

Maritime works –

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

Publishing and copyright information

The BSI copyright notice displayed in this document indicates when the document was last issued.

© BSI 2010

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.

Contents

Foreword v

1	Scope	1
2	Normative references	1
3	Terms, definitions and symbols	2
3.1	Terms and definitions	2
3.2	Symbols	4
4	General design of berth structures	5
4.1	General	5
4.2	Choice of structure	11
4.3	General site considerations	13
4.4	Durability	18
4.5	Services and equipment	19
5	Design situations for quays and jetties	22
5.1	General	22
5.2	Actions	22
5.3	Deflection control and deformations	24
6	Embedded retaining walls in quay and jetty construction	26
6.1	General	26
6.2	Suitability of embedded retaining wall structures	26
6.3	Materials for embedded retaining walls	27
6.4	Section modulus of wall	31
6.5	Actions and effects of actions on embedded retaining walls	31
6.6	Design of embedded retaining wall structures	32
6.7	Toe level	43
6.8	In-situ capping	44
6.9	Vertical actions applied to a wall	45
6.10	Berthing and mooring actions normal to the wall	47
6.11	Longitudinal imposed actions	47
6.12	Walings and tie-backs	48
6.13	Anchorage for quay walls	50
6.14	Corners	55
6.15	Earthworks	56
6.16	Construction sequence	57
6.17	Services and crane tracks	58
7	Gravity walls in quay and jetty construction	60
7.1	General	60
7.2	Types of wall	60
7.3	Design of gravity walls	60
7.4	Concrete blockwork walls	63
7.5	Precast reinforced concrete walls	68
7.6	Concrete caissons	70
7.7	Cellular sheet-piled structures	74
7.8	Double-wall sheet-piled structures	79
7.9	In-situ mass concrete walls constructed under water	82
7.10	In-situ reinforced concrete walls	83
7.11	Diaphragm walls	83
7.12	Monoliths	85
8	Suspended deck structures	88
8.1	General	88
8.2	Suitability	88
8.3	Applications	89
8.4	Types of structure	89
8.5	Overall stability	92
8.6	Earthworks at marginal quays	93
8.7	Types of bearing pile	94

8.8	Installation of piles	96
8.9	Types of superstructure	98
8.10	Design methods	100
8.11	Loading considerations	104
8.12	Design of piles	107
8.13	Construction tolerances	112
9	Dolphins	112
9.1	Types of structure	112
9.2	Actions	117
9.3	Design of flexible dolphins	118
9.4	Design of rigid dolphins	118
9.5	Mooring equipment	120
9.6	Access and working space	120
10	Roll-on/roll-off terminal ramps	121
11	Pedestrian access	121
11.1	General	121
11.2	Pontoons	121
11.3	Walkways	121
11.4	Stairways	122
11.5	Ladders	124
11.6	Handrailing	127
11.7	Toe kerbs	127
11.8	Wearing surfaces and durability	127
11.9	Life-saving equipment	128

Annexes

Annex A (normative)	Partial factors and combinations of actions	129
---------------------	---	-----

Annex B (informative)	Types of action	135
-----------------------	-----------------	-----

Bibliography	136
--------------	-----

List of figures

Figure 1 – Soldier-piled wall: typical cross-section	30
Figure 2 – Anchored embedded retaining wall structures	33
Figure 3 – Effects of flexibility of single-anchored or propped embedded retaining wall structure on earth pressure and resistance	34
Figure 4 – Active pressure distribution on single-anchored or propped embedded retaining wall structure where fill is placed before dredging	35
Figure 5 – Active pressure distribution on single-anchored or propped embedded retaining wall structure where fill is placed after dredging	36
Figure 6 – Location of anchorages in relation to failure planes in retained fill	37
Figure 7 – Embedded retaining wall with auxiliary anchorage: failure planes	38
Figure 8 – Cantilevered embedded retaining wall structure	39
Figure 9 – Simplified distribution of active earth pressure and passive earth resistance on cantilevered embedded retaining wall structure	40
Figure 10 – Embedded retaining walls with relieving platform	41
Figure 11 – Pressure distribution on embedded retaining wall in front of relieving platform	42
Figure 12 – Staggered toe elevations for sheet-piled walls	44
Figure 13 – Examples of deadman anchorages	50
Figure 14 – Examples of cantilever anchorages	52
Figure 15 – Example of a raking pile anchorage	53
Figure 16 – Examples of ground anchors	54
Figure 17 – Anchoring arrangements at corners	55
Figure 18 – Examples of crane rail foundation arrangements	59

Figure 19 – Concrete blockwork walls	64
Figure 20 – Angle of friction on virtual back of blockwork wall	67
Figure 21 – Precast reinforced concrete wall	68
Figure 22 – Concrete caisson	70
Figure 23 – Examples of joints between caissons	73
Figure 24 – Cellular sheet-piled structures	74
Figure 25 – Modes of failure of cellular sheet-piled structure	77
Figure 26 – Distribution of active earth pressure and passive earth resistance on a cellular sheet-piled structure	78
Figure 27 – Double-wall sheet-piled structure	80
Figure 28 – Distribution of active earth pressure and passive earth resistance on a double-wall sheet-piled structure	80
Figure 29 – In-situ mass concrete wall	82
Figure 30 – In-situ reinforced concrete wall	83
Figure 31 – Gravity diaphragm wall	84
Figure 32 – Monolith	86
Figure 33 – Examples of joints between monoliths	87
Figure 34 – Examples of suspended deck structures	90
Figure 35 – Access trestles	99
Figure 36 – Fender support arrangements	106
Figure 37 – Examples of pile/deck connections	108
Figure 38 – Examples of flexible dolphins	114
Figure 39 – Examples of rigid dolphins	116
Figure 40 – Typical stairway arrangement for quay walls	123
Figure 41 – Typical emergency ladder dimensions	126

List of tables

Table 1 – Operational deflection limits for maritime structures	25
Table 2 – Pile systems for suspended deck structures	89
Table A.1 – Partial factors for use in BS 6349-2	132
Table A.2 – ψ factors for combinations	133
Table A.3 – Formulae for combinations of actions	134

Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 138, an inside back cover and a back cover.

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.

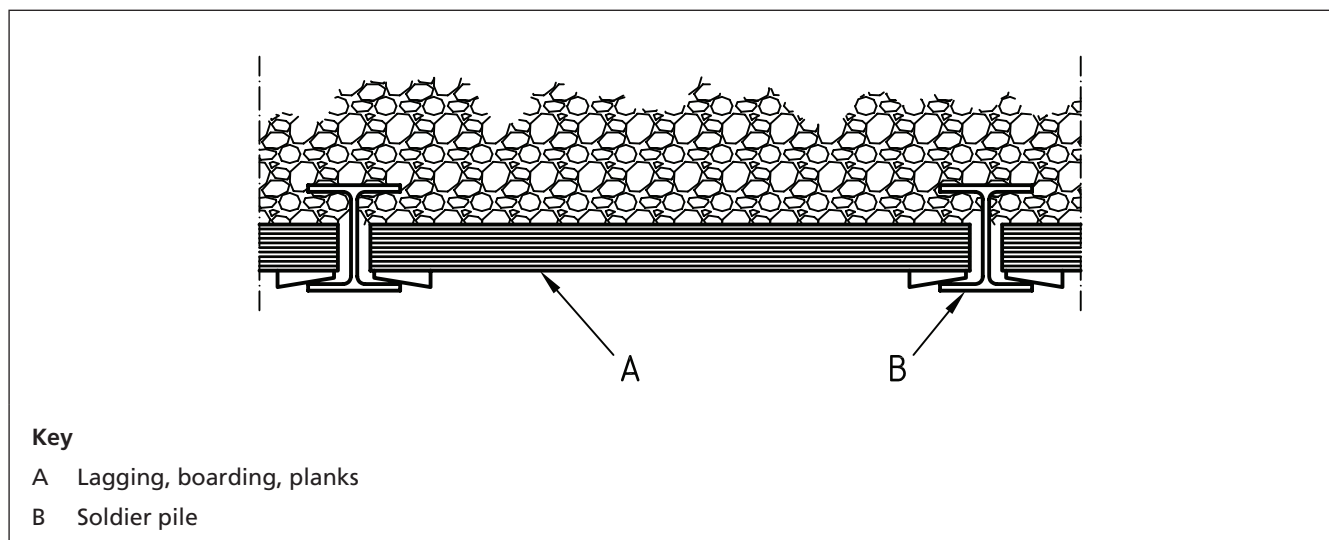
The penetration of the piles into the seabed should be calculated to take into account the consequences of failure of the ground support. The heads of all piles should be temporarily supported until the capping is cast and the anchors installed.

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

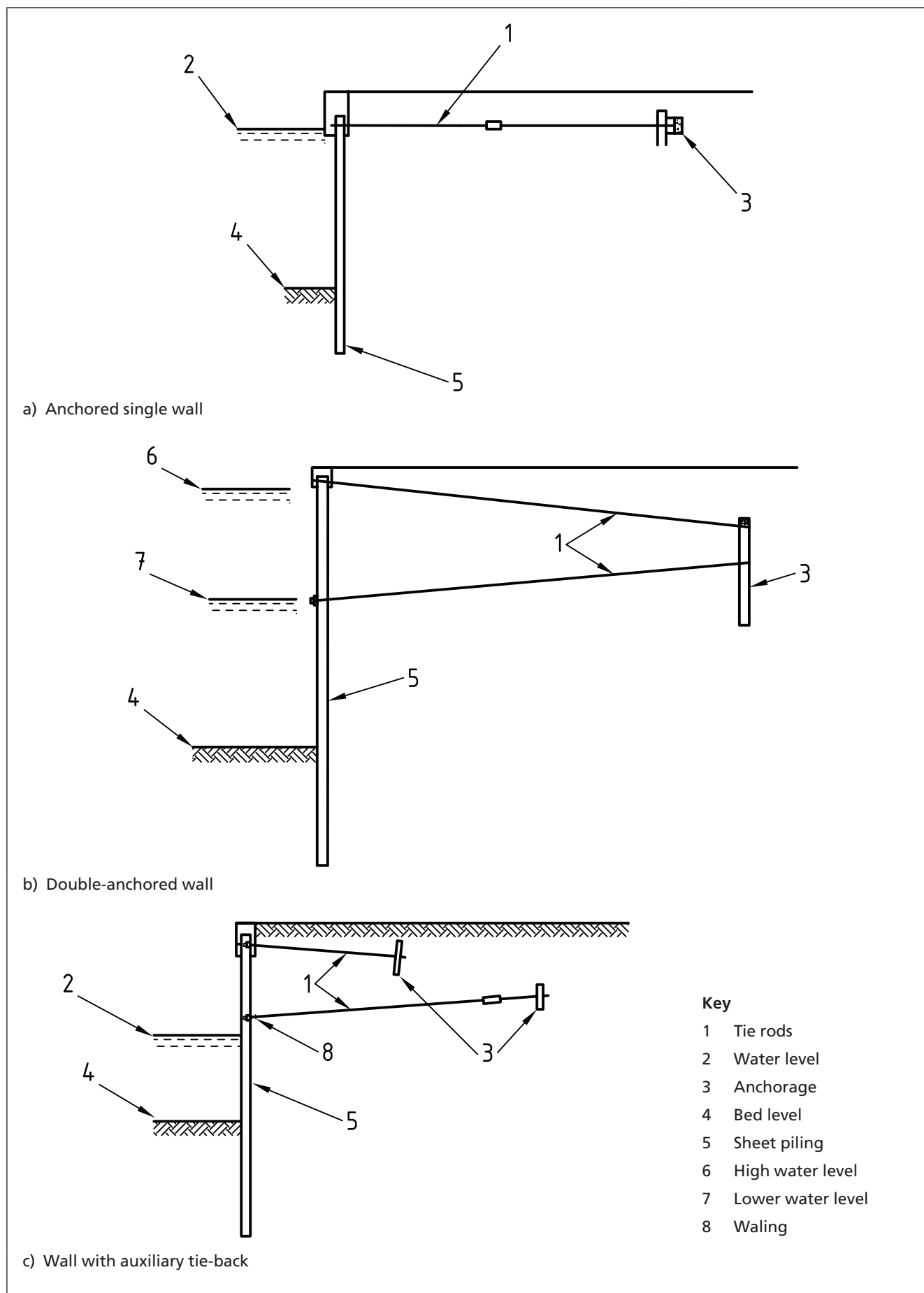
6.6.1 Anchored or propped single-wall structures

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)].

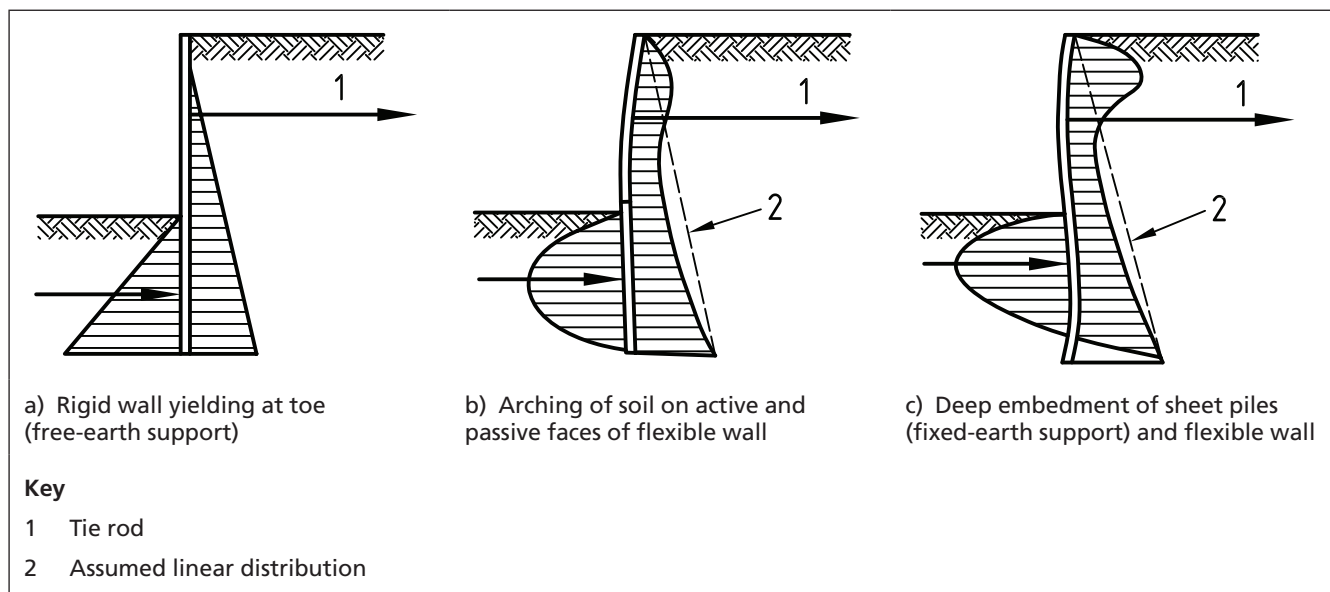
Figure 2 Anchored embedded retaining wall structures



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.

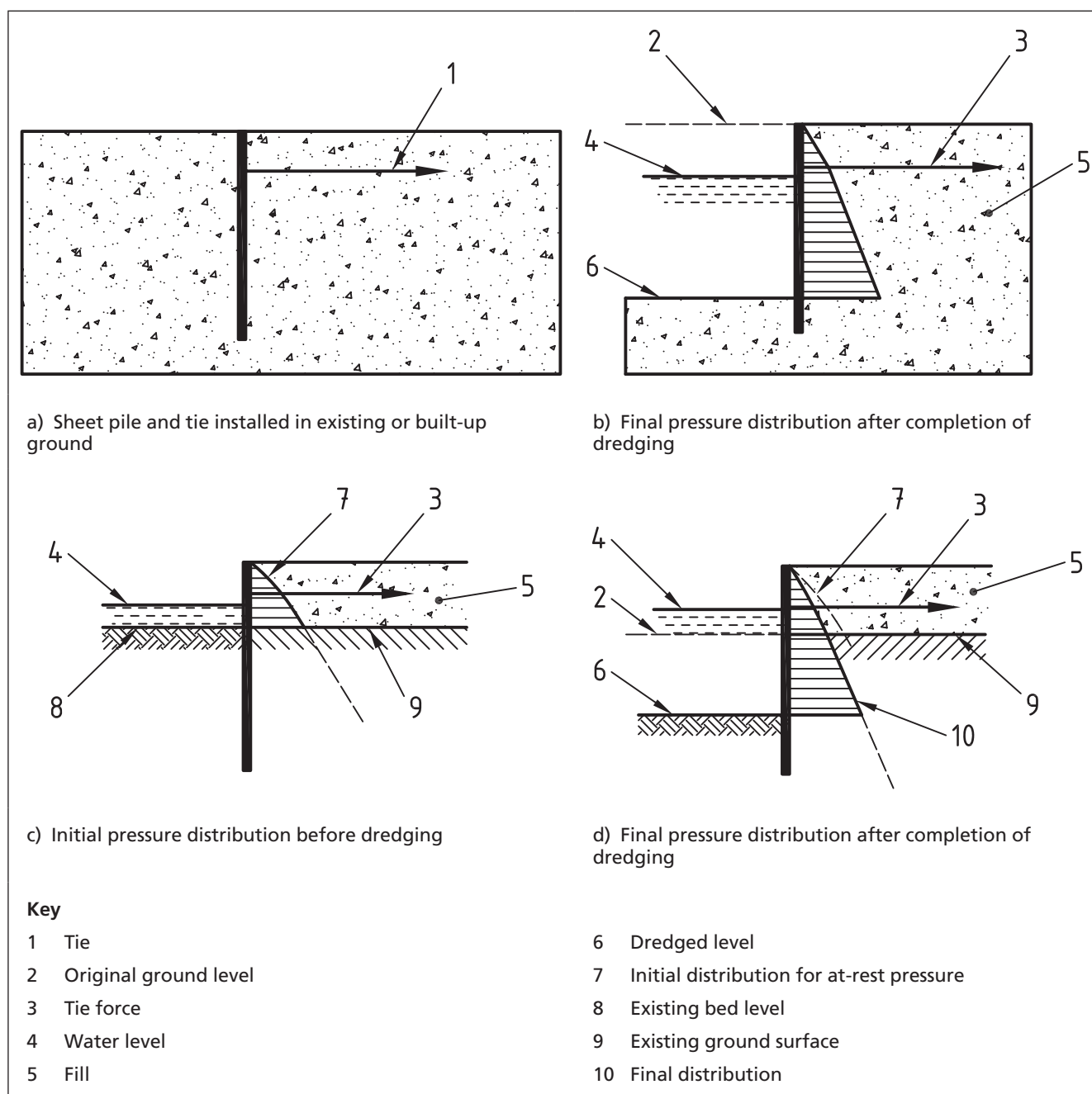
Figure 3 Effects of flexibility of single-anchored or propped embedded retaining wall structure on earth pressure and resistance



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.

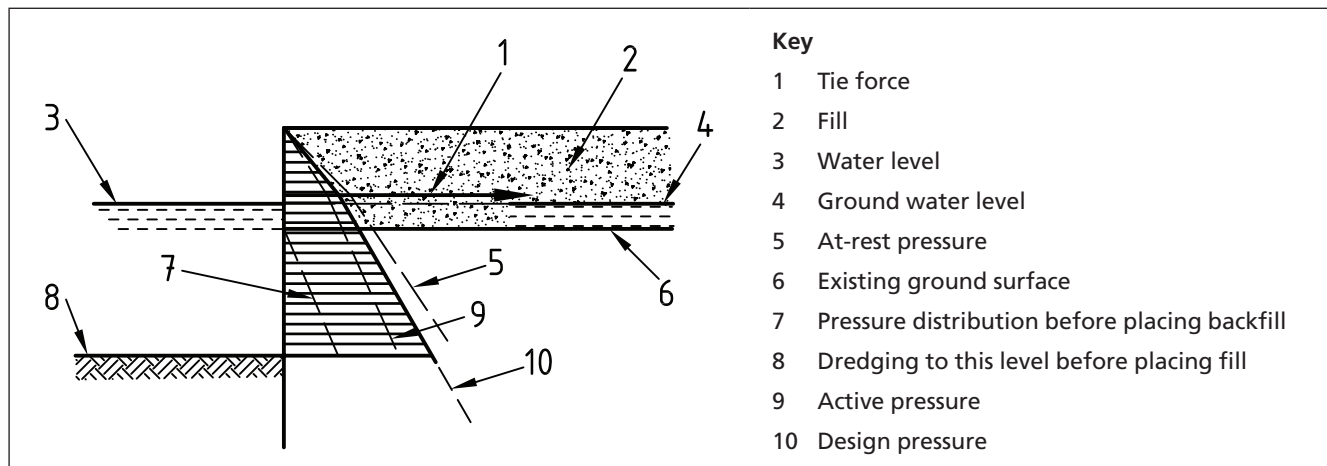
Arching conditions should be taken into account in relation to the sequence of construction of an anchored sheet-piled retaining wall. Figure 4 illustrates two scenarios where dredging takes place after the installation of the embedded wall and tie rods. Figures 4a) and 4b) indicate the situation where 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 a 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