairway or 1.5 m, whichever is the lower. Where access to a stairway is provided from a vessel by gangway, the landings should be designed to accommodate the gangway and to allow safe access to and from and past the gangway.

Recessed mooring rings, or similar devices, should be provided at each landing level, at positions which will not cause mooring lines to obstruct pedestrians using the stairs.

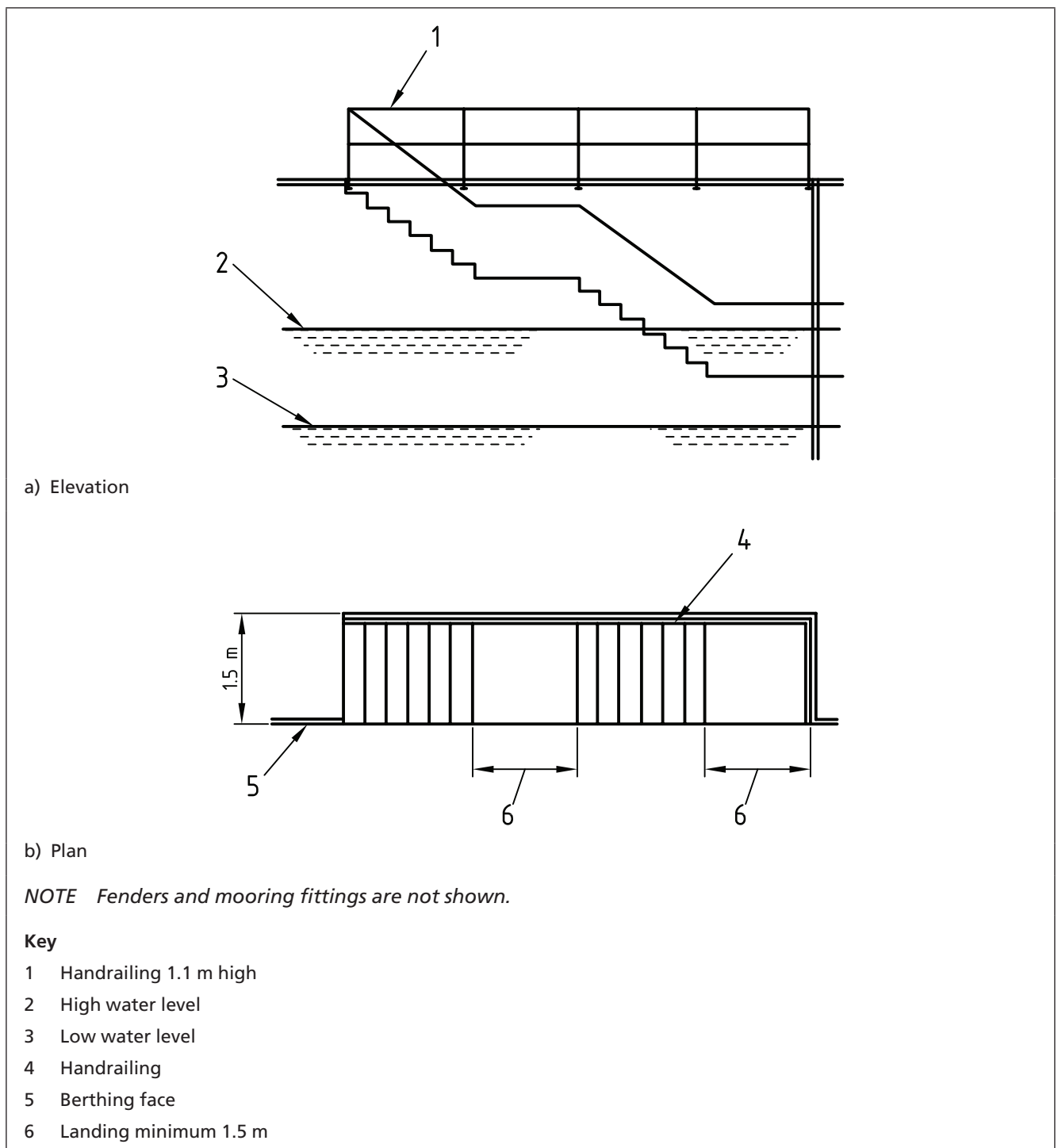
Stairways should be adequately illuminated. Illumination should conform to BS EN 12464-2.

NOTE 2 Guidance on illumination levels is given in CIBSE lighting guides LG1 [25] and LG6 [26], and CIE publications 129/1998 [27] and 015/E 2005 [28].

Fendering should be provided at the berthing face next to and below stairways. It should be arranged to prevent vessel rubbing strips being caught during both rising and falling tides.

Handrailing and wearing surfaces are covered in 11.6 and 11.8 respectively.

Figure 40 Typical stairway arrangement for quay walls



11.5 Ladders

11.5.1 General

Ladders are usually fabricated in steel, stainless steel or reinforced rubber, although aluminium may also be used. In determining the material for a ladder, account should be taken of the maintenance and design life of the ladder, and the possibility that the ladder might be used for temporary mooring of small vessels. Account should be taken of the likelihood of marine fouling, which can make the ladder slippery, and of the need for future maintenance of the ladder.

Ladders located where ice is common might need to be removable.

Ladders on berth faces should where practicable be placed in recesses to ensure a clear distance of at least 100 mm between the cope line and the rungs.

Where ladders are to be used as a primary means of access to or from vessels, measures should be taken to provide physical protection from vessels of persons using the ladder where practicable. This might require the ladder to be recessed by up to 0.6 m.

If the recess is deep enough to create a hazard, it should be surrounded with a kerb or covered with a removable lid.

If ladder recesses are not provided, fendering should be provided on or near each side of the ladder to prevent damage by small craft or ships. At ladders which might serve small craft, moorings rings or similar devices should be installed on both sides of the ladder together with suitable fendering.

Ladders might be required to provide access to various parts of a structure for inspection purposes. These should conform to 11.1 where practicable.

Ladders should always be adequately illuminated to permit location of the ladder in emergency. Where the ladder forms part of an access route for regular working, it should conform to BS EN 12464-2.

NOTE Guidance on illumination levels is given in CIBSE lighting guides LG1 [25] and LG6 [26], and CIE publications 129/1998 [27] and 015/E 2005 [28].

11.5.2 Emergency ladders

Emergency ladders should be provided for safety, allowing for a person who is in the water to climb onto a structure, or for access onto a structure.

For safety, emergency ladders should be provided at all berths and along access trestles and, in addition, at all structures where access by small craft is required. The recommended maximum spacing of emergency ladders at berth faces is 30 m: a similar spacing may be adopted on access trestles. Emergency ladders should also be installed at return walls except where stairs are provided. Ladders should be positioned where they will not be obstructed by mooring lines.

Emergency ladders should be designed in consideration of good ergonomic practice and should be in general accordance with BS 4211, except in the following respects:

- a) no safety hoop is required;
- b) stringers should not extend above cope level in any area where they might cause obstruction, for instance to mooring lines;
- c) no landings are required;
- d) the following gives minimum section sizes but consideration should be given to increased sizes where a more robust approach is required:
 - stringer thickness: 12 mm drilled and welded;
 - rungs: 25 mm diameter or square (square rungs should be mounted with the corner uppermost).

Figure 41 shows typical emergency ladder dimensions.

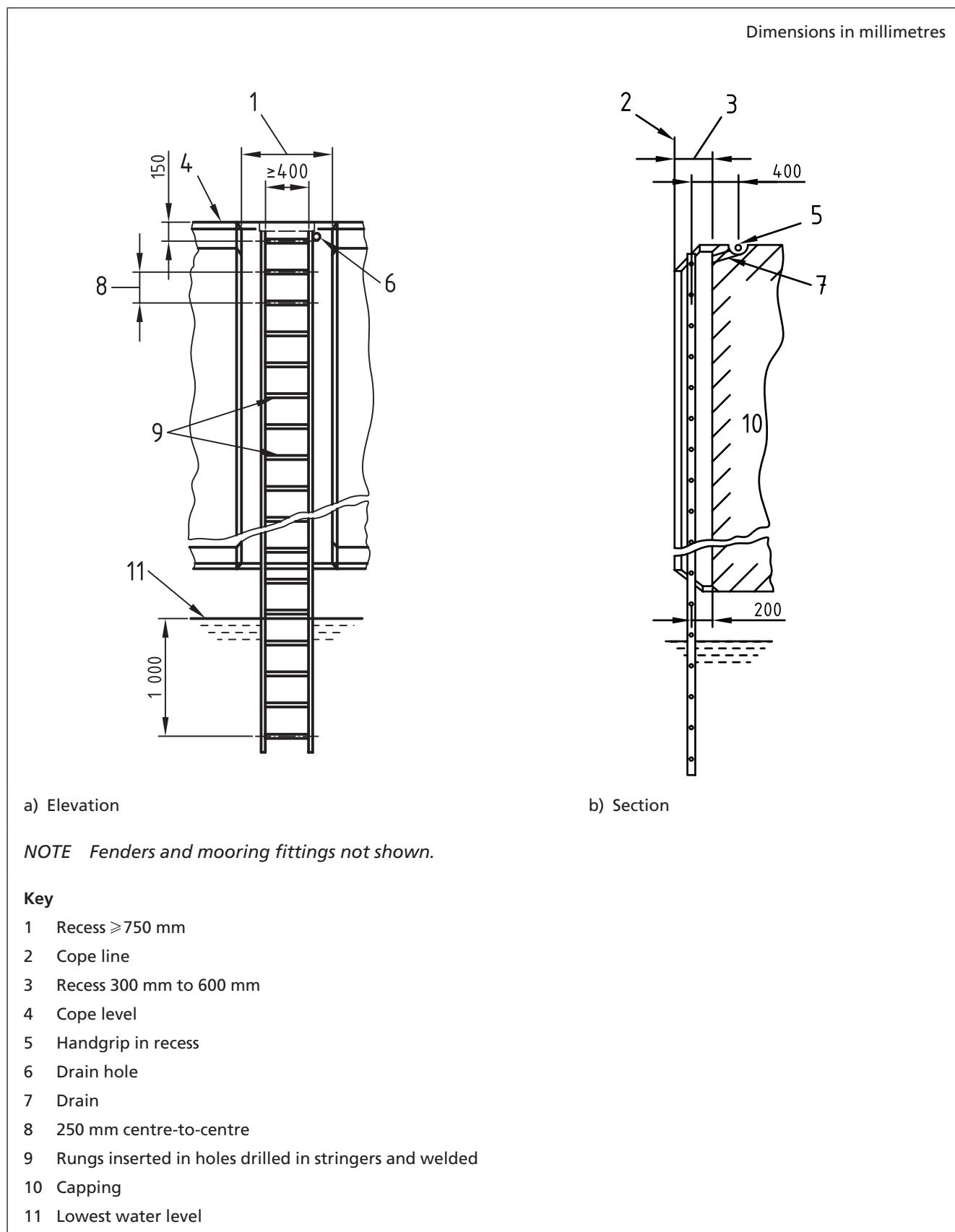
At the top of all emergency ladders with stringers that do not extend above cope level, a handgrip of the same length and section as the rungs should be fixed with its top recessed in the cope: if a kerb is provided at the cope, the handgrip may be fixed at the level of the top of the kerb. The hand grip should be located (350 ± 50) mm from the centre line of the ladder and should have a minimum clearance of 60 mm from the hand grip to any surface. Any hand grip recess should be free draining and should not allow the build-up of material that would impact on its safe use.

The top rung of emergency ladders that do not have extended stringers should be 0.15 m below cope level, and the bottom rung 1.0 m below lowest water level.

For all emergency ladders with stringers that extend above cope level, the top rung should be level with the cope.

Additional top rungs should be provided within the space between the top rung of any ladder and the cope or recess. These additional rungs should be spaced at a distance of (70 ± 5) mm.

Figure 41 Typical emergency ladder dimensions



11.6 Handrailing

Handrailing should generally be provided to the water edge of public accessed structures, to both sides of walkways, to the shoreward side of stairways, to the edges of access trestles, and around the landward edge of other marine structures where fouling of mooring ropes does not occur.

Handrailing is usually fabricated from mild steel, although aluminium may also be used. Handrailing should be designed in accordance with BS EN ISO 14122-3.

NOTE Guidance on handrail design is also given in BS 5395-1 and BS 8300.

Where members of the general public have access, mesh panels should be provided.

Handrailing should be positioned in such a way that obstruction of mooring lines being handled is kept to a minimum. On walkways, this may involve sloping the top rail downward at ends of runs. The height of the top rail should be at least 1.1 m above the walkway. An intermediate kneerail at approximately mid-height should be provided, and a continuous toe plate at least 100 mm high, incorporating drainage holes where necessary, should be installed at walkway level. The gap between the handrail and the kneerail and the toe plate should be not more than 0.5 m.

Where permanent handrailing would interfere with mooring lines, a demountable system should be employed.

11.7 Toe kerbs

Kerbs, which may be of timber or concrete, should be provided along the top perimeter of stairway recesses and, if required by the port operator, on all cope edges.

Kerbs should be at least 200 mm high. The front edge should be set back from the cope line by a distance equal to at least half the kerb height, or further if access is needed for mooring equipment. The ends of kerb runs and the top arrises of kerbs should be shaped smoothly to prevent snagging and abrasion of mooring lines. Drainage slots should be provided where required.

11.8 Wearing surfaces and durability

Steelwork used for the installations covered by this clause should generally be hot-dip galvanized for durability. If items within the splash and intertidal zones are coated, it should be by one of the methods described in BS 6349-1:2000, Clause 68. Stainless steel may be used instead of steel with protective coating.

Aluminium elements should be designed taking account of the recommendations in BS 6349-1:2000, 59.3, and BS EN 1999-1.

Where more than one metal or metallic coating is used in proximity, care should be taken to prevent bimetallic corrosion from occurring.

Concrete surfaces should have a brushed finish. Timber and steel surfaces might require treatment to ensure adequate skid resistance.

11.9 Life-saving equipment

Life-saving equipment should be installed on all maritime structures at maximum intervals of 100 m. In particular, it should be provided at jetty heads and isolated dolphins. Lifebuoys (equipped with at least 30.0 m of buoyant line) or throwing lines should be fitted at regular intervals along handrails and at other convenient points which will not obstruct or be damaged by port operations. On working quays where it is impractical to have freestanding, upright pole supports for life-saving equipment, life-saving equipment may be mounted on quayside cranes or lighting standards. In some circumstances it might be necessary to have portable equipment.

A further safety measure which should be adopted is to provide hand-holds at an intermediate distance between ladders (see 11.5). The distance between hand-holds and ladders should be not more than 15 m, and they should be available at all states of the tide. The purpose of the hand-holds is to allow a person who is in the water to gain some support until they can be rescued. They may take the form of chains, fibre ropes, rubber tyres, fenders or other suitable material suspended at the seaward face of the quayside. Hand-holds may be incorporated in the side faces of fender panels.

NOTE Further guidance is given in COP 25 [7].

Annex A (normative) Partial factors and combinations of actions

A.1 General

This annex gives recommendation for dealing with the following types of actions, which are specific to the design of maritime structures:

- wave actions, which should be considered to be variable actions in accordance with BS EN 1990;
- ship berthing actions, which should be considered to be variable actions in accordance with BS EN 1990;
- mooring actions, which should be considered to be variable actions in accordance with BS EN 1990;
- ship accidental impact actions, which should be considered to be accidental actions in accordance with BS EN 1990;
- combinations where wind loads on structures, buildings, linkspans, walkways or cranes and wave loads on maritime structures are present at the same time in a design situation. The main characteristics of these combinations are:
 - wind loads are assessed in accordance with BS EN 1991-1-4 or BS 6349-1 as appropriate;
 - wave loads are assessed based on the wave predictions given in BS 6349-1 based on the use of the maximum wave heights.

A new rule for combinations for ultimate limit state is included, using a frequent value for the variable action in the combination of the actions which is outside the normal usage demanded by BS EN 1990. (Normally only the combination value for the variable actions is used in ULS analysis.) This is to represent the low probability of a maximum wind gust and maximum wave acting on the structure at exactly the same time. The combination of wind load plus wave load in conjunction with other possible loadings is also checked as an accidental load (in order to assure adequate stability in the very unlikely event that the two extreme situations actually do coincide).

A.2 Principles

The general approach to ultimate limit state design using the Eurocodes is based on the following approach.

- a) The actions on a structure are identified.
- b) The structural form materials and the properties are decided on.
- c) A set of design action combinations is produced that are enhanced by partial factors.
- d) These are applied to the structural model, and effects in the form of internal stresses, bending moments, shear forces etc. are derived.
- e) The resistances of the structure and its various elements, in the form of bending moments at which the elements or overall structure will fail, are identified and partial factors applied to the resistances to produce a reduced capacity value.
- f) If the reduced capacity value is greater than the effects, then the structure is deemed to be sufficiently strong.

The following general procedure should be used for identifying actions and combinations of actions for maritime structures.

- 1) All the appropriate actions applied to the structures should be identified.
- 2) Design situations should be identified which include mutually compatible sets of actions.
- 3) All such combinations should include permanent actions and variable actions and may also include internal actions such as pre-stress.
- 4) Partial factors for the individual permanent actions and the variable actions should be taken from Table A.1 (see **A.3.1**).

NOTE 1 There are different sets of partial factors for different considerations, the main ones being:

- *EQU: the set for the assessment of overall equilibrium states, such as overturning stability where the strength of structural members or the ground do not govern;*
- *STR: the set governing the assessment of structural strength where the structural strength governs;*
- *GEO: the set governing the assessment of failure or excessive deformation of the ground where the soil or rock strength governs;*
- *FAT: the set for dealing with the fatigue failure of structure or structural members;*
- *UPL: the set for dealing with the loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy);*
- *HYD: the set for dealing with hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients.*

EQU, STR, GEO and FAT are defined in BS EN 1990. UPL and HYD are defined in BS EN 1997.

- 5) If the combination includes the permanent actions and only one variable action, then the partial factors listed in Table A.1 are the only factors required to derive the design actions that are used in the analysis.
- 6) If the combination includes permanent actions and more than one variable action, then one of the variable actions should be designated as the leading variable action and the others are considered to be the accompanying variable actions. The permanent actions and leading variable action should be enhanced by the partial factors listed in Table A.1, but the accompanying variable actions should be multiplied by the ψ factors, which are factors for the combination value ψ_0 , frequent value ψ_1 or the quasi-permanent value ψ_2 of the variable actions. These represent the reduced probability that the maximum effects will occur simultaneously when dealing with more than one variable action. The ψ factors are listed in Table A.2 (see **A.3.2**).
- 7) Where appropriate, the design should be assessed for a variety of design situations by considering different variable actions as the leading variable action, since this will result in a different set of structural effects. This means that a different variable will be reduced by the ψ factor in each of the different design situations.

- 8) The formulae given in Table A.3 should be used to determine the way in which the partial factors and ψ factors should be applied for combinations of permanent actions, pre-stress actions and variable actions (see **A.3.3**).

NOTE 2 This system of partial factors and ψ factors replaces the traditional approach where groups of load cases are considered separately and complete listings of partial factors are provided for each case.

A.3 Limit states

A.3.1 Partial factors for actions

The partial factors given in Table A.1 should be used as follows.

- Static equilibrium for overall global factors (i.e. not involving the strength of the structure or the ground) should be verified using the design values for EQU, Set A.
- Design of structural members not involving geotechnical actions should be verified using the design values for STR, Set B.
- Design of structural members (footings, piles, basement walls etc.) should be verified by calculating using the least favourable of the effects from STR/GEO Set B and STR/GEO Set C.
- Partial factors for the assessment of geotechnical effects due to uplift arising from water effects (buoyancy) and hydraulic gradients are not included in Table A.1 and are instead to be found in BS EN 1997.

Table A.1 Partial factors for use in BS 6349-2

Action	Symbol (from BS EN 1990)	EQU (Set A)		STR/GEO (Set B)		STR/GEO (Set C)	
		Unfavourable	Favourable	Unfavourable	Favourable	Unfavourable	Favourable
Permanent actions	$\gamma_{G,sup}$	1.05	—	1.35	—	1.0	—
Permanent actions including geotechnical actions (unfavourable)	$\gamma_{G,inf}$	—	0.95	—	1.0	—	1.0
Deck surfacing	$\gamma_{G,sup}$	1.75	0	1.75	0	1.0	1.0
Cargo loads	γ_Q	1.5	0	1.5	0	1.3	0
Ship ramp loads	γ_Q	1.2	0	1.2	0	1.2	0
Road and traffic actions (including pedestrians)	γ_Q	1.35	0	1.35	0	1.15	0
Crane loads ^{A)}	γ_Q	1.35	0	1.35	0	1.15	0
Wind loads	γ_Q	1.4	0	1.4	0	1.3	0
Temperature	γ_Q	1.3	0	1.3	0	1.3	0
Differential settlement	γ_Q	1.2	0	1.2	0	1.0	0
Snow load	γ_Q	1.4	0	1.4	0	1.3	0
Water current loads	γ_Q	1.4	0	1.4	0	1.3	0
Wave loading	γ_Q	1.4	0	1.4	0	1.3	0
Ship berthing loads	γ_Q	1.4	0	1.4	0	1.3	0
Mooring loads	γ_Q	1.4	0	1.4	0	1.3	0
Forces from ship propulsion	γ_Q	1.5	0	1.5	0	1.3	0
Earth and geotechnical water pressures	γ_Q	1.5	0	1.5	0	1.3	0
Abnormal berthing loads	γ_Q	1.2	0	1.2	0	1.2	0
Erection case	γ_Q	1.2	0	1.2	0	1.2	—
Earth and geotechnical water pressures	γ_Q	1.5	0	1.5	0	1.3	0
Pre-stress	γ_P	See BS EN 1992 or relevant Eurocode	—	See BS EN 1992 or relevant Eurocode	—	See BS EN 1992 or relevant Eurocode	—

NOTE The cases for UPL and HYD limit states are dealt with in BS EN 1997. The cases for FAT limit states are dealt with in BS EN 1993-1-9 and BS EN 1993-2.

^{A)} Alternatively, crane loads can be derived by obtaining separate information on actions due to self weight, wind and cargo handling and using the partial factors to produce a rational design situation for analysis.

A.3.2 Combination factors

The ψ factors shown in Table A.2 should be used for combinations of variable actions (see A.3.3 for formulae).

Table A.2 ψ factors for combinations

Action		Ψ_0 Factor for the combination value of a variable action	Ψ_1 Factor for the frequent value of a variable action	Ψ_2 Factor for the quasi permanent value of a variable action
General port cargo loadings etc.	Cargo loads	0.7	0.50	0.3
	Ship ramp loads	0.7	0.5	0
Traffic actions (including pedestrians)	Port vehicle loadings	0.75	0.75	0
	Tandem system ^{A)}	0.75	0.75	0
	Uniformly distributed load ^{A)}	0.75	0.75	0
	Pedestrian, cycle tracks	0.4	0.4	0
	Crane loads	0.75	0.75	0
	Unusual vehicles ^{B)}	0	0	0
Environmental loads	Wind loads	0.6	0.2	0
	Temperature	0.6	0.6	0.5
	Snow load	0.5	0.2	0
	Hydrostatic loads (excluding ground water effects)	0.7	0.5	0.3
	Water current loads	0.6	0.2	0
	Wave loading	0.6	0.2	0
Ship and mooring loads	Ship berthing loads	0.75	0.75	0
	Mooring loads	0.6	0.2	0
	Forces from ship propulsion	0.75	0.75	0
Others	Earth and ground water pressures	1.0	1.0	1.0
	Differential settlement	1.0	1.0	1.0
	Erection case	1.0	—	1.0
	Pre-stress	See BS EN 1992 or relevant Eurocode	See BS EN 1992 or relevant Eurocode	See BS EN 1992 or relevant Eurocode

A) Gr1a is a load combination that comprises the TS and UDL systems in accordance with BS EN 1991-2.

B) This assumes that special vehicles will be loading the structures in controlled conditions.

C) Hydrostatic loads for floating structures are regarded as permanent loads and therefore do not feature here.

A.3.3 Combination formulae for design situations

The formulae given in Table A.3 for ultimate limit states, accidental load limit states, seismic limit states and serviceability limit states should be used for combinations.

Table A.3 Formulae for combinations of actions

Design situation	Limit state	Permanent actions	Pre-stress	Leading accidental or seismic action	Leading variable action ^{A)}	Accompanying variable actions ^{A)}	Eq. no. ^{B)}
Ultimate limit states							
Persistent and transient	Unfavourable	$\Sigma \gamma_{Gj,sup} G_{kj,sup} + \gamma_P P$	—	—	$+ \gamma_{Q,1} Q_{k,1}$	$+ \Sigma \gamma_{Qi} \psi_{0,i} Q_{k,i} \dots$ ^{C)}	6.10
	Favourable	$\Sigma \gamma_{Gj,inf} G_{kj,inf} + \gamma_P P$	—	—	$+ \gamma_{Q,1} Q_{k,1}$	$+ \Sigma \gamma_{Qi} \psi_{0,i} Q_{k,i} \dots$ ^{C)}	6.10
Accidental	Unfavourable	Either: $\Sigma G_{kj,sup}$	$+ P$	$+ A_d$	$+ \psi_{1,1} Q_{k,1}$	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.11b
		or: $\Sigma G_{kj,sup}$	$+ P$	$+ A_d$	$+ \psi_{2,1} Q_{k,1}$	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.11b
	Favourable	Either: $\Sigma G_{kj,inf}$	$+ P$	$+ A_d$	$+ \psi_{1,1} Q_{k,1}$ ^{D)}	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.11b
		or: $\Sigma G_{kj,inf}$	$+ P$	$+ A_d$	$+ \psi_{2,1} Q_{k,1}$ ^{D)}	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.11b
Seismic	Unfavourable	$\Sigma G_{kj,sup}$	$+ P$	$+ A_{Ed}$ ^{E)}	—	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.12b
	Favourable	$\Sigma G_{kj,inf}$	$+ P$	$+ A_{Ed}$ ^{E)}	—	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.12b
Accidental construction (execution)	Unfavourable	$\Sigma G_{kj,sup}$	$+ P$	$+ A_d$	—	$+ \Sigma \psi_{2,i} Q_{c,k}$ ^{F)}	A2.2
	Favourable	$\Sigma G_{kj,inf}$	$+ P$	$+ A_d$	—	$+ \Sigma \psi_{2,i} Q_{c,k}$ ^{F)}	A2.2
Serviceability limit states							
Characteristic	Unfavourable	$\Sigma G_{kj,sup}$	$+ P$	—	$+ Q_{k,1}$	$+ \Sigma \psi_{0,i} Q_{k,i}$	6.14b
	Favourable	$\Sigma G_{kj,inf}$	$+ P$	—	$+ Q_{k,1}$	$+ \Sigma \psi_{0,i} Q_{k,i}$	6.14b
Frequent	Unfavourable	$\Sigma G_{kj,sup}$	$+ P$	—	$+ \psi_{1,1} Q_{k,1}$	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.15b
	Favourable	$\Sigma G_{kj,inf}$	$+ P$	—	$+ \psi_{1,1} Q_{k,1}$	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.15b
Quasi-permanent	Unfavourable	$\Sigma G_{kj,sup}$	$+ P$	—	—	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.16b
	Favourable	$\Sigma G_{kj,inf}$	$+ P$	—	—	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.16b

Key

+ To be combined with

Σ The combined effect of

A) Variable actions are listed in Table A.2. Any one of the variable actions in a design situation could be classified as "Leading", the others then being regarded as "Accompanying".

B) Refers to equation numbers in BS EN 1990:2002+A1:2005.

C) For the case where wind and waves are acting simultaneously on the structure for the Set A, Set B and Set C ULS cases, ψ_0 should be changed to ψ_1 to allow for the low probability that the actual maximum wave will coincide with the actual design gust levels on which wind loads in BS EN 1991-1-4 are based. An additional check using the characteristic values of both loads should be performed but treated as an accidental load case.

D) The choice between $\psi_{1,1}$ or $\psi_{2,1}$ should be related to the relevant accidental design situation.

E) $A_{Ed} = \gamma_I A_{Ek}$ where the seismic effects are as defined in BS EN 1998.

F) $Q_{c,k}$ is the characteristic value of the construction loads as defined in BS EN 1991-1-6.

Annex B (informative) Types of action

B.1 Permanent actions

The permanent action is the self-weight of the structural elements of the structure and also the self-weight of non-structural materials attached to the structure. For some design analyses it might be preferable to consider the weight of the elements in air and to treat the uplift due to hydrostatic actions separately. This can be particularly important when considering the effect of artesian water.

B.2 Non-structural material loads

The non-structural material loads are the weights of all materials forming loads on the structure that are not structural elements. They are regarded as permanent actions. Typical examples are the fill material on a relieving platform, surfacing, fixed equipment for cargo handling and quay furniture. The self-weight of large, slow-moving cranes on fixed tracks, such as container cranes, can be included in this category. In any analysis the effect of removing the non-structural material loads has to be taken into account, since it can diminish the overall stability or diminish the relieving effect on another part of the structure.

B.3 Imposed loads

Imposed loads are subdivided in BS 6349-1:2000, 47.2 into the following categories:

- a) cyclic loads, which are regarded as variable actions;
- b) impulsive loads, which are regarded as accidental actions;
- c) random loads, which are regarded as variable actions;
- d) static and long-term cyclic loads.

The first three categories are dynamic actions and it might be necessary to consider them separately to calculate the response of the structure. Cyclic and random actions are mostly environmental, but cyclic actions can be caused by vibrations from traffic or machinery.

B.4 Soil and differential water actions

Soil and differential water actions are the dominant actions affecting the stability of an earth-retaining structure. The disturbing actions are affected by the surcharge and imposed loads on the retained soil. BS EN 1997-1 gives rules for dealing with actions and the design of structures involving soil structures and pressures.

B.5 Environmental actions

Because of their long-term nature, environmental actions, such as the effects of snow, ice, temperature, current, tide and time-averaged wind, are not considered as dynamic actions.

Dynamic actions are induced by wave actions of all types, vortex shedding in steady currents and vortex shedding in wind.

Seismic actions create a specific design situation with respect to the use in the Eurocodes (see Annex A in the present part of BS 6349, and BS EN 1990:2002+A1:2005, Table A1.3).

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