

Code of practice for

Maritime structures —

Part 2: Design of quay walls, jetties and dolphins

UDC 624.034.3/.4+69.034.3/.4

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- Association of Consulting Engineers
- British Ports Association and the National Association of Ports Employers
- British Steel Industry
- Concrete Society
- Department of the Environment (Property Services Agency)
- Department of Transport (Marine Directorate)
- Federation of Civil Engineering Contractors
- Health and Safety Executive
- Institution of Civil Engineers
- Institution of Structural Engineers
- Oil Companies International Marine Forum

This British Standard, having been prepared under the direction of the Civil Engineering and Building Structures Standards Committee, was published under the authority of the Board of BSI and comes into effect on 29 January 1988

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The following BSI references relate to the work on this standard:
Committee reference CSB/17
Draft for comment 84/10645 DC

ISBN 0 580 15898 5

Amendments issued since publication

Amd. No.	Date of issue	Comments

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Foreword

This Part of BS 6349 has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee.

This code of practice contains material which is both for the information and guidance of engineers and material which forms recommendations on good practice. As such conformity with its recommendations is not obligatory and variations from its recommendations may well be justified in special circumstances and engineering judgement should be applied to determine when the recommendations of the code should be followed and when they should not.

A code of practice is intended for the use of engineers having some knowledge of the subject. It embodies the experience of engineers successfully engaged on the design and construction of the particular class of works so that other reasonably qualified engineers may use it as a basis for the design of similar works.

It is not intended that it should be used by engineers who have no knowledge of the subject nor that it should be used by non-engineers.

A code of practice represents good practice at the time it is written and inevitably, technical developments may render parts of it obsolescent in time. It is the responsibility of engineers concerned with the design and construction of works to remain conversant with developments in good practice, which have taken place since publication of the code.

Following suggestions from the Maritime and Waterways Board of the Institution of Civil Engineers, the Standards Committee for Civil Engineering Codes of Practice set up an ad hoc panel to make further studies. The panel's report, presented in 1975, concluded that existing British codes were inadequate for the special aspects of maritime structures and that there was a need for such a code. A format was proposed which divided the work into two distinct stages.

It has been assumed in the drafting of this British Standard that the execution of its provisions is entrusted to appropriately qualified and experienced people, for whose guidance it has been prepared.

The standard will be issued in seven Parts as follows:

- *Part 1: General criteria;*
- *Part 2: Design of quay walls, jetties and dolphins;*
- *Part 3: Design of dry docks, locks, slipways and shipbuilding berths, shiplifts and dock and lock gates¹⁾;*
- *Part 4: Design of fendering and mooring systems;*
- *Part 5: Recommendations for the dredging of water-ways and for land reclamation¹⁾;*
- *Part 6: Design of buoy moorings, floating and buoyant structures and anchorage systems¹⁾;*
- *Part 7: Design of breakwaters and training walls¹⁾²⁾.*

In BS 6349-1, recommendations are given to assist clients and engineers to obtain the basic data relevant to the design of any maritime structure.

This Part of BS 6349 incorporates those parts of the text of Part 1 that deal with the design of sheet-piled walls and, since these are design rather than general matters, **51.1**, **51.2** and **51.6** of BS 6349-1:1984 will be deleted by amendment. Design aspects of sheet-piled walls that are common to land-based and maritime structures will be covered in BS 8002¹⁾; **51.3** of BS 6349-1:1984 will be deleted by amendment. The design of anchorages in general is covered in DD 81 and in BS 8002¹⁾. Specific aspects of maritime anchorages are covered in this Part and in Part 3 of BS 6349; clause **53** of BS 6349-1:1984 will be deleted by amendment.

¹⁾ In preparation.

²⁾ To be issued initially as a Draft for Development.

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The full list of the organizations who have taken part in the work of the Technical Committee is given on the inside front cover. The Chairman of the Committee is Mr. J.T. Williams OBE, C Eng, FICE, F I Struct E and the following people were members of the Technical Committee.

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Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 108, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

³⁾ Past member.

Section 1. General

1.1 Scope

This Part of BS 6349 provides recommendations and guidance on the design of quay walls, jetties and dolphins. Section 2 gives general guidance and recommendations on the design of berthing structures. Section 3 discusses loading considerations for quays and jetties. Section 4 gives recommendations and guidance on the types of sheet wall, and section 5 on the types of gravity wall, commonly used in quay and jetty construction. Section 6 covers the design of suspended deck structures supported on individual bearing piles, barrettes or small caissons. Section 7 deals with the design of dolphins. Section 8 covers the general arrangement of and structural considerations for roll-on/roll-off terminal ramps including rigid and adjustable shore ramps and their supports. Section 9 covers pedestrian access, including walkways, stairways, ladders and handrails.

NOTE The titles of the publications referred to in this Part of BS 6349 are listed on the inside the back cover. References in the text to other publications are identified by numbers in square brackets and are listed in Appendix A.

1.2 Definitions

For the purposes of this Part of BS 6349 the definitions given in BS 6349-1 apply together with the following.

1.2.1

marginal berth

a berth parallel to the shore

1.2.2

breakwater berth

a berth on the lee side of a breakwater

1.2.3

quay (wharf)

a berthing structure backing on to the shore or reclaimed land

1.2.4

apron

an area of open land adjacent to a berth

1.2.5

pier

a structure projecting from the shore at which berths are provided

1.2.6

jetty

a structure providing a berth or berths at some distance from the shore

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

1.2.7

jetty head

a platform at the seaward end of a jetty or causeway

1.2.8

sheet wall

a retaining wall of thin cross section that resists loads by bending

1.2.9

gravity wall

a retaining wall of heavy cross section that resists horizontal loads by means of dead weight

NOTE The dead weight may be augmented by ground anchors.

1.2.10

suspended deck structure

a quay or jetty comprising a deck supported on bearing piles

1.2.11

berthing line

the line of the face of the fenders or (where no fenders exist) the berth structure, in the undeflected position

1.2.12

relieving platform

a platform built below deck level and supported on bearing piles, the principal function of which is to reduce lateral soil pressures over the upper portion of a sheet wall

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

1.2.13

access (approach) trestle

a bridge connecting a jetty head to the shore to provide vehicular or pedestrian access and/or support to pipes or conveyors

1.2.14

dolphin

an isolated structure or strong-point used either to manoeuvre a vessel or to facilitate holding it in position at its berth

1.2.15

berthing (breasting) dolphin

a 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.

1.2.16

mooring dolphin

a 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.

1.2.17 berthing beam

an isolated piled structure with a continuous capping situated parallel to a berth and having a similar function to two or more berthing dolphins

1.2.18 warping (lead-in) dolphin

a dolphin provided at one end of a structure or at a change in direction of a berth to support vessels being manoeuvred between two adjacent positions (similar to a berthing dolphin)

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

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

NOTE Bridging structures may be provided to accommodate relative level differences due to loading conditions of the ships and water levels.

1.2.20 walkway

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

1.2.21 cope

the top edge of a quay or jetty adjacent to a berth

1.2.22 false deck

a structural slab overlaid with fill and surfacing

1.3 Symbols

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

Symbols relating to the loading of suspended deck structures are defined in section 6: those relating specifically to limit state design of suspended deck structures are as defined in BS 5400.

D	Pile diameter.
d_1	Depth of penetration below level of zero net horizontal pressure.
d_y	Effective embedment of sheet piles.
F_H	Horizontal soil friction force.
F_V	Vertical soil friction force.
F_1	Soil force (similarly F_2 and F_3).
f_{cu}	Characteristic concrete cube strength.

H_1	Tie rod height (similarly H_2).
H_R	Retained height of structure.
h_S	Equivalent height of surcharge.
K_A	Coefficient of active earth pressure.
l_e	Effective length of anchorage.
l_o	Ineffective length of anchorage.
M	Relevant moment for staggered points.
M_S	Mass of cellular structure and contained soil.
P_A	Total active soil force per unit length of wall.
P_{AO}	Total active soil force on rear of failure wedge per unit length of wall.
P_p	Passive soil force per unit length of wall.
P_S	Surcharge pressure.
P_0	Active soil force on rear of failure wedge.
P_1	Pile reaction (similarly P_2 , P_3 , P_4).
R	Resultant force on relevant failure plane.
R_p	Resultant of passive resistance.
s	Half the stagger height.
T_1	Tension in tie rod (similarly T_2).
t	Thickness.
W	Weight of soil wedge.
Y	Depth to level of zero net horizontal pressure.
γ	Density of soil above water level.
Δ'	Deflection (similarly Δ_2).
δ	Angle of wall friction.
ϕ_r	Angle of soil shearing resistance.

1.4 Partial safety factors

For the purposes of this Part of BS 6349, the values given for safety and partial safety factors for loads and materials are based on those given in BS 5400, in so far as these are applicable.

Section 2. General design of berthing structures

2.1 General

2.1.1 Planning

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

Guidance on overall port planning is given elsewhere [1, 2, 3, 4, 5].

2.1.2 Dredging and filling

Where structures are planned in association with dredging operations, consideration should be given to the use of dredged material for reclamation. Depending upon the suitability of the material considerable economies in the cost of the berth structure may also be achieved.

Maintenance dredging may 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 may be economic benefits in providing a dredged depth greater than the minimum required so that maintenance dredging with its disruption to operations is reduced. Careful consideration should be given to the dredging tolerances which will be achieved during construction and during maintenance dredging (see 2.2.6).

2.1.3 Function of structure

A berthing structure is provided to accommodate a particular vessel or range of vessels. Dolphins may be required at berths where the vessel length exceeds the length of quay or jetty. Economies may often be made by using dolphins to resist the berthing and mooring loads, thereby permitting a lighter form of structure to be used for the quay or jetty itself. This is particularly applicable to oil terminals serving very large crude carriers (VLCCs), where maximum horizontal loads are severe and imposed vertical loads are light. Reference should be made to BS 6349-4 for specific recommendations on dolphins. The need for the berth to be contiguous with the shore, the apron width required, and the area of land required behind the berth are all matters which depend on the method of handling and storing the cargo. Bulk cargo may require extensive areas of land for storage, but this may 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 should be direct and as short as possible: in general, a marginal type of berth satisfies these requirements. The need to provide an area of land behind a berth often involves reclamation and this may have a strong influence on the type of marginal structure to be adopted.

2.1.4 Access and safety

Safety aspects should be carefully considered for all maritime structures. Points to be considered include access by emergency services, escape routes from hazardous areas and provision of fire-fighting (see 2.5.2) and lifesaving equipment. Further guidance on safety aspects is given elsewhere [6, 7].

Depending upon the nature of the cargo to be handled, anti-pollution equipment may be required.

At VLCC terminals, consideration should be given to the provision of speed-of-approach monitoring aids, current meters and anemometers. At all maritime terminals at least one tide gauge should be provided.

Access or means of access should, wherever possible, be provided to allow inspection of structures for maintenance purposes.

2.1.5 Berth orientation

Where there is freedom of choice as to the orientation of the berth or where the berth is in an exposed position, the orientation should be chosen so that the predominant winds, waves and currents have least effect on the operation of the berth and the structure has least detrimental effect on the coastal or estuarial regime. A physical or mathematical model may be necessary to study these effects.

Berths should not be broadside-on to strong prevailing winds and waves. If broadside winds are likely, the navigational and cargo handling aspects should be considered.

When considering waves at the berth location, the effects of period and direction should be assessed (see 31.4 of BS 6349-1:1984).

The direction of the current at tidal locations will usually vary. The importance of the effects of current on navigation will depend on the strength of the current, the availability of tugs, the loading condition of the ships and whether movements are restricted by tide levels.

Offsetting currents can help a ship to leave the berth but may be unacceptable if a fully laden ship has to berth against them. In cases where unacceptable currents cannot be avoided they should be redirected by the use of training walls. The direction from which vessels will approach the berth should be considered. If they are always to lie alongside in the same direction (e.g. head inwards or head outwards), the configuration of mooring points and fixed or rail mounted cargo handling equipment may sometimes be refined.

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

2.1.6 Berth geometry

NOTE The influence of services and fixed or rail-mounted cargo handling equipment on berth geometry is dealt with in 2.5.

2.1.6.1 Berth length. The length of the berth is a function of the ships' length and should take account of any changes in ship size expected during the life 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 considered. The length allowed between ships berthed in line will depend on the method of berthing ships, but a minimum distance of 15 m is commonly adopted. For fishing or pleasure craft a berth length of 1.15 times the vessel length is desirable.

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

2.1.6.2 Depth alongside. The depth of water required alongside the berth is a function of the operating draught of the vessels expected to use the berth within its lifetime, the tidal range and, at exposed berths, the effects of wave action. Waves due to passing ships, wind action causing roll and the effect upon draught of listing during loading/unloading should also be considered. If shipping patterns show that ships using the berth are never at full draught, a reduced depth may be acceptable.

The underkeel clearance in calm conditions should generally be at least 0.5 m for the deepest draught vessel, but suitable allowances for trim, listing and wave-induced motion should be made and additional clearance may be required where the seabed is rocky. At berths where the movement of the largest ships to be accommodated takes place at the higher states of the tide, the underkeel clearance may be achieved by dredging a berth box in front of the structure. The box width should be at least 1.5 times the beam of the largest vessel, and its length at least 1.2 times the overall length of the vessel, located symmetrically about the midpoint of the berth.

Where appropriate, protection against scour caused by propellers and bow thrusters should be provided (see 2.3.5).

The need to design for future berth deepening should also be considered.

2.1.6.3 Cope level. The level of the cope along the berth face may be determined by that of existing berths or the working area behind the berth. With a virgin site the level of the cope has to be set on practical and economic grounds.

The optimum cope level can be selected by determining the risk of flooding and then appraising the effect and implications of such flooding. An economic analysis can be carried out to compare the capital investment cost with the cost of flood damage or the cost of providing insurance to cover such damage.

A low cope may be appropriate at berths exclusively used by small craft.

The effect of cope level upon crane height and Ro-Ro ramps should be considered.

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

For a berth in an open harbour or exposed location, a statistical analysis can be prepared to determine the level and frequency of high water levels and wave heights. The application of joint probability methods for computing extreme sea levels is described elsewhere [8].

2.1.6.4 Width of apron. No firm guidelines can be given on the width of berth aprons, which depends on the method of handling cargo. Whether the berth is for specialized or multi-purpose use will largely influence the apron width required. Modern general cargo berths with transit sheds on the berth require a distance between berth face and shed of 20 m to 50 m. For certain specialized cargo such as fish, apron widths may be reduced to minimize handling, depending on the handling methods. Passenger and naval berths will also have different apron width requirements.

Berths should be built with adequate back-up areas to provide storage for the volumes of cargo which are expected to be handled across the berth during its lifetime. 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 may range up to 300 m or more.

The apron should be wide enough to allow cargo handling equipment to be operated efficiently and safely on it and special attention should be given to the provision of areas required for turning vehicles and for access by emergency vehicles.

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

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

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

Additional clearances should be allowed for bulbous bows when the angle between the cope line and the vessel's longitudinal axis is likely to exceed 7°. Reference should be made to BS 6349-4 for guidance on approach angles.

Consideration should also be given to preventing ships striking structures in the spaces between individual fender units. This is particularly important where vessels may turn alongside the berth when there are currents which may delay or prolong the manoeuvre.

Maritime structures cannot normally be designed to withstand the very large vertical loads which can be imposed by a ship or other large floating object trapped under or hung up on a projection during a rising or falling tide. Careful design to avoid angular projections and to provide any horizontal projection with a flare can reduce the likelihood of these unusual and damaging vertical loads occurring. Facing and deck units should be secured to the structure since, in general, gravity alone will not be sufficient to keep them in place.

2.2 Choice of structure

2.2.1 General

For any important structure, a variety of different types should be compared and a choice made on the basis of the capital and maintenance costs or ease of construction. The use of standardized designs to meet various conditions may not be economic. The typical designs given in this code are not intended to exclude the use of alternative structural arrangements, including hybrids of two or more of the arrangements described. Many examples of original design are given in [9].

2.2.2 Types of structure

Maritime structures can be either solid or open-piled. An open-piled structure can be either rigid or flexible (see 4.7.1 of BS 6349-1:1984).

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

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

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

The proximity of existing structures should be taken into account when selecting the type of structure to be adopted for a new quay.

Changes to the existing maritime regime at the berth and at adjacent locations are likely to be smaller for open-piled than for solid structures, as they present less obstruction to current flow and waves. At sites where siltation is likely to occur, a solid structure may increase current velocities, thereby reducing the volume of material deposited along-side, but possibly causing adverse effects elsewhere. A solid structure can cause unacceptable disturbances for a ship at berth, due to reflected waves, but this may be reduced by the provision of perforations or a partly open face. Where open structures are used for marginal berths, the back-of-berth fill is usually retained behind a revetted slope. This slope can be used to absorb wave energy and reduce reflection.

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

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

2.2.3 Seabed conditions

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

2.2.4 Local construction materials

Included in any geotechnical survey should be a study of the locally available natural materials for construction. The availability of rock or general fill within a short haul distance may have a strong influence on the choice of structure.

2.2.5 Method of construction

The method and sequence of construction and availability of major construction plant may determine the type of structure finally selected. The maximum use of prefabricated sections may be adopted to achieve an early completion date, but the size and weight of units have to be within the handling capacity of available plant. When the facility is part of a large project, a design involving the mobilization of specialized equipment and the establishment of a large fabrication plant for the project as a whole may have cost advantages.

Construction from on shore by the creation of a temporary bund, or a construction which works progressively forward from the shoreline or on a jack-up pontoon may be necessary if the weather conditions are too rough for the use of floating plant, either throughout the year or seasonally.

2.2.6 Construction difficulties

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

The tolerances normal in building work on land often cannot be achieved in these conditions.

In particular, it should be remembered that a pile driven over water is seldom supported over its full length and, although it may be correctly located on the seabed, the head may deviate from the correct position. Deck units to be placed on piles should be designed to accommodate the permissible deviation in the piles.

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

When precast units are to be placed on a gravel bedding layer on the seabed, the final levelling and preparation of the bedding has to be carried out by divers, often with poor visibility. As far as possible divers' work should be kept simple.

Adequate protection of the works during construction should be provided to avoid damage caused by floating craft, waves and currents.

2.3 General site considerations

2.3.1 Settlement

The effects of settlement on port operations should be carefully considered. In general, only small amounts of settlement will be acceptable in quays and jetties, unless special foundations (see 4.18) are used to minimize the effect of large settlements on operations.

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

2.3.2 Fill material

Fill placed behind quays should be of free-draining granular material. This will ensure that maximum natural consolidation is achieved in the submerged zone, where additional compaction is not usually carried out. Above water level, the fill may be compacted by conventional means, and the effects of heavy compaction, which is generally required at ports, should be carefully assessed. Such effects may include lateral soil pressures on the wall subsequently approaching at-rest values, resulting in additional stress in the wall or anchorage, or in additional seaward deflections.

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

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

Guidance on fill materials is given in 50.2.7 of BS 6349-1:1984.

2.3.3 Drainage and filters

Drainage through quay walls to minimize tidal lag (see 51.5 of BS 6349-1:1984) usually takes the form of weepholes or flap-valves. Some types of quay wall allow drainage through the wall at open joints. These should be sealed with a suitable filter to prevent loss of fill material.

Weepholes should be designed to prevent loss of fill. This can be achieved by using vertical slots whose width is less than the smallest size of stone in the adjacent filter layer. Design of the weepholes should take into account the possibility of blockage by marine growth.

In silt-laden waters, flap-valves should be provided in preference to weepholes, to avoid blockage. The valves should be fixed just above low water level to allow maintenance. They are connected to a series of drains constructed behind the wall.

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

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

Guidance on the design of filters is given in reference [10].

2.3.4 Wave pressures on walls

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

The height, length and angle of approach of waves should be considered when assessing the total hydrostatic pressure for which the wall is to be designed: drawdown in the wave trough is usually more important than pressure from the wave crest. At impermeable structures, the tidal lag should be increased to at least half the wave height to represent a wave trough where a standing wave can occur (see 39.4 of BS 6349-1:1984). At impermeable structures with permeable backing, the effect of wave action gradually building up water levels in the backfill should be considered. The effect of waves overtopping structures, such as breakwater quays, should also be studied in relation to a possible build up of water levels behind the wall face.

2.3.5 Scour protection

The effects of ships' propellers and bow-thrusters, waves and currents on the stability of the seabed and any underwater slopes near structures should be carefully assessed. Ships with variable-pitch propellers, particularly ferries, may cause worse scour than those with constant-pitch propellers.

Where scour is considered to be likely, protection, such as a rubble anti-scour apron on the seabed, should be provided in front of quay walls and particularly at berths where vessels will generally berth in the same position [11]. The size of rock protection for underwater slopes should be not less than that needed to resist the wash of propellers and bow-thrusters. It may be necessary to provide a filter between the rubble and the underlying material.

The effect on durability of the structure of natural bed material and small rock being disturbed by bow thrusters and thrown against the structure should be considered. Further guidance on scour protection is given in clause 54 of BS 6349-1:1984.

2.3.6 Impounded systems

At locations subject to a large tidal range, an enclosed system with access through locks may be provided in order to reduce the water level variations at the quay. The level may either be maintained above the outside high water level at all times, by means of pumping, or may vary, for example, allowing for opening of the gates at high tides with a gradual fall in level between high waters due to the spring/neap range, leakage and locking losses. In the latter case, an estimate should be made of the lowest probable water level within the system during operating conditions and this should be used in earth pressure calculations for the normal loading condition (see 3.2.3.2).

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

The risk of an accidental drawdown will depend on the number of gates at the site, whether spare gates are available and, if so, how readily they may be brought into service, and on the water area of the system in relation to the dimensions of the gates. For guidance on lock gates, reference should be made to BS 6349-3⁴⁾.

In assessing the risk that can be accepted, the consequences of drawdown should also be considered. These may include not only the cost of repairs to quay walls and other port structures, but also losses and legal liabilities to third parties.

The pore water pressure conditions resulting from drawdown will depend on the potential rate of drawdown (based on the area of the enclosed basin relative to the possible rate of outflow) and the soil conditions.

2.3.7 Temporary loads during construction

Most quays and jetties will be subjected to loading by construction plant before completion of the structure. One example of such loading is heavy plant passing along the top of filled ground behind quays: a uniformly distributed load of at least 10 kN/m^2 may be used to allow for this. Another common example is the use of bearing piles with temporary or incomplete decking to support plant: the nature of these loads is generally more critical and the particular items of plant should be considered carefully.

All maritime structures, with the exception of those built in the dry, will be exposed during construction to loading by currents, waves and wind. The effect of these loads on piles that are not restrained or anchored during construction needs to be carefully assessed.

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

⁴⁾ In preparation.

Overall stability during construction should be considered as well as the strength of parts of the structure.

2.3.8 Earthquakes

Most countries subjected to earthquakes publish national design codes or rules to cover these loads, and reference should be made to these wherever possible (see clause 40 of BS 6349-1:1984).

Earthquakes are generally assumed to impart a horizontal acceleration to the structure and the adjacent and underlying soil mass. There is an accompanying vertical acceleration which should also be considered, especially in the case of earth structures. The seismic acceleration applies to each mass of the structure and soil. Thus, in soil containing water, the combined mass of soil and water is accelerated. Earthquakes affect lateral soil pressures (resulting in higher active pressures and lower passive resistance), the stability of slopes and the shear strength of the soil, particularly loose soils in which liquefaction may occur. Where there is a possibility of resonance, particularly with high, slender structures, the accelerations used require very careful consideration.

The values of imposed loading on structures and fill used for design conditions are commonly reduced by up to 50 % under earthquake conditions, depending on operational patterns.

Detailed guidance on seismic resistant design is given elsewhere [12, 13, 14].

2.3.9 Paving and surface water drainage

Paving on areas of fill should be designed in accordance with the recommendations given in clause 63 of BS 6349-1:1984.

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

Where drainage at the quay face could cause pollution of the sea, or where the apron is very wide, surface water may be directed to gullies or channels provided in the apron. At oil terminals, kerbed areas will usually be required to contain oil spillage, which may then be drained to a tank provided for this purpose.

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

2.3.10 Ice loading

Structures to be built in cold regions may need to be designed to resist or accommodate the loads imposed by ice [5, 15].

2.3.11 Detailing

Greater economies are generally made by aiming at simplicity of construction and robustness of design than by trying to reduce the quantities of materials in the structure by complicated details. If in situ concrete is used within the tidal zone, members should be detailed so that the concrete can be easily placed and effectively compacted. For reinforced or prestressed concrete work in the tidal zone, it is generally better to use precast units with the minimum of in situ concrete connections.

2.4 Durability

2.4.1 General

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 be designed for durability. The problems of ensuring durability are dealt with in section 7 of BS 6349-1:1984.

2.4.2 Design life

The design life of a structure is chosen to represent its intended useful life. For guidance on the design life reference should be made to clause 16 of BS 6349-1:1984.

2.4.3 Materials

Materials should be chosen on the basis of their availability, quality and suitability for the particular project. In the UK, it would be unusual to have a site where the choice of materials was limited due to lack of availability or poor access. However, in overseas projects there may be a strong case for using locally available indigenous materials and using technology which is within the capability of the local plant and labour.

2.4.4 Corrosion and crack control

2.4.4.1 General. Protective measures required against corrosion will depend upon the exposure zone: in general, four exposure zones with differing corrosion conditions need to be considered:

- a) embedded;
- b) continuously immersed;
- c) intertidal and splash;
- d) atmospheric.

Reference should be made to clause 68 of BS 6349-1:1984 for a discussion on the most common measures adopted.

The danger of corrosion is usually greatest in the intertidal and splash zone, where alternate wetting and drying of the surface occurs. Under some conditions corrosion in the splash zone is more severe than in the intertidal zone and separate consideration may be necessary.

2.4.4.2 Steel corrosion. The intertidal and splash zone may be considered as two separate zones when estimating the corrosion rates of structural steel (see Table 22 of BS 6349-1:1984). However, it is generally not practicable to vary the section sizes or to provide different protective systems within the zone.

2.4.4.3 Concrete and crack control. It is not normal to vary the quality of concrete in the different zones but the maximum allowable crack widths may need to be reduced in the intertidal and splash zone. At the serviceability limit state, crack widths anywhere in the structure should be limited in accordance with BS 8110-2 to a maximum of 0.3 mm. In the intertidal and splash zones or other areas where crack control is considered important, crack widths should be restricted in accordance with the recommendations of BS 5400-4 and BS 5337.

For massive structures in which stresses are low and only nominal reinforcement is provided, crack widths may not be important [16]. However, in hot, arid climates, mass concrete quay walls may need to be designed with a higher cement content in the splash zone to reduce surface degradation.

In continuous in situ concrete superstructures that are partly submerged or that are continuously supported on a submerged retaining wall, differential shrinkage cracks should be avoided by careful detailing.

The provision of a waterproof membrane to minimize the effects of de-icing salts on decks in areas subject to freezing conditions should also be considered. The membrane should be protected by a wearing course.

2.4.5 Maintenance

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

Access for inspections and maintenance work under suspended decks may be facilitated by the provision of occasional permanent fixings on piles and deck soffits to enable small boats to be moored and staging to be supported. For major trestle structures, a permanent access structure for maintenance work may be required.

It may be necessary to take a facility out of service during maintenance, in which case the loss of revenue may be substantial. Each design should be assessed on its own merits and the design should be appropriate to the maintenance capability of the user. However, where this capability is not established, it should be assumed in all designs that there will be no regular maintenance of the structure except for those areas easily accessible from land.

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

2.5 Services and equipment

2.5.1 Structural considerations

Requirements for the provision of services and equipment on the berth should be fully considered and finalized at the earliest possible stage.

The support system for gantry cranes and other mechanical handling plant may dictate the geometry of the berth structure. Consideration should be given to differential movement of crane rail tracks. If a change in track centres is possible, caused, for example, by wall movements or lateral wheel loads, a strut/tie member should usually be provided between the track foundations. Differential vertical movement at structural joints may be accommodated by providing a short transition support to the rail. In critical cases, provision for future adjustment of rail levels should be built into the track. The depth required for flush fixing of the rail track in the deck is also significant when choosing between different forms of construction. The recommendations given in clause 64 of BS 6349-1:1984 and in BS 5744 should be taken into account in the design of crane rail tracks. Trenches for power supplies to heavy equipment may be required.

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

2.5.2 Services

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

- a) fresh water;
- b) salt water;
- c) chilled water;
- d) steam;
- e) bunkering oil;
- f) compressed air;
- g) electrical power;
- h) telephone;
- i) sewerage;
- j) specialist gases.

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

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

Consideration should be given to providing adequate means of access for fire-fighting services to all areas.

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

Pits and ducts may have to accommodate trace heating to prevent solidification of some liquids.

The provision of electrical outlets for one or more systems of electrical power may be required at all berths.

Ship-to-shore telephone points should usually be provided at each berth, preferably near either end to be conveniently placed for the ships' superstructures.

The other services listed above are required only at a few berths, such as those used for fitting-out vessels.

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

Guidance on earthing and bonding, and on fire-fighting equipment, at marine oil terminals is given elsewhere [17].

2.5.3 Equipment

Provision should be made for mooring devices, such as bollards, capstans, fairleads and mooring hooks, and for lifesaving equipment and safety ladders.

Where necessary, the following facilities should be provided:

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

Detailed minimum equipment requirements for berth structures are given in reference [7].

Compliance with other local regulations, if any, should also be checked.

Section 3. Loading considerations for quays and jetties

3.1 Design methods

3.1.1 General

There is a strong argument for the use of limit state design for quay walls and jetties, because the wide variety of different forms of construction which could be used do not have consistent margins of safety against failure if designed for working stresses. In some cases, such as relieving platform structures or piled structures comprising raking piles loaded in compression by a heavy concrete capping and unable to resist tension, safety margins may be very small. Limit state methods are, however, not generally used in British codes of practice for the design of foundations and earth-retaining structures. To be consistent with existing codes it is, therefore, necessary to treat the design of the superstructure separately from those elements of the structure providing support or retaining soil.

During this transition stage in design when there is no unified approach to the design of the elements of a structure it is necessary to consider very carefully the loads imposed on the structure and to ensure that the appropriate factors are applied to them. The worst credible load combinations should be examined and the sensitivity of the design to changes in load combination checked. The probable mode and consequences of failure of the structure should be ascertained.

3.1.2 Earth-retaining structures

Reference should be made to BS 8002⁵⁾ for general guidance on the design of earth-retaining structures.

Reinforced and prestressed concrete sections in retaining walls should be analysed by either limit state or working stress methods. Steel and timber sections are analysed by working stress methods, although limit state analysis may also be used for steel.

3.1.3 Foundations

Reference should be made to BS 8004 for general guidance on the design of foundations, for both gravity and piled structures.

The bearing capacity of the soil is expressed as the ultimate bearing capacity divided by a suitable factor of safety, in accordance with BS 8004, to take into account the settlement expected and the ability of the structure to take up this settlement. The loads applied to the foundation should be multiplied by a partial factor of 1.0 when calculating the loading intensity under the foundation.

Reinforced and prestressed concrete sections in the foundation are analysed by limit state methods. Steel and timber sections are analysed by working stress methods, although limit state analysis may also be used for steel.

3.1.4 Suspended decks

For suspended decks, the analysis should be based on limit state codes, such as BS 5400, where the deck is in concrete or steel. For timber, it is necessary to adopt unfactored loads and design to permissible working stresses.

3.2 Loads

3.2.1 General

Reference should be made to section 5 of BS 6349-1:1984 for the loads and soil pressures and to section 6 of BS 6349-1:1984 for the hydrostatic forces to which a maritime structure is subjected. For fendering and mooring systems, reference should be made to BS 6349-4.

Loads can be grouped under the general headings given in 3.2.2 for the initial design but it may be necessary to consider the loads within a group separately when they are unusual or of particular significance.

3.2.2 Types of load

3.2.2.1 Dead load. The dead load is the effective weight of the structural elements of the structure. For some design analyses it may be preferable to consider the weight of the elements in air and to treat the uplift due to hydrostatic forces separately. This may be particularly important when considering the effect of artesian water.

3.2.2.2 Superimposed dead load. The superimposed dead load is the weight of all materials forming loads on the structure that are not structural elements. Typical examples are the fill material on a relieving platform, surfacing, fixed equipment for cargo handling and quay furniture. The self-weight of large, slow-moving cranes on fixed tracks, such as container cranes, may be included in this category.

In any analysis the effect of removing the superimposed dead load has to be considered, since it may diminish the overall stability or diminish the relieving effect on another part of the structure.

3.2.2.3 Live loads. Live loads are subdivided in 47.2 of BS 6349-1:1984 into the following categories:

- a) cyclic;
- b) impulsive;
- c) random;
- d) static and long-term cyclic.

⁵⁾ In preparation.

The first three categories are dynamic loads and it may be necessary to consider them separately to calculate the response of the structure.

Cyclic and random loads are mostly environmental but cyclic loading can be caused by vibrations from traffic or machinery.

The position and combinations of the live static and long term cyclic loads should be chosen so that their damaging effect on the structure is the most severe.

3.2.2.4 Soil and differential water loads. Soil and differential water loads are the dominant loads affecting the stability of an earth-retaining structure. The disturbing forces are affected by the surcharge and live loads on the retained soil.

3.2.2.5 Environmental loads. Because of their long term nature, environmental loads, such as the effects of snow, ice, temperature, current, tide and time-averaged wind, are not considered as dynamic loads.

Cyclic loads are induced by regular wave trains and vortex shedding in steady currents. Wave slam forces are considered as impulsive loads, while random loads include direct wave loads, wave-induced loads, seismic loading and turbulent wind loads.

3.2.3 Load combinations for overall stability

3.2.3.1 General. The loading conditions given in 3.2.3.2 to 3.2.3.4 should not be considered as exclusive, and any other critical conditions which might possibly occur should also be analysed.

Although some combinations of loads are mutually exclusive, the probability of two or more large loads being applied to the structure simultaneously should be assessed. Depending on the consequences of failure, in most cases, it will not be economic to design for the simultaneous application of all possible extreme loads. However, where there is a very low probability that two large loads will occur simultaneously, the structure may be analysed using reduced factors of safety.

3.2.3.2 Normal loading conditions. Normal loading refers to any combination of loads that may reasonably be expected to occur during the design life of the structure, associated with normal operating conditions. This should include any foreseeable modifications to the structure, earthworks, paving, storage patterns, handling equipment or dredged depth.

The maximum normal value of each type of load described in 3.2.2 should be considered in combination, having regard to the provisions of 3.2.3.1. Examples of normal loading conditions are:

- a) overdredging of seabed within specified tolerances;
- b) increase in hydrostatic head due to drawdown in an impounded basin occurring during planned inspections at intervals not exceeding 1 year;
- c) water levels in the tide range mean high water springs (MHWS) to mean low water springs (MLWS);
- d) environmental loads that generally have a return period of 1 year or that impose restrictions on port operations, but excluding earthquakes and tsunamis;
- e) normal berthing operations as described in BS 6349-4;
- f) loads derived from average soil properties in accordance with BS 8002;
- g) loads due to containers, using diversification factors⁶⁾ for stacks more than one container high.

3.2.3.3 Extreme loading conditions. Extreme loading refers to any combination of loads that may be expected to occur during the design life of the structure, associated with the most severe credible load that could physically be applied, excluding accidental loads, such as that due to an uncontrolled berthing.

The likelihood of more than one extreme load occurring at any time should be assessed. The secondary effects of such a load on other types of load, for example, the effect on soil properties of earthquakes or flooding, should be carefully considered. However, the possibility of long term changes in conditions, such as deterioration of drainage systems, which could apply during application of the most credible load, should also be considered.

Examples of extreme loading conditions are:

- a) deepening of the seabed due to scour;
- b) an increase in the hydrostatic head due to drawdown in an impounded basin occurring as a result of occasional inspections or accidental damage to gates;
- c) astronomical water levels outside the tide range for normal loading conditions;

⁶⁾ These factors will be included by amendment in clause 44 of BS 6349-1:1984.

- d) environmental loads with a return period equal to the design life of the structure;
- e) abnormal berthing operations, as described in BS 6349-4;
- f) loads derived from the upper or lower quartile of soil properties, whichever is the more severe;
- g) loads due to containers without the use of diversification factors;
- h) earthquakes and tsunamis (see also **2.3.8** for the associated reduction in live loading).

3.2.3.4 *Temporary loading during construction.* The loads that may be expected to be applied at each stage of construction should be carefully considered (see also **2.3.7**).

Section 4. Sheet walls in quay and jetty construction

4.1 General

The design of sheet walls is covered generally in BS 8002, which should be used in conjunction with this section.

NOTE Cellular sheet-piled and double-wall structures are not sheet walls as defined in 1.2.8 and are covered in section 5.

Sheet 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 surcharge loads. The sequence of construction of sheet walls is a major design consideration (see also 4.17).

The following structural forms may be used in sheet walls:

- a) sheet piles;
- b) in situ concrete piled walls;
- c) diaphragm walls;
- d) soldier piles and sheeting.

4.2 Suitability

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

Sheet walls may be used in weak soil but, it may be more economical where weak soil lies below dredged level to replace the soil with sand before wall construction to avoid the large penetration required for stability. If the depth of soft clay is very great, a sheet wall may be unsuitable and another form of construction, such as a suspended deck structure (see section 6), should be used instead.

Where the seabed is mainly of rock, pretreatment of the bed or pile toe may be necessary to allow sheet piles to be used.

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

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

4.3 Types of structure

4.3.1 Anchored (or propped) single-wall

Resistance to active pressure from the retained soil is provided by the bending strength of the sheet wall, which spans vertically between anchors (or props) at one or more levels, and the distributed lateral resisting pressure on the lower embedded part of the sheet wall, where passive soil resistance is available. Figure 1(a) and Figure 1(b) show examples of walls with one and two tie levels, respectively.

Anchorage may be limited to a single level at or above MHWS because of the difficulty in installing anchors and walings below MLWS or at some intertidal level. They should be located at sufficient depth to allow underground services to run above them.

Walls with a substantial cantilever above tie level may require auxiliary tie-backs to the top. These may be connected to the main anchorage or to a separate anchorage [see Figure 1(c)].

4.3.2 Cantilevered single-wall

Cantilevered single-wall construction is limited to low retaining walls, generally with a retained height of soil of less than 5 m, since resistance to active pressure of the retained soil is provided only by the passive resistance of the embedded portion of the piles (see Figure 2). The soil yields in mobilizing this resistance, with consequent large deflection of the tops of the piles.

4.3.3 Sheet wall with relieving platform

A sheet wall with relieving platform is applicable where soils are weak and/or a greater height of wall is required.

Two basic types of relieved wall may be provided, as follows.

- a) Figure 3(a) shows the most common form of relieved wall in which the relieving platform is constructed behind the sheet wall on existing or filled ground. To achieve maximum relief of lateral pressures on the wall the width of the platform should be such that its rear edge intersects the failure plane of the soil, and the platform built as low as practicable.

b) Figure 3(b) shows a variant in which diaphragm walling is used both for the sheet wall and as an alternative to the bearing piles. The transverse diaphragm walling may be continuous or in the form of barrettes (see also 6.7.4.3). Ground anchors should be used where the overturning resistance is insufficient.

Lateral pressure on the sheet wall may be further reduced by sloping the ground under the platform or by taking advantage of an existing slope [see Figure 3(c)].

The sheet wall may either extend to cope level, or may be stopped off at the platform level and a reinforced concrete retaining wall built up from the platform slab to cope level.

The vertical loads are supported by the sheet wall and the bearing piles. The raking piles or diaphragm wall elements provide a rigid anchorage to the sheet wall, uplift forces being reduced by the load from the platform. Where it is desired to reduce the vertical soil load on the wall, the platform may be designed as a partially hollow reinforced concrete section [see Figure 3(d)].

4.4 Types of sheet wall

4.4.1 General

Reference should be made to BS 8002 for detailed recommendations for each type of sheet wall and to 61.3 of BS 6349-1:1984 for general considerations regarding sheet piles.

4.4.2 Sheet-piled walls

4.4.2.1 Timber. Timber sheet piles may provide an economical wall for moderate heights of retained material and where driving conditions are not too severe. Examples of suitable applications are bulkhead walls behind suspended decks and quays for small craft. Most timbers require protective treatment against rot and marine borers: see 60.2 of BS 6349-1:1984. Rubbing strips should be provided where abrasion is expected.

The joints between timber sheet piles are not generally tight, so the joint should be chosen with care if driving in existing ground where retained material could escape. Typical joint arrangements are shown in Figure 4.

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

The main advantage of properly designed concrete sheet piles is their durability, although the concrete mix may have to be modified if sand abrasion is expected. Sheet piles may also be used to advantage when heavy vertical loads are to be carried, since they will reduce the tension due to bending of the piles. However, the weight of the sheet piles, the care required during handling, the difficulty of forming extensions and the usually poor interlock at joints are all factors that in many cases will dictate against their use. If there is a danger of loss of material, the joints may be sealed by providing a filter behind the wall or grouting after driving. A typical joint arrangement is shown in Figure 5.

Prestressed piles should generally be used in preference to reinforced concrete to save weight and to produce a more economical design. They are more durable, stronger in bending and better able to resist tension set up during handling, driving or in service.

Where abrasion due to floating fenders is likely, the concrete quality should be selected accordingly.

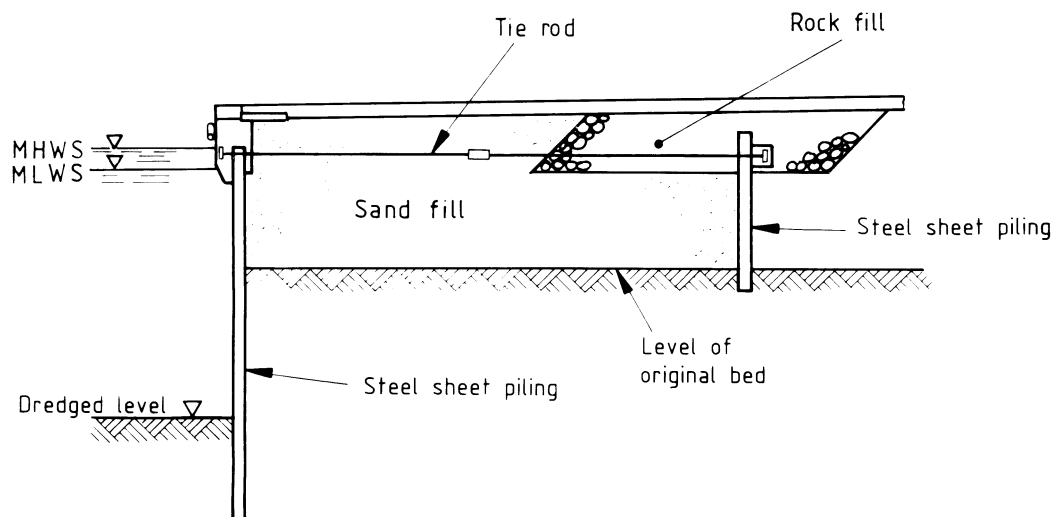
The construction tolerances given in 6.13 may be applied to concrete sheet piles.

4.4.2.3 Steel. Steel sheet piles are the most widely used sheet wall elements in quays and jetties. They are relatively light and easy to handle, they can be supplied in long lengths and can be extended and cut without undue difficulty. Guidance on welded joints is given elsewhere [18].

This type of pile can be driven to considerable depth with low displacement in a wide range of ground conditions and into weathered rock. Where hard driving is encountered, the pile section required may exceed the section needed to resist bending. This should be taken into account in assessing the reduction of moments. With various forms of pretreatment, steel sheet piles may also be installed in solid rock, in a trench backfilled with concrete or by preblasting the rock.

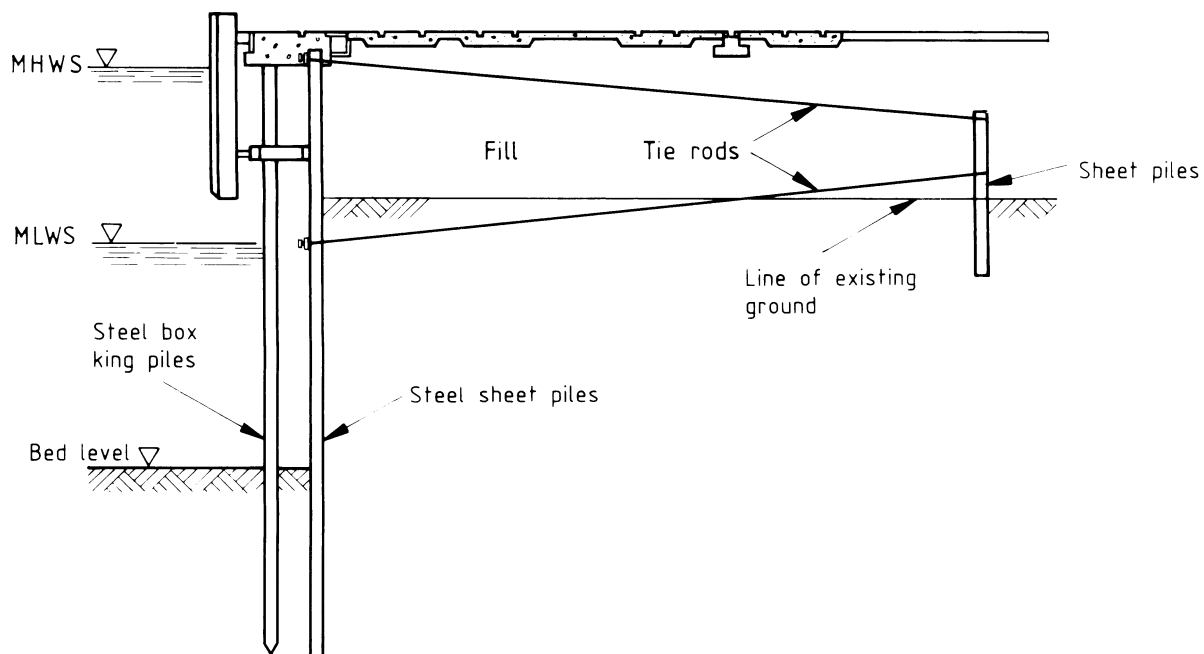
The interlocks are generally tighter than for timber or concrete piles. The principal disadvantage of steel sheet piles is corrosion, which should be allowed for in design.

Some types of floating fenders may cause abrasion of steel sheet piles or their protective coatings and this should be considered in selecting the form of fendering to be adopted.



Source: Bertlin & Partners/National Ports Council.

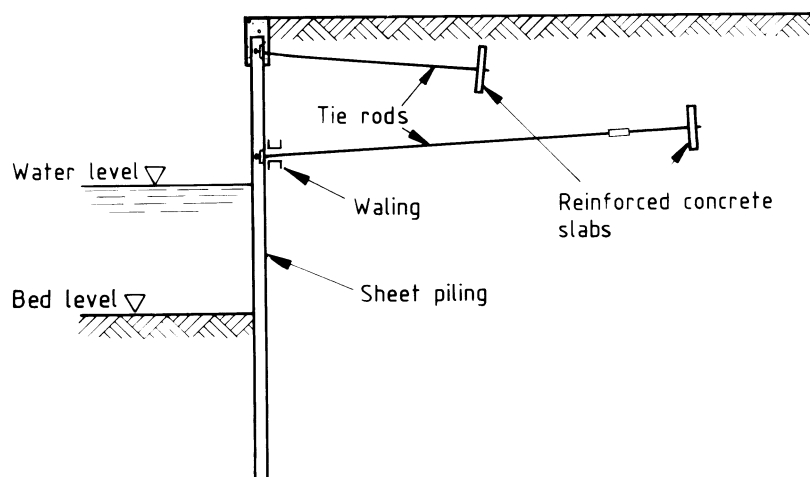
(a) Anchored single-wall



Source: Posford, Pavry & Partners/National Ports Council.

(b) Double-anchored wall

Figure 1 — Anchored sheet wall structures



Source: SACILOR, France.

(c) Wall with auxiliary tie-back

Figure 1 — Anchored sheet wall structures (concluded)

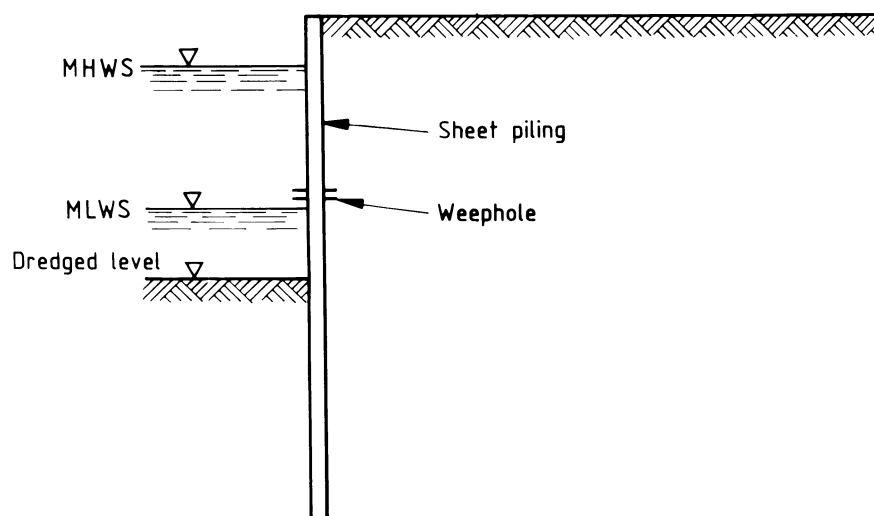
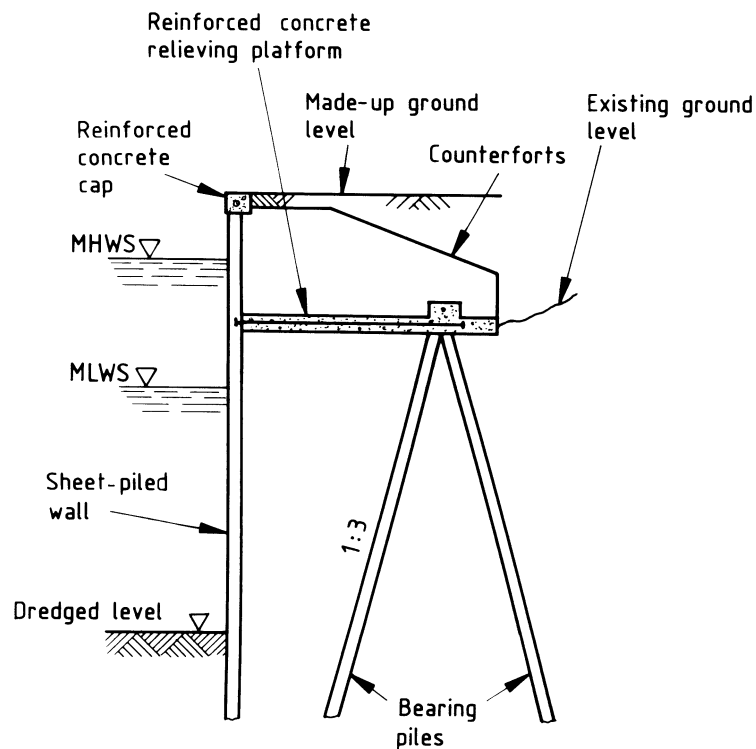


Figure 2 — Cantilevered sheet wall structure

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

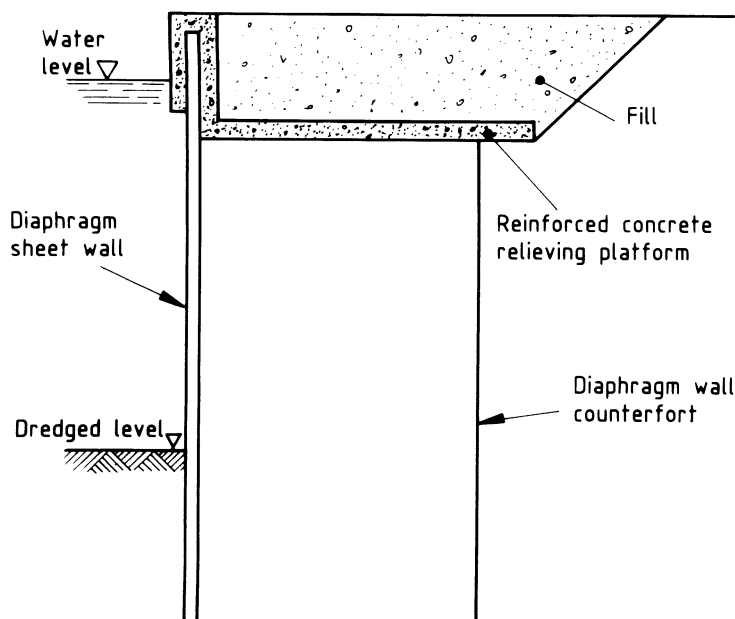
Figure 6 shows examples of interlocking steel pile sections for use in sheet walls.

The construction tolerances given in 6.13 may be applied to steel sheet piles.



Source: British Steel Corporation,
BSC Sections and Commercial Steels,
Scunthorpe.

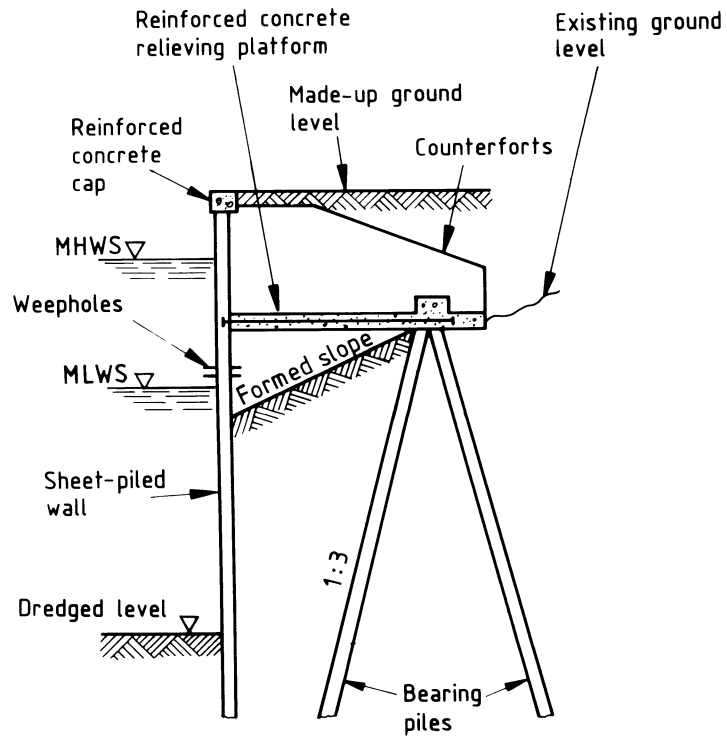
(a) Platform on bearing pile supports



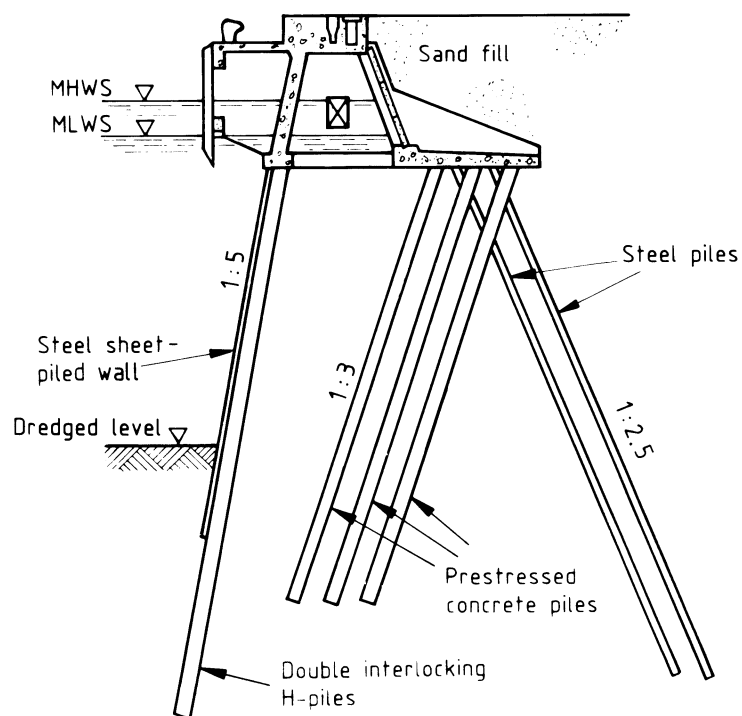
Source: ICOS (Great Britain) Ltd.

(b) Platform on diaphragm wall supports

Figure 3 — Sheet walls with relieving platform



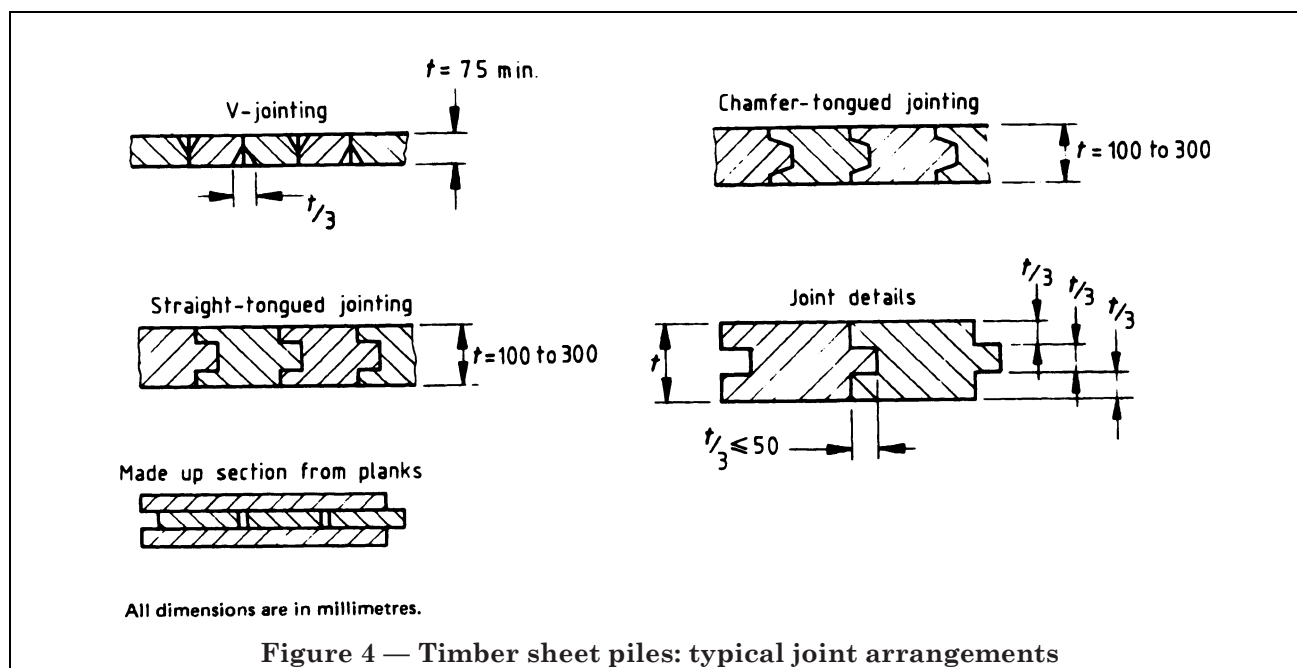
(c) Void under platform



Source: Rotterdam Public Works Administration/National Ports Council.

(d) Battered wall with partially hollow platform

Figure 3 — Sheet walls with relieving platform (concluded)



4.4.3 In situ concrete piled walls

Sheet 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 bentonite is used during excavation. Figure 7 shows typical arrangements.

In contiguous pile walls, care is needed to ensure that material is unable to escape through gaps between the piles. In secant pile walls, the piles are usually 1.0 m to 1.2 m in diameter, installed at centres equal to approximately 0.9 times the diameter: to ensure overlap, the verticality tolerance in the direction of the wall should generally not exceed 1 in 200. Construction tolerances for in situ concrete piles should, in other respects, be in accordance with 6.13.

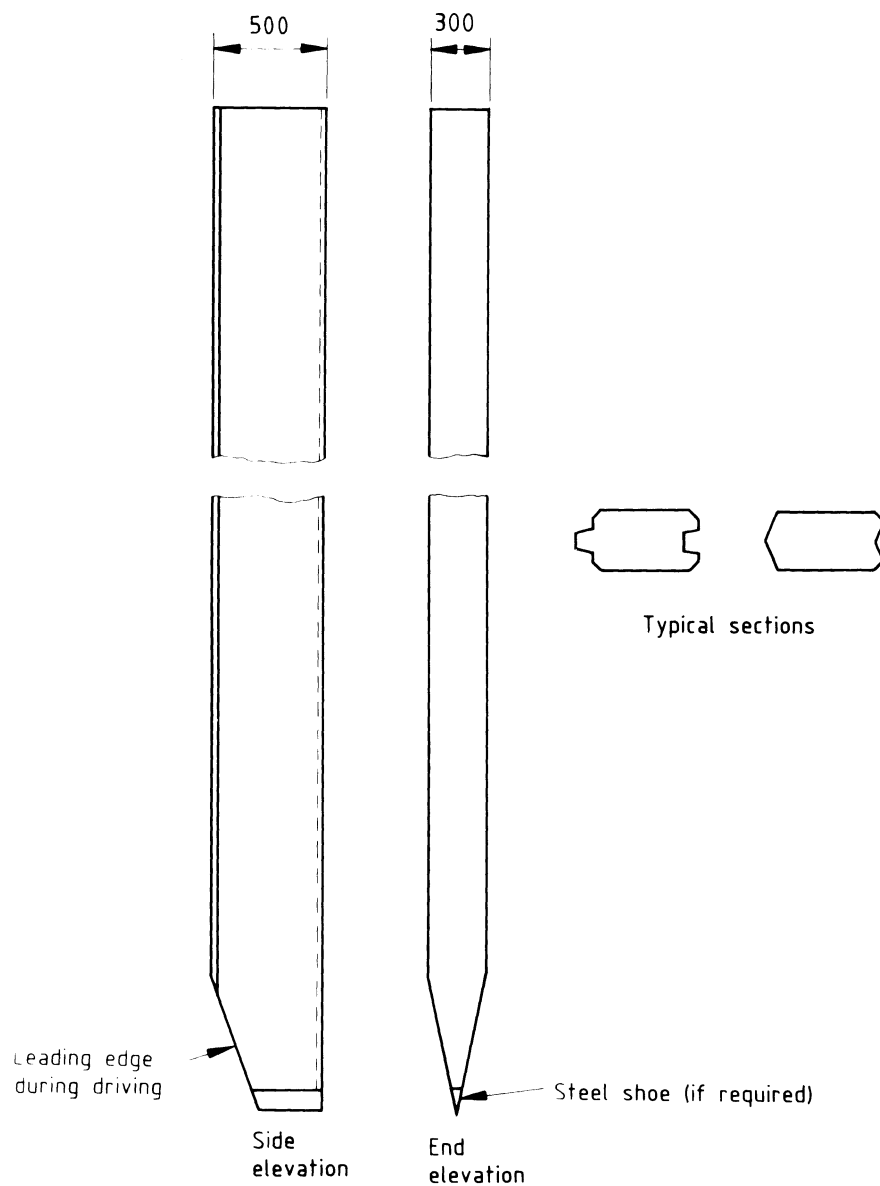
Concrete for in situ piled quay walls is usually placed by tremie. The surface finish of this type of wall is generally irregular, depending on the soil. It should, if necessary, be improved above water level by adding an in situ concrete facing, or by extending the capping down to this level.

4.4.4 Diaphragm walls

Sheet walls for in situ concrete diaphragm walling are often economical to build where there is a sufficient width of existing ground or artificial embankment available for use as a working area. They may 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. Diaphragm walls are used for high walls or where heavy vertical loads are imposed on the wall. They are particularly suitable at sites where the toe levels can be predetermined to within ± 0.5 m, to assist the advance fabrication of reinforcement cages. Figure 8 shows examples of typical panel arrangements.

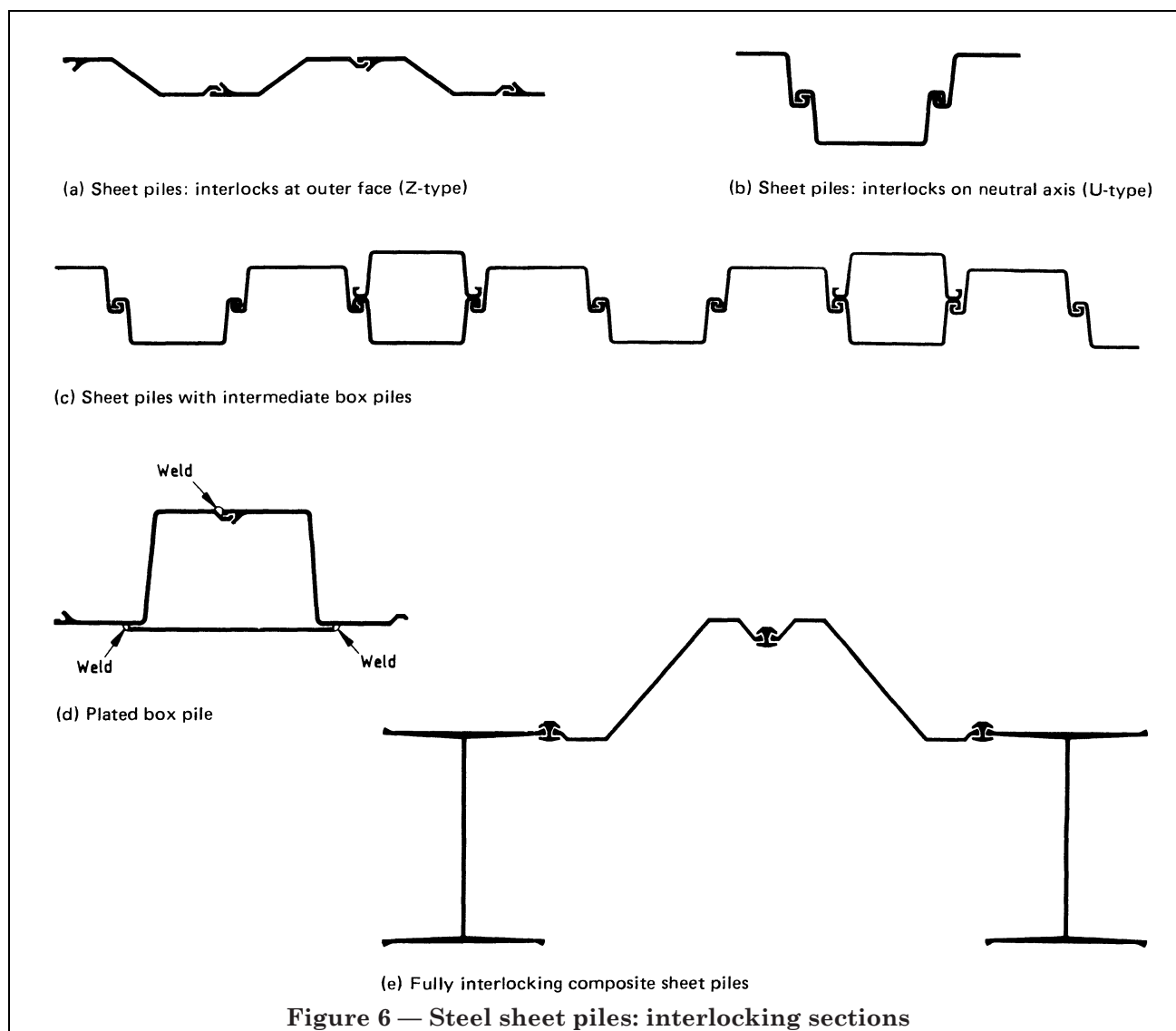
The ground level should be at least 1.0 m to 1.5 m above maximum ground water level to ensure stability of the bentonite-filled trench. Slurry trench construction is covered in clause 55 of BS 6349-1:1984.

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



All dimensions are in millimetres.

Figure 5 — Concrete sheet piles: typical details

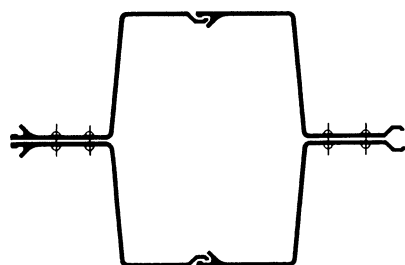


Diaphragm walls are usually of reinforced concrete, but prestressed walls can be built using tendons anchored at the top of the wall and looped at the base. Care is necessary in detailing the reinforcement to provide adequate space between bars [19]. A concrete cover of at least 75 mm should be adopted. The comments in 4.4.3 on surface finish also apply to diaphragm walls.

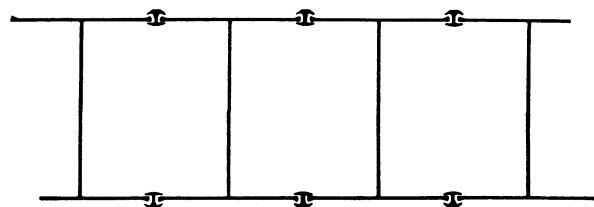
Joints between panels have usually, in the past, been formed using a circular stop-end. Square stop-ends incorporating a joggle to assist in achieving composite action between panels and in sealing the joint are now more generally available. This shape has the advantage, which is particularly important in short panels, of leaving a greater proportion of the panel length available for occupation by the reinforcement cage.

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

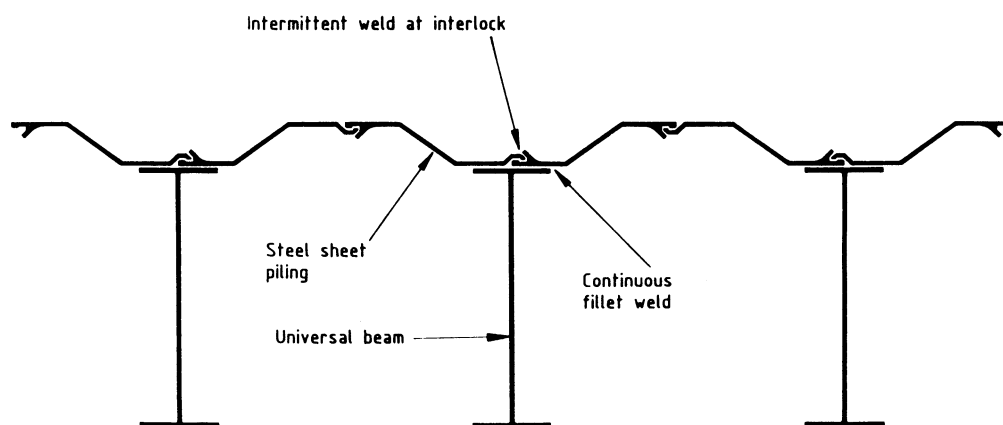
Reference [19] gives construction tolerances for diaphragm wall construction.



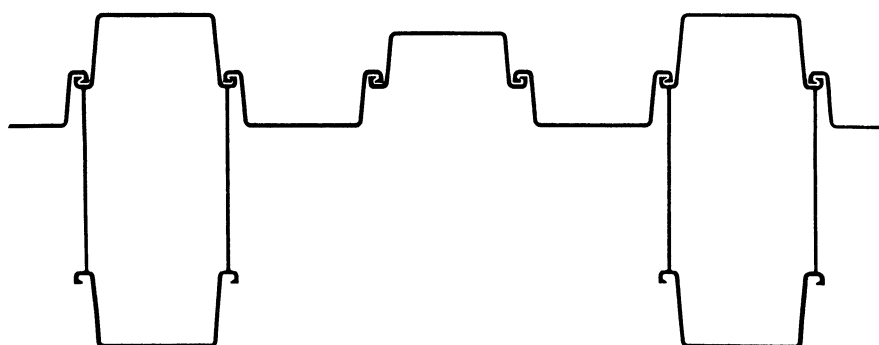
(f) Double box pile



(g) Interlocking beams and columns for continuous walls

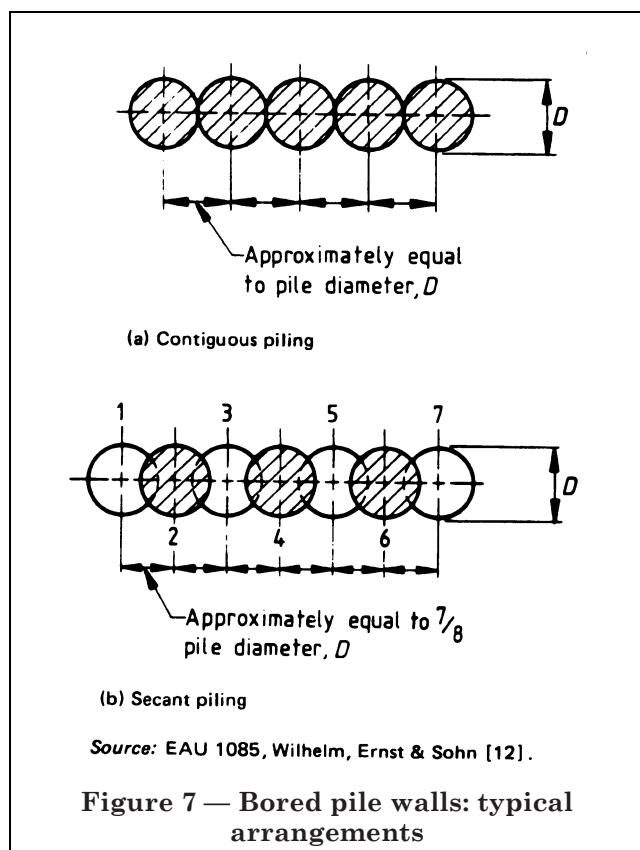


(h) High modulus composite sheet piles



(i) Sheet piles with intermediate composite box piles

Figure 6 — Steel sheet piles: interlocking sections *(concluded)*



4.4.5 Soldier piles and sheeting

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

The piles may be of steel or concrete, either precast or in situ; prestressing will normally be advisable to reduce weight and improve durability.

Preformed piles may be installed, by driving or placing in a preformed hole, in any seabed material, but may require an excessive penetration in soft clays and silts. In situ concrete piles are subject to the usual limitations of bored piles.

The penetration of the piles into the seabed should be carefully considered in view of the consequences of failure of the ground support. The heads of all piles should be temporarily supported until the capping is cast and the anchors installed.

The sheeting in quay walls of this type is usually of precast concrete planks. The sheeting should be secured to the soldier piles to prevent differential movement and consequent loss of material. Figure 9 shows an example of sandwich construction, in which two outer skins comprising concrete planks are used with an in situ concrete infill: this method overcomes the problem of sealing the joints to prevent loss of material, but does not permit drainage of the fill, which in most designs occurs through gaps between the planks.

Construction tolerances for soldier piles should be in accordance with 6.13.

4.5 Section modulus of wall

4.5.1 Steel sheet piles

The section modulus of sheet walls in vertical bending due to soil loads can, for most types of wall, be taken as equal to the full modulus, calculated assuming zero vertical slippage at interlocks.

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

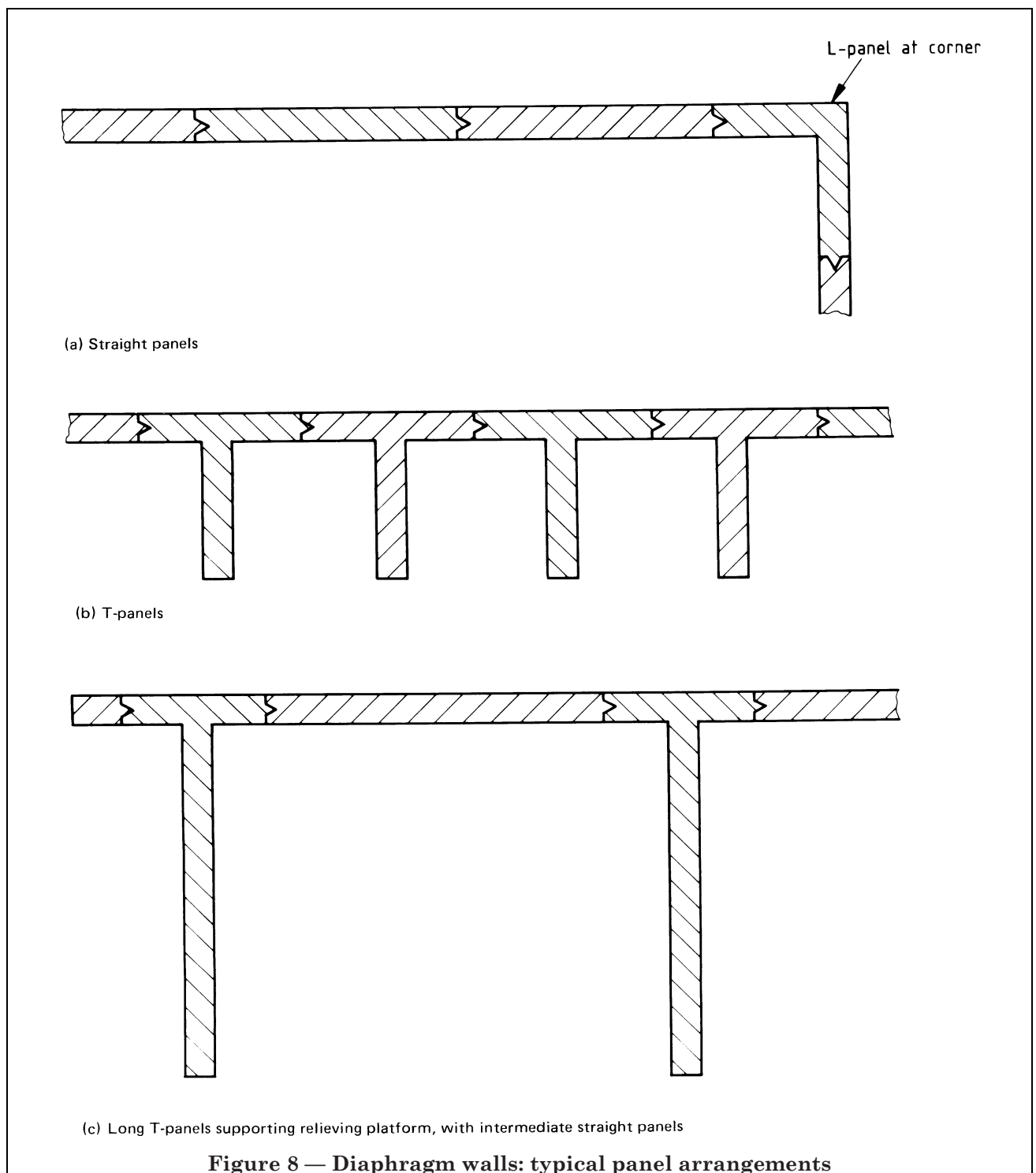
Consideration should be given to deflections which may arise whilst backfilling until the interlocks develop sufficient friction to enable the full combined modulus to be developed.

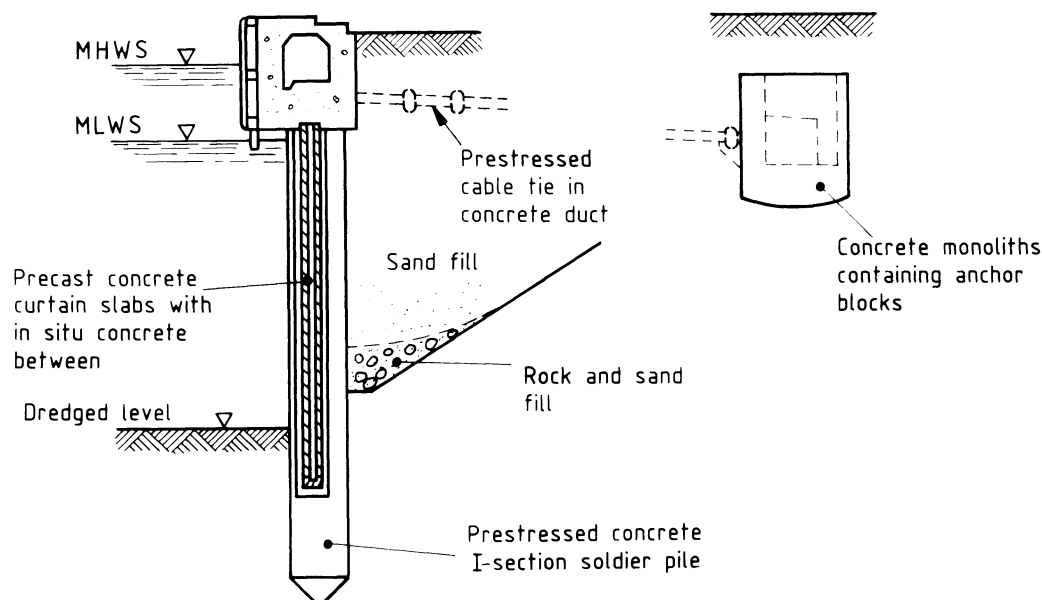
4.5.2 Composite steel sheet piles

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

4.5.3 Diaphragm walls

In diaphragm walls, the panel joints should be adequately keyed to ensure uniformity in distribution of soil pressure. It is not usual, however, to assume any transfer of load across joints when analysing the wall. Where T- or L-panels are used, the flanges of these elements should be designed as horizontal ribs or cantilevers, and the web/flange connection designed accordingly.





Source: Rendel, Palmer & Tritton/National Ports Council.

Figure 9 — Soldier-piled wall: typical cross section

T- or L- elements and straight panels in the same wall should each be designed using their own section moduli.

4.6 Materials and stresses

4.6.1 Materials

For a description of the materials used in the construction of sheet walls reference should be made to BS 6349-1.

4.6.2 Stresses

In the case of reinforced and prestressed concrete sheet walls, the working stresses obtained by analysis for normal loading conditions should be multiplied by a partial factor of not less than 1.5 to obtain the ultimate stresses in the wall. Analysis of the reinforced concrete should be carried out using limit state methods. Crack widths (see 2.4.4.3) should be calculated for the serviceability limit state.

The working stresses to be adopted in the design of steel and timber sheet walls under normal loading conditions should not exceed the maximum permissible stresses given in BS 8002. These stresses should be based on the wall section at the end of the design life.

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

In the case of steel walls, limit state analysis may be used as an alternative.

Partial factors for loads and factors of safety should, in general, be increased where soil conditions are not fully established or where ground conditions are variable. Under extreme loading conditions, the partial factors for loads and factors of safety may be reduced to not less than 1.2.

The recommendations given in BS 8002 regarding driving stresses in sheet piles should be followed.

Where bentonite is used in the excavation of in situ concrete sheet walls, the permissible bond stress of deformed steel reinforcement may have to be reduced [19].

Walings should be designed to similar stresses as given above for sheet walls.