

BS 6349-2:2019



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Maritime works

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



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Summary of pages

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Foreword

Publishing information

This part of BS 6349 is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 30 June 2019. 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This part of BS 6349 supersedes [BS 6349-2:2010](#), which is withdrawn.

Relationship with other publications

BS 6349 is published in the following parts:

- Part 1-1: *General – Code of practice for planning and design for operations*;
- Part 1-2: *General – Code of practice for assessment of actions*;
- Part 1-3: *General – Code of practice for geotechnical design*;
- Part 1-4: *General – Code of practice for materials*;
- Part 2: *Code of practice for the design of quay walls, jetties and dolphins*;
- Part 3: *Code of practice for the design of shipyards and sea locks*;
- 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

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

- reorganization of the clauses to consolidate common recommendations in [Clause 4](#);
- rationalization arising from the publication of [BS 6349-1-2](#), which now deals with the assessment of actions;
- deletion of the clause that consisted solely of a reference to [BS 6349-8](#).

This publication can be withdrawn, revised, partially superseded or superseded. Information regarding the status of this publication can be found in the Standards Catalogue on the BSI website at bsigroup.com/standards, or by contacting the Customer Services team.

Where websites and webpages have been cited, they are provided for ease of reference and are correct at the time of publication. The location of a webpage or website, or its contents, cannot be guaranteed.

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.

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

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 documents are referred to in the text in such a way that some or all of their content constitutes provisions of this document¹. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

[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 6349-1-1](#), *Maritime works — Part 1-1: General — Code of practice for planning and design for operations*

[BS 6349-1-2](#), *Maritime works — Part 1-2: General — Code of practice for assessment of actions*

[BS 6349-1-3](#), *Maritime works — Part 1-3: General — Code of practice for geotechnical design*

[BS 6349-1-4](#), *Maritime works — Part 1-4: General — Code of practice for materials*

[BS 6349-4](#), *Maritime structures — Part 4: Code of practice for design of fendering and mooring systems*

[BS 6349-5](#), *Maritime structures — Part 5: Code of practice for dredging and land reclamation*

[BS 8002](#), *Code of practice for earth retaining structures*

[BS 8004](#), *Code of practice for foundations*

[BS 8300-1](#), *Design of an accessible and inclusive built environment — Part 1: External environment — Code of practice*

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

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

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

[BS EN 1991](#) (all parts), *Eurocode 1 — Actions on structures*²

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

[BS EN 1993](#) (all parts), *Eurocode 3 — Design of steel structures*²

[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*²

[BS EN 1998](#) (all parts), *Eurocode 8 — Design of structures for earthquake resistance*²

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

² This part of BS 6349 gives dated references to BS EN 1991-1-1:2002, BS EN 1991-2:2003, BS EN 1993-1-1:2005, BS EN 1997-1:2004, and BS EN 1998-2:2005+A2:2011. It also gives an informative reference to BS EN 1993-5:2007.

[BS EN 1999](#) (all parts), *Eurocode 9 — Design of aluminium structures*

[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 12063, *Execution of special geotechnical work — Sheet pile walls*

BS EN 12464-2, *Lighting of work places — Part 2: Outdoor work places*

[BS EN ISO 14122](#) (all parts), *Safety of machinery — Permanent means of access to machinery*

BS EN ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures*

BS ISO 12488-1, *Cranes — Tolerances for wheels and travel and traversing tracks — Part 1: General*

NA to BS EN 1991-1-1:2002, *UK National Annex to Eurocode 1 — Actions on structures — Part 1-1: General actions — Densities, self-weight, imposed loads for buildings*

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 6349-1-1](#), BS EN 1990 and the following apply.

3.1.1 access 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 combined wall

retaining wall composed of primary and secondary elements that act in combination

NOTE Combined walls are commonly referred to as combi-walls.

3.1.6 cope

top edge of a quay or jetty adjacent to a berth

3.1.7 cut thread

thread formed in a cylindrical bar using a cutting tool to remove material and form troughs

NOTE The major thread diameter cannot therefore be greater than that of the parent bar.

3.1.8 dolphins

3.1.8.1 dolphin

isolated structure or strongpoint used either to manoeuvre a vessel or to facilitate holding it in position at its berth

3.1.8.2 berthing 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.

3.1.8.3 lead-in dolphin

dolphin provided at the approach to a structure or change of direction of a berth, to guide a vessel to the desired berthing line or sea lock entrance or to protect a vulnerable structure

3.1.8.4 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.

3.1.9 embedded retaining wall

retaining wall of thin cross-section that resists actions by bending

3.1.10 false deck

structural slab overlaid with fill and surfacing

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 can 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 that extends into the water area from the shore and provides a berth or berths

NOTE A jetty can be connected to the shore by an access trestle or causeway, or can be of the island type.

3.1.14 jetty head

platform at the seaward end of a jetty or causeway

3.1.15 marginal quay

quay that backs on to the shore or reclaimed land

3.1.16 quay

structure used for loading and unloading ships

3.1.17 relieving platform

platform built below deck level and supported on bearing piles, the principal function of which is to reduce lateral earth pressures over the portion of an embedded retaining wall below platform level

NOTE A relieving platform is usually supported on the retaining wall.

3.1.18 rolled thread

thread formed in a cylindrical bar using a cold forging process where the parent material is rolled through a set of dies displacing material to form the thread profile

NOTE The major thread diameter is therefore larger than that of the parent bar and the minor diameter smaller.

3.1.19 soldier-piled 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.

3.1.20 suspended deck structure

quay or jetty comprising a deck supported on bearing piles

3.1.21 upset end

end that is created by locally increasing the end diameter of a cylindrical bar by hot forging to permit threads to be formed such that the minor diameter is greater than that of the parent bar

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.

d_z	depth from ground level to intersection between the embedded wall and the lower surface of the zone included in the disturbing force, in metres (m)
F_H	horizontal soil friction action, in kilonewtons (kN)
F_V	vertical soil friction action, in kilonewtons (kN)
F_1	soil action (similarly F_2 and F_3), in kilonewtons (kN)
H_R	retained height of structure, in metres (m)
l_e	effective length of anchorage, in metres (m)
l_o	ineffective length of anchorage, in metres (m)
M_S	mass of cellular structure and contained soil, in tonnes (t)
P_A	total active soil action per unit length of wall, in kilonewtons per metre (kN/m)
P_P	passive soil action per unit length of wall, in kilonewtons per metre (kN/m)
δ	angle of wall friction, in degrees (°)
ϕ'	angle of shearing resistance in terms of effective stress (defined in BS EN 1997-1), in degrees (°)

4 General design of quay walls, jetties and dolphins

NOTE [BS 6349-1-1](#) provides recommendations and guidance concerning the overall planning and design of maritime structures, including environmental and operational matters.

4.1 General

4.1.1 Planning and risk assessment

Before a structure is designed, a planning process should be undertaken to assess and define the types of operations to be undertaken in normal and extreme operating conditions. Hazard identification (HAZID) and risk assessment should be performed to identify hazards and required mitigation measures to be incorporated into the design. The risk assessment should be used as part of the assessment of actions on structures, including any accidental actions.

NOTE 1 Additional guidance is given in HSG 177 [\[1\]](#).

For marine oil and petrochemical terminals, major hazards from accidental fire and blast and spillage should be included in the risk assessment where appropriate.

NOTE 2 Guidance on crude oil and petroleum products at terminals is given in OCIMF ISGOTT [\[2\]](#). Liquefied gas, LNG and other gaseous products are covered in SIGTTO publications Liquefied gas fire hazard management [\[3\]](#) and LNG operations in port areas [\[4\]](#). Marine oil and petrochemical terminals are covered in PIANC Working Group Report 153 [\[5\]](#).

Following the planning process and risk assessment, a design basis document should be produced, which should define, as appropriate:

- a) details of design vessels, cargo handling equipment and other relevant operational criteria;
- b) all the actions that are expected to be applied to each zone of the structure in all relevant operating conditions;
- c) combinations of the actions and whether these are dependent or independent;
- d) material properties;
- e) the strategy for operation and maintenance;
- f) the time for which it is acceptable for the berths to be out of action while maintenance operations are taking place;
- g) seismic performance requirements for both the design of the structure and temporary works required for its construction (see [4.3.10](#));
- h) requirements in respect of the area behind the berth to reflect the type and method of handling and storing the cargo, including such matters as:
 - 1) any need for the berth to be contiguous with or connected to the shore and vehicular traffic routes;
 - 2) the required apron width; and
 - 3) other specific requirements of access, cargo handling systems, piping, conveyors, utilities, drainage (including spill containment), firefighting and emergency response, safety and amenity provisions for workers;

NOTE 3 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.

- i) special measures for handling liquefied gases and dangerous goods;

NOTE 4 Attention is drawn to SIGTTO publication LGHP4 [6] and the Dangerous Goods in Harbour Areas Regulations 2016 [7].

- j) requirements in respect of the handling of cold products, including provision of fireproofing and cold splash protection (where necessary for protection of structural elements) and impoundment following spillage;

NOTE 5 Guidance is given in NFPA 58 [8], NFPA 307 [9], API STD 2510 [10] and API Publication 2218 [11].

- k) cryogenic effects such as embrittlement of concrete, steel and other materials in contact with sub-zero temperature liquefied gases, e.g. liquid natural gas (LNG) and liquid petroleum gas (LPG) (operating temperature of $-162\text{ }^{\circ}\text{C}$ and $-42\text{ }^{\circ}\text{C}$ respectively).

NOTE 6 Guidance for the design/selection of concrete and steel reinforcement materials for structures in cryogenic temperature situations is given in EEMUA publications 207 [12] and 208 [13], and ACI 376-11 [14].

The design basis document should be agreed by the developer or operator of the marine facility and should form the basis of the design and any assessment of structure performance in future.

Relevant criteria from the design basis document should be incorporated into the facility operating manual as set out in [BS 6349-1-1](#).

NOTE 7 If the risk assessment shows that a hazard can be mitigated by suitable operational controls included in the facility operating manual, then designing the structure specifically to accommodate the actions arising from that hazard might not be necessary.

4.1.2 Function of berth structure

A berth structure should be designed to accommodate the requirements given in the design basis document defined in [4.1.1](#) for a particular range of vessels and cargo handling operations, based upon the operational conditions defined by the facilities operator, including those set out in [BS 6349-1-1](#).

Dolphins should be provided where necessary for berthing and mooring, including at berths where the vessel length exceeds the length of quay, or at jetties with a loading platform. The spacing and position of fenders and mooring points for continuous and island type berths should be in accordance with [BS 6349-4](#).

NOTE 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. This is particularly applicable to bulk terminals serving oil tankers, very large crude carriers (VLCCs), LNG carriers or other bulk vessels where marginal quays are not required for cargo handling over the length of the vessel. Specific recommendations for dolphins are given in [Clause 8](#).

4.1.3 Berth location and orientation

Berth location and orientation should be selected based on a thorough and quantitative definition of prevailing meteorological and oceanographic conditions, operational requirements, and safety and environmental considerations as set out in [BS 6349-1-1](#). In particular, the berth orientation should be selected based on assessment of:

- predominant wind and wave conditions;
- current conditions, including the effect of the proposed berth structure and associated works on the existing current regime;

- sediment transport regime and potential for adverse effects of scour, either at the structure or affecting other structures, facilities or sensitive environmental receptors in the area of development;
- vessel and cargo type;
- operational philosophy, including use of tugs.

When assessing the impact that waves have at the berth location, the effects of wave height, period and direction should be taken into account, together with the vessel size and type and the specific requirements of cargo handling equipment and operations.

NOTE Assessing berth orientation involves optimization based on a number of physical and environment constraints but, in general, an orientation aligned with prevailing winds, currents and waves is the ideal. If there are unacceptable currents that cannot be avoided, it might be possible to construct training walls to modify the current regime.

The effect of the proposed structure on the movement of sediment should be assessed and the impact should be minimized. Such an assessment should include 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 transport. In addition to the effects of the structure(s) on the overall regime, the local effects should be examined to identify any unacceptable current changes, local scour or siltation of the seabed at the berth.

4.1.4 Dredging and filling

Dredging and reclamation, including environmental design, should be carried out in accordance with [BS 6349-5](#). The designer should seek to minimize the environmental impact of dredging and filling operations.

Maximum use should be made of fill material arising from other construction, excavation or dredging activities in the vicinity.

NOTE 1 Depending upon the suitability of the dredged material, the actions applied to berth structures can vary considerably. Some dredged material is inferior in quality, and a balance needs to be struck between the environmental needs and the stability of the structure. These can have a significant effect on the design of the structure.

NOTE 2 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, to reduce disruption to operations resulting from maintenance dredging.

Account should be taken of the dredging tolerances which are expected to be achieved during construction and during maintenance dredging, especially the possible depth of over-dredging that could destabilize the quay structure.

4.1.5 Berth geometry

4.1.5.1 Berth length

COMMENTARY ON 4.1.5.1

The length of berth structure that needs to be in contact with a vessel depends on the vessel characteristics and type of cargo to be handled. General cargo ships, container 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 that 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 the connection point.

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.

When determining the length of the berth, account should be taken 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 and the degree of control afforded by tugs, if any, the orientation of the berth to the prevailing wind and waves, and the degree of exposure of the berth.

NOTE Guidance on berth length and separation is given in [BS 6349-1-1](#).

Sufficient distance should be allowed between vessels to accommodate the mooring arrangements for the adjacent vessels, including the provision of sufficient mooring points for both vessels.

The berth layout should be designed to accommodate the mooring line and cargo transfer arrangements of the design vessels, where known.

4.1.5.2 Depth alongside

COMMENTARY ON 4.1.5.2

The depth of water required alongside the berth is related to the operating draught of the vessels. If shipping patterns show that ships using the berth are never at full draught, a reduced depth might be acceptable.

When determining the depth of water required alongside the berth, the following factors should be taken into account:

- any changes in draught of the vessels that might reasonably be expected over the lifetime of the berth;
- loaded draught;
- tidal range;
- effects of wave action;
- wash due to passing ships;
- wind action causing roll;
- effect upon draught of listing during loading/unloading;
- whether vessels are tidally restricted or have to manoeuvre at low water.

Allowances for any expected overdredge and underkeel clearance should be included in the design.

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

NOTE Guidance on assessing depth requirements is given in [BS 6349-1-1](#) and dredging tolerances in [BS 6349-5](#).

An assessment should be made of the likely extent of scour caused by propellers and bow-thrusters (see [4.8.1](#)). Based on this, either the design bed level used for the design of the structure should be adjusted to allow for this scour, or scour protection should be provided.

4.1.5.3 Cope and deck level

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

- the risk of flooding and the effect and implications of such flooding;

- the effects of sea level changes due to:
 - astronomical tide;
 - surge from meteorological effects;
 - long-term trends due to climate change;
- the effect of wave overtopping;
- long-term settlement of the berth structure;
- vessel characteristics including:
 - freeboard;
 - cargo loading and offloading requirements;
 - mooring and fender requirements;
 - means and locations of ship-to-shore access;
- heights and clearances required for crane and cargo handling equipment;
- heights and clearances for passenger and personal access equipment.

At cargo berths within an impounded dock, the quay level should be at least 1.5 m above the maximum impounded water level.

The level of the existing cope along the berth face may be used as a starting point for existing berths, but should not be adopted for new works without re-examining the factors above, including long-term sea level rise and the latest trends in vessel size.

For certain types of Ro-Ro berths, the cope level might need to be lower and should be set to match the requirements of the design vessels.

NOTE 1 Generally this is achieved by providing localized ramps at the specific position where ship ramps are expected to land.

NOTE 2 A low cope might be appropriate at berths exclusively used by small craft.

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 and the quantity of overtopping for events of different return periods. Elevations of such exposed areas requiring personnel or public access should be set such that overtopping quantities are consistent with safe access for the design return periods, taking into account restrictions in access that will be enforced in certain events.

NOTE 3 Where a suspended deck level is not critical to operational requirements, the height of the deck can be set such that an air gap is maintained during an extreme design event.

NOTE 4 The application of joint probability methods for computing extreme sea levels is described in Defra R&D Report SC060064 [15].

4.1.5.4 Width of apron

COMMENTARY ON 4.1.5.4

The width of berth aprons depends on the type and method of handling cargo, and is largely influenced by the use of the berth, i.e. whether it is for specialized or multi-purpose use. 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 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 back-up areas that are sufficient to provide storage for the volumes of cargo which are expected to be handled across the berth during the design working life.

The apron should be wide enough to enable:

- safe and efficient operation of cargo handling equipment;
- vehicle access and turning (including emergency vehicles);
- pedestrian access where appropriate.

4.1.5.5 Clearances at berth face

The berth structure and fendering should be arranged to minimize the risk of the structure being struck by a vessel (see [BS 6349-4](#) for guidance on clearances). The front legs of cranes, hose handling gear and other handling equipment should be set back to reduce their risk of being struck by an overhanging bow or stern or by flying bridges, and to provide pedestrian access between bollards and crane structures.

NOTE 1 The flare of container ships and naval vessels can mean that large clearances are necessary. At container terminals, it might be necessary to provide operational mitigating measures, such as to position cranes amidships during berthing to prevent them from being struck.

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 2 Some information on approach angles is given in [BS 6349-4](#).

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

Maritime structures are not normally designed to withstand the very large vertical loads that can be imposed by a ship or other large floating object being 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 that can trap or hang up a ship should be avoided. Where this is not practicable, any such horizontal projections should be provided with a flare.

4.2 Design situations

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 (BS EN 1990 to BS EN 1999).

The design situations and partial factors should be either:

- in accordance with [BS 6349-1-2](#); or
- derived from the rules for development of partial factors given in BS EN 1990, provided that a suitable statistical assessment has been made.

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. [BS 6349-1-2](#) has been developed for maritime-specific actions.

4.3 Actions

4.3.1 Self-weight and imposed actions

4.3.1.1 Self-weight

COMMENTARY ON 4.3.1.1

The distribution of self-weight loads in the structure or superstructure depends on the sequence of construction.

The self-weight loads to be resisted by each loadbearing member at each stage of construction should be calculated and the support condition of the member appraised.

For structures with relatively high vertical loads and very small horizontal loads (e.g. trestles in sheltered water), especially in deep water, an equivalent horizontal load in accordance with BS EN 1993-1-1:2005, **5.3.2(7)** should be included in the design.

4.3.1.2 Fixed permanent actions

Permanent actions from footings of fixed structures or superstructures should be allowed for in the design.

NOTE Fixed permanent actions can be assumed to disperse at 45° to the vertical from the edge of the contact area down to the neutral axis of the deck member.

4.3.1.3 Imposed loads on deck

Imposed loads (including horizontal and secondary actions) should be determined in accordance with [BS 6349-1-2](#) and included in the design of the superstructure in the appropriate loading case.

Actions imposed by Ro-Ro ramps should be taken into account where appropriate.

4.3.1.4 Maintenance actions

Account should be taken of the plant required for the maintenance of structures and any mechanical handling equipment.

4.3.2 Earth pressure actions

Earth pressure actions should be calculated in accordance with [BS 6349-1-3](#), [BS 8002](#) and [BS EN 1997](#).

4.3.3 Actions from vessels

4.3.3.1 Berthing

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

Berthing facilities for VLCCs, and LPG/LNG carriers (other than small size), should where practicable be separated from the loading platform to avoid impact loads on the platform.

NOTE 1 Potentially disproportionate damage can arise from collisions with loading platforms for petrochemical or other hazardous liquids.

Impact loads should be taken by berthing dolphins.

The type of fendering to be used should be decided at an early stage of design. The implications of reactions due to design and accidental berthing should be taken into account.

NOTE 2 Accidental berthing covers the conditions in excess of the design berthing energy, as described in [BS 6349-4](#).

4.3.3.2 Mooring

Mooring actions should be calculated in accordance with [BS 6349-1-2](#) and [BS 6349-4](#), including actions from breast, head and stern lines, and from spring lines. In addition to the variations in inclination, direction and degree of applied actions, combinations of actions on a structure should be taken into account.

NOTE 1 These can include actions applied by two vessels simultaneously.

NOTE 2 Mooring actions from vessels' lines are applied via quay fittings such as bollards, hooks, fairleads or rings fixed to the top of the deck, to the front face or on a raised platform on the deck.

NOTE 3 At exposed sites, physical or mathematical model tests might be required to determine the optimum mooring system and the actions resulting, taking into account the elasticities of the mooring lines and fenders and the response of the dolphins (see [BS 6349-1-1](#)).

4.3.4 Environmental actions

4.3.4.1 Wind

Wind actions acting directly on maritime structures should be calculated in accordance with BS EN 1991-1-4.

Wind actions on ships that are transferred to maritime structures via mooring lines should be calculated in accordance with [BS 6349-1-2](#).

4.3.4.2 Temperature

Where structures, or parts of structures, are restrained from movement, the resulting effects, and the means of resisting them, should be in accordance with [BS EN 1992](#) and [BS EN 1993](#) for concrete and steel respectively.

4.3.4.3 Ice

An assessment should be made to decide whether ice actions will be applied to the structure. Actions caused by ice at the water line should be assessed and taken into account in the design.

NOTE 1 The design calculations might need to be supported by (probabilistic) ice modelling and physical testing.

NOTE 2 Where piles are used, the geometric design can be optimized to reduce the loads.

NOTE 3 The design against local buckling might require additional wall thickness or reinforcement.

NOTE 4 Guidance on the interaction between ice and structures is given in Arctic offshore engineering [16].

4.3.5 Wave pressures on walls

Wave pressures on walls should be designed in accordance with the actions given in [BS 6349-1-2](#).

NOTE 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.

At impermeable structures with permeable backing, the effect of wave action gradually building up water levels in the backfill should be taken into account where appropriate. 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.

The presence of high ground water levels that might exist on landside of the wall due to adjacent impounding docks, flooding, springs and seepage should be included in the design conditions as appropriate.

4.3.6 Wave pressures on suspended decks

Suspended decks should either be high enough such that waves cannot strike the underside of the deck, or designed to withstand direct wave actions in accordance with [BS 6349-1-2](#).

4.3.7 Tidal lag actions

Tidal lag actions should be determined in accordance with [BS 6349-1-3](#).

4.3.8 Actions during execution

The designer should assess how the maritime structures will be constructed, and should design for actions applied by construction plant before completion of the structure where these are critical. If it is not possible to estimate or specify such actions with any accuracy when the design is prepared, they should be checked before construction commences.

Actions occurring during each stage of execution should be estimated, where possible, and the structure designed accordingly.

NOTE 1 For example, where bearing piles with temporary or incomplete decking are used to support plant, these actions can be more critical than those applied during operation even if the actions are smaller.

Actions from specific plant, including construction plant, should be allowed for in the design. Where plant could pass along the top of filled ground behind quays, a uniformly distributed load of at least 10 kN/m² should be used to allow for this.

NOTE 2 Guidance on execution actions is given in BS EN 1991-1-6.

For multi-stage construction typical of precast elements (e.g. jetty cross-heads or precast deck slabs) and tied walls, the staged construction actions 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, are exposed during construction to actions by currents, waves, wind and possibly also seismic events. The effect of these actions on piles that are not restrained or anchored during construction should be taken into account when determining the size of the piles.

An assessment should be made of the following, and mitigating measures taken where necessary:

- overall stability;
- deformations occurring during construction;
- the strength of all parts of the structure.

The nature of the fill behind retaining walls, and the extent to which the fill might or might not be fluidized during construction, should be taken into account in the design.

NOTE 3 In most cases, the selection of a well graded granular backfill material can avoid the occurrence of fluidization, although pore water pressures might be higher during construction than when the wall is complete.

The designer should provide clear guidance to the contractor as to what construction sequence and actions have been allowed for in the design. Where practicable, this information should be shown on the construction drawings.

4.3.9 Accidental actions

Account should be taken of the possible consequences of credible accidental actions, e.g. vessel impact on the berth structure, both to the structure itself and to structures alongside. The structure should be designed for accidental loads accordingly.

NOTE Where appropriate, the structure might incorporate elements designed to break off or collapse under accidental actions, or the whole structure might be designed to collapse in a way which provides additional energy absorption.

4.3.10 Seismic actions

The design should take into account the behaviour of the structure that would be expected in the event of an earthquake.

NOTE 1 Suspended deck structures with no raking piles sway during an earthquake, while blockwork and caissons displace.

NOTE 2 Flexible structures are designed to resist earthquake actions by bending of the piles. The use of raked piles for structures that have a high probability of being subject to earthquake actions is not recommended, because experience has shown that the rigidity of such structures generates very high local stresses, which often lead to failure.

The design of maritime structures to resist seismic actions should be in accordance with [BS 6349-1-2](#).

Design for seismic actions should take into account both the design low and high water level cases for quay walls, as the added mass load at high water can be significant.

In high seismic zones, temporary works should allow for reduced earthquake loading based on the construction exposure period. The reduced return periods should be calculated using BS EN 1998-2:2005+A2, Annex A.

NOTE 3 ASCE/SEI 37-14 [[17](#)] can be used in countries that do not have their own national code.

4.4 Deflection control and deformations

COMMENTARY ON 4.4

Deflections in maritime structures during the design life of the facility can be large compared with structures on the land.

The design should limit the long-term movements of the structure such that the geometry of the quay wall remains suitable for the berthing of ships and conforms to other operational requirements.

For jetties and platforms carrying normal highway vehicles or normal cargo handling operations, the operational deflections should be controlled in accordance with [Table 1](#), in addition to the normal structural limits defined in the following standards where applicable:

- BS EN 1992-1-1 for deflections of concrete structures;
- BS EN 1993-1-1 for deflections of steel structures (limits are given in the National Annex);

NOTE 1 BS EN 1992-1-1 and NA to BS EN 1993-1-1 make reference to a number of issues such as glazing and plaster that are usually not relevant to maritime works.

- BS EN 1993-2 for deck plates and longitudinal girders;
- BS EN 1993-6 for deflections for crane rails;

NOTE 2 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.

Table 1 — Operational deflection limits for maritime structures other than flexible dolphins

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 (application for bridges)	—

The fixing arrangements should take into account temperature and settlement actions. Deflections of flexible pile dolphins should be limited to those that are compatible with operational and safety considerations.

For trestle jetties carrying sensitive items such as petrochemical or similar pipelines, the restraint, structural capacity and operational performance of the items should be taken into account when deciding on the limits of acceptable deflections.

NOTE 3 Pipelines can be articulated both along their length and between trestle and shoreline such that unnecessary effects do not result.

Differential deflections of cross-heads supporting such pipelines should be such 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 overstressing and rupture.

NOTE 4 These deflections can arise from elastic movements of the trestles due to loading changes, lateral wind loads, seismic and temperature distortions.

NOTE 5 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.

4.5 Choice of structure

4.5.1 General

For any structure, a variety of different types of construction should be compared and a choice made on the basis of:

- the structure's functional requirements;
- technical feasibility of the concept;
- safe operation;
- constructability;
- capital and whole life costs;
- operational efficiency;
- maintenance;
- potential environmental effects.

NOTE The typical designs given in this part of BS 6349 are not intended to exclude the use of alternative structural arrangements, or hybrids of two or more types of structure.

4.5.2 Types of structure

COMMENTARY ON 4.5.2

Maritime structures can be either solid or open-piled. An open-piled structure can be either rigid or flexible. Solid structures include all embedded and gravity walls with a solid vertical berthing face, and are most commonly used for quays where fill material has to be retained, but are also used for jetties and dolphins.

Open structures have a suspended deck supported on piles. Flexible structures can be designed with only vertical piles and without external horizontal restraint. Rigid structures can be designed with raking piles, bracing or with struts to the shore. The degree of flexibility depends on the overall configuration, framing and relative stiffness of members and their supports. A flexible type of structure might be unsuitable to accommodate cranes or solid bulk handling equipment.

When selecting the type of structure to be adopted for a new quay, account should be taken of the effect of the movements of the structure on:

- fixed superstructures such as bridges 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;
- other structures in close proximity.

The following factors should be taken into account when deciding between open and solid structures.

- Solid structures:
 - can usefully retain backfill;
 - reflect waves causing greater disturbance at the berth and within the harbour;
 - obstruct current flow;
 - create a greater scour potential as reflected currents cause greater erosion;
 - eliminate cross-currents at the berth;
 - are suitable for marginal quays;
 - provide rigidity, often required for mechanical handling equipment.
- Open structures:
 - require a wall or revetment where needed to retain backfill;
 - attenuate short period waves if backed by a sloping revetment, causing less disturbance at the berth and within the harbour;
 - allow free current flow;
 - experience less scour potential as current reflection is reduced;
 - allow cross-currents at open berths;
 - are suitable for jetties and offshore loading structures;
 - are more vulnerable to the effects of fire when used for flammable liquids;
 - are more vulnerable to the effects of sub-zero temperatures from chilled liquefied gases.

The design of quay structures that retain the ground should allow for movement during or after construction due to the mobilization of earth pressures.

NOTE These movements are in addition to natural settlements and depend on the type of structure and the ground conditions.

Loading platforms and trestle structures for liquid cargo berths should be designed to meet the limiting values for the differential displacement of the piping systems.

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

4.5.3 Seabed conditions

The geotechnical and bathymetric survey data should be assessed together to determine suitable founding levels for different types of structure and to determine the extent to which dredging is necessary.

Environmental design for dredging and reclamation should be in accordance with [BS 6349-5](#).

4.5.4 Local construction materials

A study of the locally available natural materials for construction should be carried out, taking into account the quality, quantity and environmental aspects. The choice of structure should take into account this study, and where appropriate should use suitable locally available material.

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

4.5.5 Method of construction

The method and sequence of construction, and the availability of major or specialized construction plant, should be taken into account when determining the type of structure.

NOTE When construction requires specialized plant, the same plant might be required to carry out remedial works.

When selecting the method of construction, the designer should aim to minimize the extent of offshore, subsea and diving work, and to improve the safety and efficiency of the operation, by adopting as many of the following approaches as are applicable:

- constructing in the dry;
- construction from land to sea using a progressively constructed bund or jetty structure;
- maximizing the use of precast elements;
- modularization of the structural elements;
- minimizing the extent and amount of offshore in-situ concrete works;
- using jack-up and heavy lift plant.

The method and plant should be selected to take into account:

- the foundation and seabed conditions;
- the sea conditions expected during the construction period;
- the land availability for temporary and permanent works;
- environmental, planning and other non-constructional constraints.

4.5.6 Constructability

The choice of structure should make allowance for the fact that the structure is often constructed over water from temporary working platforms, as fast currents and wave action increase the difficulty of accurate construction.

The detailing of structures constructed over water should allow for larger construction tolerances compared to construction on land.

NOTE 1 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 construction tolerances of the piles.

The designer should where practicable avoid temporary slopes on which soft material can build up as a result of deposition of material in suspension.

When precast structures and structural elements are to be placed on a prepared bedding layer on the seabed, the structure foundation should, as far as possible, be designed to minimize and simplify the bed levelling in order to avoid or at least minimize the need for divers' work.

NOTE 2 Underwater excavation can leave an undulating or stepped surface on which a layer of fine material can be deposited. Where a trench is dredged to remove soft materials in the vicinity of the quay structure, this is particularly prone to soft materials being deposited in the trench. If not removed, this can form a potential failure plane under foundations. A layer of gravel or rubble can be placed on the seabed prior to filling to prevent the formation of such a failure plane below fill placed on the surface layer.

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

4.6 Construction tolerances

COMMENTARY ON 4.6

The construction tolerances normal in building work on land often cannot be achieved in maritime conditions. For instance, 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.

4.6.1 Piles

4.6.1.1 Pile head

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

NOTE The ICE specification for piling and embedded retaining walls [18] gives further guidance on tolerances.

4.6.1.2 Pile centreline

The inclination of vertical piles should not exceed 1:100 from the vertical in sheltered waters and 1:75 at exposed sites.

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

4.6.1.3 Precast deck units

Precast deck units to be placed on piles should be designed to accommodate pile tolerances.

Where precast units are placed upon other precast units, the detailing of connecting reinforcement should allow for fixing tolerances to minimize the risk of clashing.

4.6.2 Embedded walls

Construction tolerances for embedded walls should be in accordance with BS EN 12063 where applicable.

NOTE The ICE specification for piling and embedded retaining walls [18] gives further guidance on tolerances for embedded walls.

4.6.3 Secant piled walls

For secant piled walls, the verticality tolerance in the direction of the wall should generally not exceed 1 in 200.

NOTE The ICE specification for piling and embedded retaining walls [18] gives further guidance on tolerances for secant piled walls.

4.6.4 Diaphragm walls

Tolerances for diaphragm walls should be in accordance with BS EN 1538 where applicable.

NOTE The ICE specification for piling and embedded retaining walls [18] gives further guidance on tolerances and joints between panels.

4.6.5 Blockwork walls

The tolerance on block casting for blockwork walls should be ± 3 mm on all faces from the specified shape.

Where practicable, blocks should be placed in plan to within a maximum of ± 25 mm of the specified position. The tolerance of the level of each block should be determined according to the placing pattern and whether an in-situ or a precast capping beam is used. For bonded blocks, the differential vertical tolerance between adjacent blocks should be not greater than ± 25 mm.

NOTE For column blockwork and sliced blockwork, such a vertical tolerance is not required, as these forms of wall can allow settlement.

4.6.6 Caissons

The gaps between caissons should be determined by the sea state and weather window. Suitable provision should be made for sealing the gaps.

NOTE Caissons in sheltered water can be placed with gaps as tight as 150 mm, whereas in exposed ocean conditions, gaps up to and exceeding 1 000 mm are more practical.

Caissons should be placed to a vertical tolerance of 1:100.

4.6.7 Monoliths

Tolerances for monoliths (see 6.10) should be defined as part of the design, taking into account constructability.

NOTE Typical tolerances for monoliths are ± 0.3 m for position on plan and 1:100 for verticality.

4.6.8 Crane rails

Crane rails should be installed in accordance with the tolerances set out in BS ISO 12488-1 or as specified by the developer or operator.

4.6.9 Dredging

The design should take into account the tolerance on the dredged level given in BS 6349-5.

4.7 Environmental impact

COMMENTARY ON 4.7

The construction of maritime structures has the potential to cause environmental impact. This could include pollution from suspended solids as part of a dredging operation, or a change in water quality following the construction of a quay wall.

The environmental impacts and required control measures would normally be identified as part of the environmental impact assessment.

The design should minimize and mitigate adverse environmental impacts. In addition, the design should where practicable maximize any potential benefits by working with nature. Appropriate environmental controls should be identified at the design stage and implemented during construction.

4.8 Control of external water effects

4.8.1 Scour protection

An assessment should be made as to whether scour caused by the effects of ships' propellers and bow-thrusters, waves and currents could occur and be detrimental to the structure.

Scour protection should be designed to prevent erosion that would otherwise undermine or destabilize quay structures.

For gravity structures placed on a prepared bed, the design should take account of the potential for scour of the bedding material by currents, wave action and from vessels especially along the front and corners of structures.

For slopes under open-piled structures, scour protection should be provided where necessary to protect the stability of the slopes from wave, current and propeller wash actions.

NOTE 1 This might be in the form of rock filter and armour layers or mattresses. Where some risk of rotational failure of the slope exists, a toe trench filled with rock or sand might be necessary to enhance the slope stability.

NOTE 2 After driving the piles, armouring can be placed over the seaward slope, provided that care is taken to avoid damage to the piles by bending or abrasion.

The size of rock protection for underwater slopes should be not less than that needed to resist the wash of propellers and bow-thrusters. A filter between the rubble and the underlying material should be provided as necessary.

Account should be taken of the effect on the structure's durability caused by natural bed material and small rocks being disturbed by bow-thrusters and thrown against the structure.

NOTE 3 Guidance on the extent of scour protection required and its design can be found in PIANC Report 180 [19]. Further guidance on scour protection is given in the CIRIA rock manual [20], which includes some guidance on the assessment of the effect of jet propeller wash.

4.8.2 Impounded systems

COMMENTARY ON 4.8.2

At locations subject to a large tidal range, an enclosed system with access through locks can be provided in order to reduce the water level variations at the quay. The level can either be maintained above outside high water levels, by means of pumping, or can 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.

For structures within impounded systems, 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.

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) of the sea or river outside the impounded system.

NOTE 1 The water level within an enclosed system can be reduced below the lowest probable normal water level due to deliberate drawdown, accidental damage to the lock gate, sabotage or abandonment. The probability of an accidental drawdown occurring depends on the number of gates at the site, whether spare gates are available and, if so, how readily the gates can be brought into service, and on the plan water area of the system in relation to the dimensions of the gates.

NOTE 2 Guidance on lock gates is given in [BS 6349-3](#).

In assessing the risk of drawdown that can be accepted, the consequences of drawdown should be taken into account, including 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.

NOTE 3 Pore water pressure conditions resulting from drawdown depend on the potential rate of drawdown (based on the plan surface area of the enclosed basin relative to the possible rate of outflow) and the soil drainage conditions.

4.8.3 Surface water drainage

The upper surfaces of all quay aprons and jetties should be designed to allow rainwater and spray to drain away. Where the upper surface is constructed with a crossfall towards the edge of the structure, the crossfall should be typically between 1:60 and 1:100, depending on the 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 should be directed to gullies or channels provided in the apron. These should be connected to oil interceptors and/or silt traps as appropriate.

NOTE Kerbed areas are often 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 slots for crane and train rail tracks, and to all ducts, trenches, galleries and connection boxes for services.

4.9 Specific recommendations for quay walls

4.9.1 Settlement

At sites where highly compressible materials exist, the effects of quay wall construction, piling, dredging, filling and future transient loads on the reclaimed ground should be 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.

Any settlement that could occur as a result of the construction of the wall and any backfilling or reclamation should be taken into account in the design of the wall. This should include the potential for self-weight settlement within the fill, and settlement of the underlying materials.

The effects of long-term settlement on port operations should be taken into account.

NOTE In general, only small amounts of settlement are acceptable in quays and jetties, unless measures are taken to minimize the effect of large settlements on operations when the facility is commissioned.

4.9.2 Fill material

Where practicable, fill placed behind quay walls and in front of and behind deadman anchorages should be granular material capable of free drainage. Less permeable fill may be used where the use of such material has been assessed and taken into account in the design. Loss of material through joints in the wall should be prevented by providing a suitable filter behind the wall or by sizing the fill material such that it will not pass through the joints.

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

The need to replace weak soils should be assessed in order to minimize vertical and horizontal movements that might affect the port operations or the stability of the structures.

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

Soil replacement below final seabed level should take place before quay wall construction and piling commences.

All soil replacement at in-situ concrete walls should be completed in advance of wall construction.

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

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

NOTE 3 Guidance on fill materials is given in [BS 6349-1-3](#) and [BS 6349-5](#).

4.9.3 Drainage and filters

COMMENTARY ON 4.9.3

Drainage is often required behind quay walls to minimize tidal lag (see [BS 6349-1-3](#)). This has traditionally taken the form of weepholes or openings with flap-valves. Some types of quay wall allow drainage through the wall at open joints.

Where a drainage system is provided, the detailing of the quay wall should be compatible with the assumptions regarding tidal lag used in the design. Where a drainage system is not provided, the wall should be designed to resist the hydrostatic water pressure as described in [BS 6349-1-3](#).

Drainage through the wall should be provided with suitable filters to prevent loss of fill material.

Where drainage of fill behind quay walls is required, openings with flap-valves should be provided in preference to weepholes, to avoid flow of water into the fill behind the wall. The valves should be fixed just above low water level to allow maintenance without the use of divers.

Where the use of weepholes is unavoidable, the design should take into account the possibility of blockage by marine growth or by migrating sand/fill material fines.

Drainage should be provided to the fill above relieving platform slabs where inundation of the surfacing might occur.

NOTE 1 Drainage of backfill may be achieved using pipework, gravel drains, drain fabric or a rockfill backfill. Well graded rockfill is often used behind mass concrete walls to reduce lateral pressures on the wall.

Where rockfill or similar is used behind a quay wall, a filter should be provided between the rockfill and the general fill to prevent the loss of general fill leaching into the rockfill or through the wall.

NOTE 2 Guidance on the design of filters is given in the CIRIA rock manual [20].

The design of the drainage system should allow for future maintenance. Appropriate measures should be provided to allow for monitoring the effectiveness of the drainage system following construction, to verify that the design assumptions for tidal lag are not being exceeded.

Relieving platforms and false decks should have surfacing constructed with cross-falls to minimize the quantity of rainwater entering the fill. As a precaution, drainage holes should be incorporated in the concrete at the underside of the fill.

NOTE 3 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.

NOTE 4 A suitable means of measuring tidal lag is to install a series of piezometers.

4.9.4 In-situ capping

COMMENTARY ON 4.9.4

Embedded retaining walls or quays and blockwork gravity walls are usually provided with an in-situ concrete capping. This serves as a cope and may be used as a fender support. The capping may also be used to distribute vertical and horizontal imposed loads, to support crane rails or to function as a waling. However, for sheet piled retaining walls, timber and steel capping beams can also be used in certain circumstances.

4.9.4.1 General

The seaward face of the capping should be dimensioned in relation to the face of the wall to give the necessary clearances (see 4.1.5.5). Where necessary, reinforcement in accordance with BS EN 1992 should be provided to resist shrinkage and thermal stresses in the beam resulting from the restraint imposed by the wall and the soil.

Movement joints should generally be provided at 15 m to 30 m intervals.

NOTE The capping may be of composite construction, in which a precast concrete facing unit is provided on the seaward face and the remainder is cast in situ. This method has the advantages of ensuring a good quality, durable finish on the exposed face, and of avoiding the need for formwork over water.

Precast units should be designed to allow for irregularities in the wall, especially in the case of troughed steel sheet piles.

4.9.4.2 Specific recommendations for sheet pile walls

In-situ capping should generally be at least 0.5 m high, the actual height depending on the wall cut-off level in relation to cope level and on fendering arrangements.

The capping should be provided with lateral reinforcement above the sheet piles. Where troughed steel piles are used and where vertical actions are to be resisted, the transfer of vertical actions to the sheet piles should be checked.

Links should be provided to each part of the capping section.

NOTE 1 This might require passing bars through holes in the piles or welding them to the piles.

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

Cracking in the capping near the joints of sheet pile walls can be minimized by debonding the capping and the wall for a short distance each side of the joint. The capping should be provided with suitable means to transmit actions across movement joints where this is required.

For cappings built on sheet pile walls, a minimum pile embedment of 75 mm should be used.

NOTE 2 Where the capping or superstructure is stiff in relation to the wall, the bending stresses that would be set up in the wall when a rigid connection is used may be eliminated by means of a hinged joint between wall and capping.

Where a hinged joint between the wall and capping is used, this joint should be designed such that all horizontal and vertical actions are effectively transmitted and that soil is unable to be lost through the joint.

Where separate bearing piles are provided to resist vertical actions, these should support the capping.

4.10 Durability

COMMENTARY ON 4.10

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 6349-1-4](#).

The design working life of a structure is chosen to represent its intended useful life. Guidance on the design working life is given in [BS 6349-1-1](#).

4.10.1 Materials

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

4.10.2 Detailing

Simplicity of construction and robustness of design should generally be adopted in preference 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 such that the concrete can be easily placed and effectively compacted.

For reinforced concrete work in the tidal zone, precast units should be used where practicable, with the minimum of in-situ concrete connections.

4.10.3 Maintenance

COMMENTARY ON 4.10.3

Because most parts of a maritime structure are underwater or underground, maintenance inspections are difficult. Intertidal zones are exposed for short periods of time and inspection is impeded by marine growth and other deposits on the structure. While underwater zones are more safely inspected by remotely operated vehicles (ROVs), limited visibility and ROV navigation limitations might require inspection by a diving team. Such inspections are often challenging due to the presence of obstructions, piles, currents, waves, marine growth, limited visibility and problems of communication between the diver and the engineer. Inspection of areas above the tidal zone, such as the soffit of a suspended slab, can be difficult with limited headroom and rock armour slopes even if accessible from a boat or floating platform at low tide.

Guidance on inspection of marine structures is given in ASCE Manual of Practice No. 130 [\[21\]](#).

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 [\[22\]](#), PIANC Report 153 [\[5\]](#), PIANC Report 119 [\[23\]](#) and PIANC Report 17 [\[24\]](#).

Attention is drawn to The Diving at Work Regulations 1997 [25] and HSE L104 [26].

The design for the durability of a structure should take account of the effects of the operation and maintenance strategy (see [4.1.1](#)), including the frequency of inspection and maintenance.

The design should be appropriate to the maintenance capability of the operator. Where a marine maintenance capability is absent, the design should assume that there is no regular maintenance of the structure, except for those areas easily accessible from land.

4.11 Services and equipment

4.11.1 Functional arrangements

Arrangements for the provision of services and equipment on the berth should be finalized at the earliest possible stage in the design. The berth structure and geometry should take account of the support system for gantry cranes and other mechanical handling plant. The tolerances of crane rails and gauge should be selected such that short-term and long-term quay and rail movements are within the operational limits of the cranes.

Differential vertical movement at structural joints may be accommodated by providing a short transition support to rails. In critical cases, provision for future adjustment of rail levels should be built into the rail track system. The depth required for flush fixing of the rail track in the deck should be taken into account when choosing between different forms of construction. Crane rail tracks should be detailed in accordance with [BS 6349-1-4](#).

Trenches for power cables to heavy equipment should be provided where necessary.

Where practicable, services should be incorporated within the body of the structure to avoid exposure to the environment under the deck. If underslung services are provided, they should be fixed to resist wave and current actions as applicable.

Access should be provided for installation, maintenance and replacing services.

Where services have to cross, the cross-over points and outlet points should be accommodated in the structural design.

NOTE 1 Where there are a number of services, it is probable that the service lines will have to cross, and the depth required can control the designed section depth.

NOTE 2 A services gallery, with full height head-room for access, is often an acceptable solution.

4.11.2 Services

The design should make provision for utilities and services as required for the particular berth usage.

NOTE These could include:

- *bunker water;*
- *fire-fighting water – usually it is preferable to provide this set back from the quay edge as this is safer for fighting fires on ships;*
- *fuel supplies with hazard risk:*
 - *bunker fuel oil;*
 - *diesel;*
 - *jet fuel – military/naval;*
 - *LNG;*
 - *future fuels;*
- *shore power – often with a very high demand;*

- *communications – increasingly replaced by wireless systems.*

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.

Ducts for rail-mounted crane power supplies should be constructed integrally with the crane rail beam where there is any possibility of differential movement between beam and fill.

4.11.3 Equipment

The design should make provision for facilities as necessary for effective and safe operation of the berth.

NOTE 1 Facilities that might be needed include:

- *mooring equipment*
- *fendering;*
- *cranes and mechanical handling equipment;*
- *crane or railway tracks/wheel stops or buffers/storm anchors;*
- *lighting;*
- *navigation lights;*
- *fire monitoring and fire-fighting equipment (foam and/or water);*
- *mooring rings and/or posts for small craft;*
- *safety railing/kerbs;*
- *access routes, stairways (see also 9.4) and gangways, including hydraulic gangway towers where appropriate;*
- *inspection and access openings;*
- *cathodic protection (impressed current transformers);*
- *vessel approach aids;*
- *ladders;*
- *safety chains;*
- *lifesaving equipment.*

NOTE 2 Minimum equipment requirements for berth structures are given in HSE L148 [27], and in Safety and health in ports [28]. For oil and gas facilities, requirements for mooring equipment are given in the OCIMF Mooring equipment guidelines [29].

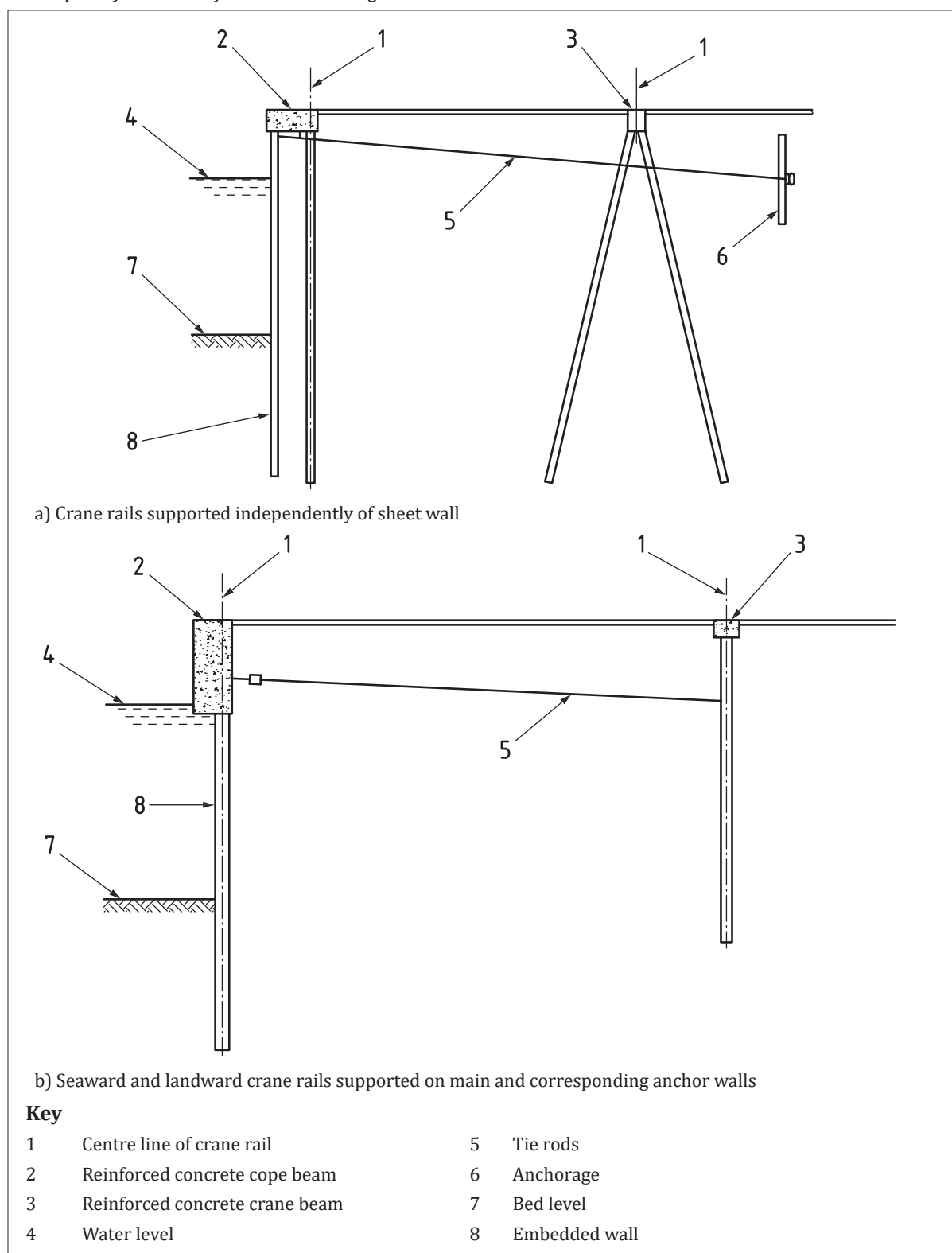
If tie beams are used to maintain crane rail gauge, rails should not be fixed until the tie beams have been constructed and all construction-related wall deflections have ceased. In any case suitable allowance should be made in crane rail recesses to accommodate long-term movements.

NOTE 3 Figure 1 shows examples of crane rail foundation arrangements.

4.11.4 Lighting

Illumination should conform to BS EN 12464-2.

NOTE Guidance on illumination levels is given in CIBSE lighting guides LG01 [30] and LG06/16 [31], and CIE publications 129/1998 [32] and S 015/E 2005 [33].

Figure 1 — Examples of crane rail foundation arrangements

5 Embedded retaining walls

COMMENTARY ON CLAUSE 5

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 [5.8](#)).

The following structural forms may be used in embedded retaining walls:

- *sheet piles;*
- *combined walls;*
- *high modulus walls;*
- *in-situ concrete piled walls;*
- *diaphragm walls;*
- *soldier piles and sheeting;*
- *barrettes (see also [7.6.3.4](#)).*

Most forms of embedded retaining wall have low permeability relative to the retained soil, and provisions for drainage of the fill are generally required to reduce tidal lag (see also [BS 6349-1-3](#)).

5.1 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.

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

NOTE 1 Cellular sheet-piled and double-wall structures are not embedded retaining walls as defined in [3.1.9](#), and are covered in [Clause 6](#).

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.

5.2 Suitability of embedded retaining wall structures

COMMENTARY ON 5.2

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 will also be one of these types.

Embedded retaining walls are also 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 offset of the berth face is to be kept to a minimum.

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

Embedded retaining walls may be used in weak soil, but it might be preferable, where weak soil, such as soft clay, lies below dredged level, to replace the weak soil with granular fill before wall construction to avoid the large penetration required for stability. If the depth of the weak soil is very great, an embedded retaining wall might be unsuitable and another form of construction, such as a suspended deck structure (see [Clause 7](#)), can be used instead.

The suitability of embedded retaining wall structures should be assessed as part of the selection and design process.

5.3 Materials for embedded retaining walls

5.3.1 General

Materials used in the construction of embedded retaining walls should be in accordance with [BS 6349-1-4](#).

5.3.2 Sheet pile walls

COMMENTARY ON 5.3.2

Interlocking U- and Z-shaped steel sheet piles are the most widely used embedded retaining wall elements in quays and jetties. Other steel elements with the same geometry (i.e. tubular piles, box piles, H piles, H piles with welded-on sheet piles) may be installed alongside one another to create high modulus walls, and the combination of different elements (i.e. tubes with intermediate U or Z sheet piles) may be used to create combined (combi) walls. Precast concrete and timber sheet piles may also be used for the construction of walls of moderate height and where driving is not too hard. However, these are rarely used for quay structures in the UK for the reasons discussed in [BS 6349-1-4](#).

Where appropriate, the effects of abrasion on sheet pile properties should be taken into account, as some types of floating fenders can cause abrasion of concrete or timber and protective coatings of steel sheet piles.

5.3.3 Combined walls

COMMENTARY ON 5.3.3

The primary elements of combined walls 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).

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

5.3.4 In-situ concrete piled walls

COMMENTARY ON 5.3.4

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. In addition, in loose granular soils, there is an enhanced risk of over-break from loss of the surrounding soil into the pile excavation. This could lead to the creation of voids outside the pile, and a loss of concrete integrity.

In contiguous pile walls, material should be unable to flow through gaps between the piles. In secant pile walls, the piles should be installed at centres of approximately 0.9 times the diameter to create an overlap.

NOTE Typical arrangements are shown in BS EN 1536:2010+A1:2015, Figure 6.

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.

5.3.5 Diaphragm walls

COMMENTARY ON 5.3.5

Embedded retaining walls in the form of in-situ concrete diaphragm walling are sometimes 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 dowelled into solid rock, but are generally unsuitable where flowing or artesian water is present during construction. In addition, in loose granular soils, there is an enhanced risk of over-break from the loss of the surrounding soil into the wall excavation. This could lead to the creation of voids outside the diaphragm wall panel, and a loss of concrete integrity.

Diaphragm walls can be used for high walls or where heavy vertical loads are imposed on the wall, where the site is suitable.

Diaphragm walls are usually of reinforced concrete. They 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.

To assist with the advance fabrication of reinforcement cages, the toe levels should be predetermined to within ± 0.5 m.

The ground level should be at least 1.0 m to 1.5 m above maximum ground water level to maintain stability of the trench when support fluid is used. Environmental issues connected with the use of support fluids or pumping of concrete in water-bearing soils should be taken into account. Slurry trench construction should be in accordance with [BS 6349-1-3](#).

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

Panel sizes should be limited by the weight of reinforcement cages to be handled; tandem lifts should be provided where necessary. The panel size should be assessed with reference to the volume of concrete, which has to be poured in a single operation.

NOTE 2 Panel lengths are typically 4.5 m to 5.0 m, the practical limits being 2 m and 10 m.

Where the diaphragm wall is of reinforced concrete, the reinforcement should be detailed to provide adequate space between bars, particularly at tie bar anchor locations and reinforcement laps. A minimum concrete cover of at least 75 mm should be adopted, with additional due allowance for the effect of over-break to achieve the minimum cover. The recommendations in [5.3.4](#) on surface finish should also be met for diaphragm walls.

NOTE 3 Further guidance is given in the ICE specification for piling and embedded retaining walls [18] and EFFE/DFI best practice guide to tremie concrete for deep foundations [34].

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.

5.3.6 Soldier piles and sheeting

COMMENTARY ON 5.3.6

Embedded retaining walls constructed using soldier piles and horizontal sheeting are not widely used in maritime works, owing to the extent of 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.

This type of wall might also be appropriate for shallow walls in locations with high tidal ranges where most of the sheeting can be installed using intertidal working.

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 in any seabed material by driving or placing in a preformed hole, but might require an excessive penetration in soft clays and silts. In-situ concrete piles are subject to the risk to the integrity of freshly placed wet concrete, and the imitations of supporting open bores and the need to place concrete under water. However, these limitations may be mitigated to some degree by the use of continuous flight auger (cfa) bored piles. Further guidance on the selection of wall type can be found in [BS 8002](#) and [BS 8004](#).

The sheeting in quay walls of this type is usually of precast concrete planks, but may be timber where there are particular environmental impact requirements.

Where soldier piles and horizontal sheeting are used, the penetration of the piles into the seabed should be calculated to take into account the consequences of failure of the ground support.

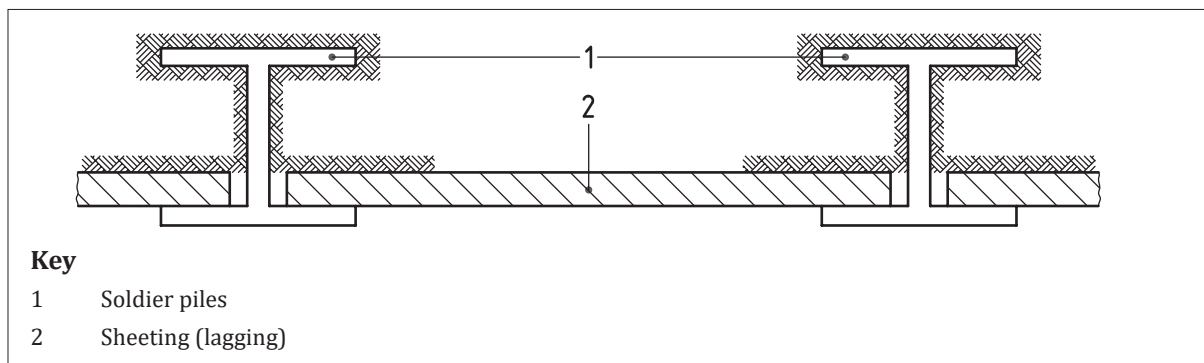
The heads of all piles should be temporarily supported until the capping is cast and the anchors installed, where used.

Where the sheeting is of precast concrete planks, it should be secured to the soldier piles to prevent differential movement and consequent loss of material.

NOTE [Figure 2](#) shows an example of such a construction.

Appropriately sealed joints should be provided to prevent loss of material.

Figure 2 — Soldier-piled wall: typical cross-section



5.4 Bending resistance of wall

5.4.1 Steel walls

5.4.1.1 Steel sheet piles and high modulus piles

The bending resistance of steel sheet pile and high modulus walls should be determined in accordance with BS EN 1993-5, taking into account the effects of pile shape, interlock fixing, wall form and soil type.

When interlocked piles are not connected by crimping or welding, 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 used.

NOTE Where the section modulus of a steel sheet pile 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.

5.4.1.2 Combined steel sheet pile and soldier-piled walls

The bending resistance of combined walls should be determined in accordance with BS EN 1993-5. The designer should take into account the contribution of both primary and secondary elements to the bending resistance of the wall.

In combined steel sheet pile walls in which the intermediate (secondary) sheeting is required to contribute to the bending resistance, the interlocks should be welded to provide the full integral capacity of the secondary sheeting.

Soldier-piled walls should be designed on the assumption that all horizontal actions are transmitted to the soldier piles.

NOTE If the modulus of the primary elements far exceeds that of the secondary elements, it may be assumed that all horizontal actions are transmitted to the primary elements and no welding of interlocks is then required.

5.4.2 Diaphragm walls

The section modulus for a diaphragm wall should be calculated in accordance with [BS EN 1992-1](#).

5.5 Effects of actions on embedded retaining walls

5.5.1 General

NOTE 1 The actions on a retaining wall are summarized in BS EN 1997-1:2004, 2.4.2.

The geotechnical parameters should be established in accordance with [BS 6349-1-3](#) through an appropriate geotechnical investigation.

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.

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 steel embedded retaining walls does not usually coincide with the most severe corrosion zone, and this should be taken into account in the analysis of steel sheet pile walls.

Crack widths should be calculated for the serviceability limit state.

NOTE 2 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 3 Further guidance is given in the ICE specification for piling and embedded retaining walls [18].

Walings should be designed in accordance with [BS EN 1992](#), [BS EN 1993](#), [BS EN 1995](#) or [BS EN 1997](#) as appropriate to the material used.

5.5.2 Vertical actions applied to a wall

5.5.2.1 Types of action

The following vertical actions which might be imposed on embedded retaining walls should be taken into account:

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

5.5.2.2 Resistance to vertical actions

Vertical actions imposed at the cope should be resisted with minimal settlement of the structure.

If a load test is carried out, the wall should be designed taking account of effects observed during testing.

NOTE Load tests are desirable where isolated vertical actions are high.

Resistance should be achieved by one of the following methods.

- a) By the wall itself. The toe of the wall over the length of the dispersed load should have an adequate bearing capacity, which might require that the wall be taken down to rock. If plug formation is uncertain, the design should be based solely on skin friction. When assessing skin friction, the length of pile determined for vertical load resistance should take into account the additional effects of movement of the ground in resisting horizontal actions, and how this could affect the mobilization of friction on the piles.

In the case of sheet pile walls with staggered toes, the vertical resistance of both the deeper and the shallower piles should be assessed to establish the net capacity of the wall.

In soldier-piled walls, only the soldier piles should be assumed to resist the vertical actions. The effective column length of the loadbearing wall may be taken as being equal to the vertical distance between the points of zero bending moment due to horizontal actions. Load-deflection effects should also be assessed.

- b) By the provision of a combined wall with load-bearing elements. Where the embedded retaining wall has insufficient vertical resistance, a combined section may be used. In sheet pile walls, the vertical actions may be carried by H-section, box or tube piles (see also [7.10.4](#)), and in diaphragm walls, T-shaped panels may be used. Where a combined section is required to resist horizontal actions, the same loadbearing elements may be used to carry the vertical actions.
- c) By the provision of separate bearing piles. Separate bearing piles may be installed to seaward or landward of the embedded retaining wall to carry all or part of the vertical imposed loads. Bearing piles are also used to support relieving platforms.

The piles should be installed at sufficient depth that there is no danger of partial transfer of the actions to the embedded retaining wall (see also [5.6.3](#)). Bearing piles close to embedded retaining walls should be driven below the potential surfaces of sliding.