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Code of practice for

Maritime structures —

Part 2: Design of quay walls, jetties and dolphins

Committees responsible for this British Standard

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- 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

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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/m² may be used to allow for this. Another common example is the use of bearing piles with temporary or incomplete decking to support plant: the nature of these 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.

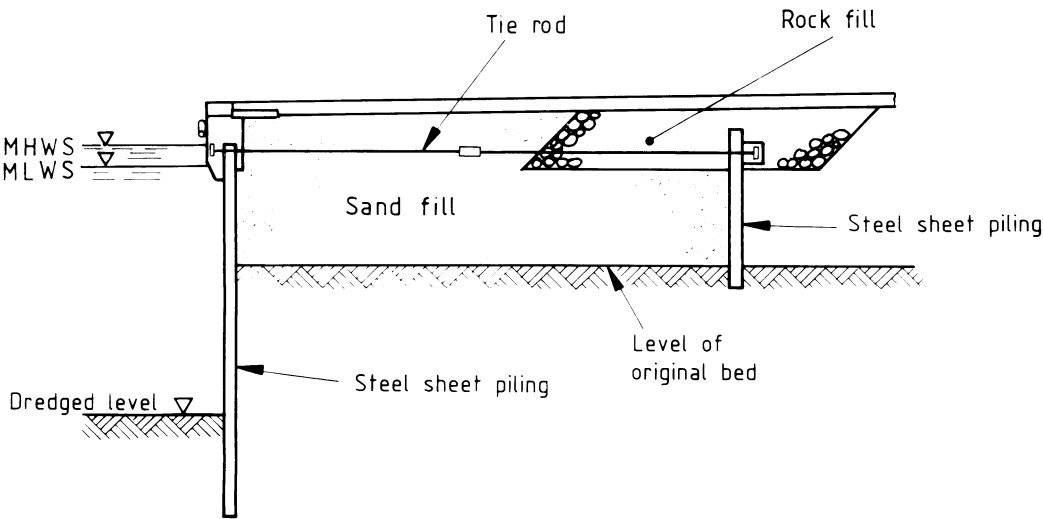
The construction tolerances given in 6.13 may be applied to concrete sheet piles.

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].

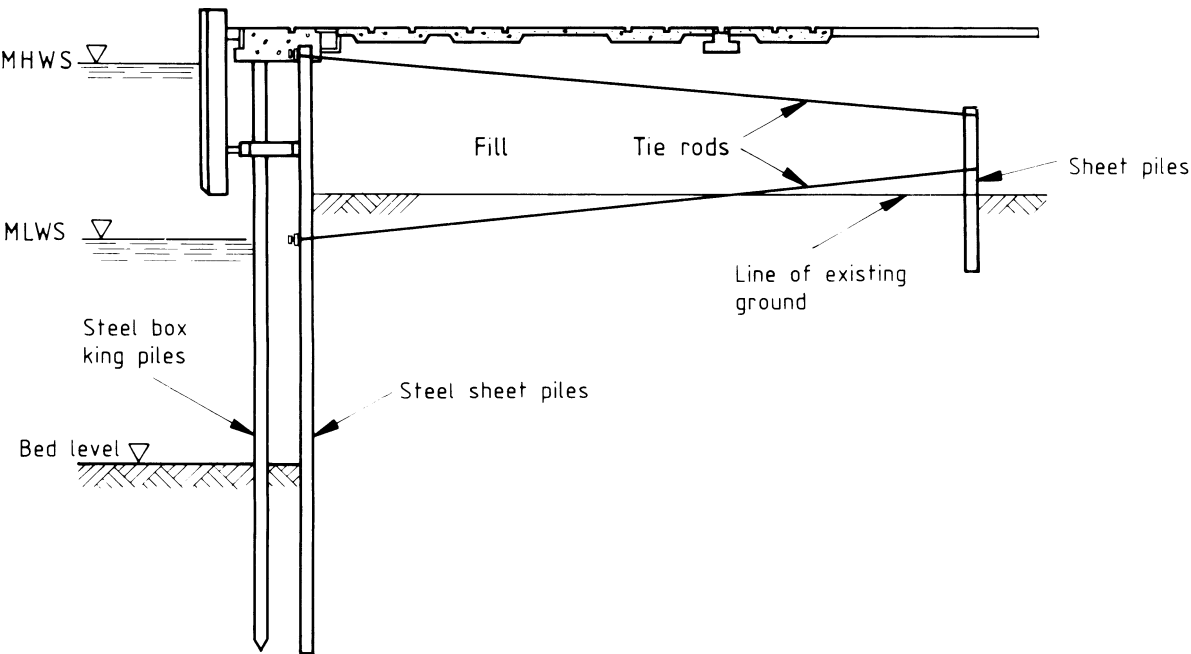
This type of pile can be driven to considerable depth with low displacement in a wide range of ground conditions and into weathered rock. Where hard driving is encountered, the pile section required may exceed the section needed to resist bending. This should be taken into account in assessing the reduction 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.

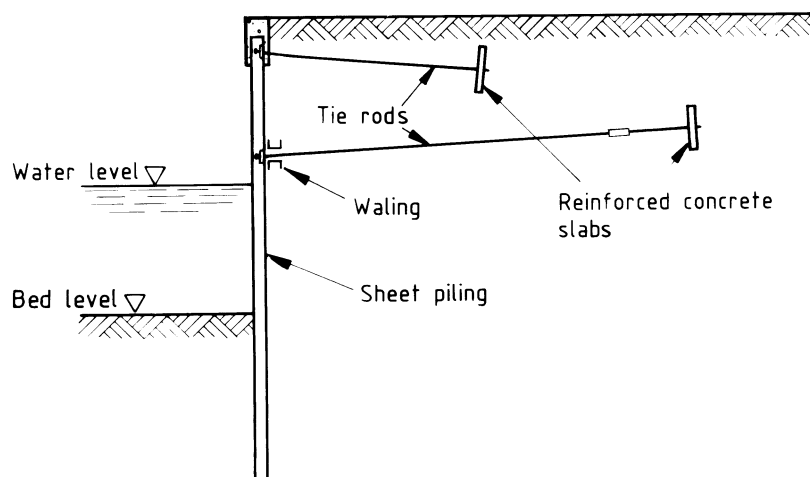


Source: Bertlin & Partners/National Ports Council.
(a) Anchored single-wall



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)

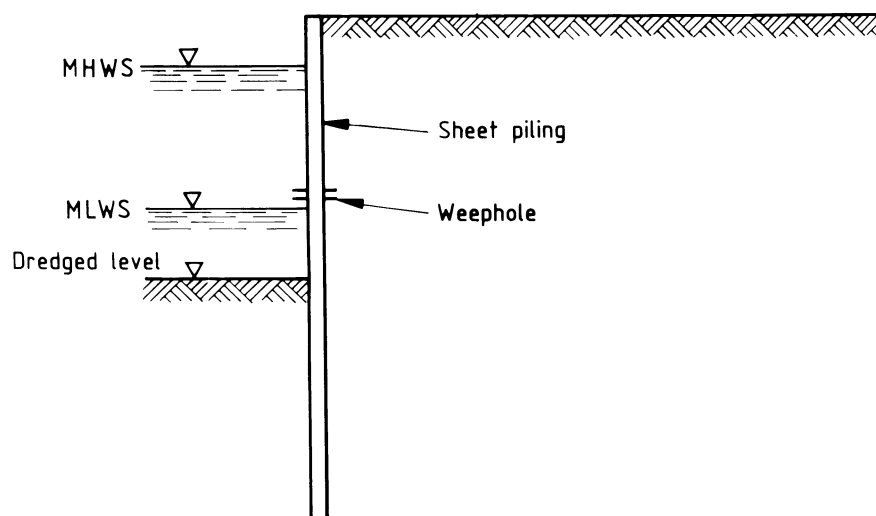
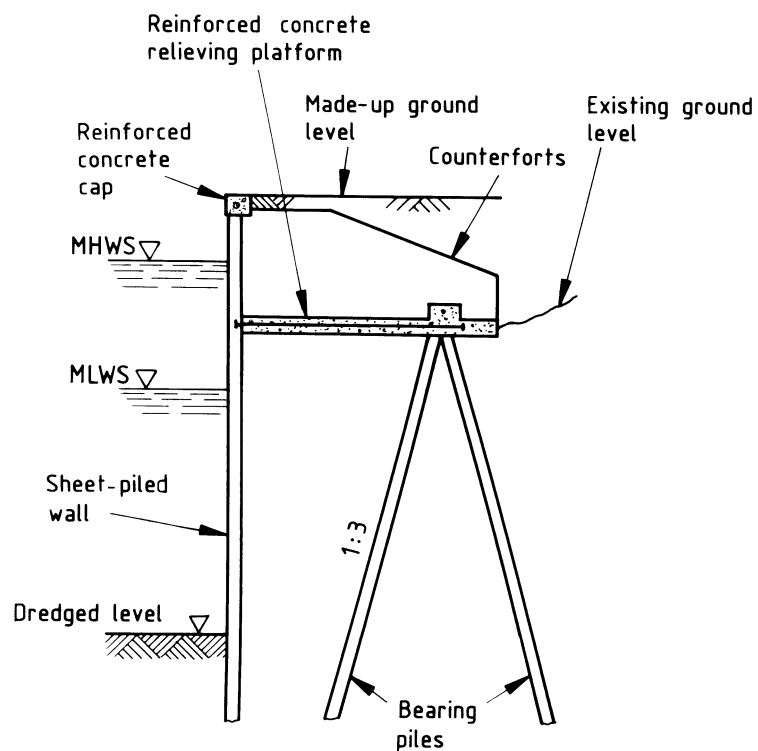


Figure 2 — Cantilevered sheet wall structure

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.

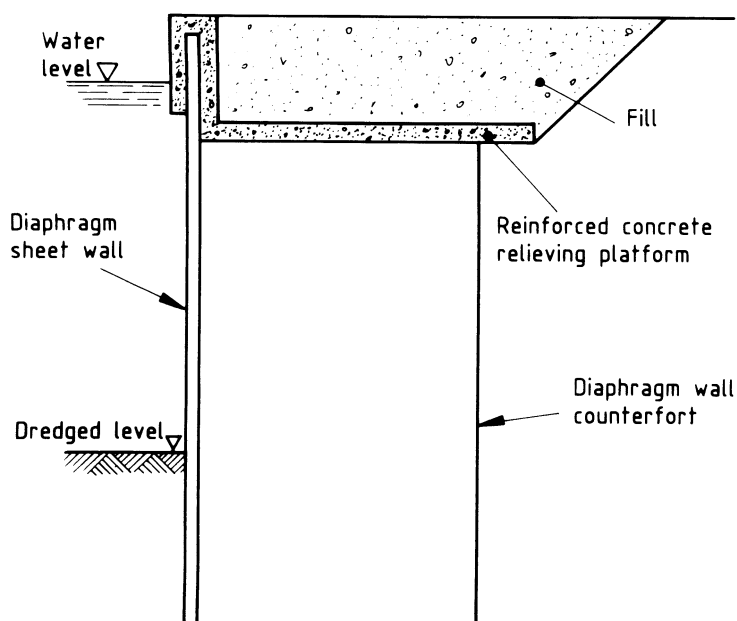
Figure 6 shows examples of interlocking steel pile sections for use in sheet walls.

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



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

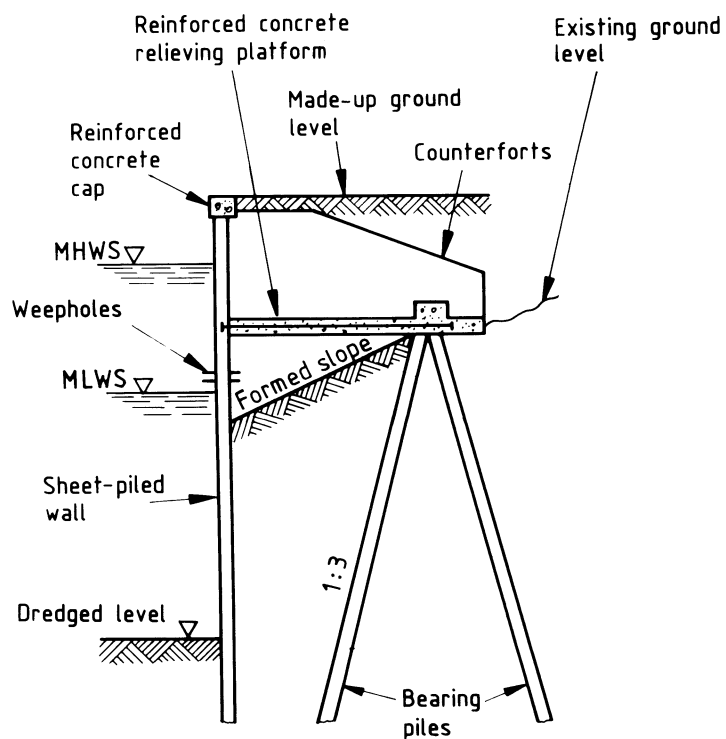
(a) Platform on bearing pile supports



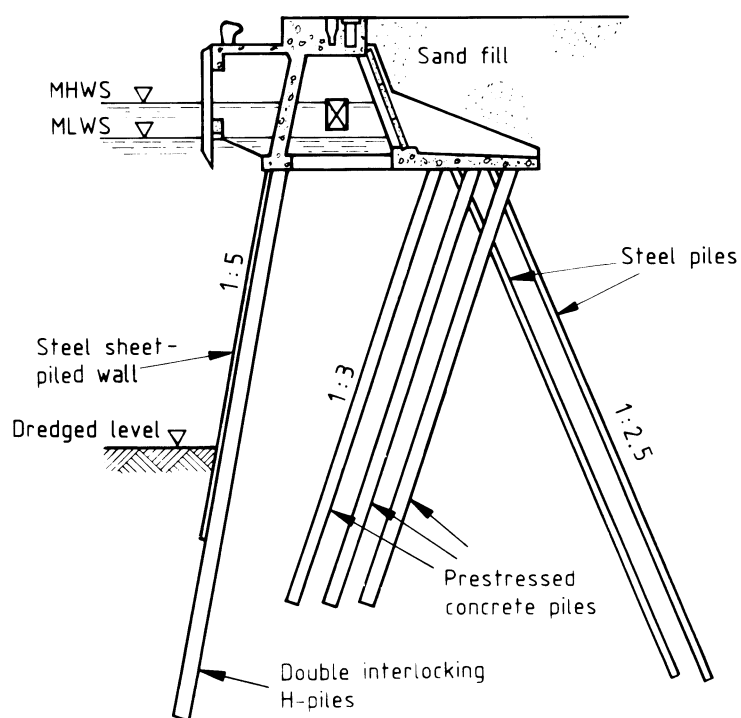
Source: ICOS (Great Britain) Ltd.

(b) Platform on diaphragm wall supports

Figure 3 — Sheet walls with relieving platform



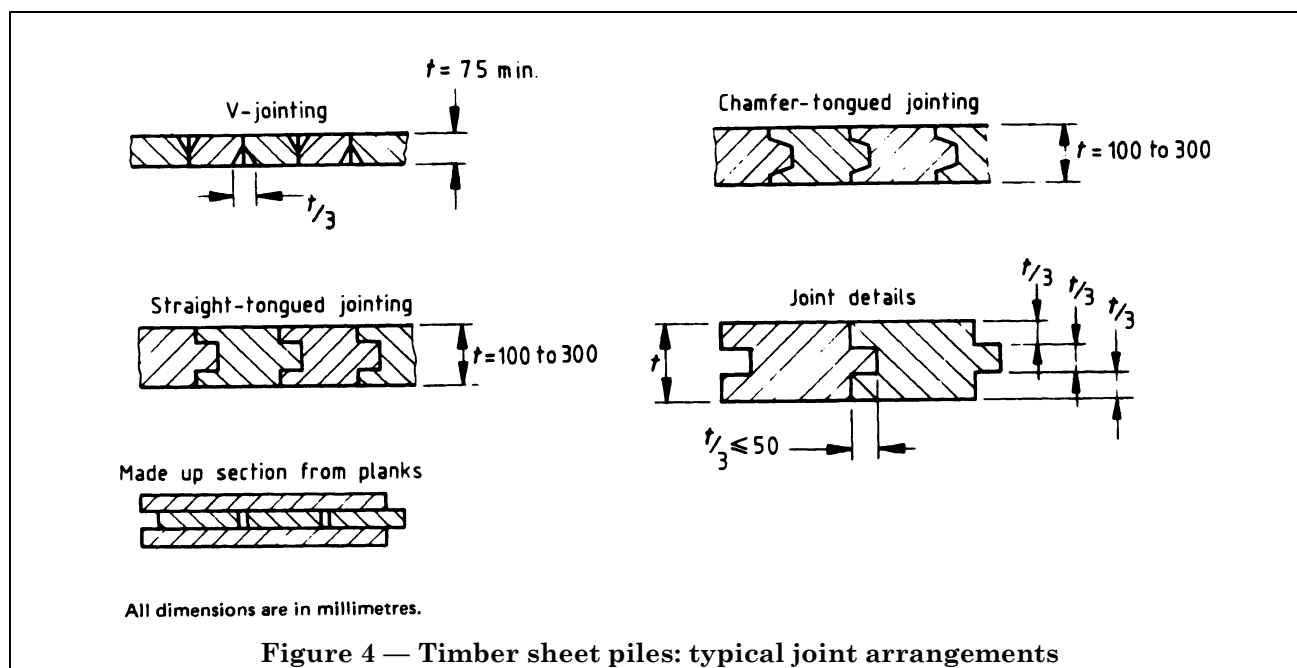
(c) Void under platform



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.

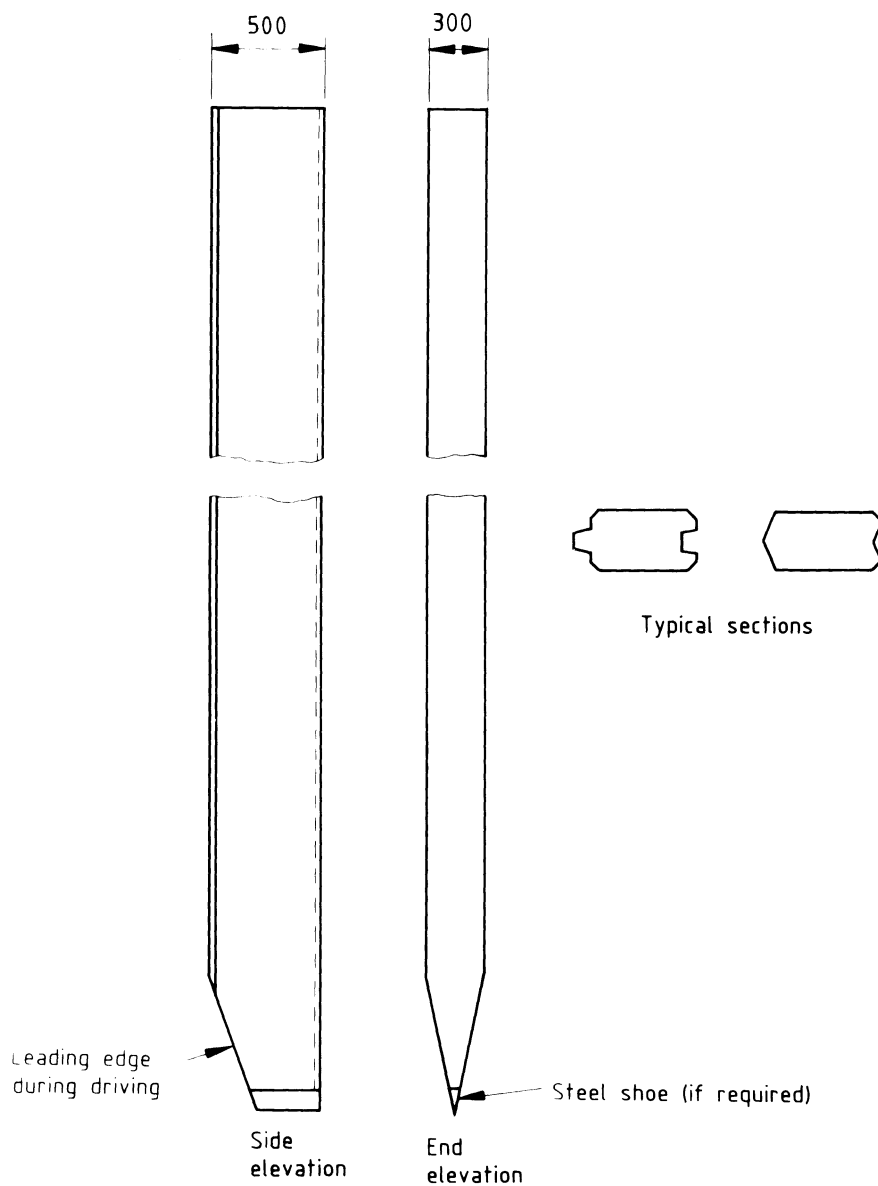
Concrete for in situ piled quay walls is usually placed by tremie. The surface finish of this type of wall is generally irregular, depending on the soil. It should, if necessary, be improved above water level by adding an in situ concrete facing, or by extending the capping down to this level.

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.

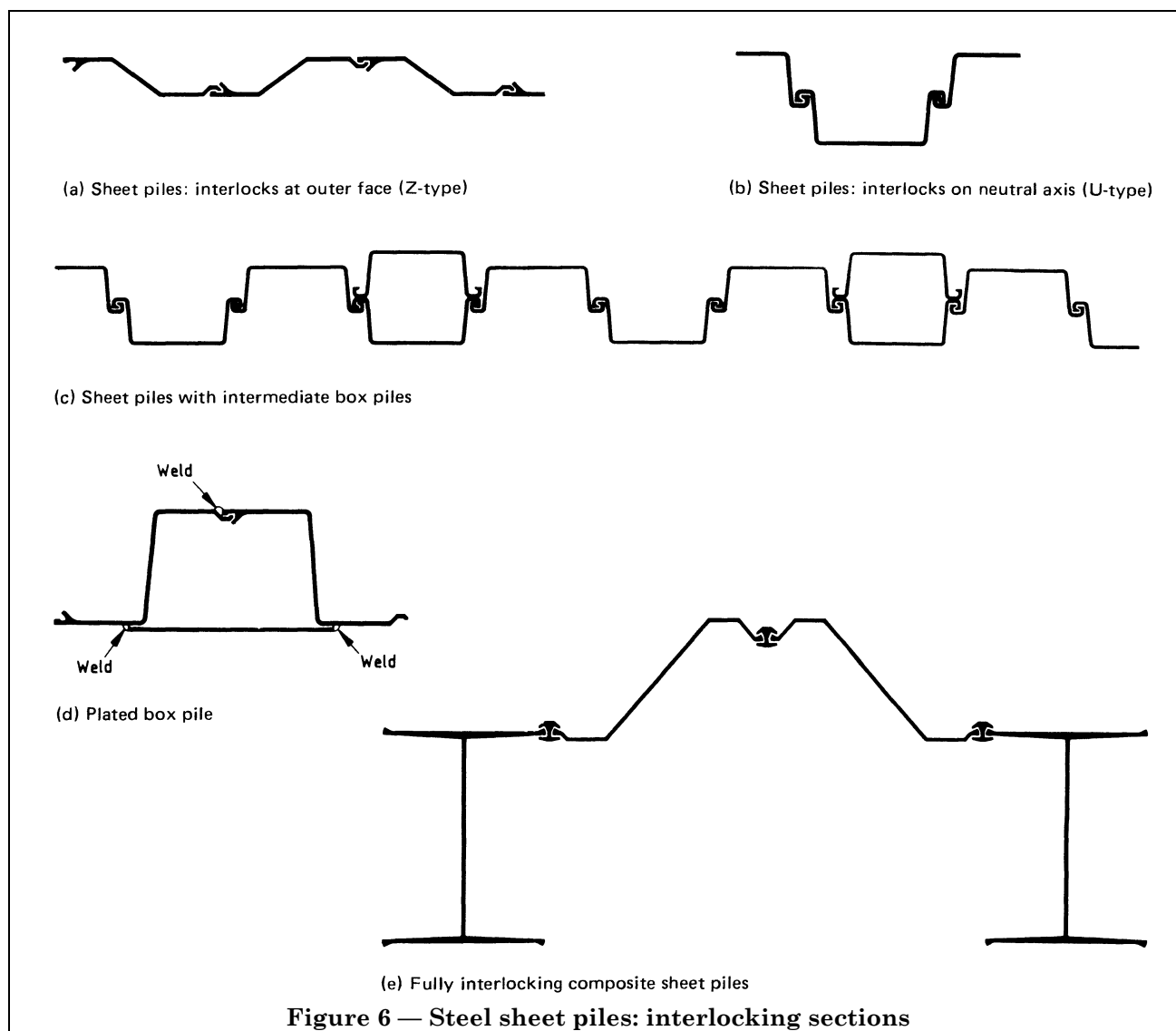
The ground level should be at least 1.0 m to 1.5 m above maximum ground water level to ensure stability of the bentonite-filled trench. Slurry trench construction is covered in clause 55 of BS 6349-1:1984.

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

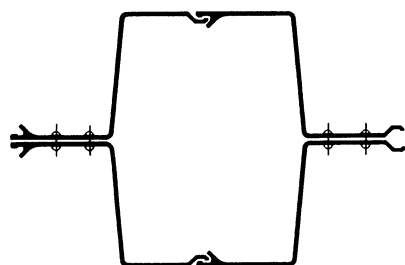


Diaphragm walls are usually of reinforced concrete, but prestressed walls can be built using tendons anchored at the top of the wall and looped at the base. Care is necessary in detailing the reinforcement to provide adequate space between bars [19]. A concrete cover of at least 75 mm should be adopted.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

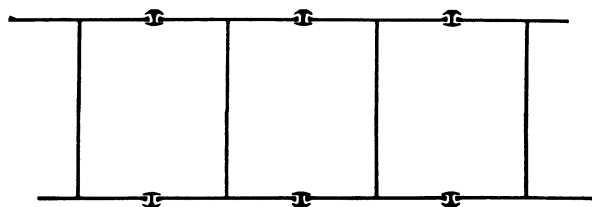
Joints between panels have usually, in the past, been formed using a circular stop-end. Square stop-ends incorporating a joggle to assist in achieving composite action between panels and in sealing the joint are now more generally available. This shape has the advantage, which is particularly important 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.

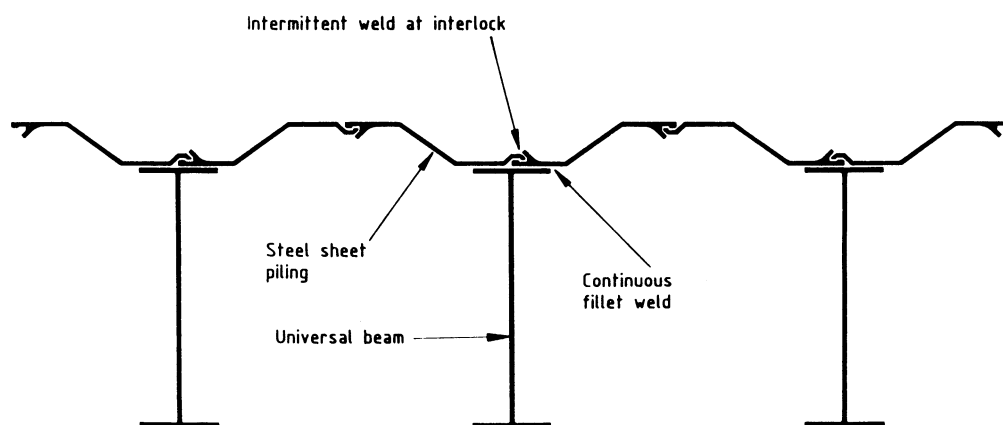
Reference [19] gives construction tolerances for diaphragm wall construction.



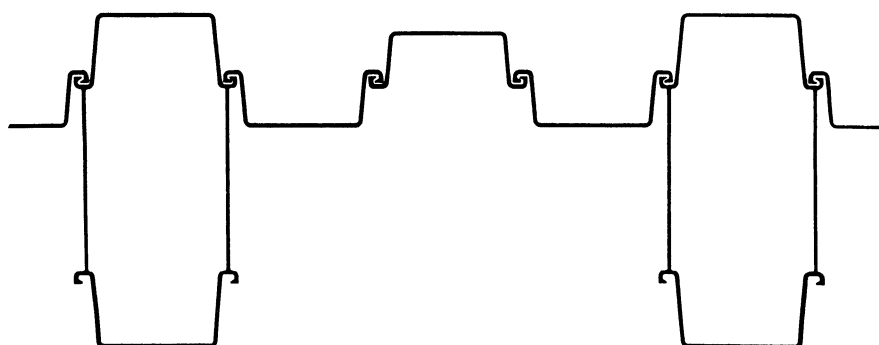
(f) Double box pile



(g) Interlocking beams and columns for continuous walls

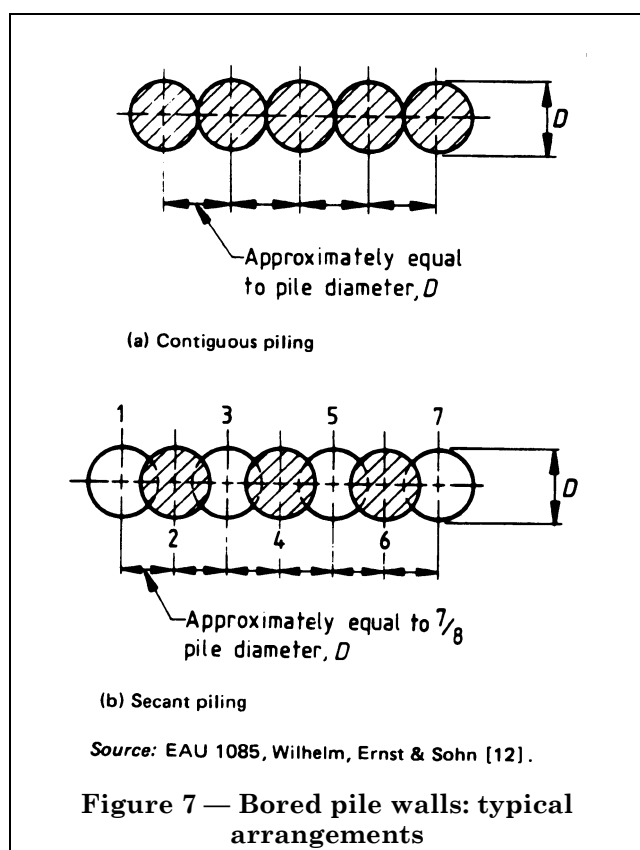


(h) High modulus composite sheet piles



(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.

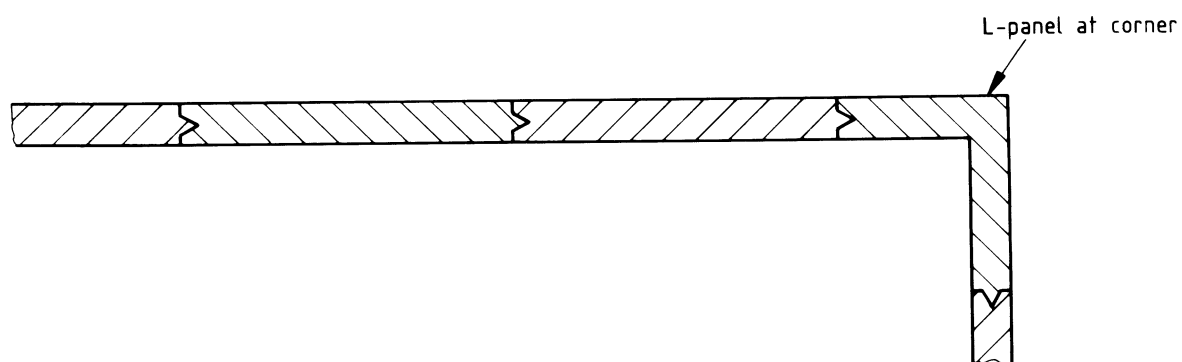
Consideration should be given to deflections which may arise whilst backfilling until the interlocks develop sufficient friction to enable the full combined modulus to be developed.

4.5.2 Composite steel sheet piles

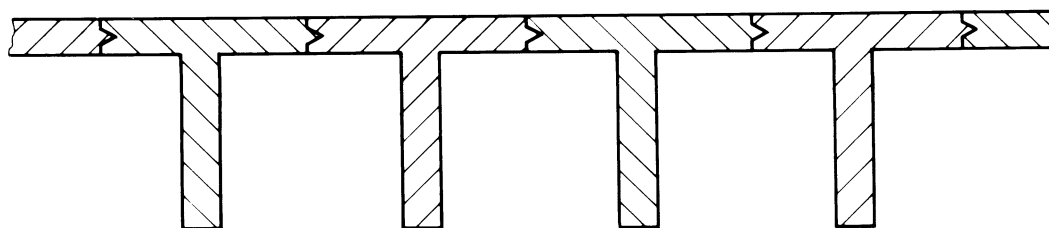
In composite steel sheet-piled walls in which the intermediate sheeting is required to contribute to the bending resistance, welding of the interlocks will in many cases be required. If the modulus of the H-section, box or tube far exceeds that of the intermediate sheeting, it may be assumed that all 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

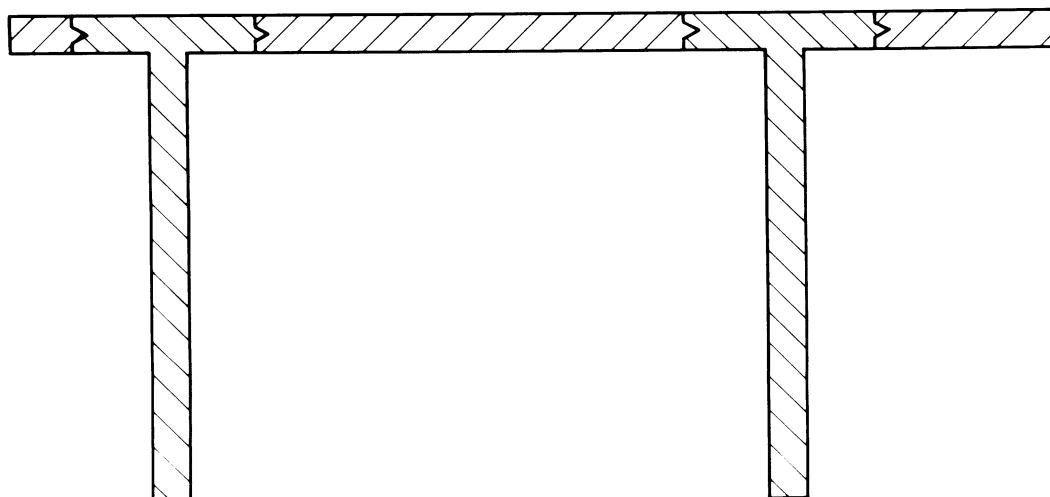
In diaphragm walls, the panel joints should be adequately keyed to ensure uniformity in distribution of soil pressure. It is not usual, however, to assume any transfer of load across joints when analysing the wall. Where T- or L-panels are used, the flanges of these elements should be designed as horizontal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



(a) Straight panels

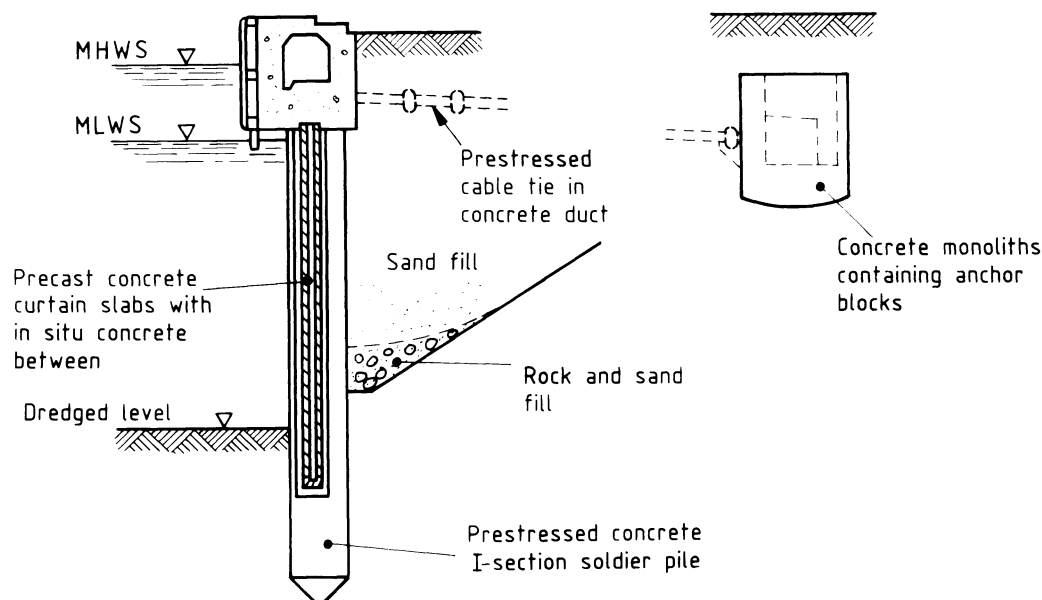


(b) T-panels



(c) Long T-panels supporting relieving platform, with intermediate straight panels

Figure 8 — Diaphragm walls: typical panel arrangements



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.

4.7 Design of sheet wall structures

4.7.1 Anchored or propped single-wall structures

4.7.1.1 Single-anchored or propped walls. Design of single-anchored or propped walls may be carried out in accordance with BS 8002 as illustrated in Figure 10 to Figure 12 (see 4.7.1.2 for alternative design methods).

The pressure distribution shown in Figure 10(a) represents a condition of free-earth support in which the rigid anchored wall yields by forward translation. The effects of flexibility in causing arching of the soil are shown in Figure 10(b). Where the sheet piles are driven deeply [see Figure 10(c)], fixity occurs in the lower portions of the piles giving a reversal of curvature in the piling.

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

Arching conditions should be taken into account in relation to the sequence of construction of an anchored sheet-piled retaining wall. When backfilling is completed before the soil in front of the wall is dredged away, the movement of the wall due to pressure from the small retained height of soil may be insufficient to develop active pressure conditions. As dredging takes place, the wall will yield and the pressure distribution will change from arched conditions at the upper level to the final assumed linear active condition. The initial and final stages are shown in Figure 11.

If, however, the soil is dredged away before placing any fill behind the wall, a linear active pressure distribution will develop over the depth within the existing ground after completion of dredging. When backfill is placed and the part above ground water level is compacted, the additional yielding of the wall and anchorages may not be sufficient to develop active earth pressures from the upper fill. In this case, provision should be made for lateral pressures from the upper part of the fill at a state intermediate between the active and at-rest condition, depending on the expected forward movement of the wall as the fill is placed (see Figure 12).

For the purpose of calculating the total active soil forces per unit length of wall P_A the active pressure distribution in terms of effective stresses should be assumed to be linear as shown in Figure 10(a).

However, for the purpose of determining the stability of the wall against overturning and for calculating anchorage forces and bending moments in the sheet wall, the effect of flexibility of the structure should be taken into account. With a flexible wall, the anchorage forces are higher and the bending moments between anchorage and seabed are lower than those calculated assuming a linear stress distribution. The wall flexibility is reduced where the wall is built into a stiff concrete capping beam. The effects of flexibility, which are to increase the anchorage forces and to raise the level of the resultant passive resistance, should be considered as shown in Figure 10(b) and Figure 10(c).

Water pressures (see 51.5 of BS 6349-1:1984) should be considered separately and added to the earth pressure for the purpose of obtaining the total lateral force on the structure.

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

The design of non-vertical sheet walls should take into account the batter of the wall, using modified earth pressure coefficients; see Figure 10.4 of [10].

Methods of calculating anchorage forces and bending moments, shear forces and deflections in the structure, taking account of flexibility, are to be found in BS 8002 for both the free-earth support method and the fixed-earth support method.

The overall stability against sliding of the structure and the mass of earth retained by and beneath the structure should be considered. Sliding may be rotational, planar, or on a composite failure plane. Methods of calculating the stability against sliding of earth masses in any of these failure modes are described in BS 6031.

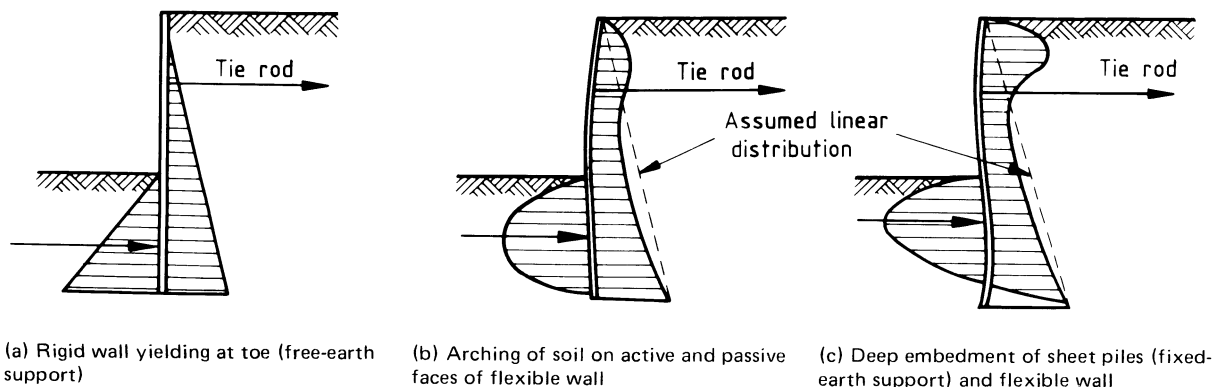


Figure 10 — Effects of flexibility of single-anchored or propped sheet wall structure on earth pressure and resistance

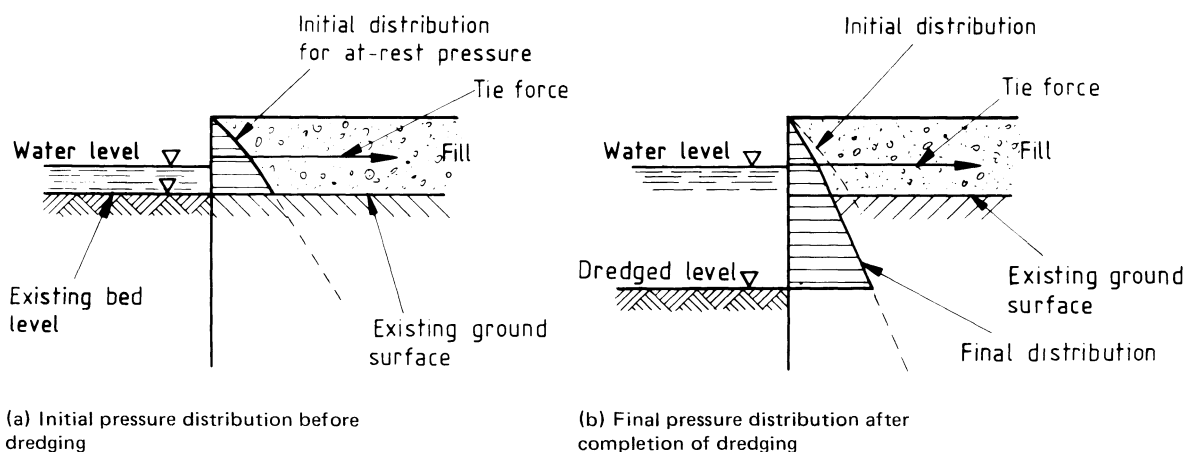


Figure 11 — Active pressure distribution on single-anchored or propped sheet wall structure where fill is placed before dredging

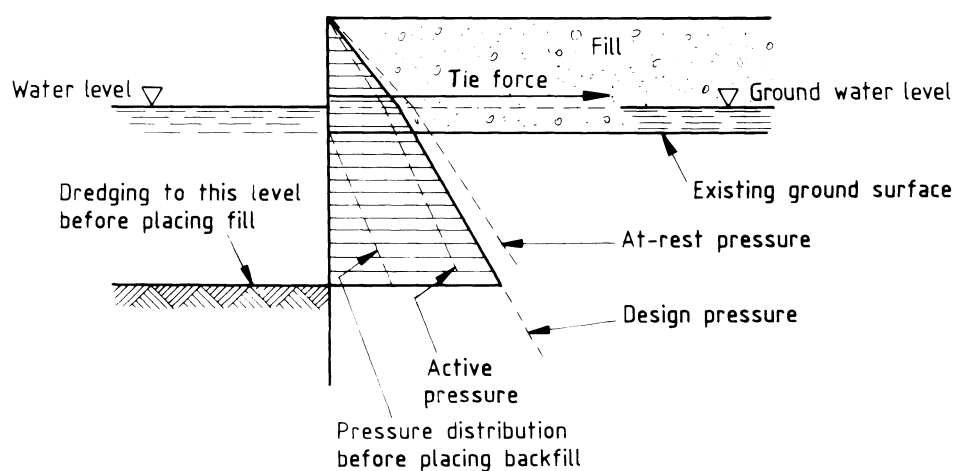


Figure 12 — Active pressure distribution on single-anchored or propped sheet wall structure where fill is placed after dredging

For cohesive soils, the long term considerations of stability are likely to be critical and the analyses should be made in terms of effective stresses under equilibrium conditions of pore water pressure, except where dredging is carried out in front of the wall, in which case, total stresses may govern, depending upon the speed of construction. In cohesive soils, the effects of soil shrinkage causing tension cracks within and landward of the anchorage zone should be considered. The effects of scour and dredging below the nominal design depth should be considered in relation to the depth of embedment required to mobilize passive resistance. It may be necessary to specify a maximum depth for overdredging. In cohesive soils, removal of overburden by dredging will cause swelling and softening of an over-consolidated clay, which should be taken into account in the selection of soil parameters.

When considering the stability against sliding of the structure and earth masses, the mooring forces and surcharge loads (see 51.4 of BS 6349-1:1984) on the structure should be included, where these act as disturbing forces. The effects of earthquakes on stability should be considered in areas subject to seismic activity.

The weight of the soil that lies above the lower failure plane should be included in the disturbing forces in relation to the length and inclination of the anchors (see Figure 13). Where cantilever anchorages are used, the lower failure plane may be assumed to intersect the anchorage at the lower point of zero shear.

4.7.1.2 Alternative design methods for single-anchored or propped walls. Methods other than that given in BS 8002 for the design of sheet walls may be adopted. Rowe's moment reduction method [20] may be used for flexible steel sheet pile walls, but not for other types of sheet wall. The method involves analysis of the wall assuming free-earth support, and application of bending moment reduction factors to take into account the flexibility of the wall related to the embedment in the soil. This method is likely to give a more economical design than Blum's modified equivalent beam method and is suitable for use in most soils except over-consolidated clays and backfilled walls, where factored earth pressures should not be used because of the high lateral stresses which occur. Care should be taken, however, to ensure that the wall deflections obtained by this method are not excessive.

Another method of analysis is that of Brinch Hansen [21]. This is an empirical ultimate load method which assumes the formation of yield hinges at failure.

For stiff wall sections, it is recommended that computer analysis is carried out using the values of horizontal modulus of subgrade reaction of the soil. Most of the existing methods for this approach are derived from [22].

4.7.1.3 Design of double-anchored walls. Yielding of double-anchored walls will, in practice, occur in a similar way to that described in 4.7.1.1 and the pressure distribution may then be modified in accordance with either a) or b) as follows.

a) *Without pressure redistribution.* If no redistribution of pressures is assumed, the approximate method given in [23] and illustrated in Figure 14(a) may be adopted to analyse the structure:

In Figure 14(a), the deflection Δ_2 of a double tie wall at the proposed level of the second tie is calculated. Then the tension T_2 in the lower tie is that force, applied at the lower tie rod level, that is necessary to produce an equal but opposite deflection Δ_2 in the single tie wall. The wall can be treated as a simple span between the upper tie rod and the resultant R_p of the passive resistance.

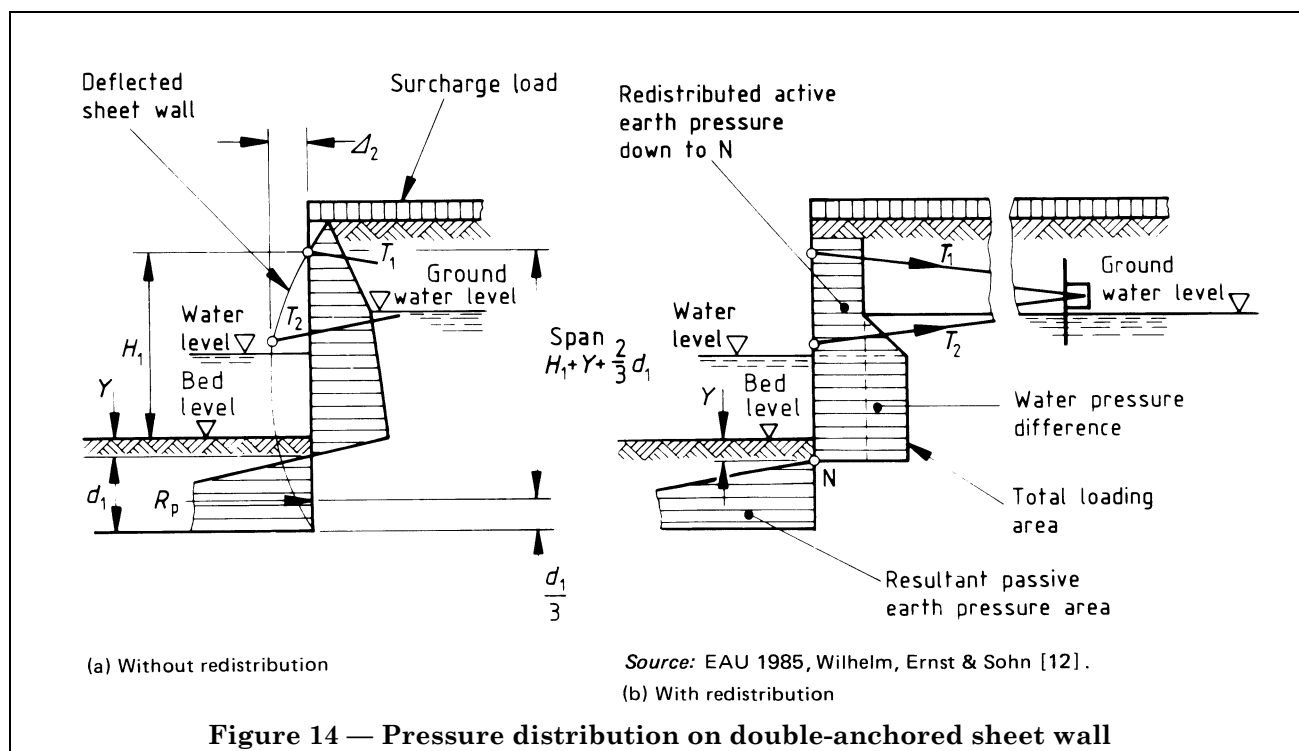
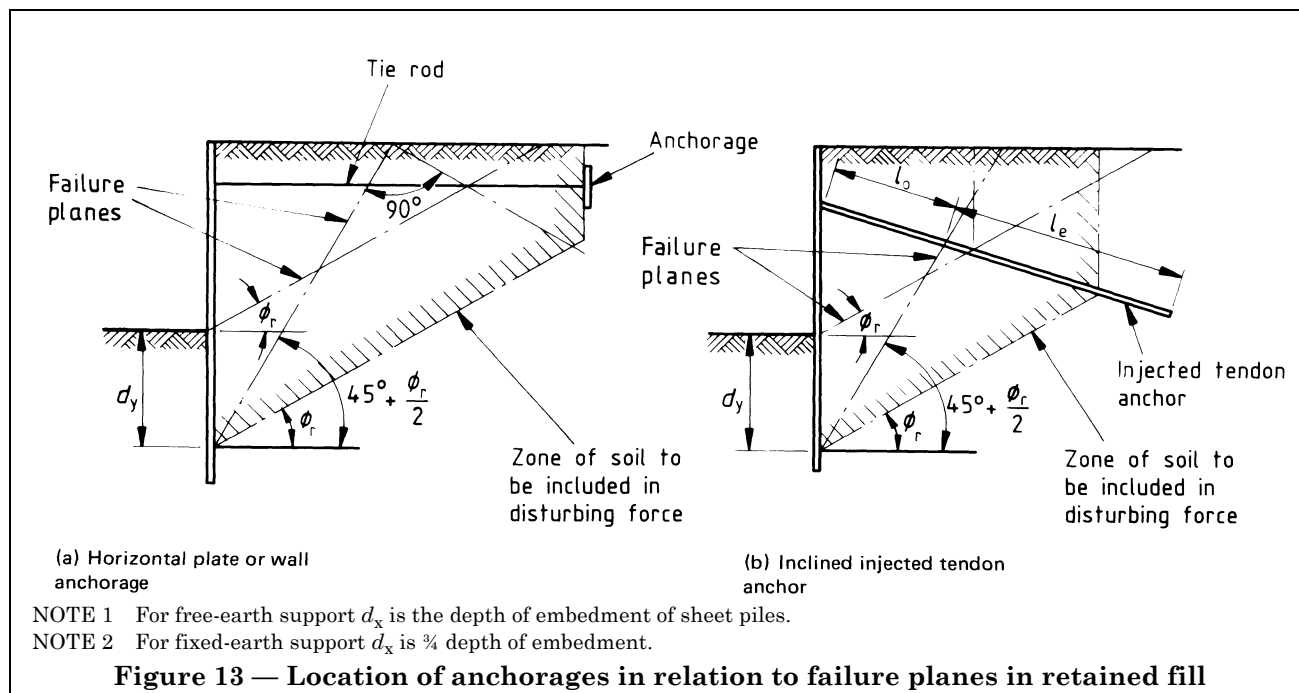
If backfilling is commenced before the installation of the lower tie rod, an initial deflection Δ' will occur in the wall. Consequently, the calculated force in the lower tie rod will be reduced by the ratio $(\Delta_2 - \Delta')/\Delta_2$, since the actual deflection Δ' cannot be counteracted.

The reduction of tension in the top tie rod due to the introduction of the lower tie rod is equal to the reaction of the simple span mentioned above with a single point load equal to the tension in the lower tie rod. Allowance should be made for the increased tension in both tie rods if they are inclined. Once T_1 and T_2 are determined, the depth of penetration can be revised by statics; however, it is wise to keep in mind that the above methods are only approximate. It is recommended that any reduction factor be omitted if a wall has two or more tiers of tie rods, and that the tie rod loads calculated be increased by 15 %.

b) *With pressure redistribution.* An approximate method involving redistribution of pressures is given in [12] : Figure 14(b) shows how redistribution takes place. This is particularly applicable to walls built in existing high ground and where the tie-backs are prestressed. A more accurate method is to use the modulus of horizontal subgrade reaction of the soil.

The top anchor and waling should be capable of resisting bollard pulls and surcharge loads applied nearby.

The overall stability against sliding of the wall and the surrounding earth masses should be considered as described in 4.7.1.1, taking the total net anchor load as the resultant of the two tie-back loads.



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

The stability of the auxiliary anchorage should be checked for the failure planes shown in Figure 15, and as described in 4.7.1.1.

4.7.2 Cantilevered single-wall structures

The distribution of active earth pressure and passive earth resistance on a cantilevered sheet wall is shown in Figure 16(a). The sheet wall should be embedded sufficiently to obtain fixed-earth support conditions at the toe: this type of wall is therefore unsuitable for use in cohesive soils, which weaken in the long term. For the purpose of this calculation the wall should be assumed to be rigid. The pressure distribution can be simplified as shown in Figure 16(b) when the force represented by the area CDE of Figure 16(a) can be replaced by a single horizontal force F_3 acting at the point A, and the line AB is assumed to be horizontal. The value of the force F_3 is such that the algebraic sum of the forces F_1 , F_2 and F_3 is zero.

The methods given in BS 8002 for calculating bending moments, shear forces and deflections in the sheet wall should be used.

The overall stability against rotational failure of the wall and the surrounding earth masses should be considered as described in 4.7.1.1.

4.7.3 Sheet wall with relieving platform

The lateral soil pressures acting on sheet walls built in front of a relieving platform conforming to the geometrical arrangement shown in Figure 3(a) may be assumed to be relieved by the platform.

Figure 17(a) shows the assumed theoretical pressure distribution for the case of a rigid wall with free-earth support. The pressure distribution will, in practice, be modified according to the flexibility of the sheet wall, the closeness of bearing piles under the platform, the toe level of the bearing piles, the degree of fixity at the wall/platform junction and the rigidity (which may generally be assumed to be infinite) of the raking pile anchorage. Relatively rigid walls are unlikely to yield sufficiently to cause active pressures to occur on the rear face, and at-rest conditions may then apply.

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

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

Active pressures acting on the back of the sheet wall should be calculated using trial wedges (e.g. Culmann wedges) to take into account the effects of the bearing piles. The landward boundary of the wedges is assumed to be the vertical plane passing through the rear of the platform [see Figure 17(b)]. Where an existing sheet wall lies on this plane, the same boundary should be assumed.

Where a void exists at the underside of the relieving platform, the theoretical pressure distribution on the back of the sheet wall may be assumed to be as shown in Figure 17(c)].

The overall stability against sliding of the wall and the surrounding earth masses should be considered as described in 4.7.1.1. The total pressure acting on the back of the relieving platform structure should be calculated neglecting any relief afforded by the platform or the piles. The effect of the relieving platform bearing piles on overall stability should be taken into account. The platform and piles should be designed in accordance with the recommendations given in section 6.

The overall stability of the diaphragm wall variant shown in Figure 3(b) should be assessed as described in section 5.

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

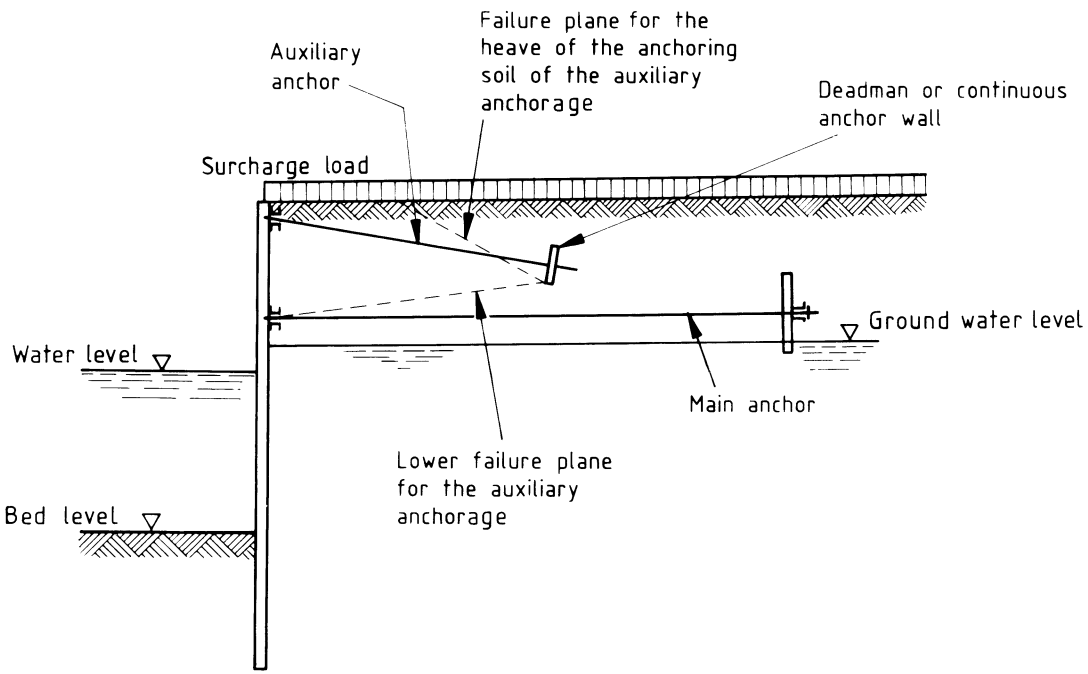


Figure 15 — Sheet wall with auxiliary anchorage: failure planes

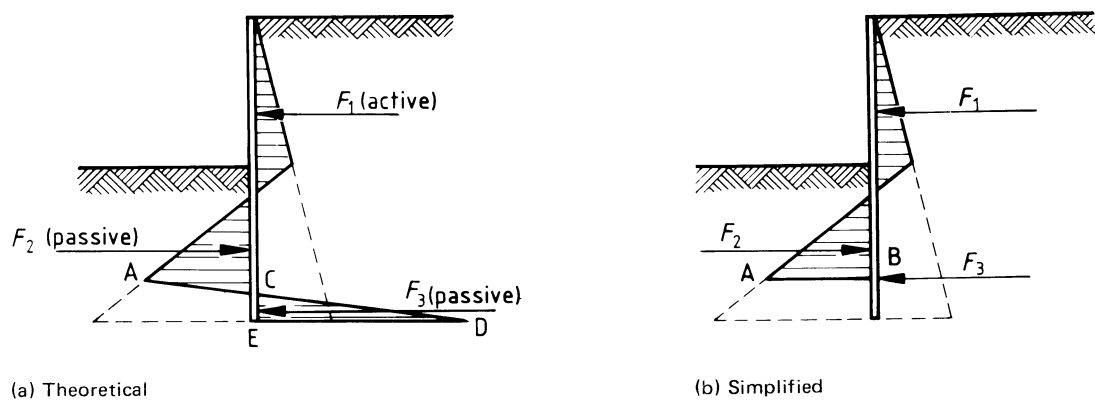
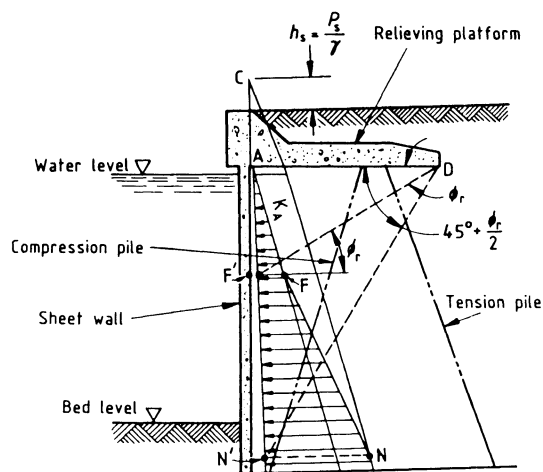
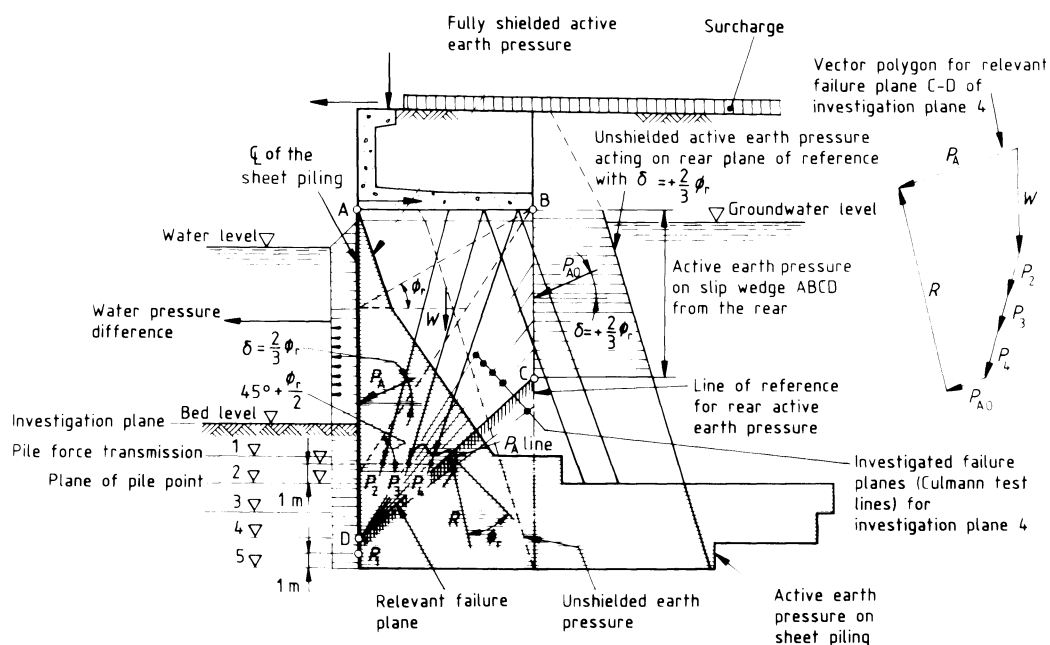


Figure 16 — Simplified distribution of active earth pressure and passive earth resistance on cantilevered sheet wall structure



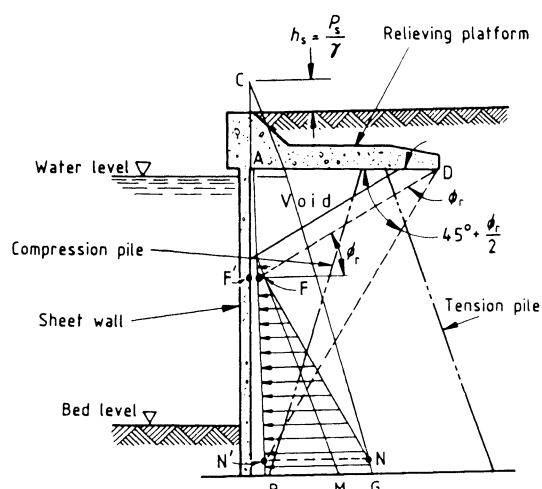
Source: Foundation Engineering, McGraw-Hill [24].

(a) Theoretical pressure distribution on back of wall



Source: EAU 1985, Wilhelm, Ernst & Sohn [12].

(b) Loads on sheet wall showing trial wedges



Source: Foundation Engineering, McGraw-Hill [24].

(c) Pressure on wall with void under platform

Figure 17 — Pressure distribution on sheet wall in front of relieving platform

4.8 Toe level

4.8.1 Minimum embedment

The minimum embedment required for overall stability of the wall should be determined in accordance with the recommendations given in 4.7. The toe elevation of elements resisting vertical loads will be influenced by the depth at which soil resistance to these loads is achieved.

Additional factors to be taken into account in assessing embedment are given in 2.1.6.2.

4.8.2 Staggered toe elevations

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

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

Figure 18 illustrates staggered toe arrangements.

4.9 In situ capping

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

The capping should generally be at least 0.5 m high, the actual height depending on the wall cut-off level in relation to cope level and on fendering arrangements. The seaward face of the capping should be dimensioned in relation to the face of the sheet wall to give the necessary clearances (see 2.1.6.5).

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

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

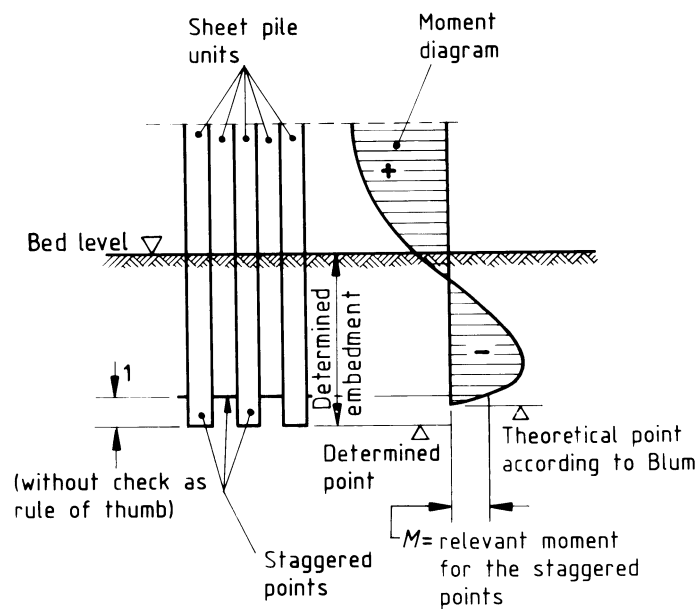
Movement joints should generally be provided at 15 m to 30 m intervals. Cracking in the capping near the joints may be minimized by debonding the capping and the sheet wall for a short distance each side of the joint.

The capping should be provided with suitable means to transmit loads across movement joints where this is required.

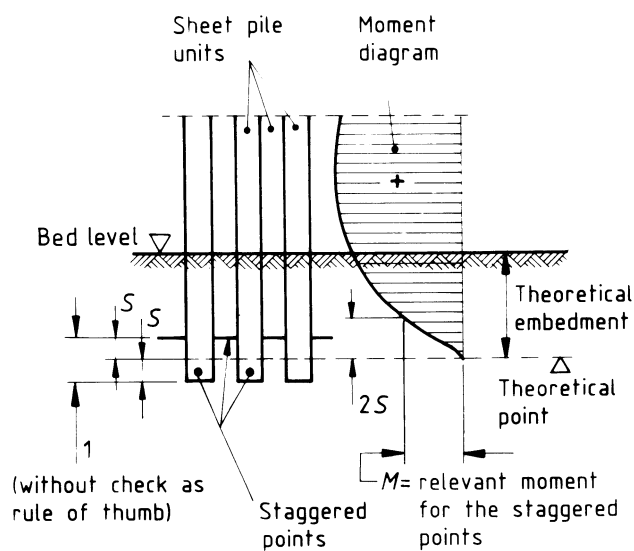
For cappings built on sheet-piled walls, a minimum pile embedment of 75 mm is recommended. Where the capping or superstructure is stiff in relation to the sheet wall, the bending stresses that would be set up in the wall when a rigid connection is used may be eliminated by means of a hinged joint between wall and capping. This should be designed to ensure that all horizontal and vertical loads are effectively transmitted and that soil is unable to escape through the joint.

The capping may be of composite construction in which a precast concrete facing unit is provided on the seaward face and the remainder is cast in situ. This method has the advantages of ensuring a good quality, durable finish on the exposed face and of avoiding the need for formwork over water. The precast units have to be designed to allow for irregularities in the sheet wall, especially in the case of troughed steel sheet piles.

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



(a) Fixed-earth condition



(b) Free-earth condition

Source: EAU 1985, Wilhelm, Ernst & Sohn [12].

All dimensions are in metres.

Figure 18 — Staggered toe elevations for sheet-piled walls

4.10 Vertical loads applied to wall

4.10.1 Types of load

Vertical loads which may be imposed on sheet walls are as follows:

- a) dead loads due to superstructure and relieving platforms;
- b) live loads due to cargo, ship ramps and handling equipment;
- c) vertical components of berthing and mooring loads;
- d) vertical components of anchor loads;
- e) vertical components of soil pressures resulting from wall friction, which may cause downdrag (this may be reduced by application of a slip layer on preformed sheet piles; see 6.8.3.1).

4.10.2 Distribution of concentrated loads

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

4.10.3 Resistance to vertical loads

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

Resistance may be achieved by one of the following methods.

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

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

In soldier-piled walls, only the piles should be assumed to resist the vertical loads.

The effective column length of the load bearing sheet wall may be taken as being equal to the vertical distance between the points of zero bending moment due to horizontal loads.

Composite action of the wall elements in resisting large concentrated loads should be ensured in steel sheet-piled walls by welding the upper portion of the interlocks, and in in situ concrete walls by grouting the joints as required.

- b) *By the provision of a composite sheet wall with load bearing elements.* Where the sheet wall has insufficient load bearing capacity, a composite section may be used. In sheet-piled walls the vertical loads may be carried by H-section, box or tube piles and in diaphragm walls T-shaped panels may be used (see also 6.12.4).

Where a composite section is required to resist horizontal loads, the same load bearing elements may be used to carry the vertical loads.

- c) *By the provision of separate bearing piles.* Separate bearing piles may be installed to seaward or landward of the sheet wall to carry all or part of the vertical imposed loads. Bearing piles are also used to support relieving platforms. The piles should be installed at sufficient depth to ensure that there is no danger of partial transfer of the loads to the sheet wall (see also 4.7.3).

For the installation of bearing piles close to sheet walls reference should be made to 51.4 of BS 6349-1:1984.

4.10.4 Uplift

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

4.10.5 Factors of safety on soil resistance

The minimum factors of safety on soil resistance to vertical loads should be in accordance with the values given in 6.12.12.1.

4.10.6 Design of load-bearing elements

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

4.11 Berthing and mooring loads normal to the wall

4.11.1 Berthing loads

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

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

4.11.2 Mooring loads

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

- a) *By the in situ capping and sheet wall.* The load is assumed to disperse through the capping and is resisted by beam action of the capping and cantilever action of the wall above anchor level. The load is transmitted to the anchorage. This method should be used only for small bollard pulls.
- b) *By the provision of additional tie-backs.* Additional tie-backs may be provided at bollards in the form of auxiliary tie-backs to a separate anchorage (see 4.7.1.4) or additional tie-backs to the wall anchorage. Distribution of the load along the anchorage may be achieved using diagonal tie-backs arranged in opposing pairs (see Figure 19). The tie-backs should be attached to the top of the sheet wall.
- c) *By the retaining wall above relieving platforms.* The retaining wall provided above relieving platforms is designed to transmit the load to the platform by cantilever action or by means of counterforts. The load is resisted by the raking piles under the platform.

4.12 Longitudinal imposed loads

Horizontal loads applied to the wall in a longitudinal direction by ships (via fenders and mooring lines) and cranes are resisted in bending by the wall elements in a plane at right angles to the direction of the wall.

The loads should, in general, be distributed along the wall by the waling or capping.

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

When sheet piles are used, the maximum biaxial bending stress in the wall may exceed the allowable values for single bending by up to 10 %. The capping and walings should be designed to resist the direct tensile and shear stresses caused by the longitudinal loads.

The loads are transmitted to the ground by friction between the rear face of the wall and the soil and by passive resistance at projecting faces, such as the webs of troughed sheet piles, composite sheet piles and T-panel diaphragm walls. The passive resistance at any level cannot exceed the frictional resistance within the soil between the projections.

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

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

4.13 Walings and tie-backs

4.13.1 General

Failures in sheet walls have generally been due to failure of the anchorage system rather than of the sheet wall itself. Design of walings and tie-backs and their connections should, therefore, take into account all factors which may tend to increase loadings.

4.13.2 Walings

Walings are required in sheet walls to transfer horizontal loads to the tie-backs. Walings should be designed as continuous beams spanning between the tie-backs. Where the tie-backs are inclined, the walings and fixings should be designed to carry the vertical load component. Other vertical loads to be resisted are the dead weight of the waling, the weight of any soil and surcharge loads directly above the waling, and the vertical component of active soil pressure acting on the vertical plane passing through the rear edge of the waling. Guidance on the design of walings is given in BS 8002.

Walings may be used to distribute longitudinal imposed loads along the wall (see 4.12). The wall/waling connections should be capable of transferring the loads to the wall. Walings should be designed to resist the direct stresses resulting, and connections to resist the additional shear stresses.

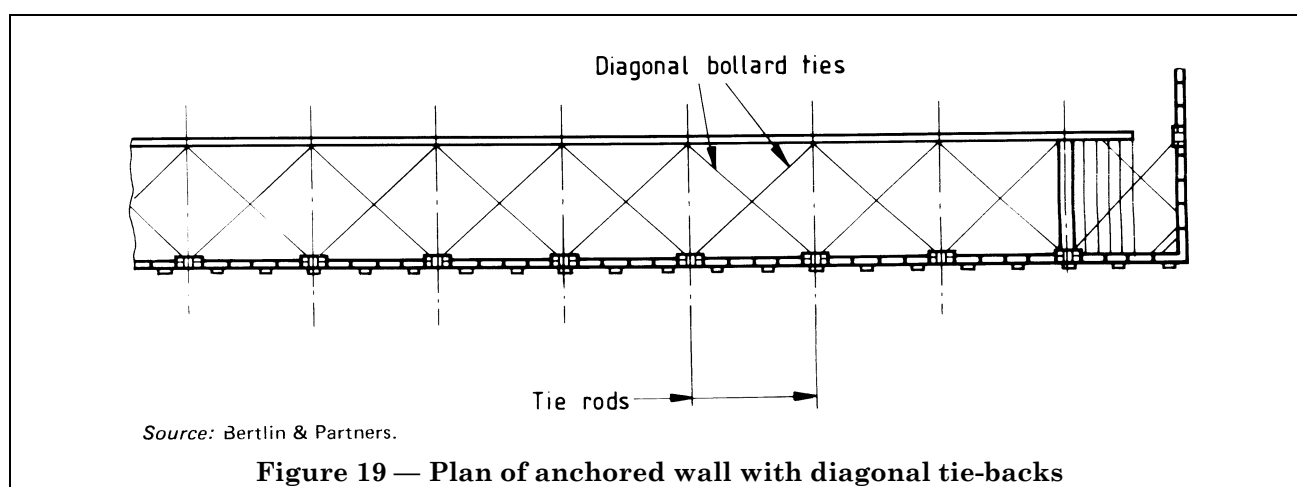


Figure 19 — Plan of anchored wall with diagonal tie-backs

Walings may be placed in front of or behind the sheet wall but, in quays and jetties built in sea water, they should normally be placed behind to prevent mechanical damage and reduce corrosion. Tie-bolts or other means are required to transmit the load from the wall to the waling. Where the capping or deck projects well in front of the wall, the waling may be placed outside and a saving can then be made by omission of the tie-bolts. If stress reversals due to horizontal loading can occur, the waling fixings should be designed accordingly. In sheet-piled anchor walls, walings should be placed behind the wall and tie-bolts are not then required.

Steel walings comprising two channel-sections should be drilled through the web of the upper channel at intervals of approximately 3 m to permit drainage of water collecting on top, except where the walings are permanently immersed.

Where the in situ concrete capping is to function as a waling, the connection between it and the sheet wall should be sufficient to transmit all internal forces. If inclined anchor piles are employed, the capping should be designed to resist any torsion resulting from vertical bending due to long term soil loading on the anchor piles: alternatively, pinned connections may be provided.

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

4.13.3 Tie-backs

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

Guidance on the design of tie rods, including the importance of corrosion protection, is given in BS 8002.

In relieving platform structures, the platform transmits the horizontal loads from the wall to the piles.

Tie-backs are commonly attached to the sheet wall at a single level. This level should generally be as far below the top of the wall as practicable, usually just above low water level, in order to reduce the vertical span, and thus the bending moments in the wall. The cantilever moment in the top part of the wall further helps to reduce the span and toe moments.

Auxiliary tie-backs may be provided to the top of the wall or full tie-backs may be provided at two levels (see 4.7.1). Where diagonal tie-backs are provided they should be arranged in opposing pairs.

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

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

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

It is usual to apply a small prestress to tie rods to limit wall movement. The amount of prestress should be not less than the maximum amount of stress relief which could occur during the life of the structure, in order to avoid the sudden imposition of load in fluctuating load conditions.

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

Tie rods may be of low carbon or high yield steel. Low carbon steel is preferable where deflection of the wall is to be minimized and where long term settlement could cause bending of the tie rods. Reference should be made to BS 8002 for the allowable stresses. In sea water conditions it is recommended that the diameter be at least 50 mm.

Guidance on the design of tendons is given in DD 81. Particular attention should be given to the effects on the soil of cyclic loading, and of long term settlement of the soil causing vertical loading on the tendon. Corrosion protection should be designed to accommodate the deformation of the tendon if settlement is expected.

Under extreme loading conditions, the factors of safety given for tie-backs under normal loading conditions may be reduced by a maximum of 25 %, depending on fatigue considerations and effects on soil properties.

4.14 Anchorages for quay walls

4.14.1 General

For recommendations on the design of anchorages for sheet walls in general, reference should be made to section 4 of BS 8002.

A factor of safety of 2.0 on the tie-back design working loads should be achieved, under all loading conditions (see also 4.13.3).

4.14.2 Deadman anchorages

Deadman anchorages include steel sheet-piled walls, concrete walls, concrete blocks and concrete slabs (see Figure 20).

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

Concrete blocks and friction slabs, which may be buried or at ground level, resist tie-back loads partly by friction at their base and partly by passive resistance of the soil in front. They are generally less efficient than walls in achieving the required horizontal resistance and will usually be limited to sheet walls where tie forces are not very large. This applies particularly where the ground water table can rise sufficiently to reduce frictional resistance. If a suitable gravity structure exists behind the wall, this may be used as an anchorage. Hollow anchor blocks may be used where the tie-backs are prestressed tendons. Stressing of the tendons can then take place in the dry compartment in the block. This arrangement may be advantageous where ground water levels are high, in which case small monoliths can be used to form the blocks (see Figure 9).

Restraint to forward movement of deadman anchorages is provided by the passive resistance of the soil against the face of the member.

Consideration should be given to the location of the anchorage in relation to potential failure planes within the mass of the soil retained by the structure. An anchorage is fully effective only if placed behind zones of potential failure.

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

Where poor ground exists in front of and behind a deadman anchorage, it should, where possible, be replaced with compacted granular material in order to improve the passive resistance and reduce the active pressure on the back [see Figure 1(a)]. Where backfill is placed against the vertical faces of buried block or wall anchorages, care should be taken to compact the fill in such a manner that the required passive resistance to forward movement of the anchorage will be obtained in a uniform manner.

Sheet-piled wall anchors may be used to resist vertical loads, such as those imposed by crane tracks, by providing intermittent deep piles and a capping of appropriate design. The provisions of 4.10 regarding vertical loads and settlement also apply in general to anchor walls.

Staggered toes may be provided on sheet-piled anchor walls to save materials, as described in 4.8. A stagger of 0.5 m may often be adopted in anchor walls at least 2.5 m high without additional calculation.

4.14.3 Cantilever anchorages

Where poor soil exists above a better quality soil, or where the ground water level is too high to suit construction of a balanced anchorage, a deeper anchor wall may be installed by driving sheet piles or forming barrettes using slurry trench construction: see Figure 21 for examples. The anchor wall is then designed as a cantilever: large bending moments and deflections can occur in this type. If the anchor wall has to be driven to a great depth, a double-wall structure (see 5.8) may be more economical than an anchored single-wall structure.

As for deadman anchorages, continuous anchor walls should be provided with weepholes.

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

4.14.4 Raking piles

Raking piles may be used to form an A-frame tied to the wall by a rod or tendon, or as bearing piles to a relieving platform (see Figure 22).

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

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

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

4.14.5 Ground anchors

Ground anchors include grouted, stressed and unstressed tendons, and piles installed at a flat rake (see Figure 23). This type of anchor is connected directly to the sheet wall or waling.

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

Tendons grouted into soil or rock are covered in DD 81. They are usually installed at an angle to the horizontal to achieve the required resistance with the minimum length of tendon. The tendons may be installed in a single row or in multiple rows.

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

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

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

Inclined anchor piles should be designed such that soil resistance is obtained only outside the active wedge, in the same way as grouted anchors. The piles should be designed to resist compressive loads due to berthing. Where the ends of the piles are not buried, the effective length should be used in design. The factors of safety given in 6.12.12 may be used for inclined anchor piles.

4.15 Corners

4.15.1 General

Special anchoring provisions have to be made at corners to achieve sufficient resistance to horizontal soil and imposed loads and to hydrostatic loads. These provisions may take the form of diagonal ties between the two walls, anchor walls in two directions, or a relieving platform. Figure 24(a) to Figure 24(d) show typical examples of each type. Corners may be rounded to reduce damage both to vessels and the wall, and to improve hydraulic conditions nearby.

4.15.2 Diagonal ties

Corners tied diagonally across the corner require the horizontal loads applied at or near corners to be transmitted to the soil by wall friction (see also 4.12). The forces in the diagonal ties cause additional tensile forces to be set up in the walings. These additional forces may be reduced by providing opposing diagonal ties back to the anchor walls.

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

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

4.15.3 Anchor walls in two directions

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

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

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

4.15.4 Relieving platform

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

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

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

4.16 Earthworks

4.16.1 Fill

Fill placed behind sheet walls and in front of and behind deadman anchorages should be granular material having suitable drainage characteristics. Loss of material through joints in the wall should, where necessary, be prevented by providing a suitable filter behind the wall (see also 2.3.2).

Material of similar quality should be used to replace weak soils, such as soft clay and silt and fine-grained sand, which could liquefy in earthquakes, from behind or in front of the wall and anchorages, in order to minimize vertical and horizontal movements. Soil replacement below seabed level should take place before piling. Material above this level may be placed before piling, in which case it will need to be dredged out after the wall is built, or after, depending on design considerations. At in situ concrete walls, all soil replacement at the wall should be done in advance of wall construction.

4.16.2 Settlement

An appropriate allowance for or reduction of soil settlement should be made (see also 2.3.1).

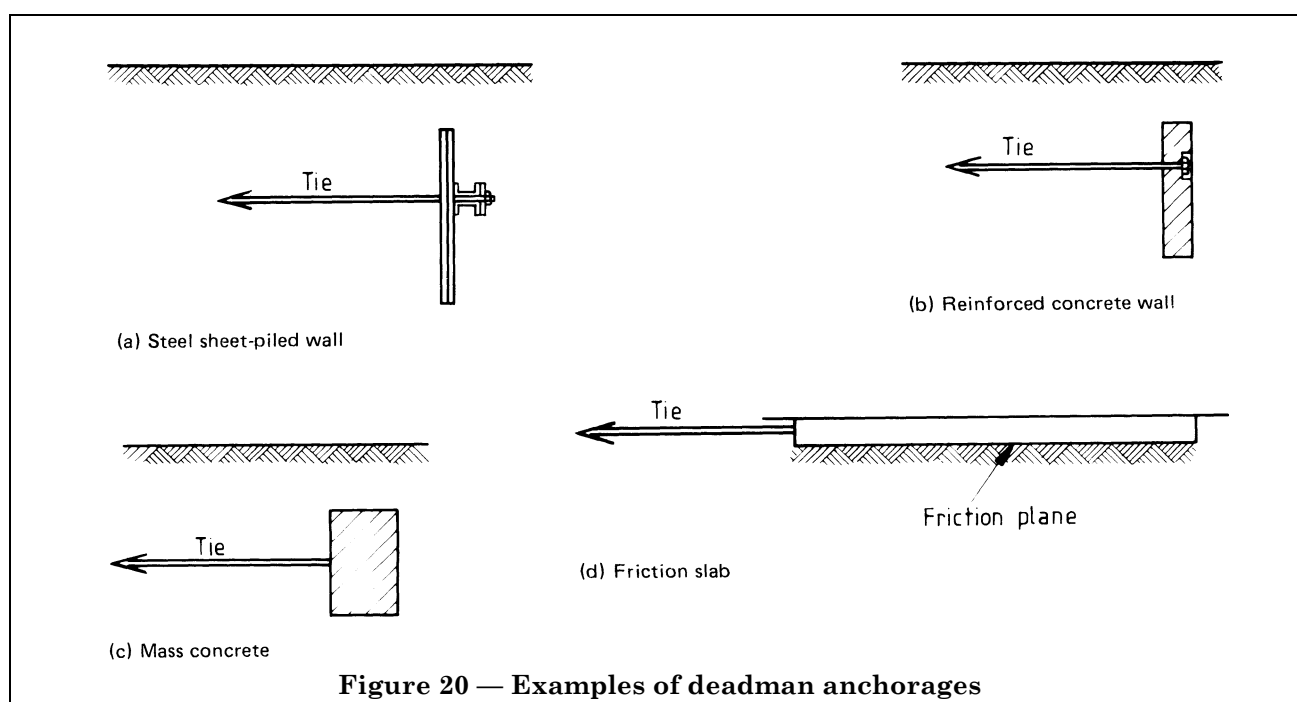
4.16.3 Drainage

Most forms of sheet wall are fairly impermeable and provisions for drainage through the wall will generally be required to reduce tidal lag (see also 2.3.3 and 51.5 of BS 6349-1:1984).

Drainage of the soil above relieving platform slabs should be provided where it is considered that inundation of the surfacing could occur. This may take the form of weepholes surrounded by a filter blanket.

4.16.4 Scour protection

Scour protection may be required in order to prevent scour of the seabed in front of sheet walls below the level for which the wall is designed (see also 2.3.5).



4.17 Construction sequence

4.17.1 Effect on design

The construction sequence is a major consideration in the design of sheet walls, because the distribution of stress on the wall is affected by each stage of dredging or excavation, filling, compaction and anchoring (see 4.7.1.1).

4.17.2 Wall construction

Sheet piles may be installed by percussion methods in most soils, by vibration in granular soil or by hydraulic methods in cohesive soil. Driving should be done in such a way that additional stresses due to deformation of the pile clutches are kept to a minimum.

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

Piles installed with a large unsupported length require temporary support to maintain line and to avoid buckling. Where the line of installed piles is irregular, the walings may be used to assist in straightening the top of the piles, but care should be taken to avoid overstressing the piles and walings resulting in locked-in stresses. U- and Z-section steel sheet piles are normally installed in pairs to achieve a stiffer and more balanced section during driving. Guidance on the installation of steel sheet piles is given elsewhere [25].

The construction of in situ concrete walls is usually carried out in existing or filled ground. A considerable width may be required for the working area alongside the wall. Guidance on the construction of diaphragm walls and the use of thixotropic liquids in excavation is given in clause 55 of BS 6349-1:1984 and on bored piles in [26].

4.17.3 Anchorage

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

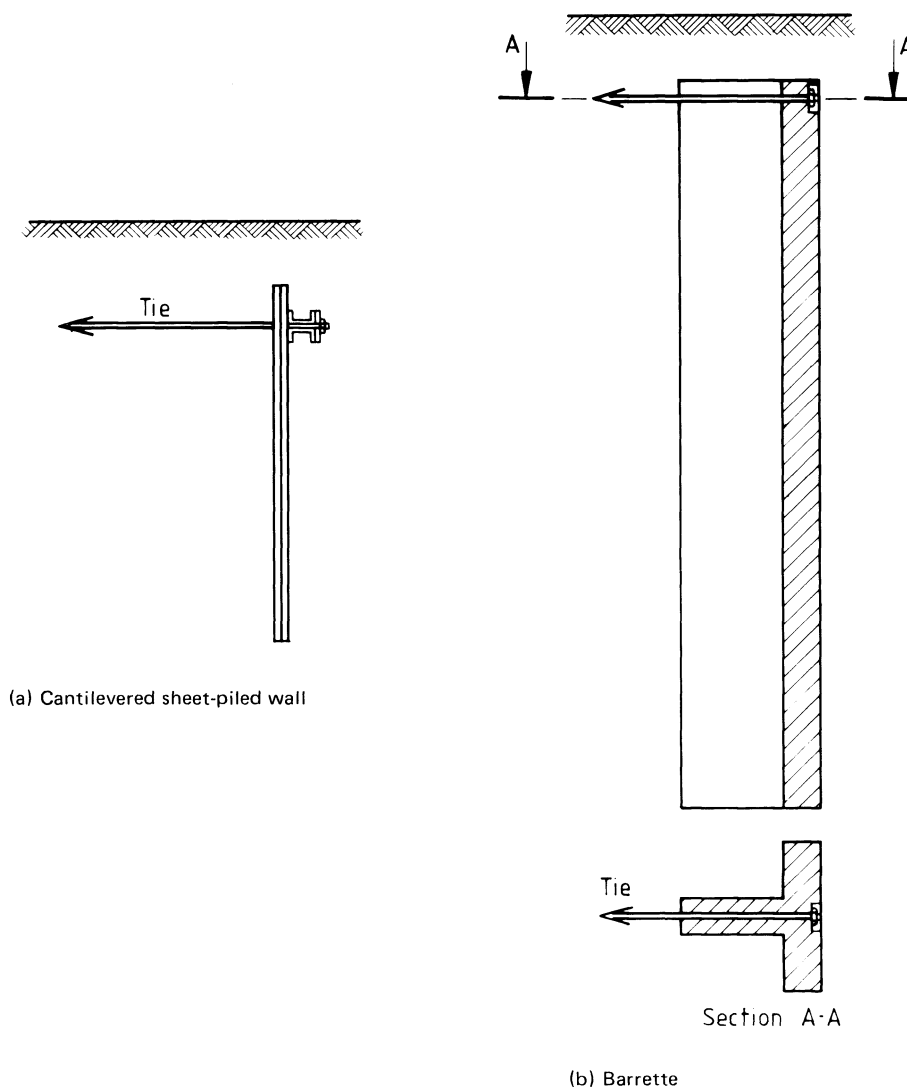


Figure 21 — Examples of cantilever anchorages

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

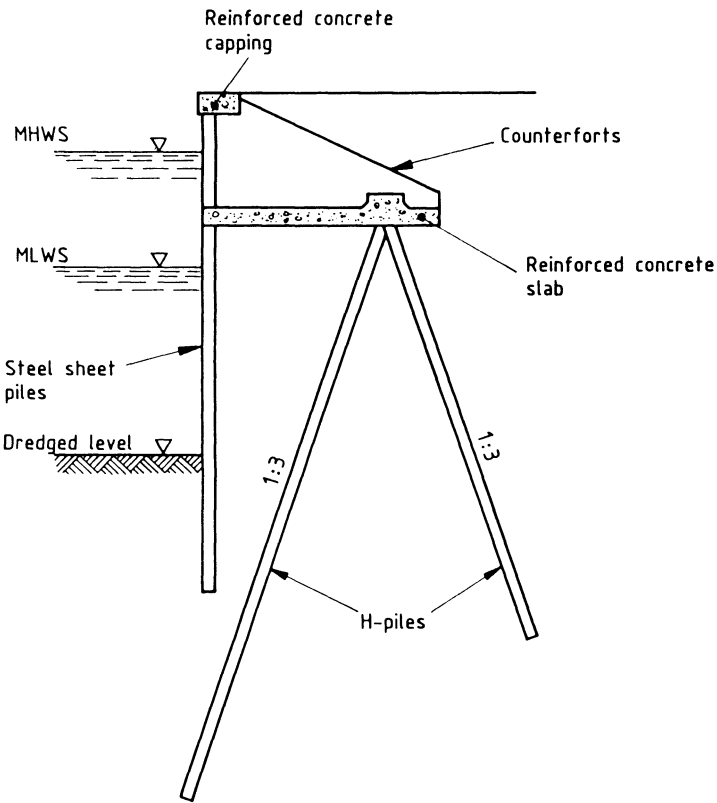
Care should be taken, in handling tie rods, not to damage them by impact or bending. Prestressing should generally be carried out at turnbuckles, since threads may be damaged if stress is applied by tightening the end nuts. The location of turnbuckles should be carefully considered.

4.17.4 In situ capping

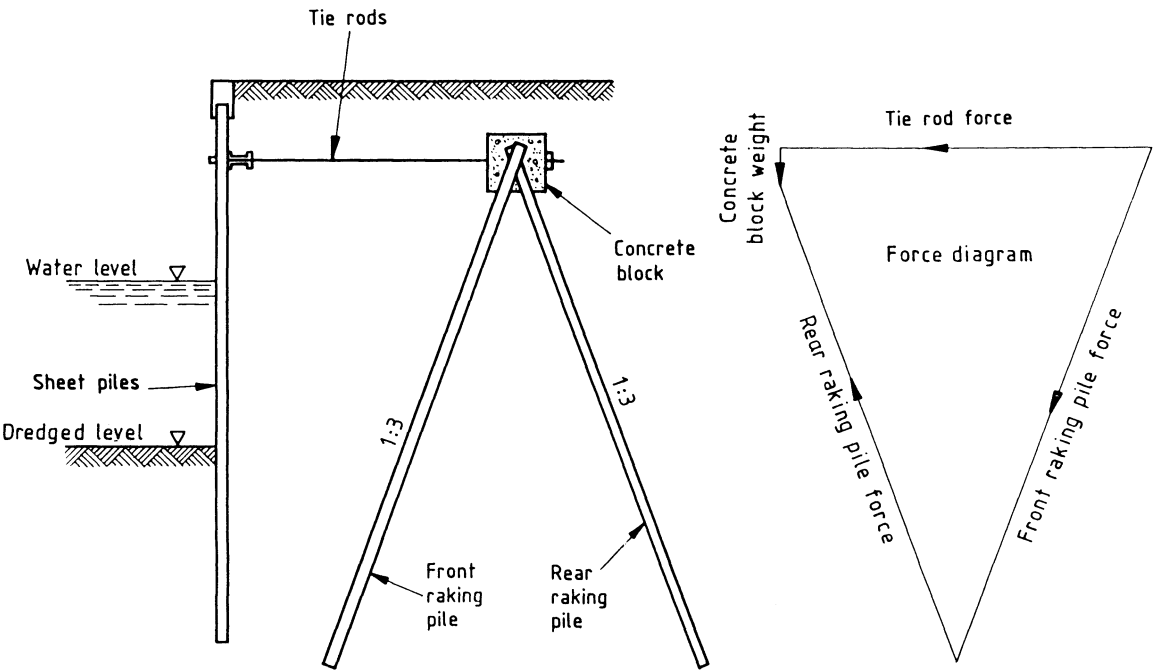
The capping may be useful in masking an irregular wall line and its width should be designed taking the driving tolerances into account. Where straightening of the wall is required, this may be achieved by temporary walings if the capping is to function as a permanent waling. The temporary support should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

4.18 Services and crane tracks

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

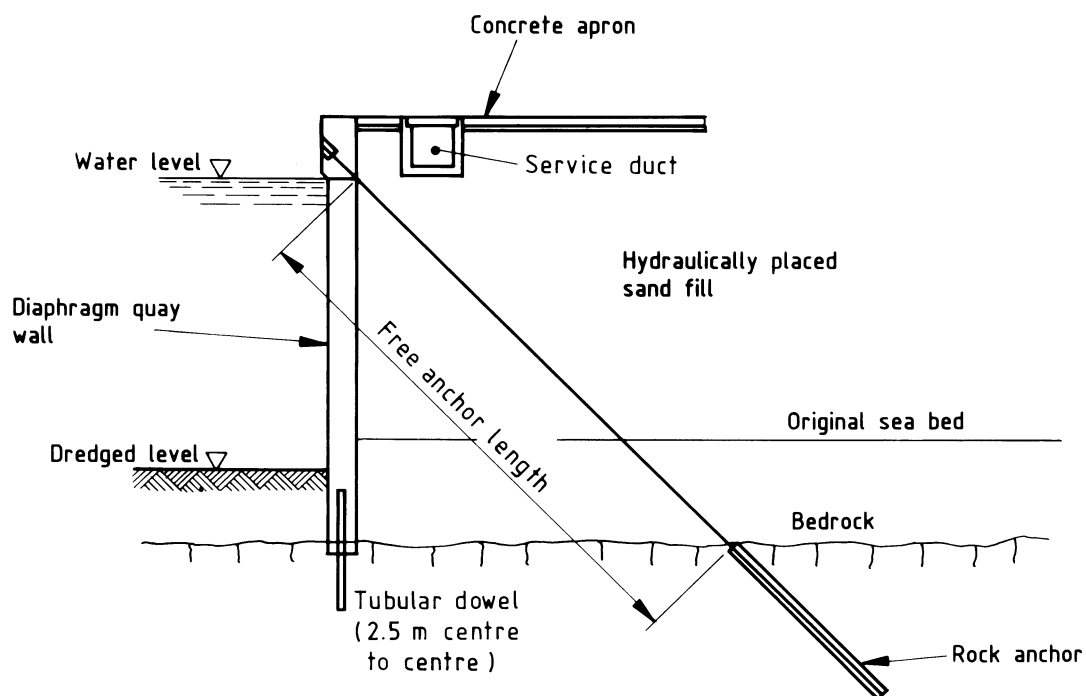


Source: British Ports Association/National Ports Council.
(a) With relieving platform



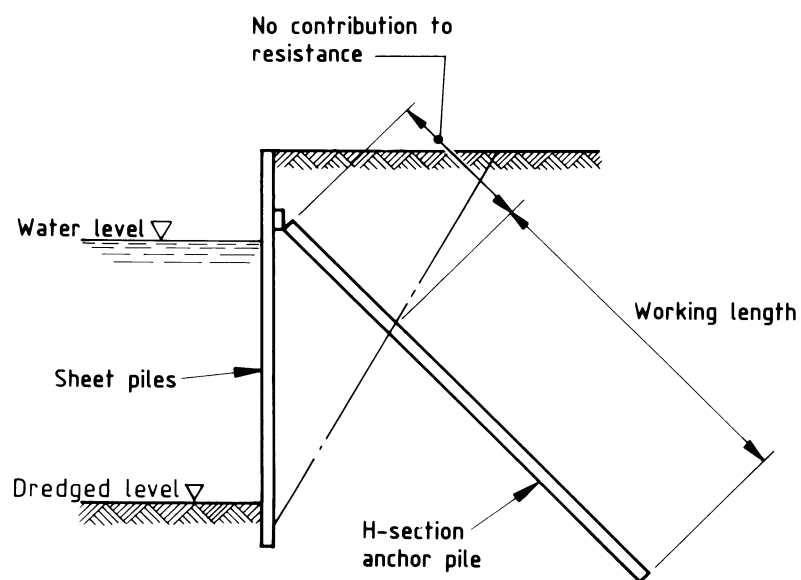
Source: British Steel Corporation, General Steels, Scunthorpe.
(b) With tie rod

Figure 22 — Examples of raking pile anchorages



Source: ICOS (Great Britain) Ltd.

(a) Prestressed tendon



(b) H-pile

Figure 23 — Examples of ground anchors

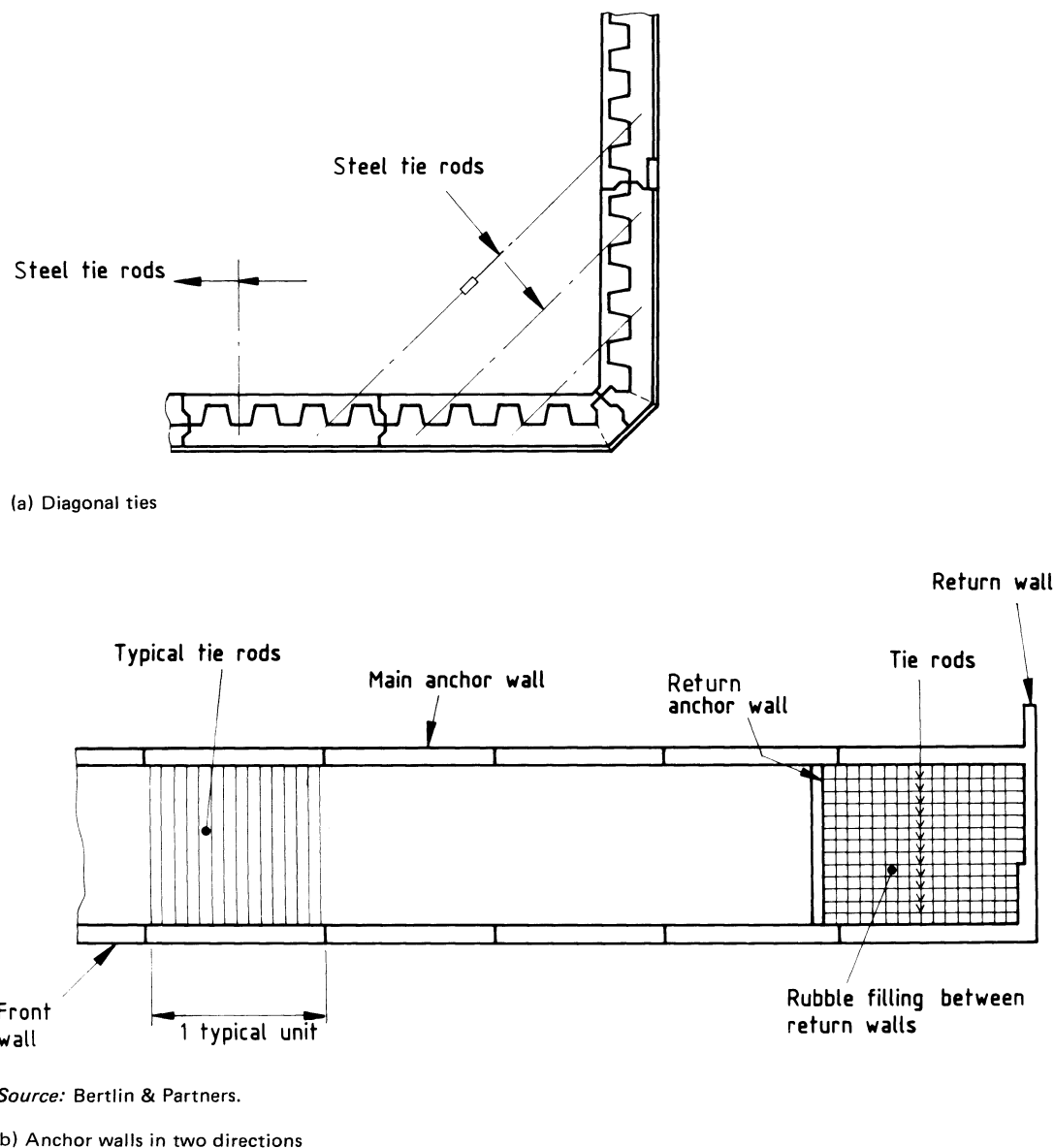
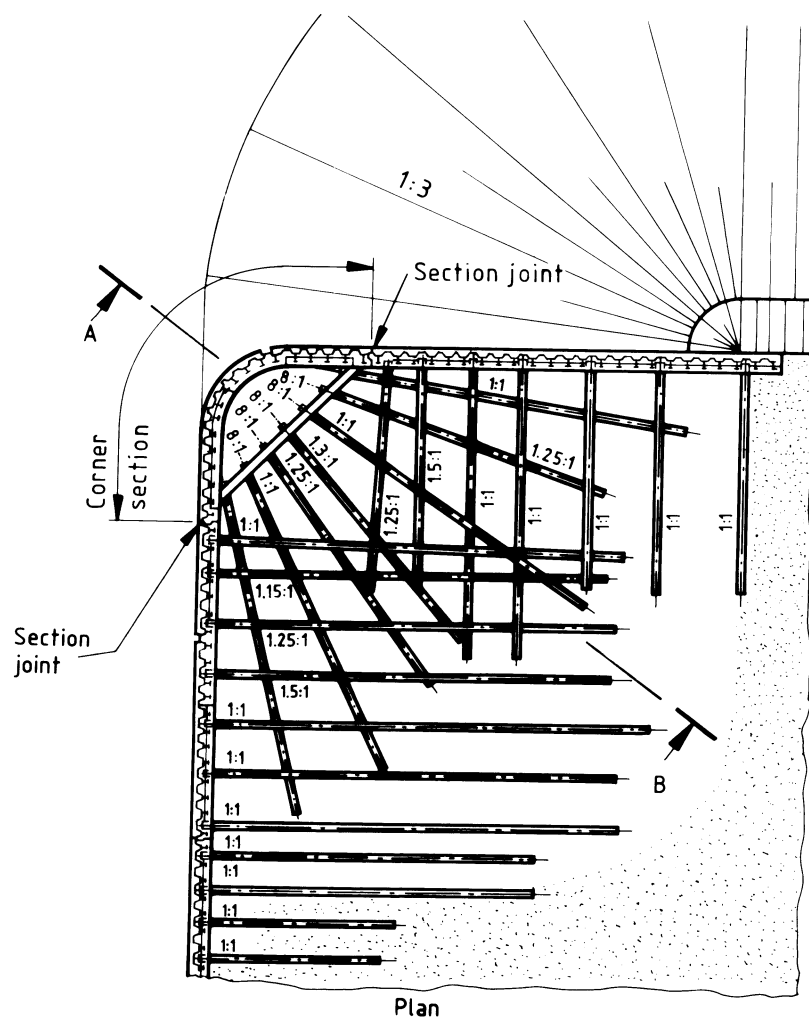
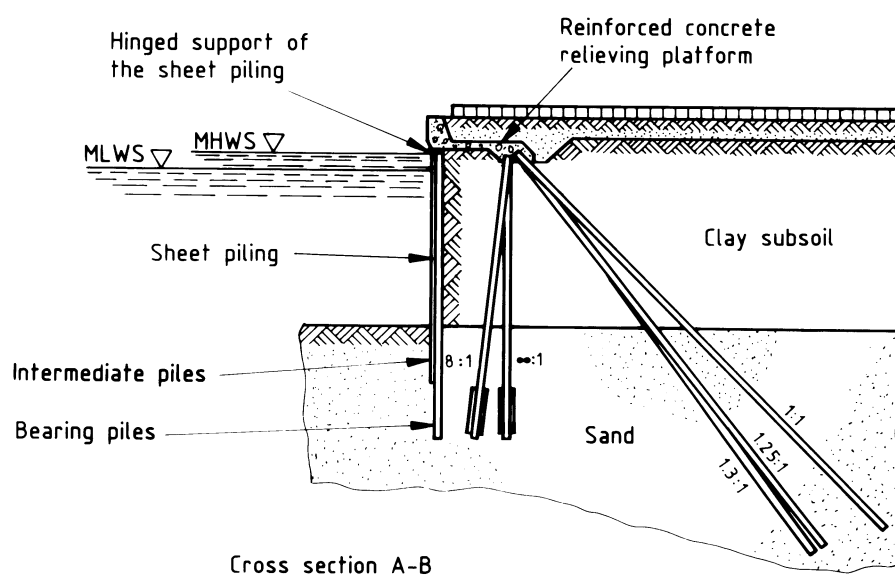


Figure 24 — Anchoring arrangements at corners

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

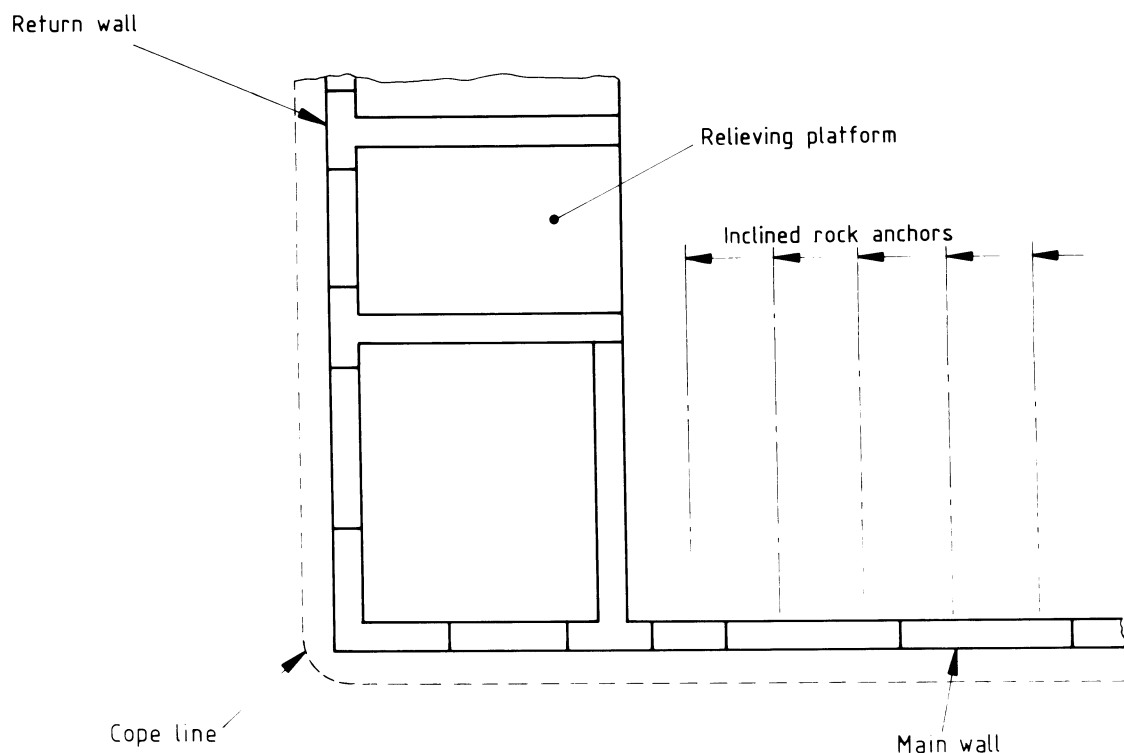
The seaward rail of quayside cranes and bulk handling equipment may be mounted on the capping directly above the sheet wall, or on a beam supported on separate bearing piles. The beam may be part of the capping or independent of it. The landward rail may be mounted on a ground beam, on beam supported on piles, on an anchor wall or on a relieving platform. Maintenance of rail centres is achieved by means of tie beams connecting the rail beams. Rails should not be fixed until the tie beams have been constructed. Figure 1(b) and Figure 25 show examples of typical crane rail foundation arrangements.



Source: EAU 1985, Wilhelm, Ernst & Sohn [12].

(c) Relieving platform supported on raking piles

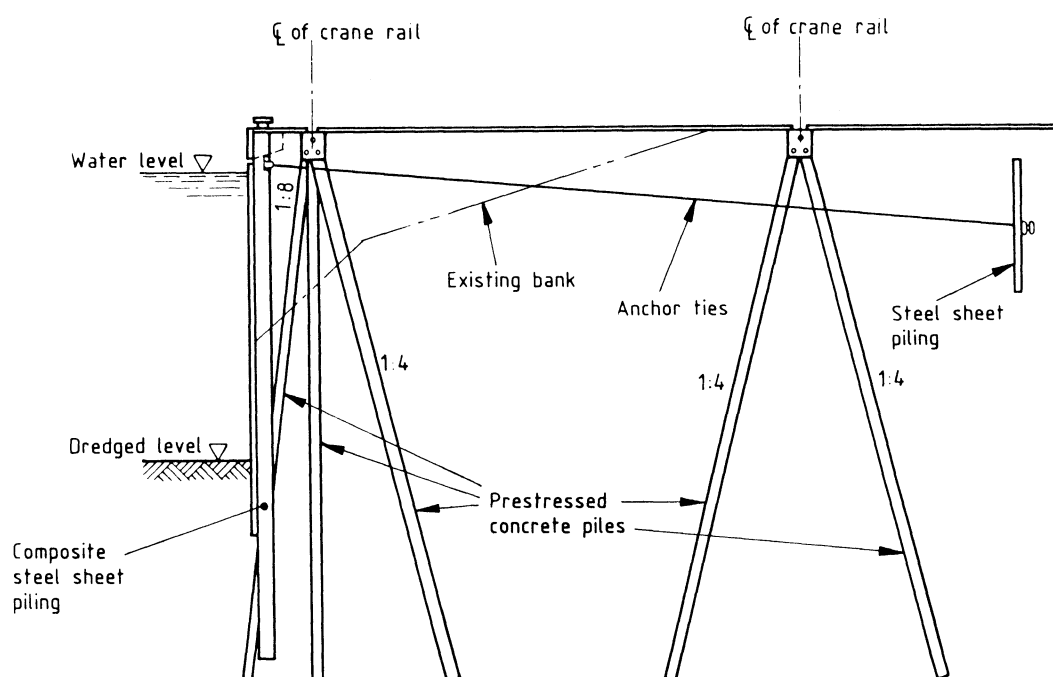
Figure 24 — Anchoring arrangements at corners (continued)



Source: Livesey Henderson.

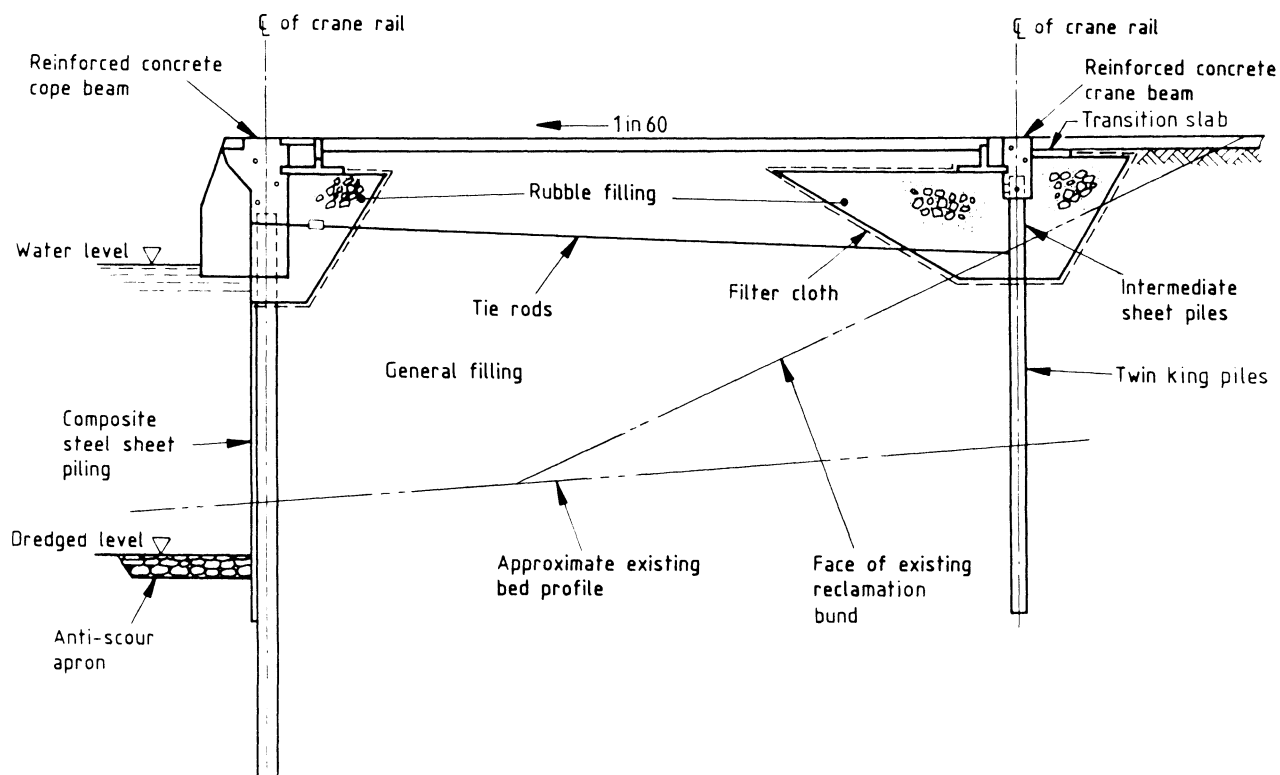
(d) Relieving platform supported on diaphragm walls (showing panel arrangement)

Figure 24 — Anchoring arrangements at corners (concluded)



Source: Amsterdam Department of Public Works/National Ports Council.

(a) Seaward crane rail supported independently of sheet wall



Source: Bertlin & Partners.

(b) Seaward crane rail supported on sheet wall

Figure 25 — Examples of crane rail foundation arrangements

Section 5. Gravity walls in quay and jetty construction

5.1 General

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

Some gravity walls are built behind a cofferdam in the dry but most walls are constructed in water by a method used only in maritime works, in which large precast units are lifted or floated into position and installed on a prepared bed under water. This form of construction is not covered by other codes of practice and is therefore dealt with in detail in 5.4.

Gravity walls used in maritime works are generally required to retain reclaimed ground, the quality of which can be selected. It is usual to use rubble or a free-draining granular fill immediately behind a quay wall so that the effects of tidal lag are minimized and earth pressures are reduced (see also 2.3).

5.2 Types of wall

Gravity quay walls can be divided into three categories, as follows.

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

Walls in category 1 would normally be constructed in water using floating plant, those in category 2 in the dry, often behind cofferdams, and those in category 3 from ground above water level, the soil in front of the wall being subsequently removed. Figure 26 to Figure 34 illustrate some common types of gravity wall in these categories.

The selection of the most appropriate type of gravity wall will be influenced by the site conditions and method of construction.

5.3 Design of gravity walls

5.3.1 Principal modes of failure

5.3.1.1 General. The principal modes of failure of a gravity structure are:

- a) deep slip;
- b) overturning;

c) sliding;

d) foundation failure.

Earth pressure calculations and general design principles should follow the recommendations of BS 8002.

The factors of safety given in 5.3.1 to 5.3.4 refer to cases where soil properties are known with confidence. Where soil strength is not fully established or where ground conditions are variable, these factors should be increased.

5.3.1.2 Deep slip. Methods of checking the stability of the earth mass in which the wall is constructed are given in BS 8002 and 6.3 of BS 6031:1981.

The factor of safety against failure is expressed as the following ratio:

$$\frac{\text{restoring moment about centre of rotation}}{\text{disturbing moment about centre of rotation}}$$

The minimum factor of safety should be within the range 1.25 to 1.50 for normal loadings and not less than 1.2 for extreme loadings.

5.3.1.3 Overturning. Either of the calculation methods given in BS 8002 may be adopted for normal loading but the edge pressure on the foundation material under the toe of the wall should not exceed the allowable value (see 5.3.1.5). The submerged weight of underwater portions of the wall should be used in calculating the resisting forces.

For transient loading, the resultant may fall outside the middle third provided that the factor of safety against overturning is not less than 1.5 but for either method the maximum edge pressure should not exceed the allowable by more than 25 %.

Under severe earthquake conditions, theoretical tension between wall and foundation may be permitted to extend to the centroidal axis of the interface. The inertia forces include the mass of the wall, the soil wedge and the water contained in any voids in the wall and in the soil.

5.3.1.4 Sliding. For base resistance to sliding reference should be made to BS 8002.

Where the walls have been sunk or driven on to a firm stratum the analysis of sliding resistance is similar to that for walls constructed on land. However, for many forms of maritime structure, the wall is founded on a granular levelling bed laid on the existing or dredged seabed. In this case it is also necessary to check for shear failure through the granular bed and at the seabed interface (see 2.2.6). Where the construction sequence involves laying the bed well in advance of the wall, it may be necessary to remove any fresh silt layer by suction cleaning immediately prior to placing the wall units. A similar problem may arise where there is a delay between placing successive layers of concrete blocks.

Walls can have a sloping or a serrated base to improve the resistance to sliding. The depth of the serrations should be comparable to the average stone size of the rubble bed; an angle of friction $\delta = \phi_r$ may then be assumed instead of $\delta = 2/3\phi_r$.

The factors of safety against sliding given in BS 8002 refer to normal loads. For economy, walls should wherever possible be designed to yield, in which case active pressures due to retained soil may be used. Where there is doubt as to the degree of movement which will occur, earth pressure at-rest should be used in the investigation of sliding: with allowance for water pressure difference, hydraulic uplift and all disturbing forces such as mooring loads.

The most critical case may be the temporary condition during construction, especially where hydraulic fill is being placed behind a wall.

Passive resistance in front of the toe of the wall should only be taken into account when there is no possibility of the material being removed by scour or dredging.

In all cases the factor of safety against sliding should be at least 1.75.

5.3.1.5 Foundation failure. The maximum pressure under the wall should not exceed the allowable bearing pressure in the underlying material. Typical values are given in Table 1 of BS 8004:1986⁷⁾. These values may be used in the preliminary design but, in the final design, analysis of settlement may show that bearing pressures should be reduced to suit the ability of the structure to accommodate settlement.

Allowable bearing pressure at the edge of the foundation may be increased by surcharging the bedding by means of a rubble anti-scour apron.

The bearing pressure under the heel of the wall due to berthing loads should be calculated taking into account the resistance provided by the fill behind the wall.

5.3.2 Distribution of concentrated loads

Typical concentrated loads are horizontal loads due to berthing and mooring operations and vertical loads from ship ramps, mobile crane outrigger pads or rail mounted cranes. The area of contact for the berthing load will depend on the profile of the ship's hull, the angle of approach and the fender system.

In general it is necessary to consider the effects of a concentrated load over the whole length between movement joints of the structure to which it is applied. When calculating bending moments in the capping, considered as a beam, it is normal to assume a 46° dispersion through the in situ concrete from the limits of the contact area to the effective centre of any tensile reinforcement. Special consideration should be given to the effect of loads applied near a construction or movement joint or the end of the wall.

For the distribution of point loads applied as a surcharge on the fill behind the wall, reference should be made to Appendix H of [27]⁸⁾.

5.3.3 Materials and stresses

The materials used in the construction of gravity walls are described in BS 6349-1. Consideration should be given to the use of a retarder in the concrete mix in large pours.

The stresses to be adopted in design should be in accordance with the appropriate British Standard or code. Depending on the material this will either be expressed as an ultimate stress (e.g. concrete) or a working stress (e.g. steel sheet piles).

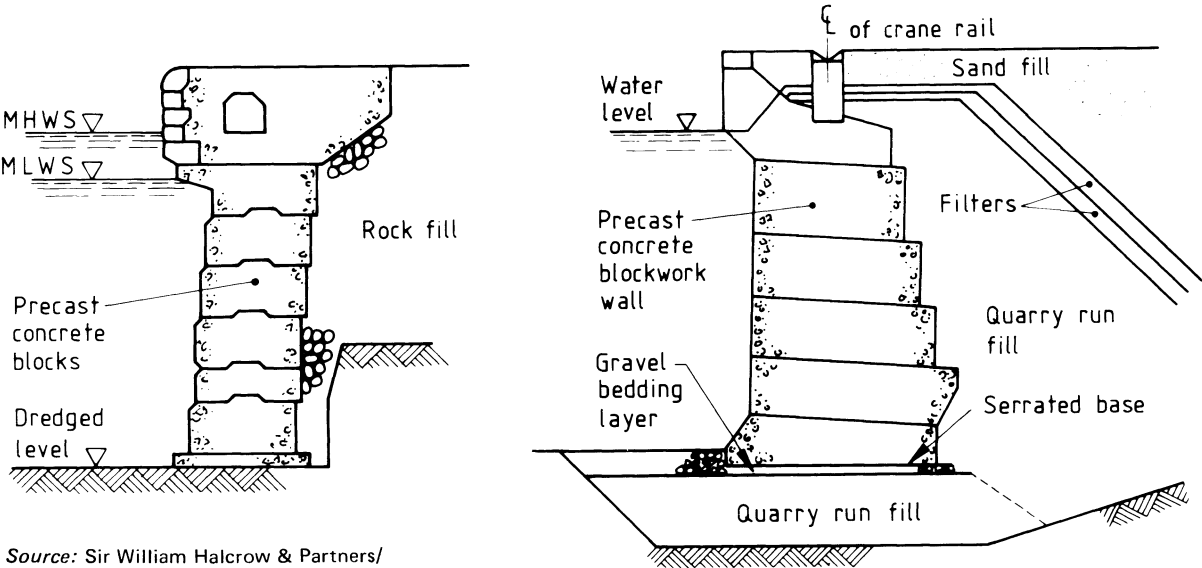
Principal tensile stresses in mass concrete should not exceed $0.25\sqrt{f_{cu}}$.

5.3.4 Detailing

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

⁷⁾ This information will also be included by amendment in BS 6349-1.

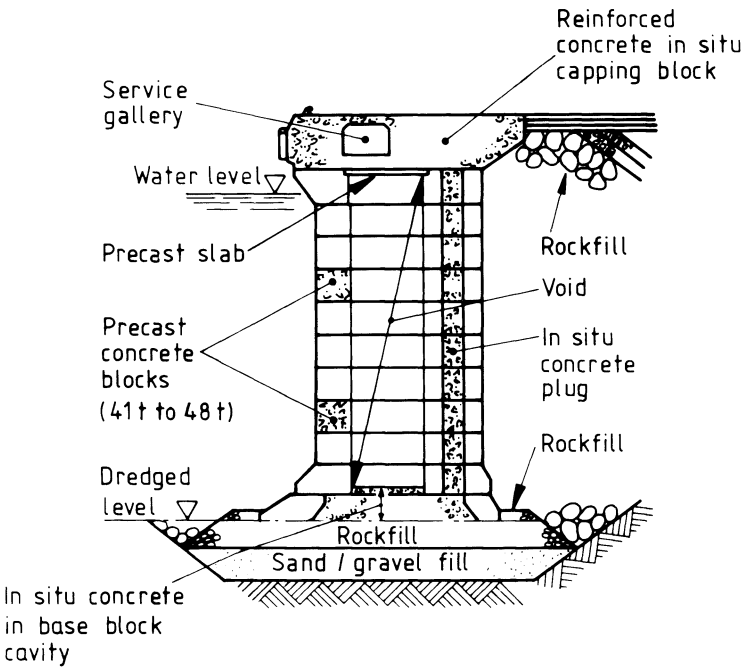
⁸⁾ Currently under revision as BS 8002.



Source: Sir William Halcrow & Partners/
National Ports Council.

Source: Bertlin & Partners.

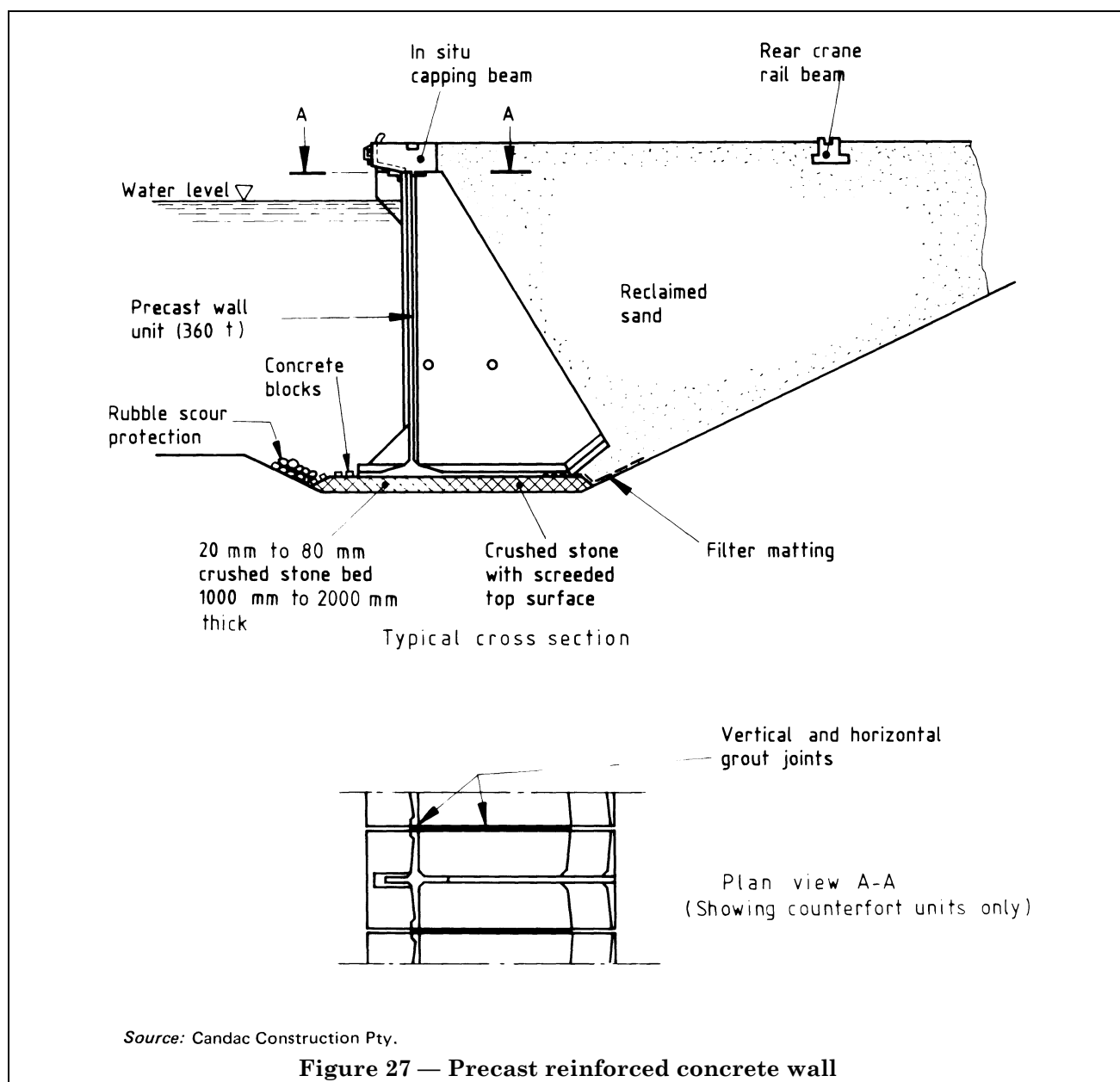
(a) Examples of bonded blockwork



Source: Sir William Halcrow & Partners.

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

Figure 26 — Concrete blockwork walls



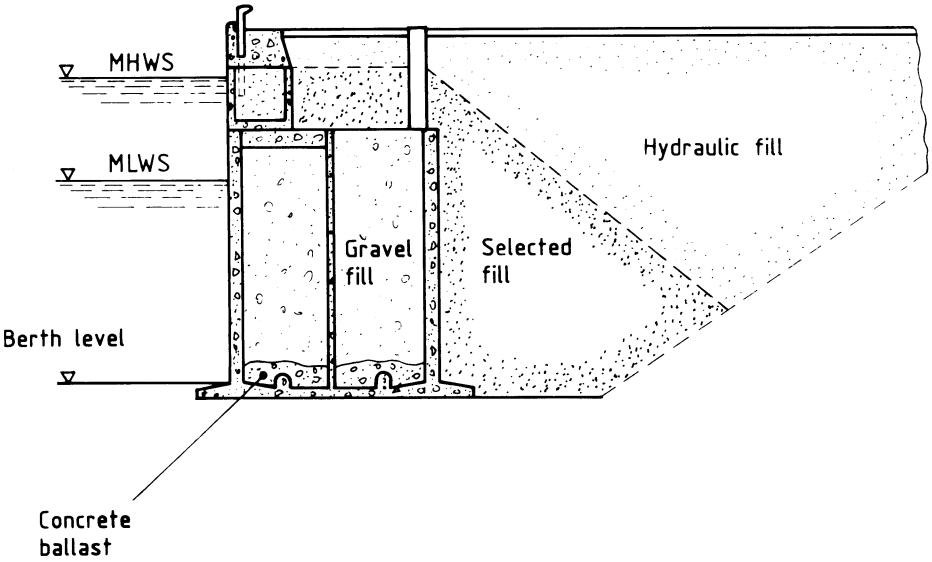


Figure 28 — Concrete caisson

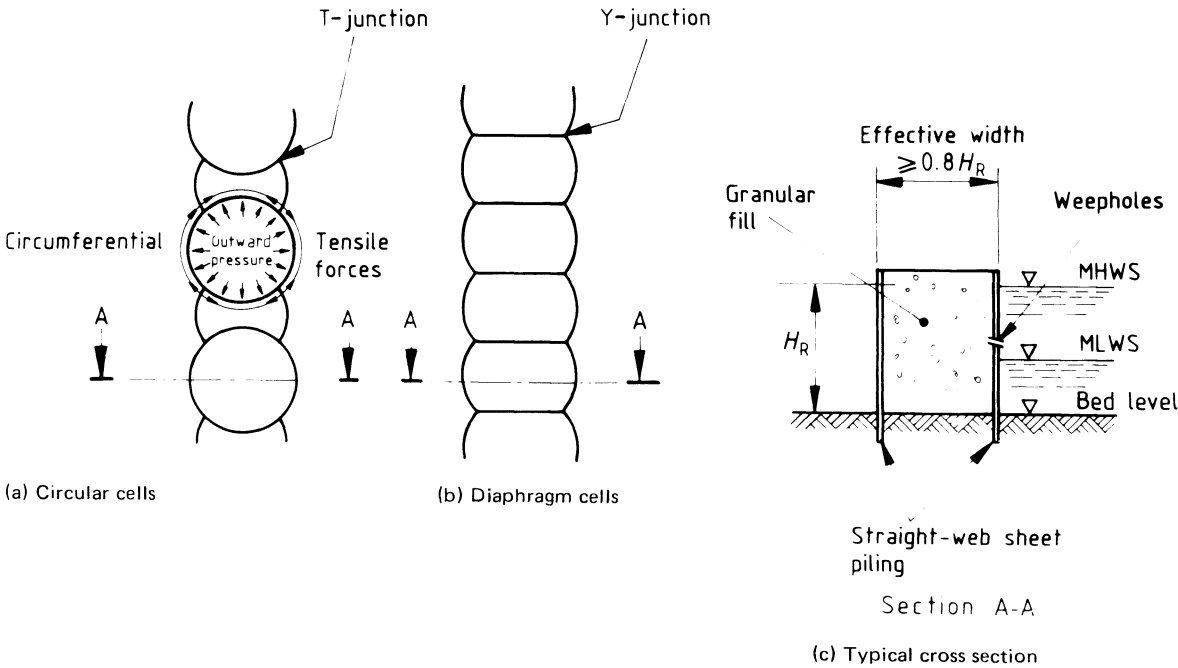


Figure 29 — Cellular sheet-piled structures

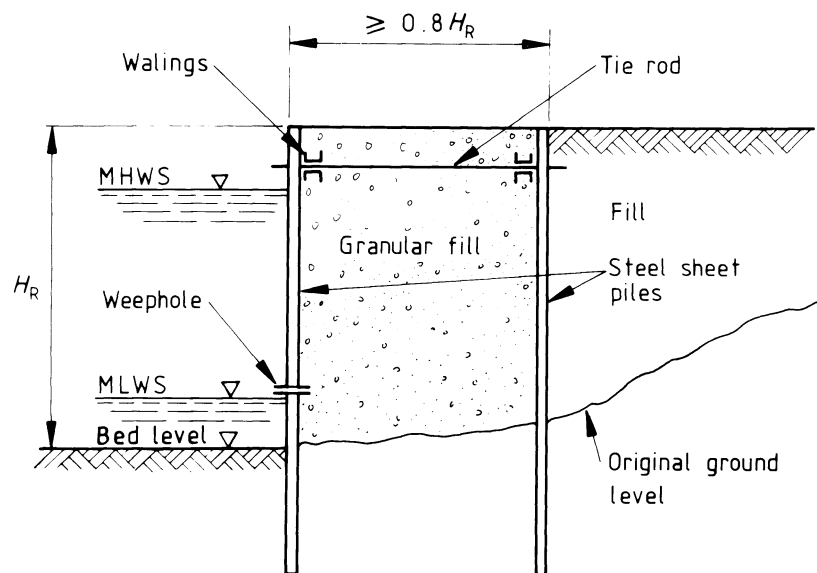
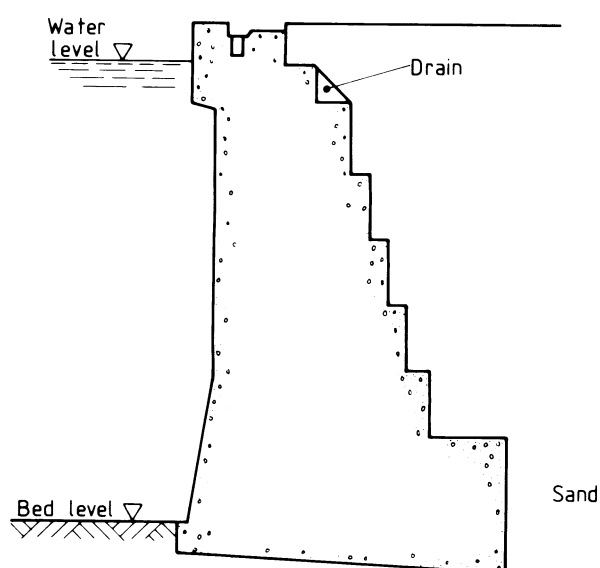
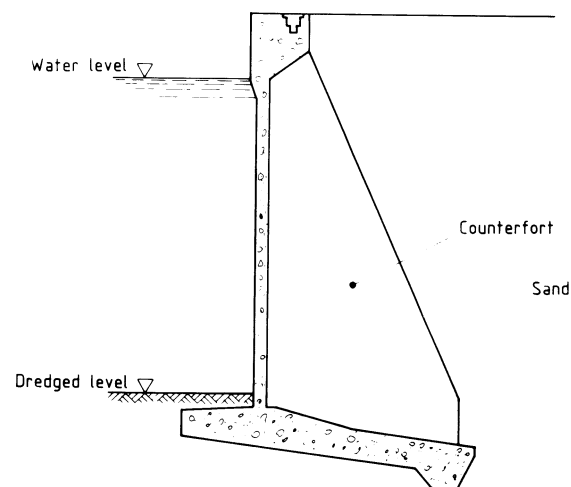


Figure 30 — Double-wall sheet-piled structure



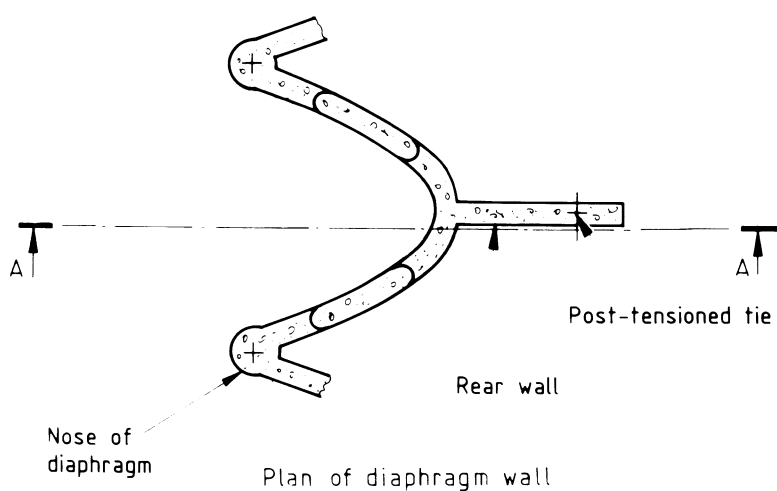
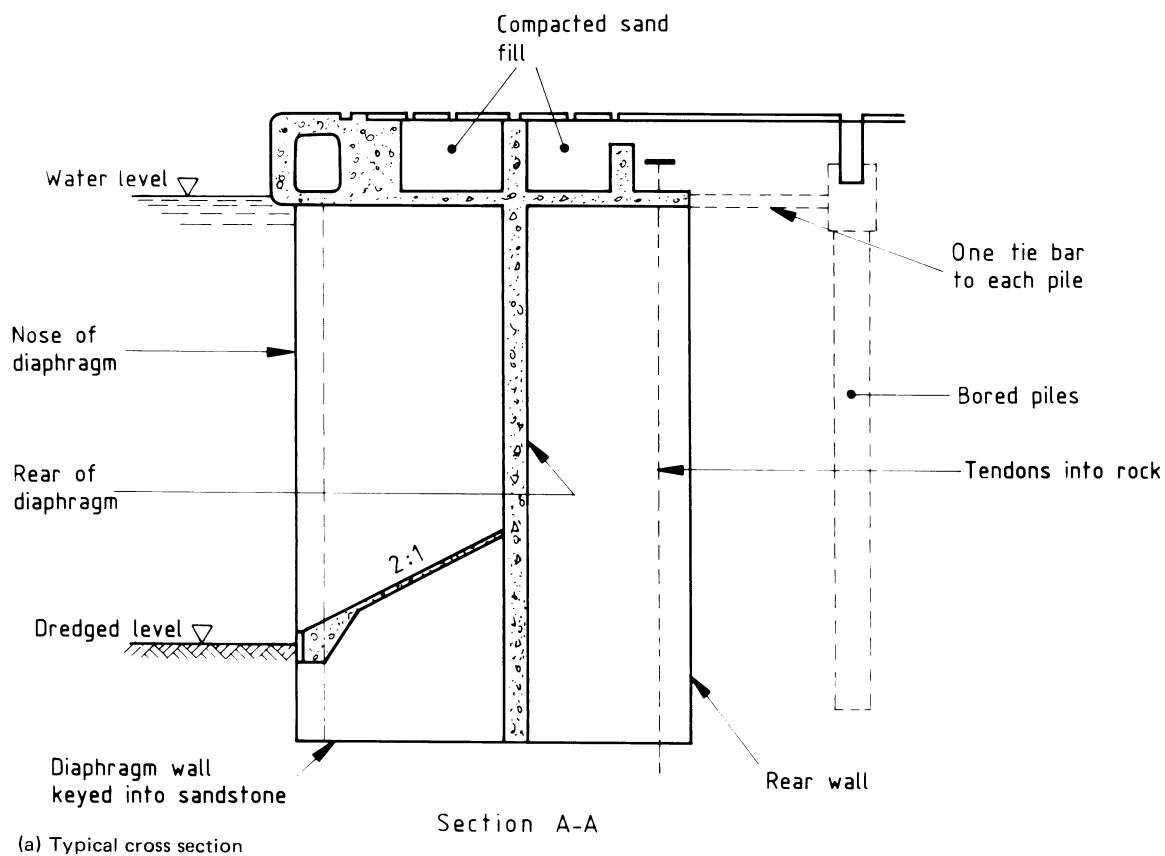
NOTE Wall may be buttressed, if desired.
Source: City of Antwerp/National Ports Council.

Figure 31 — In situ mass concrete wall



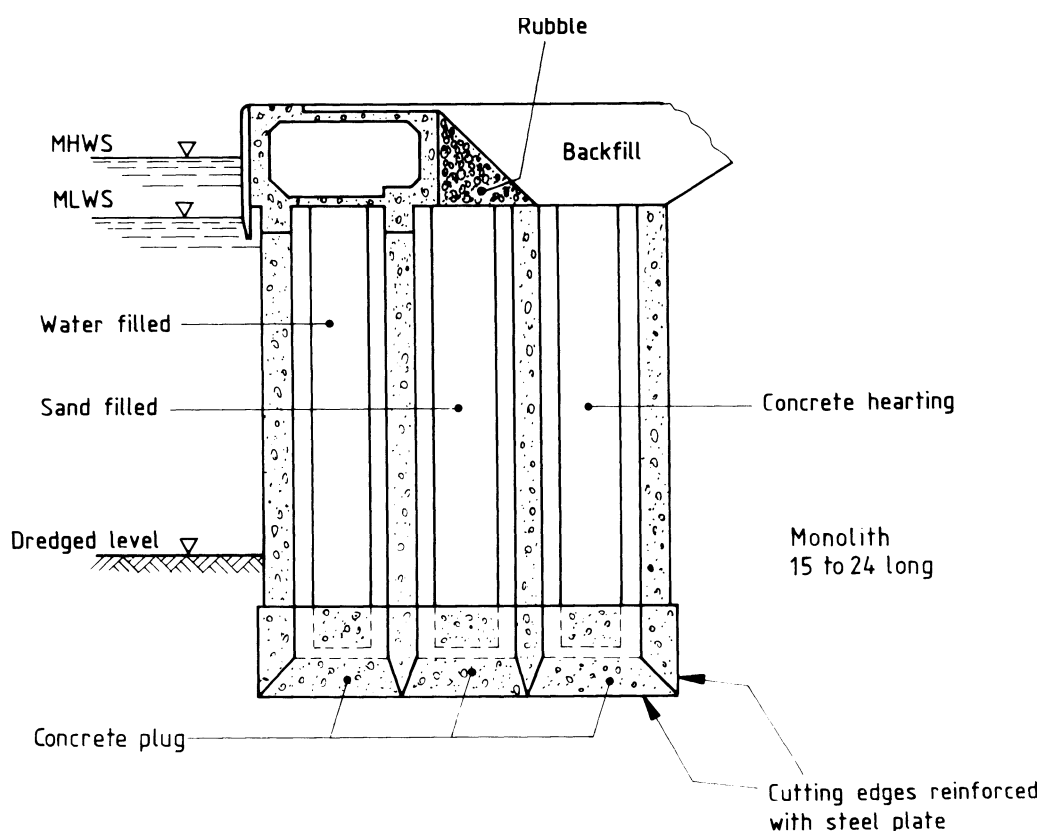
Source: City of Ghent/National Ports Council.

Figure 32 — In situ reinforced concrete wall



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

Figure 33 — Gravity diaphragm wall



Source: Coode & Partners/National Ports Council.

Figure 34 — Monolith

5.3.4.2 Services and crane rail tracks. Ducts for services and foundations for crane rail tracks can usually be accommodated in the concrete capping. Figure 26 to Figure 34 include typical arrangements of these.

5.3.4.3 Front face. BS 8002 recommends that the front faces of walls are provided with a 1 : 50 batter to avoid any illusion of the wall tilting forward. Since the front face of a quay wall is under water, a batter is not required for this reason but should be provided when necessary to compensate for forward tilting due to foundation settlement after backfilling, particularly in underwater construction.

Care should be taken to ensure that the necessary clearances are achieved (see 2.1.6.5). This generally requires the face of the in situ capping to project to seaward of the wall face. Precast facing units may be used to avoid shuttering over water.

5.3.4.4 Prevention of loss of material. Careful detailing is required to prevent loss of material from behind or in front of the wall. Filters should be used to prevent loss of fine material through joints in walls (see 2.3.3). Where a water-tight structure is required, the joints should be sealed.

Scour and wave action can remove material in front of or underneath the structure. This should be prevented by the use of a protective anti-scour apron in front of the wall.

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

5.4 Concrete blockwork walls

5.4.1 General

Heavy precast mass concrete blocks provide a robust maintenance-free structure. The blocks are usually placed to just above low water level. Above this, an in situ concrete capping is cast up to cope level.

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

5.4.2 Types of construction

5.4.2.1 Bonded blockwork. Typical examples of bonded blockwork are shown in Figure 26(a). This form of construction is less tolerant of differential settlement than sliced or column blockwork, although the effects may be reduced by completing each course of construction before placing the course above.

Bonded blocks are often keyed to provide better interlock and to assist in placing under water.

Instead of returning the wall at the ends to retain the filling, a scar (or stepped) end may be formed to protect the slope to the edge of the fill behind. This facilitates the construction of an extension without causing additional settlement in the completed wall.

5.4.2.2 Column block work. In column blockwork the blocks are stacked in vertical columns: an example is shown in Figure 26(b). The main advantage of this form of construction is that it can accommodate differential settlement.

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

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

5.4.2.3 Sliced blockwork. Blocks in sliced blockwork construction are laid at an inclination of 10° to 25° from the vertical. The short scar (or stepped) end in this form of construction permits the use of a smaller crane than other types of blockwork.

Specially shaped base blocks and abutments are required, and wedge blocks may have to be provided to correct the inclination. As a consequence a large number of different block types are required.

Blocks are provided with a tongue and groove interlock. Blocks are placed with the tongue uppermost so that the groove of succeeding blocks can be engaged and the block slid into place.

The effects of differential settlement may be partially accommodated along the inclined slip planes.

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

5.4.2.5 Arch and buttress blockwork. This type of construction is only suitable where a rock foundation exists, since differential settlement cannot easily be accommodated. Less concrete is required than for the other types of blockwork, and sliding resistance may be critical. Construction can take place in the dry by supporting lifting equipment on the previously constructed buttresses.

Two basic types of block are used in arch and buttress blockwork, one for the buttresses and one for the arches between. Blocks are placed vertically above each other.

5.4.3 Shape of blocks

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

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

5.4.4 Foundation

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

The base should be topped with a bedding layer capable of being screeded to level by divers. The base and bedding are sometimes laid at a slope to increase resistance to sliding. If settlement is expected, this may be allowed for by constructing the bedding higher than the final required level, or at a greater slope. The importance of keeping the surface clean is discussed in 5.3.1.4. The variation in level of the bedding layer surface should not exceed ± 10 mm in sheltered waters.

The base should extend at least 1.0 m on each side of the wall base to accommodate any construction tolerance and to spread the load. On the seaward face, where the bedding may be disturbed by currents or propeller action, it should be protected by a rubble or other form of anti-scour apron. The foundation in front of the rubble base may also be liable to scour, requiring an anti-scour apron.

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

5.4.5 Joints between blocks

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

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

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

5.4.6 Fill

Where the back face of the wall projects into the fill, the fill may have to be placed in stages to prevent the formation of voids.

5.4.7 In situ capping

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

5.4.8 Design of blockwork wall

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

5.4.8.2 Ground pressure. A more even ground pressure can be achieved by projecting the toe of the wall beyond the front face. To achieve the necessary clearances, the maximum toe projection may be located within the thickness of an anti-scour apron.

Further economies and a more even pressure can generally be achieved by extending some of the blocks on the landward side to counterbalance the disturbing earth forces. These blocks should generally be shaped to prevent formation of voids in the filling, although this may not be necessary where rubble backfill is used and where water movements or earthquakes are not expected.

5.4.8.3 Virtual back. Where the rear edge of any block projects significantly behind the back of the wall above, overall stability at the level of the underside of the projecting block should be calculated assuming the existence of a virtual back extending vertically upward from the rear of the block (see 1.435 of [27]⁹⁾).

The lateral pressure at failure by sliding or tilting forward should be taken as acting on concrete elements which abut or intersect the virtual back at an angle of $2/3 \phi_r$ to the horizontal (see Figure 35).

5.4.8.4 Relieving effect of overhanging elements.

Where the rear face of a capping block or precast block projects behind the rear face of the block below, partial relief of the lateral pressure over the upper part of the lower block may be assumed. This partial relief is similar to that which occurs under a relieving platform [see Figure 17(a)], and may be assumed to apply in overall stability calculations where the rear face of the lower element lies on or behind the back or virtual back of the wall. Where a large overhang occurs, as in the case of block B in Figure 35, a more thorough analysis, such as Culmann's trial wedge method, should be carried out.

5.4.8.5 Block interfaces. At the horizontal block interfaces the eccentricity of the resultant load should be limited such that the concentration of the local stresses in the blockwork does not exceed the permissible values. Except under extreme load conditions and where careful analysis has been carried out, the eccentricity should not exceed one-third of the width of the common interface. Under normal loading the local stress should not exceed $0.5 f_{cu}$, but under extreme loading a stress of $0.6 f_{cu}$ is acceptable.

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

5.4.8.6 Hydrostatic uplift. Full hydrostatic uplift should be assumed in horizontal joints. Where sealed joints are provided, the possibility of uplift should be considered.

⁹⁾ These values will be included in BS 8002.

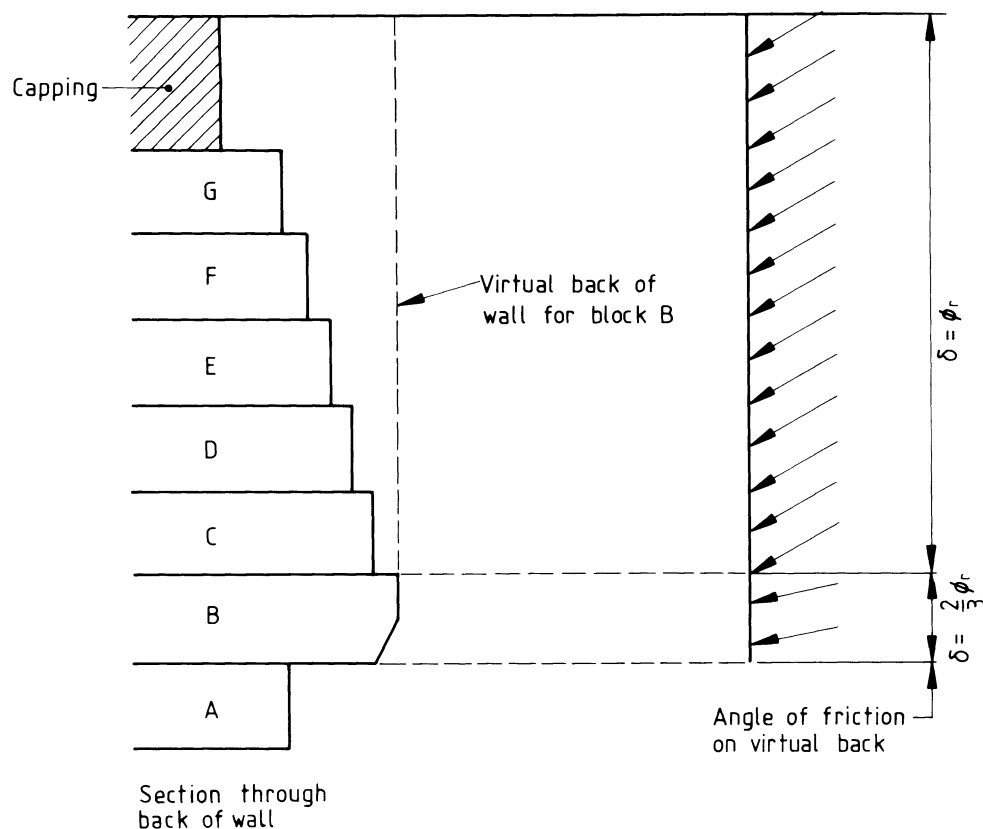


Figure 35 — Angle of friction on virtual back of blockwork wall

5.5 Precast reinforced concrete walls

5.5.1 General

Precast reinforced concrete walls are used in maritime works, both as quay walls and as bulkhead walls adjacent to suspended deck quays. Plain cantilever walls are generally uneconomic for heights in excess of 8 m and for greater heights a counterfort wall may be used.

Units more than 18 m high have been used (see Figure 27).

The units are usually designed so that the top is just above low water level after placing. An in situ concrete capping is cast above this up to cope level after filling.

Less concrete is required than for a blockwork wall, but a considerable weight of reinforcement has to be provided and attention has to be paid in design to minimizing the possibility of corrosion. Precasting of the wall units is more difficult than for blocks and may require slipforming.

The viability of using this form of construction for deepwater berths depends on the availability of suitable lifting equipment, either floating or on a jack-up pontoon, and on there being a sufficient length of wall to spread the high cost of plant and formwork. Tight construction tolerances are essential, and this type of wall is unlikely to be suitable at sites subject to appreciable wave or current action.

5.5.2 Foundation

The recommendations given for blockwork walls (see 5.4.4) apply equally to precast reinforced concrete walls. For high walls, the tolerance on level of the top of the rubble foundation should be determined in relation to the design of the joints between units.

5.5.3 Precast units

The toe of the precast units should be arranged so that it does not project into the berthing area (see 2.1.6.5).

Crack widths in the walls of the precast units should be determined for all maximum bending conditions, including during each stage of construction. Crack widths are discussed in 2.4.4.2.

Methods of lifting and handling the units should be devised during design. If necessary, lifting points should be specified.

Generous splays should be provided at internal corners to minimize local cracking. Arrises should be chamfered to minimize mechanical damage.

5.5.4 Joints between units

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

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

5.5.5 Fill and drainage

Fill material placed behind the wall units may be fine granular material or rubble. The maximum size of stone may have to be restricted on account of the high local loads which could be imposed on the wall.

If the tidal range is significant, drain holes or slots should be provided to reduce tidal lag. A suitable filter should be installed around drainage holes to prevent loss of material.

5.5.6 In situ capping

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

The capping beam may be assumed to be continuously supported vertically, and horizontally in the case of cantilever walls. For counterfort walls, the capping beam is assumed to span horizontally between counterforts.

The capping and fenders should extend sufficiently to seaward of the wall to avoid vessel impact on the wall units.

5.5.7 Design of precast reinforced concrete wall

Recommendations on the design of retaining walls with spread footings are given in BS 8002. Stability at all stages of filling as well as in service should be assessed.

The virtual back of the wall should be used in calculating lateral pressures for overall stability as described in 1.435 of [27]. The wall stem should be designed assuming $\delta = 0$.

5.6 Concrete caissons

5.6.1 General

Concrete caissons consist of open-topped cells prefabricated in the dry which are usually floated to their final location and then sunk into position on the seabed. Caissons are usually designed so that, after sinking, the top is just above low water level with due allowance for waves. The cells are filled, usually with sand and sometimes with concrete or gravel. The superstructure may consist of a solid in situ concrete capping, or a reinforced concrete edge retaining wall which is backfilled and the top surfaced with concrete paving. One possible capping arrangement is shown in Figure 28.

The positioning tolerance on caissons can usually be greater than that for blockwork or precast wall units. Caissons can therefore often be used where wave disturbance is appreciable.

Caissons, after filling, form self-stable structures which can be used to support heavy construction equipment. They can be used for both quay walls and jetties.

As the cost of temporary works is high, the viability of this form of construction depends on having a sufficient number of caissons. Water of sufficient depth is also required for manoeuvring the floating caissons between the casting area and the final location, unless additional buoyancy tanks are used.

5.6.2 Shape

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

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

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

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

Caissons without compartments may be built using precast concrete units as a shutter for placing thick mass concrete walls. After the caisson has been positioned, tremie concrete is used to form the internal base and to fill the wall voids. The caisson centre may be left unfilled to reduce bearing pressures and virtual mass during earthquakes.

Caissons should be provided with generous splays on internal corners to avoid local cracking and with chamfered arrises to minimize mechanical damage.

5.6.3 Foundation

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

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

5.6.4 Fabrication

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

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

Economies can often be achieved by casting, on land or on a pontoon, the minimum height of the caisson required to permit flotation, then launching. Fabrication is completed by casting the remaining height of the walls while afloat, the draught increasing progressively.

Casting caissons afloat demands reasonably calm water conditions.

5.6.5 Floating conditions

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

An accurate knowledge of tidal conditions is essential to the success of the positioning operation.

To assist in sinking the caisson, stopcocks may be fitted in the outer walls.

5.6.6 Fill

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

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

5.6.7 Joints between caissons

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

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

Examples of joints are shown in Figure 36.

5.6.8 Drainage and filters

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

5.6.9 In situ capping

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

5.6.10 Design of caisson wall

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

5.6.10.2 Ground pressure. The ground pressures at foundation level should be determined for all conditions of loading, assuming that the base slab of the caissons is rigid.

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

A partial loss of ground support should be assumed in the design of long caissons, which should be analysed as beams in the longitudinal direction.

Design of base slabs should take into account the possibility of loss of ground support under filled cells and of higher than normal ground pressures under empty cells, where these occur. The slabs are assumed to be supported by the compartment walls.

5.7 Cellular sheet-piled structures

5.7.1 General

Cellular sheet-piled structures consist of cells formed by interlocking straight-web steel sheet piles, driven or placed with their tops above water level. The cells are filled with granular material. The superstructure may be a solid in situ concrete capping, or a reinforced concrete edge retaining wall which is backfilled and the top surfaced. This type of structure is illustrated in Figure 29.

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

Sheet-piled cells may be founded on soft rock, granular material or very stiff clay. Where soft clays exist, they should be removed before filling the cells. This soft material should be dredged prior to cell construction at sites where the soft material layer is very thick, to avoid causing compression of the cells.

Cellular structures may be used for both quay walls and jetties. They are usually cheaper to construct than double-wall sheet wall structures because the steel resists the lateral soil loads in hoop tension rather than bending, and walings, tie rods and bulkheads are generally not required.

This type of structure is particularly liable to damage by wave action during construction as it possesses little resistance to horizontal loads before the internal filling is completed.

5.7.2 Types of construction

Cellular walls are usually of either the circular or the diaphragm type. Figure 29 shows a typical plan of each. As a general rule, the effective width of the wall should be not less than 0.8 times the retained height.

5.7.3 Materials and stresses

Sheet piles for cellular walls are of the straight-web type to transmit horizontal tension. Two single standardized sections are available in steel grades 43A and 50A in accordance with BS 4360.

Junction sections comprising T or Y shapes are generally available. Extruded Y-sections are manufactured in some countries for use where high tensile forces are to be resisted.

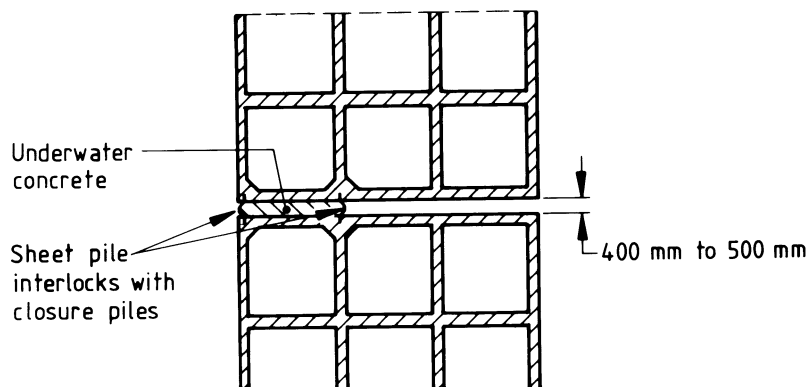
A factor of safety of 2.0 on the ultimate interlock strength at the end of the design life should be applied to the maximum working forces.

5.7.4 Construction considerations

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

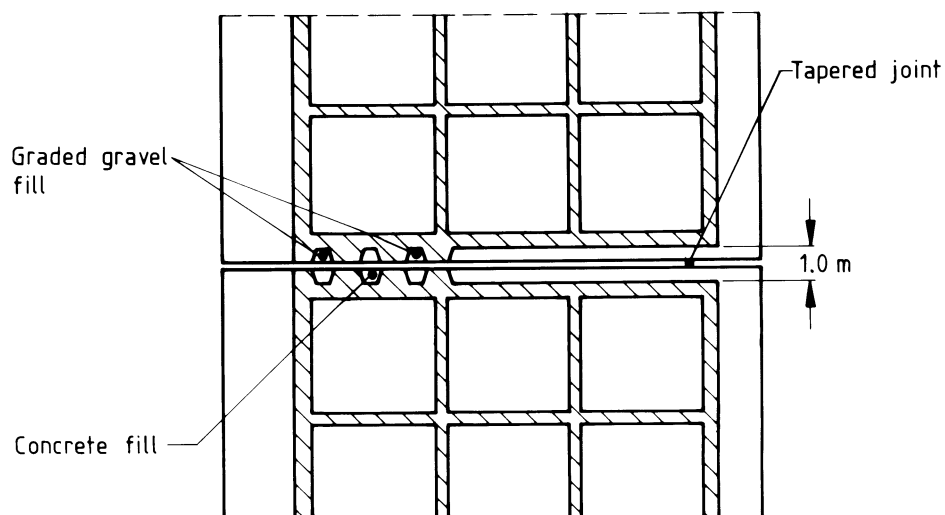
The piles should not be driven hard into rock and, where rock level is highly irregular, consideration should be given to levelling the surface to minimize differential stresses in the steel.

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



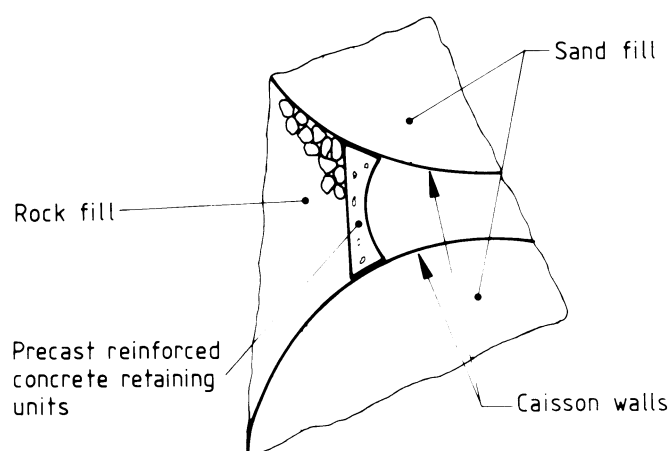
Source: EAU 1985, Wilhelm, Ernst & Sohn [12].

(a) Using sheet piles



Source: EAU 1985, Wilhelm, Ernst & Sohn [12].

(b) Using concrete shear keys



Source: Bertlin & Partners.

(c) Using precast concrete units between circular caissons

Figure 36 — Examples of joints between caissons

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

5.7.5 Fill

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

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

5.7.6 Drainage

The cells should be provided with drainage holes. Drainage and the effects of pumped hydraulic filling are covered in 5.1.5.2 of BS 6349-1:1984.

5.7.7 In situ capping

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

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

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

5.7.8 Design of cellular wall

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

Guidance on the design of cellular structures given in this subclause amplifies that given in BS 8002 and [25].

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

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

5.7.8.3 Modes of failure. Investigations of the stability of cellular sheet-piled structures should include consideration of the modes of failure shown in Figure 37.

Where the sheet piles are driven into soft clay, the possibility of heave within or beneath the cells should be investigated [28].

5.7.8.4 Modes of resistance. Where the sheet piles are driven into soil below dredged level [see Figure 38(a)], the resistance to sliding or overturning of the structure will be provided by passive resistance of the soil below dredged level, pull-out resistance of the sheet piles on the landward side, friction on the base of the filling material, and weight of the structure and infill material.

Where the sheet piles are driven on to hard rock at, or a short distance below dredged level [see Figure 38(b)], the resistance to overturning and sliding will be provided by base friction and gravitational forces only.

5.7.8.5 Stress in junction piles. In addition to calculating interlock tension in the main cells, the tension at the junctions of arcs and circular cells should be estimated since the largest circumferential forces can occur here. In large diameter cell walls, the stress at junctions may be high enough to warrant the use of special junction piles (see 5.7.3).

5.7.8.6 Economies in steel sheet piles. The landward connecting-arcs of circular-type cellular quay walls may be eliminated, provided that the average effective width of the structure is reduced to compensate for that length of wall between the cells which is effectively zero where the rear arc is eliminated. Omission of the rear arcs from diaphragm type cellular structures is unlikely to prove economic.

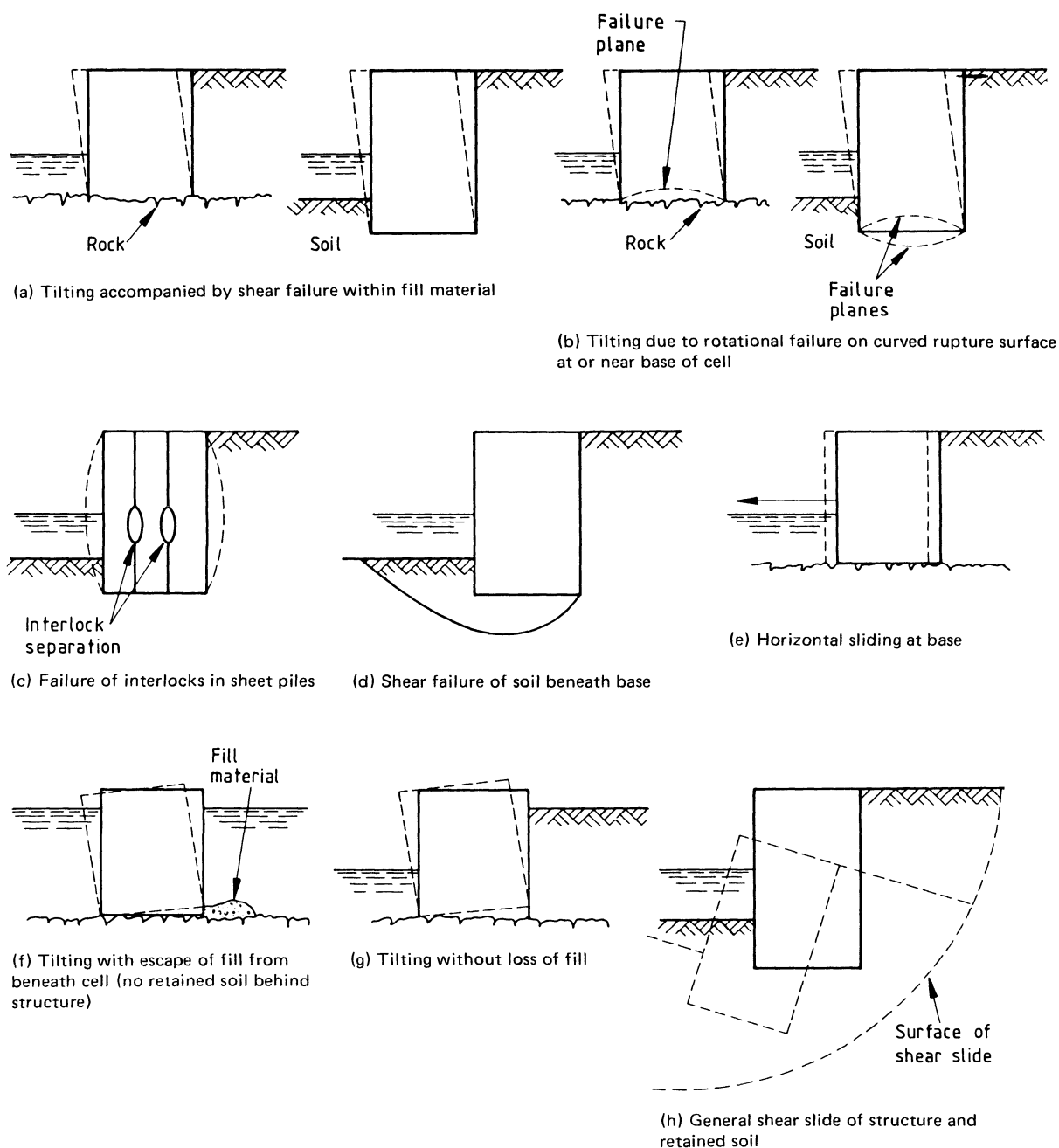


Figure 37 — Modes of failure of cellular sheet-piled structure

Provided that allowable driving stresses and interlock tension are not excessive, a lighter section of lower grade steel may be used for the sheet piles on the landward side of a quay wall, in order to reduce costs. The toes of these sheet piles may be installed at a higher elevation than those on the seaward side provided that loss of material would not result and that the total frictional resistance between the piles and the fill remains adequate. The tops of the landward sheet piles of quay walls may also be stepped down to reduce the quantity of steel.

5.7.8.7 Passive resistance of berm. Where passive resistance is provided by a berm along the front of the wall, the reduced resistance due to the slope of the berm should be taken into account.

Consideration should be given to whether the wall will move sufficiently to mobilize full passive resistance within the depth of the berm.

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

5.8 Double-wall sheet-piled structures

5.8.1 General

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

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

Double-wall structures are most suitable when founded in medium or dense granular soil or firm to stiff clay. Where the seabed is mainly of rock, pretreatment may be necessary to ensure penetration of the sheet piling. Where the double-wall structure is installed in a soft clay, it will be necessary to remove the soft clay from between the walls before placing the fill. Where the thickness of the soft layer is large, this may need to be done by dredging prior to wall construction. The possibility of heave of the soil below excavation level should be investigated [28].

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

5.8.2 Type of construction

Combined action of the two walls is achieved either by tying the tops of the walls together and providing adequate penetration of the sheet piling (see Figure 39) or, where this is not practicable, for instance in hard rock, loose sands or soft clays, by connecting the walls at a low level using tie-rods. The latter alternative will generally require underwater working.

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

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

5.8.3 Materials and stresses

Requirements for sheet piles in double-wall structures are similar to those for single sheet walls, and steel U- and Z- sections will usually be suitable. The section modulus of the piling is calculated as in 4.5.1.

For materials and stresses in the sheet piles, walings and tie-rods, reference should be made to section 4.

5.8.4 Fill

Fill placed between the double walls should have the qualities described in 5.7.5.

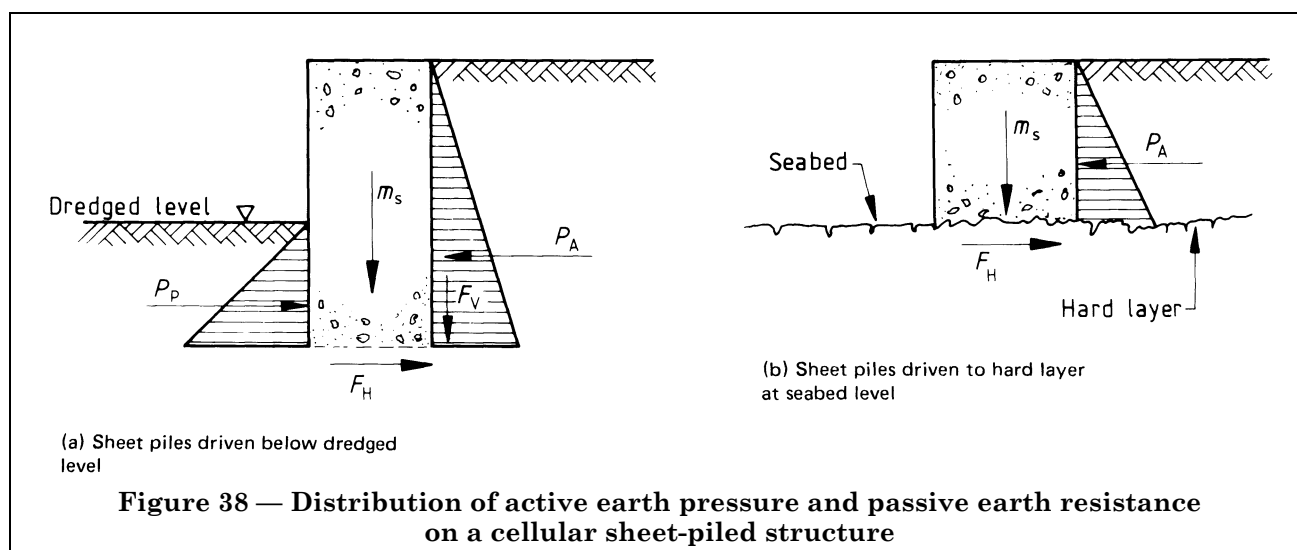
Backfill behind the structure may be coarse sand, gravel or rubble.

5.8.5 Drainage

Drainage holes should be provided in the structure. Drainage and the effects of pumped hydraulic filling are covered in 5.1.5.2 of BS 6349-1:1984.

5.8.6 In situ capping

The capping on the wall at the seaward face(s) should be designed in accordance with 4.9.



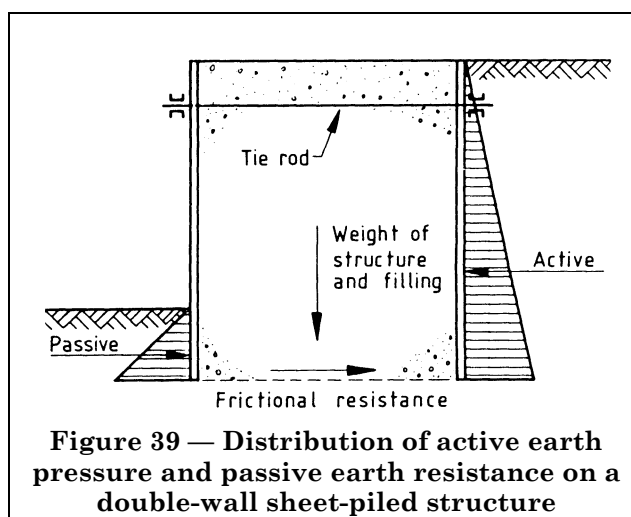
5.8.7 Design of double-wall structures

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

Guidance on the design of double-wall structures given in this section amplifies that given in BS 8002.

5.8.7.2 Lateral pressures. Where the double-wall structure is founded on soil such that lateral earth pressures are resisted partly by passive resistance of the soil below dredged level, the structure will deflect towards the water side. For these conditions, the lateral earth pressure on the landward side of the wall may be taken as active (see Figure 39). Where the two walls are installed on a hard rock surface at, or a short distance below, dredged level, the lateral earth pressure on the landward side of the wall will be in a condition intermediate between the active and at-rest states, depending on the amount of horizontal deflection of the structure.

The internal soil pressures acting on the outer wall(s) should be assumed to be 1.25 times the active values.



5.8.7.3 Internal stability. The internal stability of the structure should be checked assuming a lower failure plane within the fill, in the same way as described in 4.7.1.1 for a single sheet wall with cantilever anchorage.

5.8.7.4 Foundations. The bearing capacity of the soil beneath double-wall structures should be adequate to support the weight of the structure and infill material, any superimposed loading on the walls and the vertical forces transmitted to the landward side wall due to wall friction and the impact forces of berthing vessels, where applicable. Guidance on methods of determining the factor of safety against failure in shear of the soil beneath the structure is given in BS 8004.

5.8.7.5 Analysis of structural members. The bending moments and shear forces in the sheet piling and the forces in the connecting tie-rods should be calculated in the same way as for anchored single sheet walls.

5.9 In situ mass concrete walls constructed under water

In situ mass concrete walls may be of solid or voided construction (see Figure 31). Alternatively a buttressed-arch form of construction may be used to reduce the volume of concrete. This type will generally require ground anchors to provide the stability required, and these should be designed in accordance with DD 81.

The tensile resistance of concrete is not normally utilized in design, but where tension does occur, stresses should be in accordance with the recommendations in 5.3.3. No tension should be assumed at horizontal construction joints unless they are carefully detailed to transfer the tension and water bars are provided to prevent the ingress of water and development of hydrostatic uplift at the joint.

Tension may be acceptable in the concrete when it occurs under local or extreme random loads. In this situation, the value should not exceed that given in Table 11 of [27]¹⁰.

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

Where a sheet wall is used as a permanent shutter on the face of a mass concrete wall, ties are normally provided to anchor the sheet wall to the concrete. It should generally be assumed that full hydrostatic pressure will be developed between the sheet wall and the concrete. The ties used to anchor the sheet wall to the concrete should be designed to resist the full hydrostatic pressure.

5.10 In situ reinforced concrete walls

In situ reinforced concrete walls (other than diaphragm walls, which are discussed in 5.11) should be constructed in the dry, either behind a cofferdam or within an excavation in existing ground. Walls of this type are unlikely to be economic unless built over a substantial length in one operation.

For walls up to about 8 m high, a plain cantilever design may be used. For higher walls, a counterfort or buttress design will be required (see Figure 32). Economies may be made by employing ground anchors, which can be used to resist both overturning and sliding. Ground anchors should be designed in accordance with DD 81.

The recommendations given in 5.5 are generally applicable to this form of construction, except where specifically related to the use of precast concrete.

5.11 Diaphragm walls

Diaphragm walling can provide an economical form of construction for gravity quay walls. The walls are generally constructed in cellular or tee formations to just above water level, and an in situ concrete capping built on top.

Figure 33 shows an example of a gravity diaphragm quay wall.

The most common application for this form of construction is in sheet walls, which are covered in section 4.

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

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

In most soils, forward movement of the toe will be prevented by surface friction with only modest penetration of the panels below the bed, but slight tilting of the wall may occur in soft ground if slurry is left at the bottom of the trench. This movement is generally just sufficient to mobilize the frictional forces. It is desirable to make a full-scale (axial) jacking test on site, particularly if the nature of the soil varies with depth: the test may also be used to check the lubrication effect of any bentonite which may remain at the wall/soil interface.

¹⁰) This table will be included in BS 8002.

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

When walls of this type are built on an unyielding foundation, at-rest earth pressures are likely to apply. Additional resistance to overturning or sliding may be obtained using vertical rock anchors, in accordance with DD 81.

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

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

5.12 Monoliths

5.12.1 General

Monoliths, also known as open caissons, are built directly above their final position by alternately raising the walls and sinking the structure into the ground. They are generally constructed using mass concrete or lightly reinforced concrete.

Monolith construction commences with fabrication of cutting edges. The walls are progressively extended in the dry, keeping step with the sinking of the monolith so as to maintain the top of the walls above water level at all times. Construction can take place to any depth provided that the lateral soil pressures can be accommodated. Once the monolith walls have been completed, an in situ concrete plug is poured in the base and the monolith filled with lean-mix concrete as required. The joints between monoliths are then sealed and an in situ capping built. An example of a monolith is shown in Figure 34.

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

This form of construction is usually expensive and is likely to provide an economic solution only in very poor ground. In ground containing flowing groundwater, monoliths may be more suitable than diaphragm wall construction.

5.12.2 Shape

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

5.12.3 Sinking

The interior of the monolith is usually excavated by grabbing, with help from divers if there are obstructions under the cutting edges.

Sinking may be assisted by bentonite lubrication of the outside face, by jetting (in granular soils), and/or by kentledge on the monolith walls. In cohesive soils, the effective weight of the monolith may be increased by dewatering. In granular soils, dewatering should not be done because of the danger of causing a blow-out.

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

5.12.4 Plugging

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

5.12.5 Fill

The cells may be filled or left empty, depending upon stability considerations.

5.12.6 Joints between monoliths

The spaces between monoliths, which are typically between 0.5 m and 3 m wide, may be sealed by means of piles (see Figure 40). The space may be excavated and then filled with concrete or, in weak ground, grout intrusion of preplaced aggregate may be more suitable. Instead of a continuous concrete seal a series of precast concrete piles may be used to infill the space. If movement of the monoliths in service is expected, a flexible seal of gravel may be used instead of concrete.

5.12.7 In situ capping

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

5.12.8 Design of monolith wall

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

Overall stability may, if necessary, be improved by the addition of tie-backs (see section 4).

The distribution of ground pressure under the base should be checked and the founding level should be designed to ensure that material cannot be removed from underneath the base by scour.

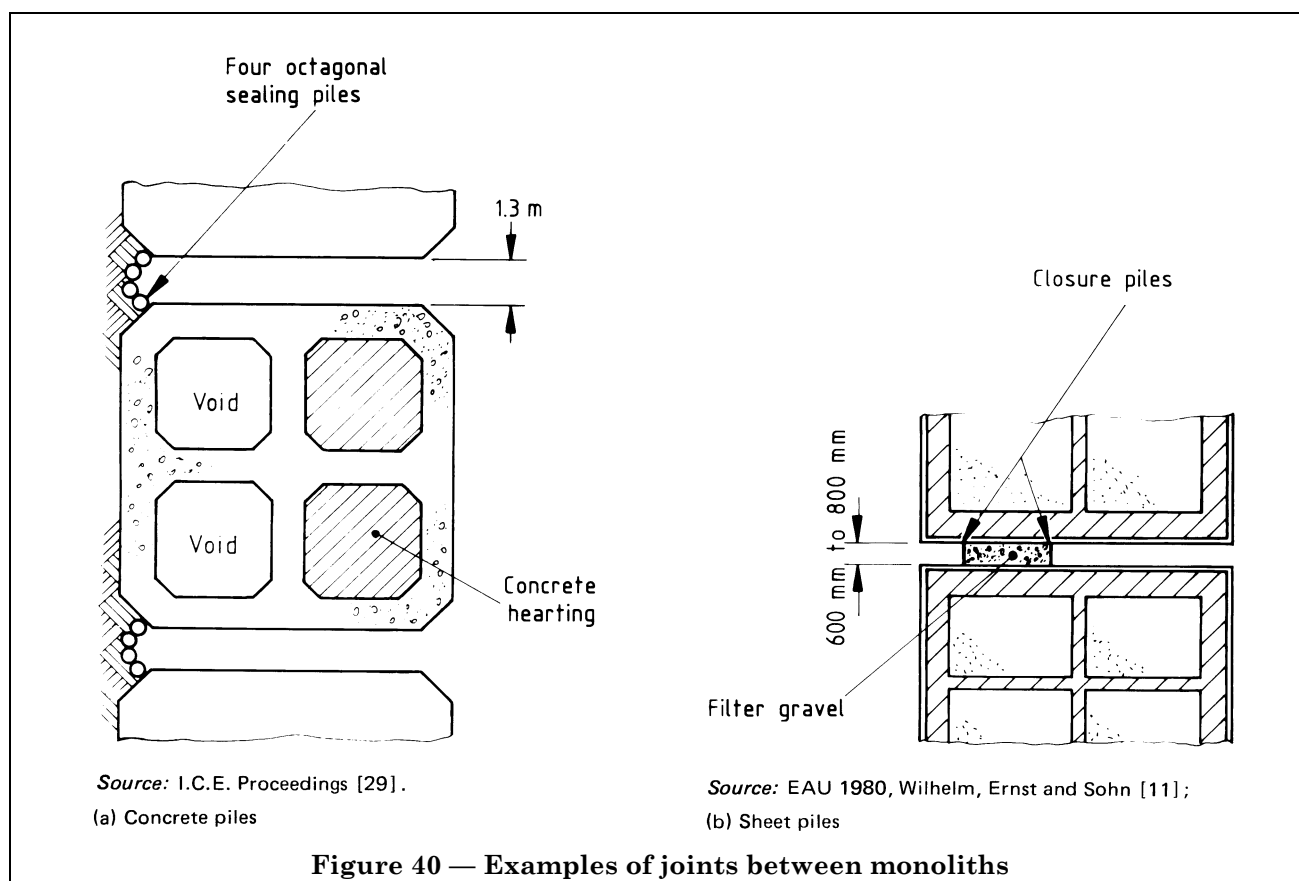
5.12.8.2 Concrete plug. The concrete plug should be designed to resist the maximum uplift pressure occurring at foundation level.

5.12.8.3 Concrete walls. The walls should be designed for all conditions of lateral pressure as stated in 5.6.10.3. The design of the monolith should also take into account the uneven bearing of the cutting edges during sinking.

In its final position, the presence of the concrete plug will ensure an even bearing of the base.

Design of the mass concrete walls should take into account the provisions of 5.3.3. Anti-crack steel reinforcement may be provided in accordance with 5.9.

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



Section 6. Suspended deck structures

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

6.1 General

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

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

Structures may be classified as rigid or flexible (see 47.1 of BS 6349-1:1984) as regards their response to horizontally applied loads. The degree of flexibility depends upon the materials used, the ground conditions and the framing of the structure.

6.2 Suitability

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

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

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

6.3 Applications

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

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

6.4 Types of structure

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

The most common pile systems, with the method of resisting horizontal loads, are given in Table 1, and illustrated in Figure 41.

Raking piles should not normally be employed in compressible ground owing to the progressively increasing vertical loads due to settlement and downdrag causing bending of these piles, unless the bending can be economically resisted.

Horizontal tie-backs (see section 4) may be provided where the upper strata are poor or where large transverse loads, such as those due to soil or earthquake, are to be resisted.

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

6.5 Overall stability

6.5.1 General

The proposed structure should be checked for overall stability by reference to BS 6031, BS 8004 and in accordance with 6.5.2 and 6.5.3.

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

6.5.2 Slope failure

The stability of the slope under a marginal quay should be analysed for all critical loading conditions. These will include the placing of fill to raise the ground level behind the quay, pile driving, the imposition of heavy loads behind the quay, and at some locations, earthquake loads. The analysis should take account of any variations which could occur in water levels and pore water pressures, including the possibility of liquefaction.

Where long-term lateral soil loads could be applied to the piles, ground improvement or other suitable measures should be taken in advance of pile driving to prevent this.

6.5.3 Settlement or uplift of structure

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

Table 1 — Pile systems for suspended deck structures

Type of structure	Application	Flexible/rigid (see 6.1)	Method of resisting horizontal loads	Figure reference
All vertical piles	Jetty heads	Flexible	Bending of piles	Figure 41(b)
All vertical piles	Marginal quays	Partly flexible and partly rigid	Bending of piles, plus passive resistance of backfill	Figure 41(a)
All vertical piles plus horizontal tie-backs	Marginal quays	Partly flexible and partly rigid	Bending of piles, plus horizontal tie-back (seaward loads), passive resistance of backfill (landward loads)	—
All vertical piles plus strut/tie members to shore	Marginal quays	Rigid	Strut/tie	Figure 41(c)
Vertical and raking piles	All	Rigid	Raking piles	Figure 41(d)
All raking piles	Access trestles and jetty heads	Rigid	Raking piles (transverse loads), longitudinal raking piles or shore anchorage (longitudinal) loads	—

6.6 Earthworks at marginal quays

6.6.1 General

Earthworks associated with suspended deck structures will be required only for marginal quays, except in the case of finger jetties requiring dredging. Further guidance may be obtained from BS 6031 and BS 6349-5¹¹⁾; points related to anchorages and sheet bulkhead walls will be found in section 4.

6.6.2 Settlement

The treatment of settlement is discussed in 2.3.1.

6.6.3 Slope stability and protection

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

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

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

Figure 41 shows typical examples of slope protection under suspended deck quays.

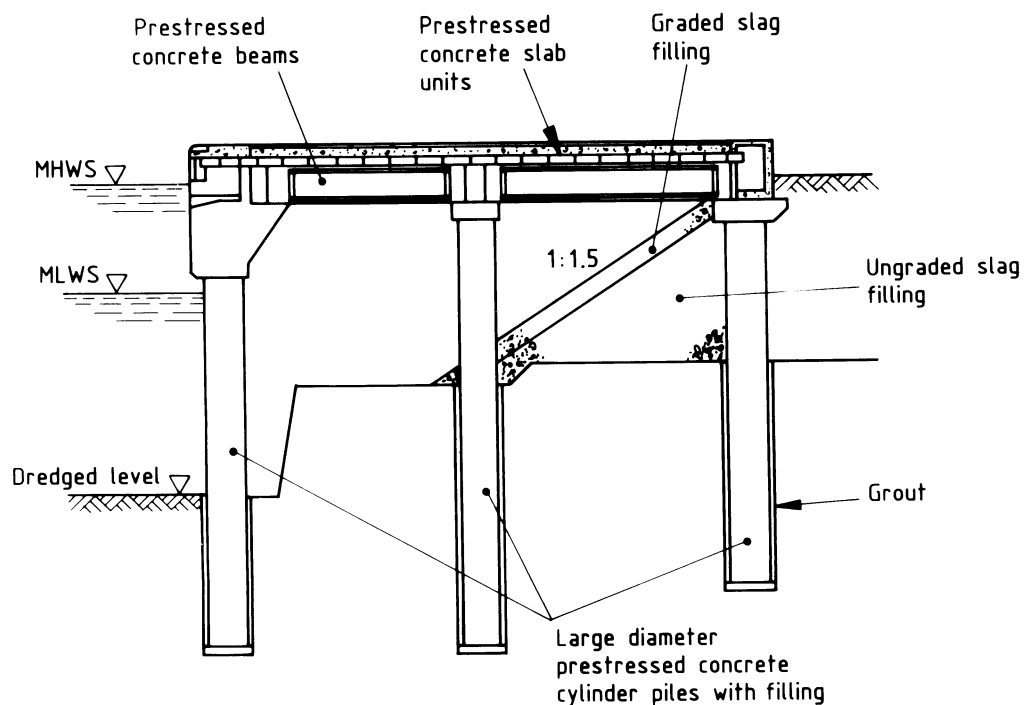
The range of levels over which a filter is required on the face of the slope should be carefully considered. A filter may also be required at the base of a rubble embankment or slope. The possibility of loss of fill from the top of the slope under the rear edge of the superstructure due to ground water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

6.6.4 Drainage

If the fill behind a suspended deck marginal quay is not a free-draining material, a drainage medium should be provided along the rear face of the superstructure to connect with the rock slope below: the drain may be in the form of a rock filter or a proprietary drainage strip. This will prevent local instability of the fill behind the quay, and thereby maintain the lateral soil pressure for resisting berthing loads on the rear face of the superstructure.

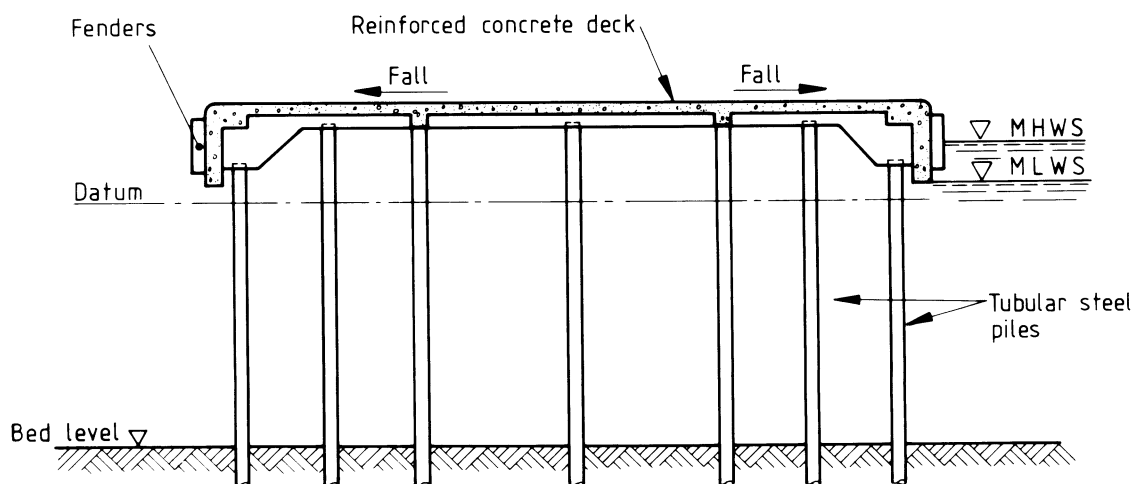
Drainage of the soil above relieving platform slabs should be provided, where necessary, to prevent inundation of the surfacing (see 2.3.3).

¹¹⁾ In preparation (see foreword).



Source: Sir William Halcrow & Partners/National Ports Council.

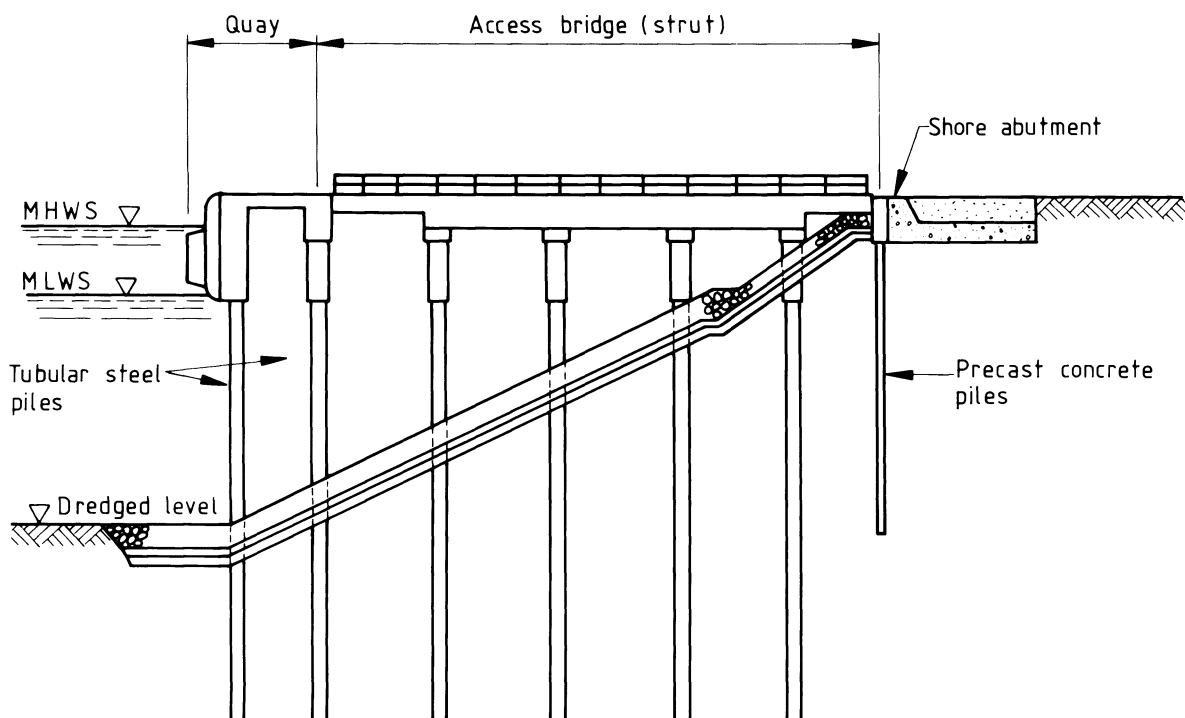
(a) All vertical piles of large diameter



Source: Parsons, Brown & Newton.

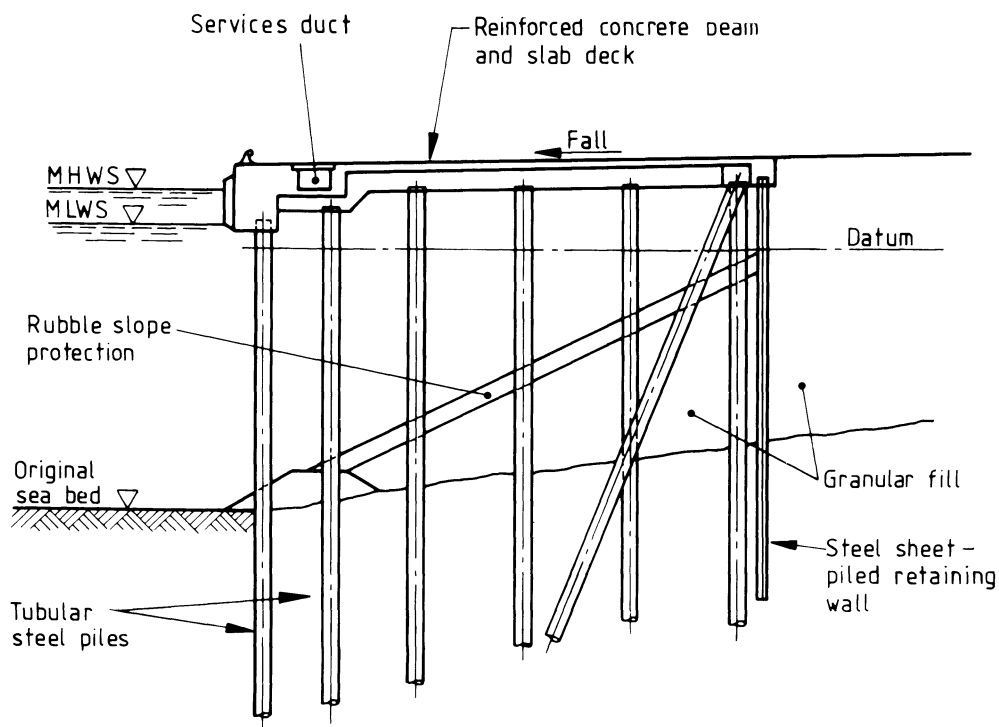
(b) All vertical piles for jetty with berthing on both sides

Figure 41 — Examples of suspended deck structures



Source: Bertlin & Partners.

(c) All vertical piles plus struts to shore



Source: Parsons, Brown & Newton.

(d) Vertical and raking piles

Figure 41 — Examples of suspended deck structures (concluded)

6.7 Types of bearing pile

6.7.1 General

For detailed provisions on the types of bearing pile described in 6.7.2 to 6.7.6, reference should be made to 6.1.2 of BS 6349-1:1984 and elsewhere [30].

6.7.2 Timber piles

Timber piles are easy to handle and cut to length, they require only simple driving equipment, and their flexibility may be useful in energy-absorbing structures. The length of timber piles is generally restricted (see 7.4.1.1.1 of BS 8004:1986 but they may be jointed to achieve deep penetration in weak ground). Timber should generally be treated (see 6.0.2 of BS 6349-1:1984) or cased with concrete muffs to prevent attack by marine borers.

6.7.3 Precast concrete piles

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

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

6.7.3.2 Prestressed concrete. The comments in 6.7.3.1 apply generally to prestressed piles. Prestressed piles, however, are better able to resist tensile stresses during handling, driving and in service than reinforced piles. A smaller cross section may therefore be possible: this may be advantageous if downdrag is a problem. A prestressed pile will have a greater load capacity than a reinforced pile of the same section.

6.7.4 Cast in situ piles

NOTE In situ cast in situ piles are not recommended for maritime structures. However, where exceptional circumstances prevail the following guidance is given.

6.7.4.1 Driven casings. Driven casings for cast in situ piles may be used in preference to precast concrete piles to save handling, to permit a larger section to be used, to reduce driving requirements, or at sites where the pile lengths are irregular. The piles should normally be used to resist axial loads only, unless control of the concrete placed through reinforcement can be assured. Reinforcement should be provided in compression piles from the head to a short distance below seabed level, and over the entire length in tension piles.

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

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

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

Bored piles may be excavated by percussive equipment in sizes up to 40 m in length, 1 200 mm in diameter and 1 : 3 rake. Augered bored piles, which will generally be formed in cohesive soils requiring little casing, may be up to 3 000 mm in diameter, 1 : 3.5 rake and lengths of 100 m or more. Concrete for bored piles should normally be placed by tremie.

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

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

Diaphragm wall construction technique is covered in clause 55 of BS 6349-1:1984.

6.7.5 Steel piles

Steel piles are relatively light and easy to handle, and can be driven in most types of ground, including many types of rock: H-piles are particularly suitable for this. H-piles are also useful as tension piles because of the large penetrations which can be achieved. Circular (and box) piles are more suitable where large axial and bending stresses are to be resisted (in corrosive conditions they are less vulnerable because they have less exposed surface). The advantages of each type may be employed in a composite pile comprising a bottom H-section, driven to achieve uplift resistance and welded to the inside of a tubular upper section which is used to resist vertical and horizontal loads.

Consideration should be given to the fracture toughness of welds, where appropriate.

Steel piles have good energy-absorption properties and may therefore be used, when installed vertically, to resist horizontally applied loads. If a concrete encasement is added for corrosion protection, considerations of maximum strain may require the permissible energy absorption to be reduced.

Steel piles may be extended without undue delay by butt-welding an extension piece, and they are therefore useful at sites where the pile lengths cannot be accurately predicted. Guidance on butt welds in steel bearing piles is given elsewhere [18].

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

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

NOTE Recommendations on fracture toughness were previously given in 6.6.3.1 of BS 6235:1982 which is withdrawn. BS 6235 made reference to BS 4360 which covers steels with specified Charpy V-notch impact values that enables appropriate grades to be selected depending on the design minimum temperature and on the plate thickness. External corrosion may be minimized by means of protective coatings, or may be allowed for in design (see 2.4).

Further guidance on steel bearing piles is given elsewhere [31].

6.7.6 Screw piles

Particular care should be taken where screw piles are used in soils having a low remoulded strength.

NOTE Recommendations on the use of screw piles were previously given in 7.4.7 of CP 2004:1972, which is withdrawn.

6.8 Installation of piles

6.8.1 General

The design, installation and testing of piles for non-maritime applications is covered in BS 8004. Clause 6.8 should be read in conjunction with 2.2.

The construction plant used for the installation of the piles will depend mainly on the type of pile, site exposure, depth of water (if any) during piling and plant available. The most likely methods of support for a particular site should be considered at the design stage, since this will influence the construction sequence, any temporary loadings on the piles and the construction tolerances which can reasonably be expected.

Raking piles can be difficult to drive from floating plant in tidal waters and the design may need to take account of any temporary works required.

The effect on piling tolerances of driving through rubble slopes should also be considered (see also 6.13).

6.8.2 Temporary bracing

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

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

6.8.3 Measures to improve performance of piles

6.8.3.1 Slip layer. Where piles pass through a stratum which will consolidate, the downdrag (or negative skin friction) induced on the pile shaft may be minimized by applying a bituminous slip layer to part of the pile shaft. On end-bearing piles, the slip layer should not be applied over a distance greater than 10 pile diameters above the pile toes. On friction piles, the slip layer should not be applied below the neutral points, that is the level at which the pile and ground settle by the same amount.

The layer may be applied to steel and precast concrete piles and to permanent steel casings.

6.8.3.2 Vertical anchors. In cases where adequate uplift-resistance in tension piles cannot be economically provided by means of embedment, a rock anchor may be installed axially through the pile shaft to achieve the required resistance (see 53.2.3 of BS 6349-1:1984). Design of the rock anchor should be in accordance with DD 81. Anchors may be stressed or of the deadman type.

An example of an anchored pile is shown in Figure 42.

6.8.4 Construction sequence

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

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

Particular factors which should be considered are:

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

6.8.5 Connections

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

6.9 Types of superstructure

6.9.1 Concrete deck

6.9.1.1 Flat slab. Flat slabs may be used where a high water level would make the construction of beams difficult and where the cost of the necessary falsework is low.

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

6.9.1.3 Two-way beams and slab. Beams may be provided in the longitudinal direction as well as transversely for the following purposes:

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

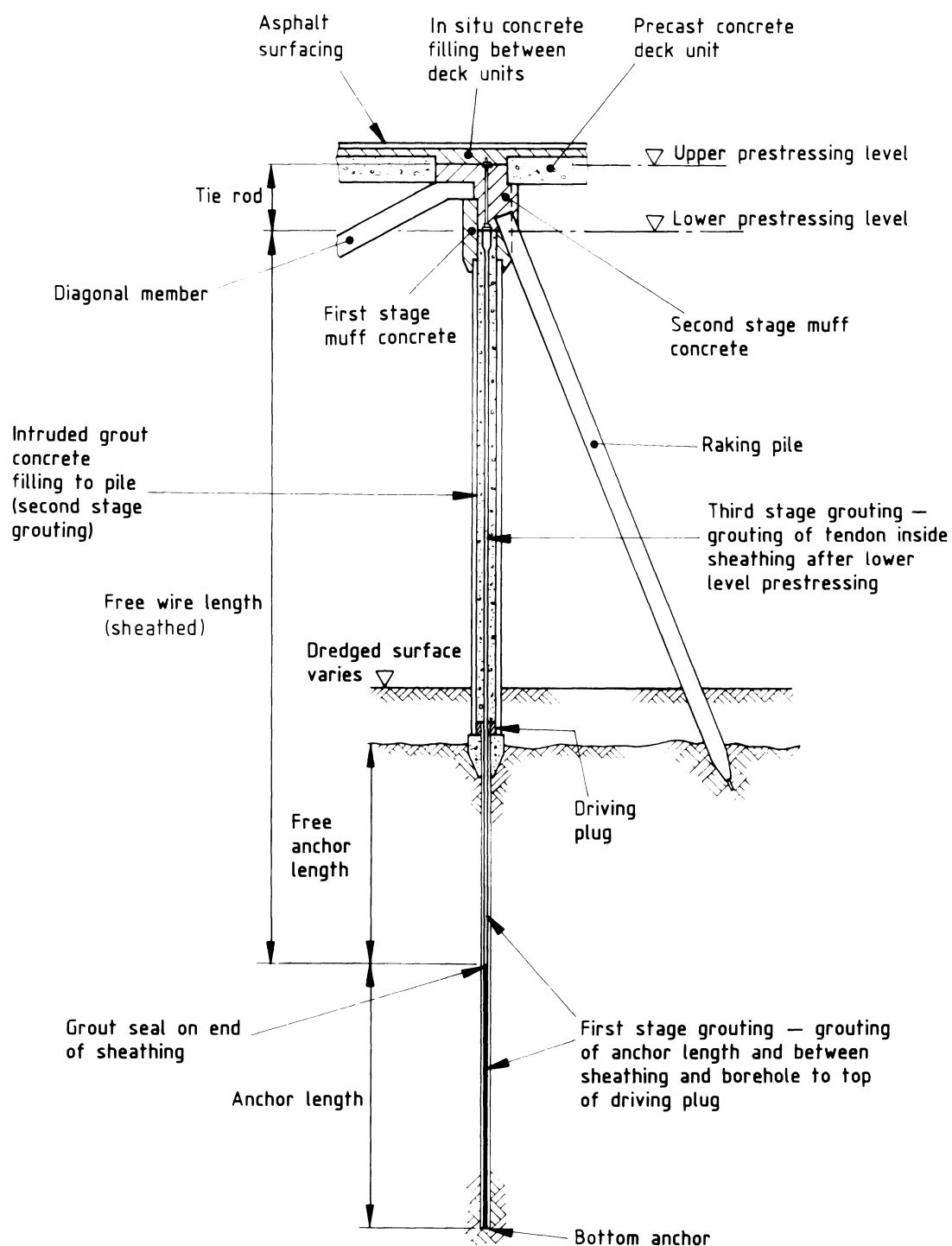
Precast units may be used in most structures of this type.

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

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

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

Examples of access trestles are shown in Figure 43.



Source: Bertlin and Partners.

Figure 42 — Vertical anchor for pile

6.9.1.5 Relieving platforms and false decks.

Relieving platforms and false decks may be constructed using one of the types described in 6.9.1.1 to 6.9.1.3. Where the soffit is near water level, beams may conveniently be constructed as upstands. If counterforts are required, they can also serve as cross-head beams.

The retaining wall along the outer edge of the deck, which may also function as an edge beam or berthing face, will usually be of in situ reinforced concrete, unless a special finish is required, in which case precast units may be used.

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

6.9.2 Steel and concrete deck

Steel beams may be used in preference to concrete, where loadings do not normally exceed highway loading or where rapid construction is required. Particular attention has to be paid to protection of the steelwork against corrosion (see clause 68 of BS 6349-1:1984). They are not usually used in quay superstructures subjected to loads in excess of highway loading because of the high cost of providing a frame of sufficient rigidity in the vertical and horizontal directions. Arrangements in common use are:

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

6.9.3 Timber deck

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

6.10 Design methods**6.10.1 Vertical loads from superstructure**

Quay and jetty structures should generally be designed so that vertical loads are resisted:

- a) by the vertical piles alone, with raking piles (where provided) resisting all the horizontally applied loads; or

- b) by the vertical piles and opposing pairs of raking piles.

All loads can then be assumed to be resisted by axial forces in the piles (i.e. no bending). The validity of this assumption should be checked using the relative elastic deformations of the piles.

The elastic deformation of differentially loaded piles will alter the forces in the superstructure. The deformation depends on the type of ground into which the pile transfers the load, and on the effective length, cross section and material of the pile.

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

6.10.2 Soil loads

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

6.10.2.2 Negative skin friction. The effects of negative skin friction should be considered. Measures to mitigate these effects are given in clause 6.5.7.1.

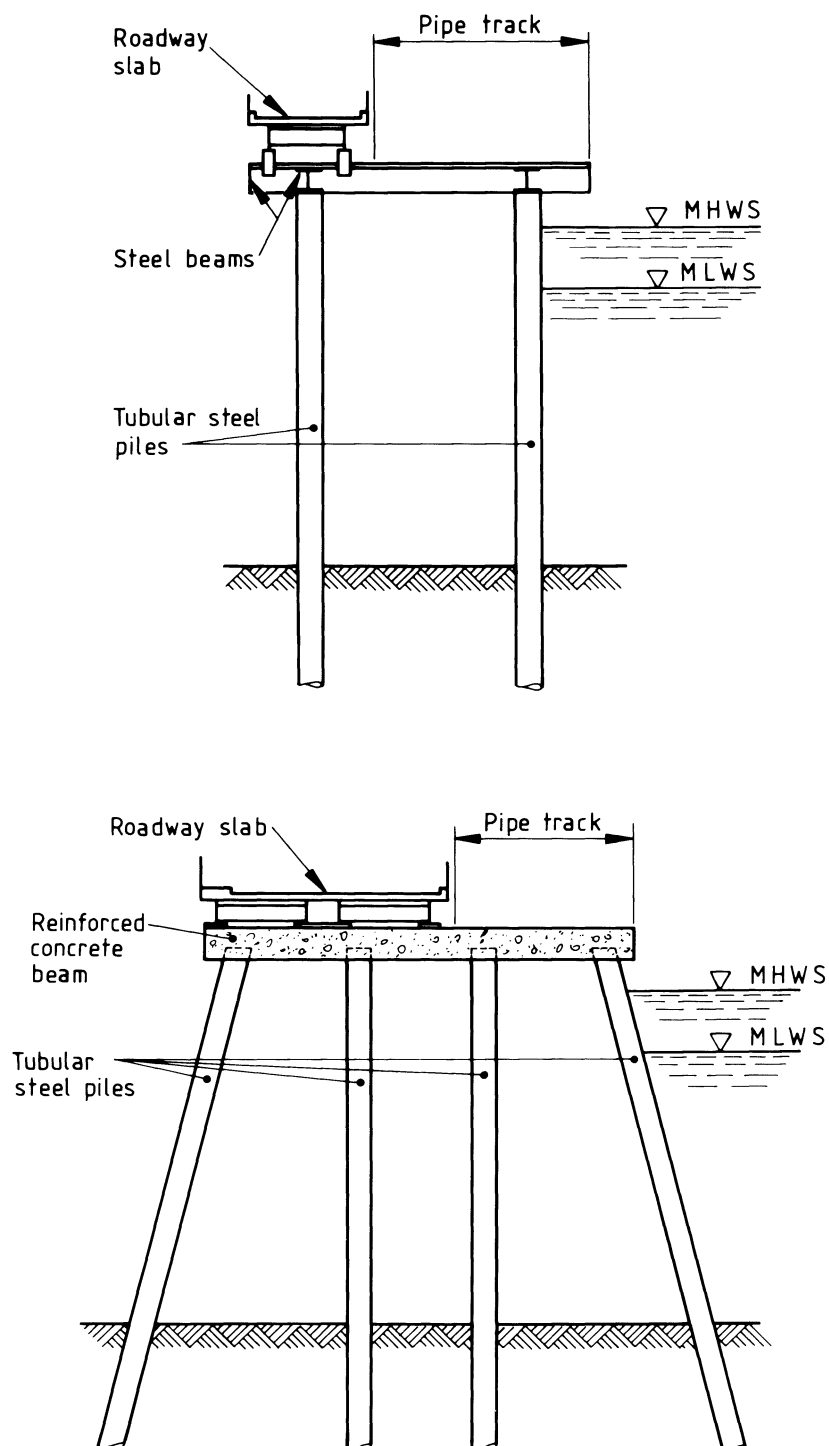
6.10.2.3 Other soil loads. Where there is danger of slope instability causing lateral loading on the embedded lengths of piles, the piles should have ample lateral strength.

6.10.3 Berthing and mooring loads

Loads are to be derived from considerations given in BS 6349-4.

6.10.3.1 Transverse loads. Movements and resulting stresses caused by berthing and mooring loads applied at right angles to the berthing face are calculated by analysis of the entire structure as a three-dimensional model, or as a two-dimensional model in which the deck, acting as a deep horizontal beam, is supported on spring supports representing the pile bents or other strongpoints and the passive resistance, if present, at the rear of the deck.

Elastic extensions of tie-backs and extensions or shortening of raking piles will result in small horizontal deflections at the heads of any vertical piles but stresses induced in vertical piles by these deflections may usually be ignored.



Source: Parsons, Brown & Newton.

Figure 43 — Access trestles

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

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

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

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

6.10.3.3 Energy absorption. Flexible structures should be designed to achieve the maximum energy absorption of berthing loads by combined use of the structure and the fendering, to reduce fendering requirements and thereby cost. The contribution of the structure will be greater where piles of high yield steel are used. For a uniform section, the following simplified formula for calculating the energy absorbed in bending ϵ may be used:

$$\epsilon = \frac{f^2 L z}{6 y E}$$

where

f is the bending stress (due to dead and live load and deflections) of extreme fibre of pile;

L is the actual pile length in bending (measured between point of apparent fixity in the ground and the underside of the superstructure whether fixed or pinned);

z is the section modulus;

y is the distance from neutral axis to extreme fibre in plane of bending;

E is the modulus of elasticity.

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

6.10.4 Other loads

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

The thermal movement of the structure should be considered when locating raking piles to resist longitudinal forces, together with the anchoring effect of fully embedded piles such as at the rear of marginal quays.

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

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

A piled or suspended screen, either solid or perforated, may be provided in front of the structure to reduce loads on the piles or on vessels moored alongside. The structure should then be designed to resist loads transmitted from the screen.

Waves (especially those of long periods) and currents cause motion in vessels berthed alongside, giving rise to bollard and fender forces (see clause 42 of BS 6349-1:1984), which are resisted as described in 6.10.3.

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

6.10.4.4 Earthquakes. General guidance on earthquake effects is given in 2.3.8.

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

The large mass of relieving platform superstructures has to be taken into account in calculating stresses.

6.10.5 Fatigue

Fatigue of suspended deck structures is not usual but may require consideration for structures subjected to repeated waves or vortex shedding in strong currents (see clause 47 of BS 6349-1:1984). Where fatigue effects are important, connections should be designed taking these into account.

6.11 Loading considerations

6.11.1 Vertical dead and live loads

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

6.11.1.2 Superimposed dead loads. Loads from footings of fixed superstructures may be assumed to disperse at 45° to the vertical from the edge of the contact area down to the neutral axis of the deck member.

6.11.1.3 Live loads on deck. Live loads (including horizontal and secondary loads) should be determined in accordance with clauses 44 and 45 of BS 6349-1:1984 and included in the design of the superstructure in the appropriate loading case. Loads imposed by Ro-Ro ship quarter ramps should be considered where appropriate¹²⁾.

6.11.1.4 Erection loads. Loads occurring during each stage of construction should be estimated, where possible, and the structure checked accordingly. These loads may exceed the loads on the completed structure. If it is not possible to estimate or specify such loads with any accuracy when the design is prepared, they should be checked before construction commences.

6.11.2 Horizontal live loads

6.11.2.1 Berthing. Berthing loads should be calculated for each class of vessel expected to use the berth during its design life: the energy is calculated as described in BS 6349-4.

The effects of vertical and longitudinal forces associated with friction between the fenders and the vessel should be included.

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

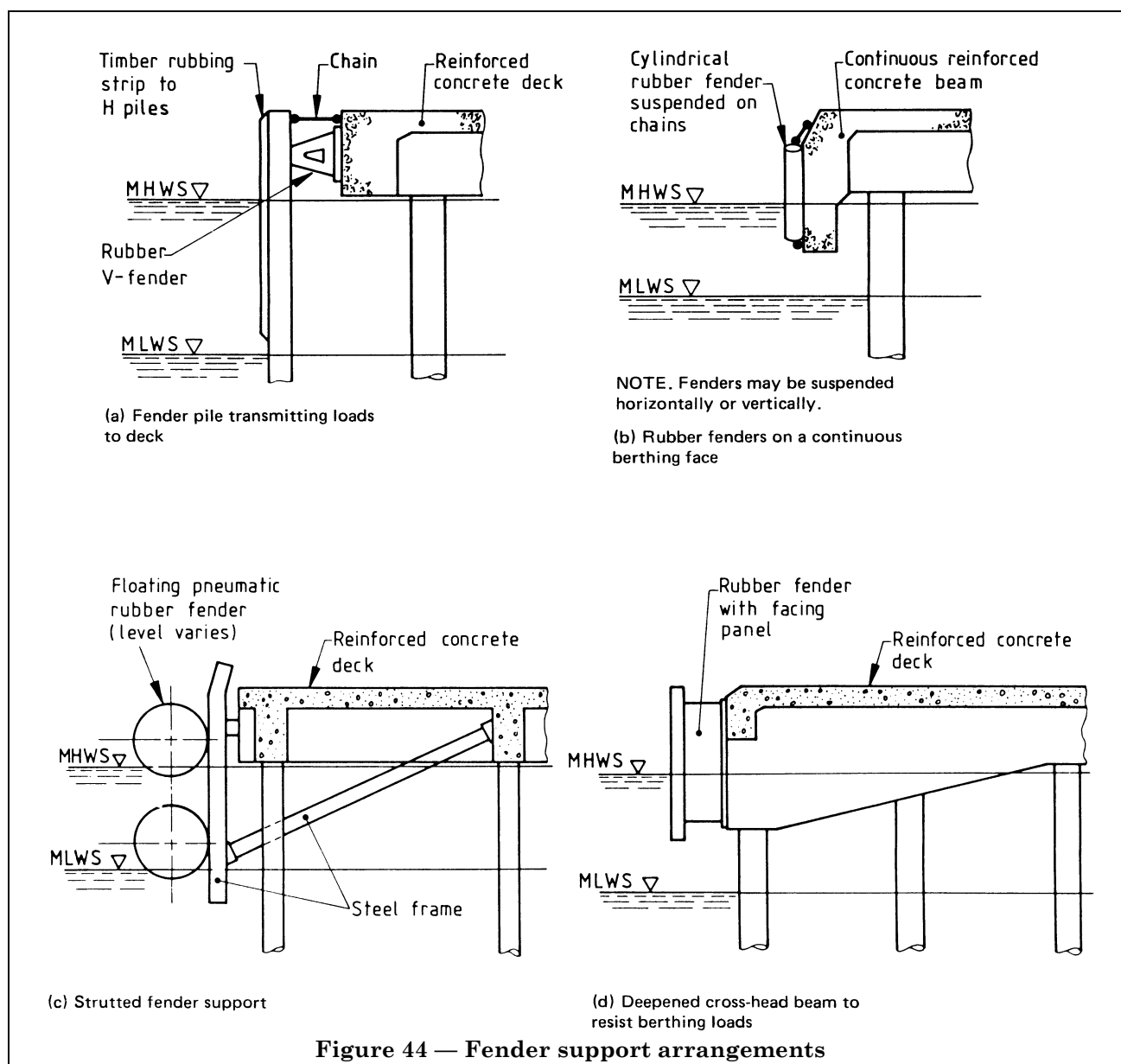
Figure 44 shows typical structural arrangements used to resist berthing loads.

6.11.2.2 Mooring. Mooring loads from vessels' lines are applied via quay fittings such as bollards, hooks, fairleads or rings fixed to the top of the deck, to the front face or on a raised platform on the deck. Except where small loads are involved, the quay fitting should be fixed to the seaward end of a cross-head beam, which is then designed to resist the total forces which may be applied over a range of directions in the horizontal and vertical directions. The forces to be resisted are given in clause 42 of BS 6349-1:1984.

6.11.3 Environmental loads

6.11.3.1 Wind. Wind loads may be calculated in accordance with 5.3 of BS 5400-2:1978. The loads are resisted by axial and bending stresses set up in the deck members.

¹²⁾ Typical loads are to be included in amendment in BS 6349-1.



6.11.3.2 Temperature. Overall changes in temperature are discussed in 6.10.4.1. Where structures, or parts of them, are restrained from movement, the resulting stresses in the deck members may be assumed to be adequately resisted by the minimum requirements given in BS 5400 for steel and concrete.

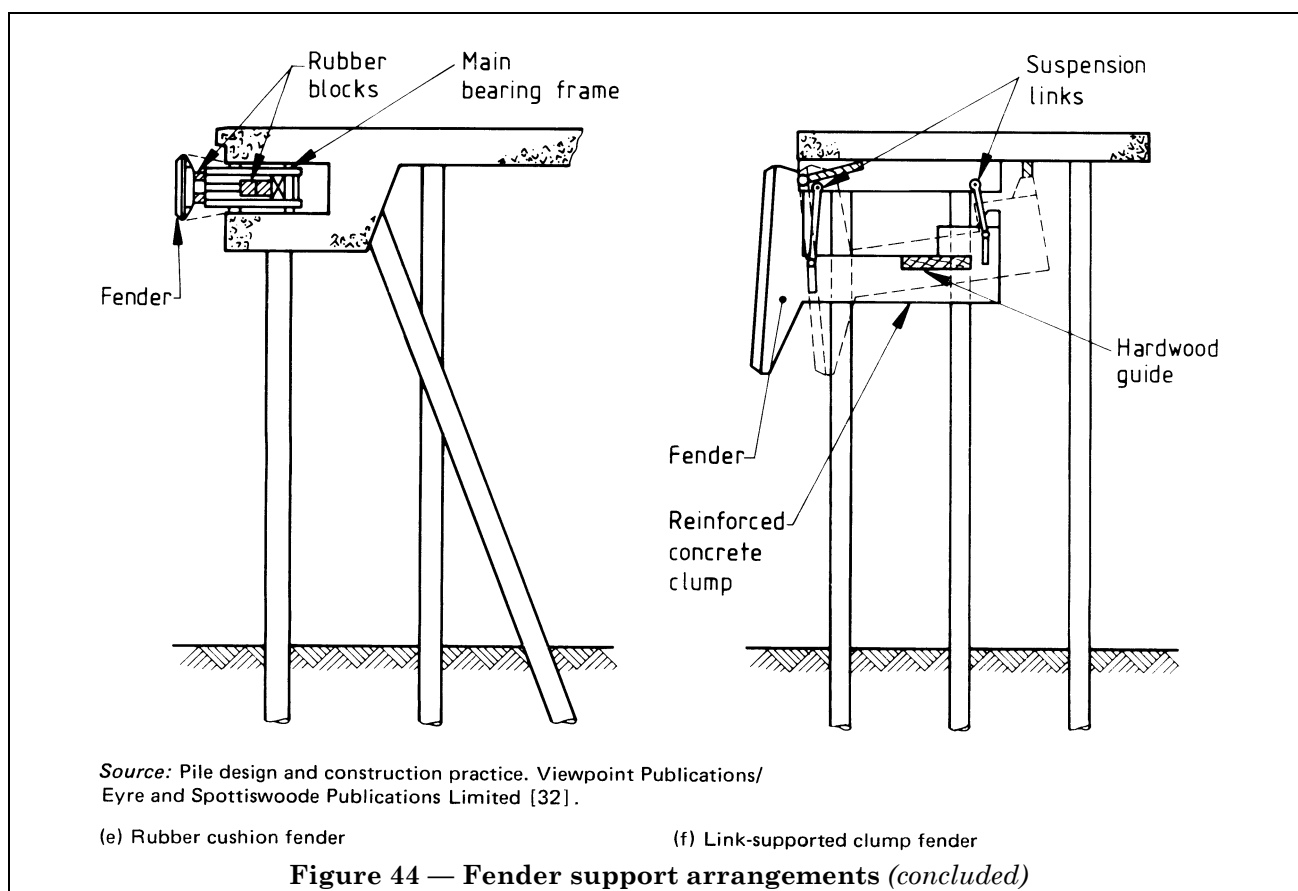
6.11.3.3 Earthquake. Flexible structures are designed to resist earthquake loads by bending of the piles.

Rigid structures resist earthquakes by means of the raking piles, strut or other resisting medium provided.

6.11.4 Loading cases

6.11.4.1 General. The superstructure should be designed to resist all combinations of loads which may realistically be assumed to act on the structure simultaneously. Loads may act directly on the superstructure or indirectly via the piles.

For both limit state and working stress design methods, the individual loads to be used in each combination are described as the nominal loads, Q_k as defined in 3.3.1 of BS 5400-1:1978.



6.11.4.2 Limit state design. The partial factor γ_{fL} for loads, for use in limit state design, is given in Table 2 for each load case. γ_{fL} is defined by the equation:

$$\gamma_{fL} = \text{function } (\gamma_{f1} \gamma_{f2})$$

(γ_{f1} and γ_{f2} are as defined in 3.3.1 of BS 5400-1:1978),

and should be applied whenever this contributes to additional stress in the structural member under consideration. Where stress relief would occur in the design member as a result of lower values of any type of load being applied to the structure a value of $\gamma_{fL} = 0.9$ should be used for that load; unless the load may be removed completely, in which case $\gamma_{fL} = 0$.

The loading cases given in Table 2 correspond to the load combinations described in 3.2.3 for overall stability, as follows:

- loading case 1 normal loading;
- loading case 2 extreme loading;
- loading case 3 temporary loading during construction.

The design loads Q^{13} , are determined as defined in 3.3.1 of BS 5400-1:1978 by the equation

$$Q^{13}) = \gamma_{fL} \cdot Q_k$$

The design load effects or stresses, S^{13} , in the structure are determined as defined in 3.3.2 of BS 5400-1:1978 by the equation

$$S^{13}) = \gamma_{f3} \text{ (effects of } Q^{13})$$

At the ultimate limit state (ULS) $\gamma_{f3} = 1.1$ for concrete and steel, except that where plastic methods are used for the analysis of a concrete structure, γ_{f3} should be taken as 1.15. At the serviceability limit state (SLS), $\gamma_{f3} = 1.0$ for concrete and steel.

Because of the difficulty in the design and construction of maritime structures γ_{f3} should be applied in all cases unless the loads under consideration can be defined precisely.

The values of the partial safety factor for materials, γ_m , should be in accordance with BS 5400-3, BS 5400-4 and BS 5400-5 i.e.:

for concrete $\gamma_m = 1.5$ (ULS) and 1.0 (SLS);

¹³⁾ Generally correct: for specific items, see Table 2 of BS 5400-3:1982.

for steel reinforcement $\gamma_m = 1.15$ (ULS) and 1.0 (SLS); and
for structural steel $\gamma_m = 1.05$ (ULS) and 1.0 (SLS)¹³.

Table 2 — Partial load factor γ_{FL} for suspended decks

Load	Limit state	γ_{FL}^a in loading case		
		1 normal	2 extreme	3 temporary
Dead : steel	ULS ^c	1.05	1.05	1.05
	SLS ^d	1.0	1.0	
Dead : concrete	ULS	1.15	1.15	1.15
	SLS	1.0	1.0	
Dead: superimposed	ULS	1.2	1.2	1.2
	SLS	1.0	1.0	
Live (see note 1)	ULS	1.4	1.2	1.2
	SLS	1.1	1.0	
Berthing or mooring	ULS	1.4	1.2 ^b	—
	SLS	1.1	1.0	
Environmental (see note 2)	ULS	1.4	1.2	1.2
	SLS	1.0	1.0	
Soil (see note 3)	ULS	1.3	1.15	1.15
	SLS	1.0	1.0	
Hydrostatic uplift (see note 4)	ULS	1.2	1.0	1.0
	SLS	1.0	1.0	
NOTE 1 Q_k should be increased as required to include the effect of dynamic loads.				
NOTE 2 To include the additional indirect loads applied to the structure (e.g. crane corner loads due to wind).				
NOTE 3 To include pressure on retaining walls (see section 4), downdrag and differential settlement.				
NOTE 4 Applied to decks and relieving platforms.				
^a γ_{FL} is to be used in conjunction with γ_{F3} (see 6.11.4.2).				
^b Where accidental loads transmitted to the structure are limited by a failure mechanism such as break-off bolts to bollards or plastic failure of a fender frame, γ_{FL} may be reduced to 1.1 for the overall design of the suspended deck.				
^c ULS: ultimate limit state.				
^d SLS: serviceability limit state.				

6.11.4.3 Working stress design. The combinations of loads for use in working stress design of suspended decks should be as given in Table 2, but using a partial load factor, $\gamma_{FL} = 1.0$. The permissible working stresses under normal loading should be in accordance with the relevant code of practice for the material used. Permissible stresses may be increased by 30 % for concrete and to 90 % of yield stress for steel under either abnormal berthing or loading due to earthquake and tsunامي.

6.12 Design of piles

6.12.1 General

It should be noted that installation stresses may be a controlling factor in the choice of pile size.

When considering the completed structure, concrete piles should be analysed by limit state methods in accordance with BS 5400-4, while steel piles are analysed by limit state methods in accordance with BS 5400-3 or with reference to allowable working stresses in BS 449-2. Timber piles should be analysed with reference to the permissible stresses in BS 5268-2.

The effect of pile joints in precast and timber piles should be studied.

For all piles, the working loads transmitted to the ground are calculated, a suitable factor of safety being applied to the ultimate soil resistance to obtain allowable values.

6.12.2 Column buckling

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

The effective column length of the piles should be determined assuming that the top of the pile is either at the level of the underside of the deck or at the level of any bracing in the plane being considered, whichever is the lower, and that the bottom of the pile is its apparent point of fixity, taking into account any future deepening, scour, overdredging or soil liquefaction (see 47.2.2 of BS 6349-1:1984). The point of fixity of piles in slopes under marginal quays should be determined taking into account the angle of the slope and the likely degree of restraint due to any rubble protection provided.

The heads of piles are usually built into the superstructure, thereby achieving the minimum effective length and maximum load-bearing capacity. Figure 45 gives examples of pile/deck connections.

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

6.12.3 Local buckling of steel piles

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

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

6.12.4 Load transfer to soil

For H-piles, in which the flange width and web depth are approximately equal, it can generally be assumed that a plug of soil forms between the flanges during driving, except in loose granular soils. The skin friction is calculated for the effective perimeter, equal to twice the sum of the width and depth of the pile section. The end resistance may be calculated using the gross pile area if plug formation occurs.

Where a plug of soil forms, the end resistance of hollow piles is calculated using the gross cross-sectional area. Steps should be taken to ensure that the internal water level is at least as high as the outside water level to prevent subsequent plug wash out. If plug formation is doubtful, the net pile area should be used.

When lateral deflections associated with cyclic loads at or near the seabed are relatively large, consideration should be given to reducing or neglecting skin friction in this zone.

6.12.5 Vertical loads

6.12.5.1 Vertical piles. The load to be resisted by the soil is the sum of the pile head load, the pile dead weight and the maximum downdrag to which the pile may be subjected during the design life.

Providing that piles are installed within the tolerances quoted in 6.13, incidental eccentricities can be ignored. However, where lateral deflections are caused by applied loads, the effect of induced eccentricities should be investigated.

The net uplift force is calculated as the maximum uplift force due to imposed loads minus the dead load. Uplift to be resisted in friction by the soil should be calculated ignoring downdrag.

The group effect of piles should be taken into account if the spacing is less than the values recommended in 7.3.4.2 of BS 8004:1986.

6.12.5.2 Raking piles. Raking piles designed to resist vertical loads should be arranged in opposing pairs of equal batter, unless they are required simultaneously to resist a lateral load. The vertical load at the head is assumed to be divided equally between each of the two piles.

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

6.12.6 Horizontal loads

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

Where large horizontal loads cause high bending moments in the stiffest piles (such as the rear piles of a marginal quay), piles of greater bending resistance may be required. This may economically be achieved for piles with long embedments, by increasing the bending resistance only where required.

Piles resisting lateral loads should be adequately embedded to obtain the required resistance from the soil. This will usually be possible without excessive penetrations if the seabed is of medium to dense granular soil, firm to stiff clay, or weak rock. Guidance on the soil resistance of laterally loaded piles is given elsewhere [32, 33, 34]. The restraint offered by rubble protection layers on slopes under marginal quays should be considered in assessing the relative stiffnesses of each row of piles.

The piles should be designed to resist loads originating from any possible direction, and non-symmetrical sections should therefore be checked for biaxial bending. Where large loads are applied predominantly from one direction (usually at right angles to the centreline of the structure), the major axis of the pile should be orientated at right angles to it.

The effect of concrete infill and muffs on pile stiffness should be considered.

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

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

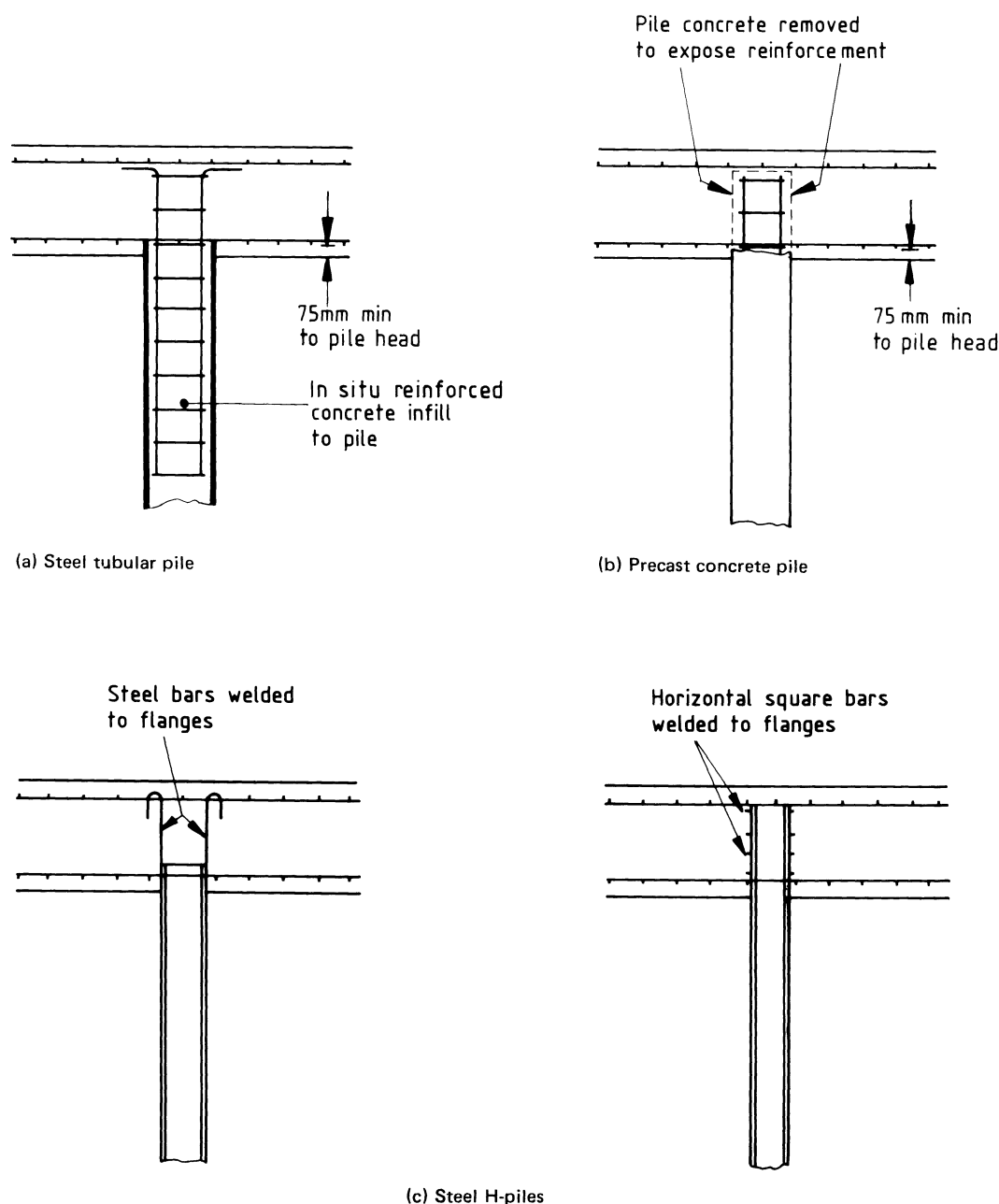


Figure 45 — Examples of pile/deck connections

6.12.7 Wave and current forces

6.12.7.1 During construction. Loads due to waves and currents acting on unbraced piles during construction should be considered. In particular, the possibility of current-induced oscillations should be investigated (see 38.3 of BS 6349-1:1984) and, if necessary, suitable measures should be taken: these may include altering the pile stiffness, fitting spoilers to the piles or bracing them.

6.12.7.2 Permanent condition. Wave and current forces on individual piles should be calculated as described in clauses 38 and 39 of BS 6349-1:1984. The piles resist the applied loads by beam action, the end conditions corresponding to the degrees of fixity at the seabed and deck.

Guidance on the analysis of the bow-string oscillation of fixed-ended piles due to current forces is given in 38.3 of BS 6349-1:1984. This analysis should be carried out where fast currents occur.

6.12.8 Tension anchors

Piles anchored to the ground to obtain the required pull-out resistance should be capable of resisting the compressive force imposed by the anchor, including any test loads. To avoid the possibility of totally destressing the anchor, the tendon should be stressed to give a minimum load equal to 1.2 times the maximum compressive load on the pile.

6.12.9 Testing

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

6.12.10 Pile movements

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

6.12.10.2 Horizontal. Horizontal movement of pile heads should be assessed for each combination of maximum horizontal loads. The possibility of two adjacent structures deflecting in opposite directions should be considered.

6.12.11 Stresses in piles

6.12.11.1 Concrete piles. Reinforced and prestressed concrete piles should be designed as columns using a partial factor of not less than 1.5 for loads under normal loading conditions and with adequate reinforcement to resist driving stresses. Under extreme loading conditions and temporary loading conditions during construction, the partial factor for loads should be not less than 1.2. Lifting stresses in precast piles should be calculated using a partial factor of 1.4 applied to the dead load, plus an allowance for impact loading.

For cast in situ piles, the average compressive stress under working load should not exceed $0.25f_{cu}$. For piles cast underwater (see 58.5 of BS 6349-1:1984) this limits the permissible average compressive stress to 6.3 N/mm^2 .

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

6.12.11.2 Steel piles. Allowable stresses should be determined taking into account the yield stress (f_y) and section properties of the pile under consideration. Stresses should be calculated taking into account the effects on the pile section of corrosion during the design life of the structure.

The compressive stress on the gross steel area due to axial loads should not exceed the allowable value calculated in accordance with clause 30 of BS 449-2:1969. However, to limit the effects of increased stresses during driving, the axial compressive stress in driven steel piles under normal loading conditions should not exceed $0.3f_y$. This value may be increased to $0.5f_y$ when an easy drive through soft ground terminates with rapid seating in a firm stratum.

The axial tensile stress in steel piles under normal loading conditions should not exceed $0.3f_y$.

The bending stresses in steel piles under normal loading conditions should not exceed the normal permissible stresses given in clause 19 of BS 449-2:1969. Under the most severe combination of extreme loads the permissible bending stress may be increased by up to 25 % subject to a maximum value of $0.8f_y$ provided that the axial component does not induce a stress greater than the limit stated above.

The shear stress in steel piles under normal loading conditions should not exceed the normal permissible stresses given in clause 23 of BS 449-2:1969.

Piles subjected to combined stresses should satisfy the requirements of clause 14 of BS 449-2:1969.

Piles may alternatively be analysed by plastic design methods (see BS 5950).

6.12.11.3 Timber piles. Working stresses should be in accordance with BS 5268-2, but may be increased as described in 6.11.4.3 for transient loads.

6.12.12 Factors of safety on soil resistance

The factors of safety given in 6.12.12.1 and 6.12.12.2 on the ultimate values of soil resistance should, in general, be applied to the total working loads of individual piles.

6.12.12.1 Bearing. Factors of safety should be in accordance with those in BS 8004. Under abnormal berthing and loading due to earthquake and tsunami, the overall factor of safety may be reduced to 1.3.

6.12.12.2 Pull-out. The minimum factor of safety on static loads under normal loading conditions should normally be 3.0 where soil properties are known with confidence. This may be reduced to 2.0 for abnormal berthing or loading due to earthquake and tsunami.

6.13 Construction tolerances

6.13.1 Pile head

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

6.13.2 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.

Section 7. Dolphins

7.1 General

Reference should be made to BS 6349-4 for dolphin layouts and fendering and mooring systems.

7.2 Types of structure

7.2.1 Function

Berthing (or breasting) dolphins are usually provided at jetties, such as those at bulk handling terminals, where the jetty head is required only to support cargo handling equipment. They may also be required at Ro-Ro terminals where there is no quay for vessels to berth alongside.

Dolphins for manoeuvring vessels may be located at dock or lock entrances or at berths having fixed cargo handling equipment, where vessels may have to be winched along the berth.

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

7.2.2 Structural stiffness

7.2.2.1 General. Dolphins may be rigid or flexible as defined in 47.1 of BS 6349-1:1984.

7.2.2.2 Flexible structures. Flexible dolphin structures usually comprise a group of vertical piles built into a heavy concrete cap or a braced frame and deck. Multiple rows of cantilever piles can also be used. Flexible dolphins may also be built using one or more large diameter vertical piles. For light loads timber piles may be used. Figure 46 shows examples of flexible dolphins.

7.2.2.3 Rigid structures. Rigid dolphin structures may be of open-piled or solid construction. Examples are:

- a) a group of raking piles (with or without vertical piles) with concrete cap;
- b) a group of braced piles supporting a platform;
- c) a group of vertical piles with concrete cap and strut/tie to shore;
- d) a concrete caisson;
- e) a steel sheet-piled cell.

Figure 47 shows typical examples of rigid dolphins.

NOTE Steel jackets are sometimes used as dolphins at deep water terminals.

7.2.3 Choice of structure

The type of structure to be adopted depends largely on the factors discussed in 2.3. With groups of piles, the resistance to uplift caused by horizontal loading should receive particular attention. The advantages and disadvantages of individual structural types are discussed in sections 4, 5 and 6. Onshore mooring points may be provided, depending on the proximity of dry land, shallow water or a suitable existing structure.

The flexibility of the structure should be considered in relation to its function and the type of operations to be carried out at the berth. Whereas berthing or lead-in dolphins may be flexible, mooring dolphins should be relatively rigid so that the tension in vessels' lines is maintained. At container and Ro-Ro terminals, consideration should be given to restricting the flexibility of berthing dolphins under conditions in which handling operations may take place.

7.3 Loads

7.3.1 Berthing loads

For berthing loads, reference should be made to BS 6349-4. The design of berthing and lead-in dolphins and berthing beams should take into account all normal variations in the elevation, direction and magnitude of applied loads.

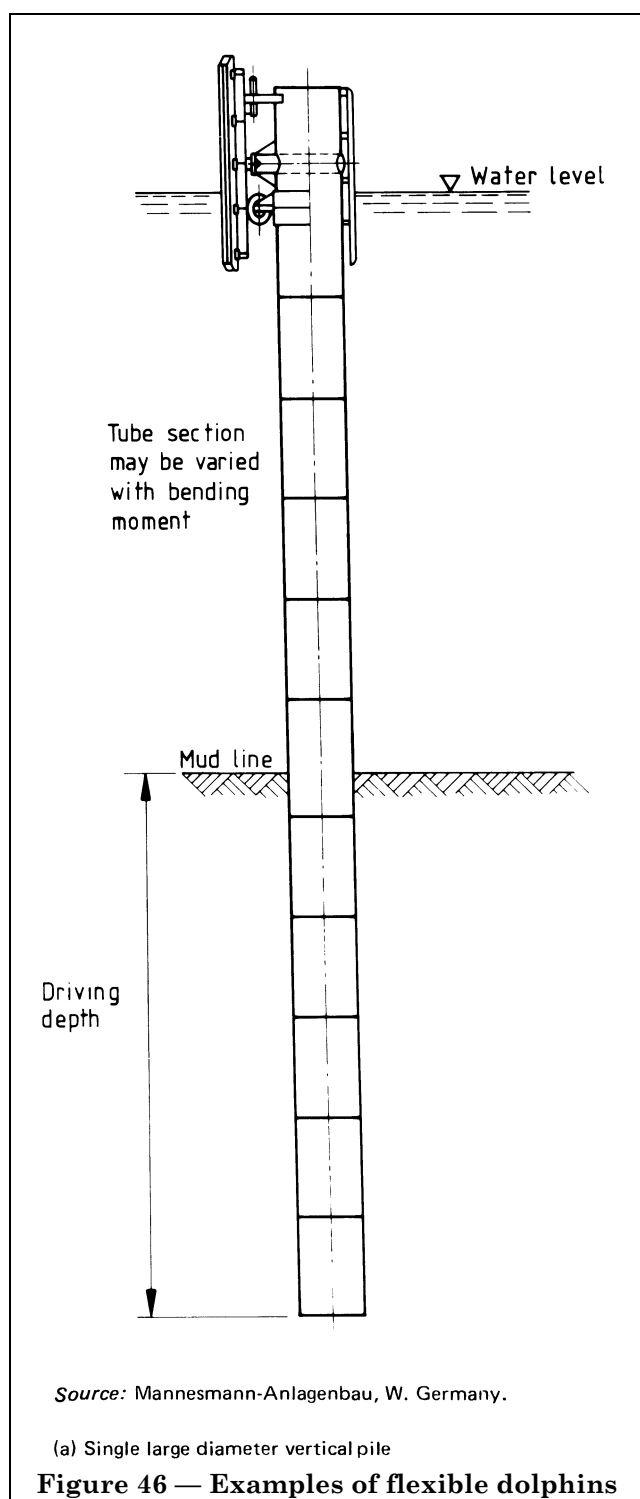
Consideration should also be given to the possible consequences of accidental vessel impact on the berthing structure, both to the structure itself and to structures alongside. Where appropriate, the structure may incorporate elements designed to break off or collapse under accidental loading or the whole structure may be designed to collapse in a way which provides additional energy absorption.

The proportion of energy absorption to be provided by the fendering will depend on the flexibility of the structure. In rigid structures, the small amount of energy absorbed by the structure should be ignored.

The choice of type of fendering to apply loads at the most suitable points on the structure may enable considerable savings in total cost to be achieved.

7.3.2 Mooring loads

For mooring loads reference should be made to BS 6349-4. They include loads from breast, head and stern lines, and from spring lines. In addition to the variations in inclination, direction and degree of applied loads, combinations of loads on a structure should be considered. These may include loads applied by two vessels simultaneously.



The structural form of the dolphin should be appropriate to the position and elevation of the mooring points.

At exposed sites, physical or mathematical model tests may be required to determine the optimum mooring system and the loads resulting, taking into account the elasticities of the mooring lines and fenders and the response of the dolphins (see 31.3 of BS 6349-1:1984).

7.3.3 Environmental loads

Guidance on environmental loads is given in clauses 34 to 40 and on movements and vibrations in clause 47 of BS 6349-1:1984. The design of dolphins at exposed sites should include consideration of cyclic loading due to waves, while vortex shedding from circular piles should be analysed for sites where currents may be sufficient to cause this effect.

The resonance of flexible structures during earthquakes should be checked.

7.4 Design of flexible dolphins

7.4.1 Flexible dolphins

Flexible dolphins are designed to absorb the kinetic energy of a berthing vessel by horizontal displacement of the pile heads. Berthing loads applied at the corner of a dolphin will result in torsion in the structure; this may be reduced by judicious positioning of the fenders, or made use of by providing torsional strength to increase energy absorption.

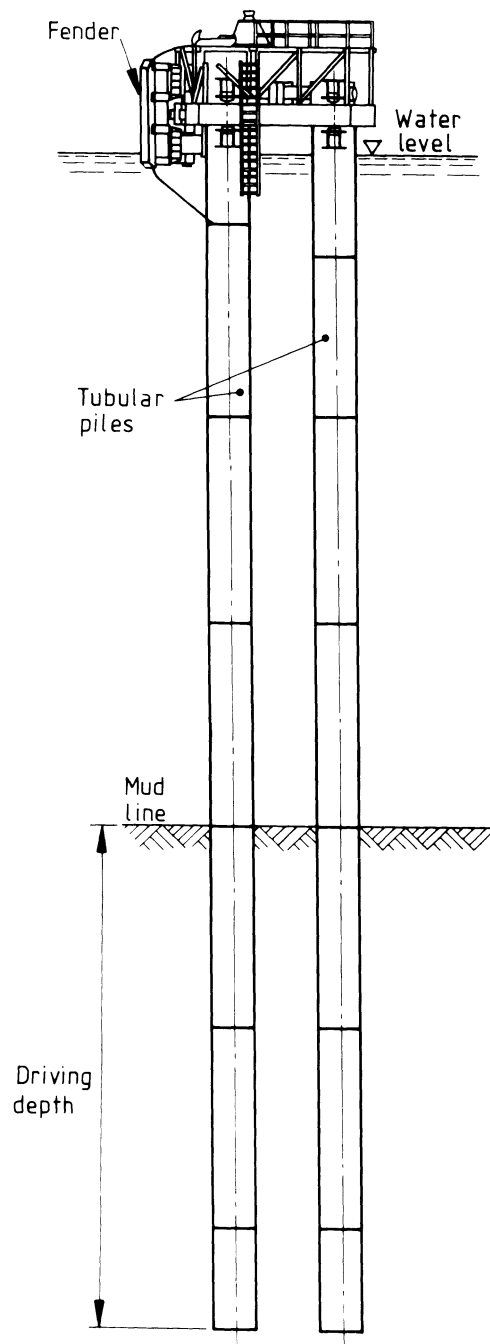
Where large diameter steel tubes are used, the design may take into account the torsional resistance of the piles in order to increase the total resistance of the structure.

The piles should be adequately embedded to obtain the required lateral and torsional resistance from the soil (see also 6.12.6.1 and [35]).

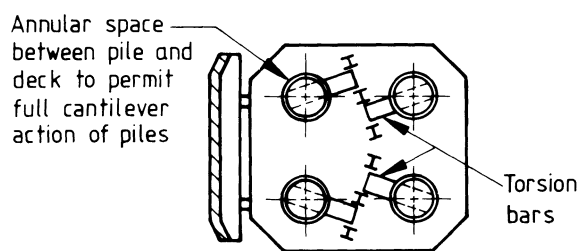
The combined deflection of the structure (and fenders if provided) under maximum load should generally not exceed 1.5 m, to avoid problems with ship handling. Hulls should not be able to strike piles when the head of the structure is deflected.

Piles for flexible dolphins usually comprise tubes of high yield steel, which are suitable on account of their high strength and energy absorbing properties. Tubes of variable thickness are often used for economy in structures constructed in deep water. Steel H-piles can be used but are more vulnerable to corrosion and are weak about their minor axis. Prestressed concrete cylinders may also be used. For berthing beams, steel H-piles are most suitable.

The design of flexible dolphins is dealt with in Part 4.

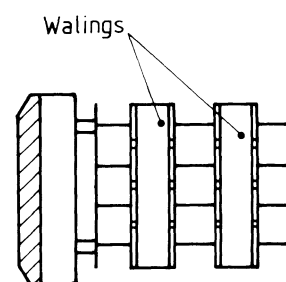


Elevation



Plan

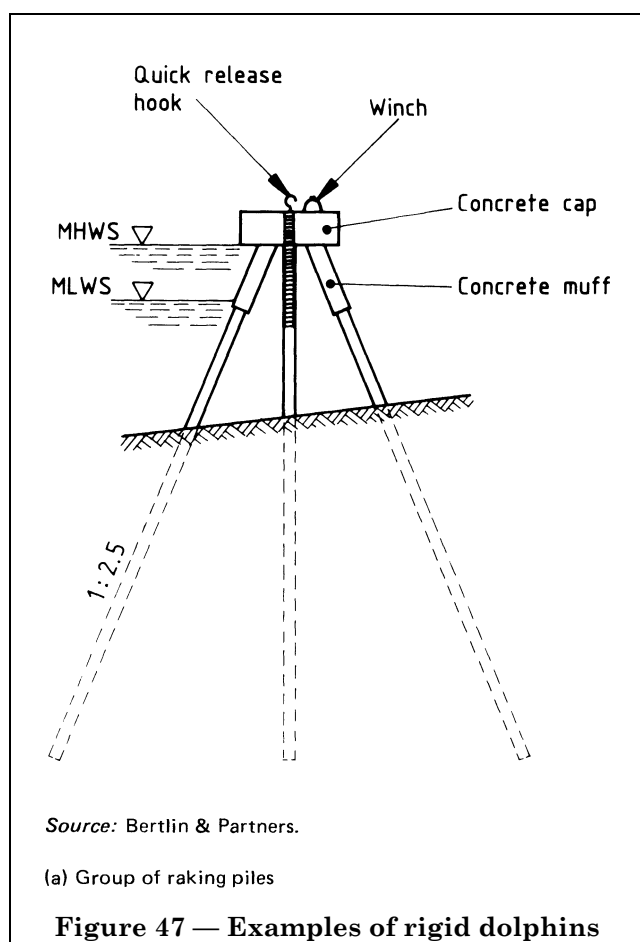
(b) Group of large diameter vertical piles with torsion bars



Plan

(c) Multiple rows of cantilever bearing piles

Figure 46 — Examples of flexible dolphins (concluded)



7.4.2 Berthing beams

Berthing beams resist horizontal loads by bending of the piles, and by bending and torsion of the beam. They should be analysed using the procedure recommended for flexible dolphins.

7.5 Design of rigid dolphins

7.5.1 Raking pile structures

7.5.1.1 Design methods. Raking pile structures are conventionally designed assuming that only axial forces are transmitted to the piles as a result of loads applied to the structure by vessels. Environmental loads such as wave and current loads will cause lateral loading of the piles. Guidance on design methods may be found elsewhere [36].

The rake of the piles should be as large as possible for maximum efficiency. However raked piles should not project into locations where they could be struck by vessels.

Where ground conditions are such that considerable downdrag may occur, the resulting stresses set up in raking piles may be excessive. This type of structure may not then be suitable.

Pile axes should preferably be arranged so that they intersect in pairs or groups at at least two locations on plan. The resultant applied load should, where possible, pass between these locations, which should be as widely spaced as possible especially in cases where the applied load has a considerable turning effect about the elastic centre of the pile group.

In many instances, particularly in the case of mooring dolphins, it will not be practicable to provide more than one pile group. By arranging the piles so that the elastic centre of the pile group lies on or near the resultant applied load, the likelihood of rotation is reduced with consequent lessening of the pile loads. Where the position of the resultant varies, the elastic centre should be located as close as possible to the intersection of the resultant loads. The effect of head and toe fixity may usually be neglected during trial designs, but in the final design should be checked fully.

Bending moments and other internal forces are best determined by computer analysis of a three-dimensional model of the structure.

7.5.1.2 Resistance to uplift. The horizontal loads applied to rigid piled dolphins are resisted by axial forces in the raking piles, and the pile head connection should be designed accordingly. Some piles may have to resist uplift. Where it is not possible to achieve the necessary resistance, the uplift may be resisted by a rock anchor installed through the pile after driving.

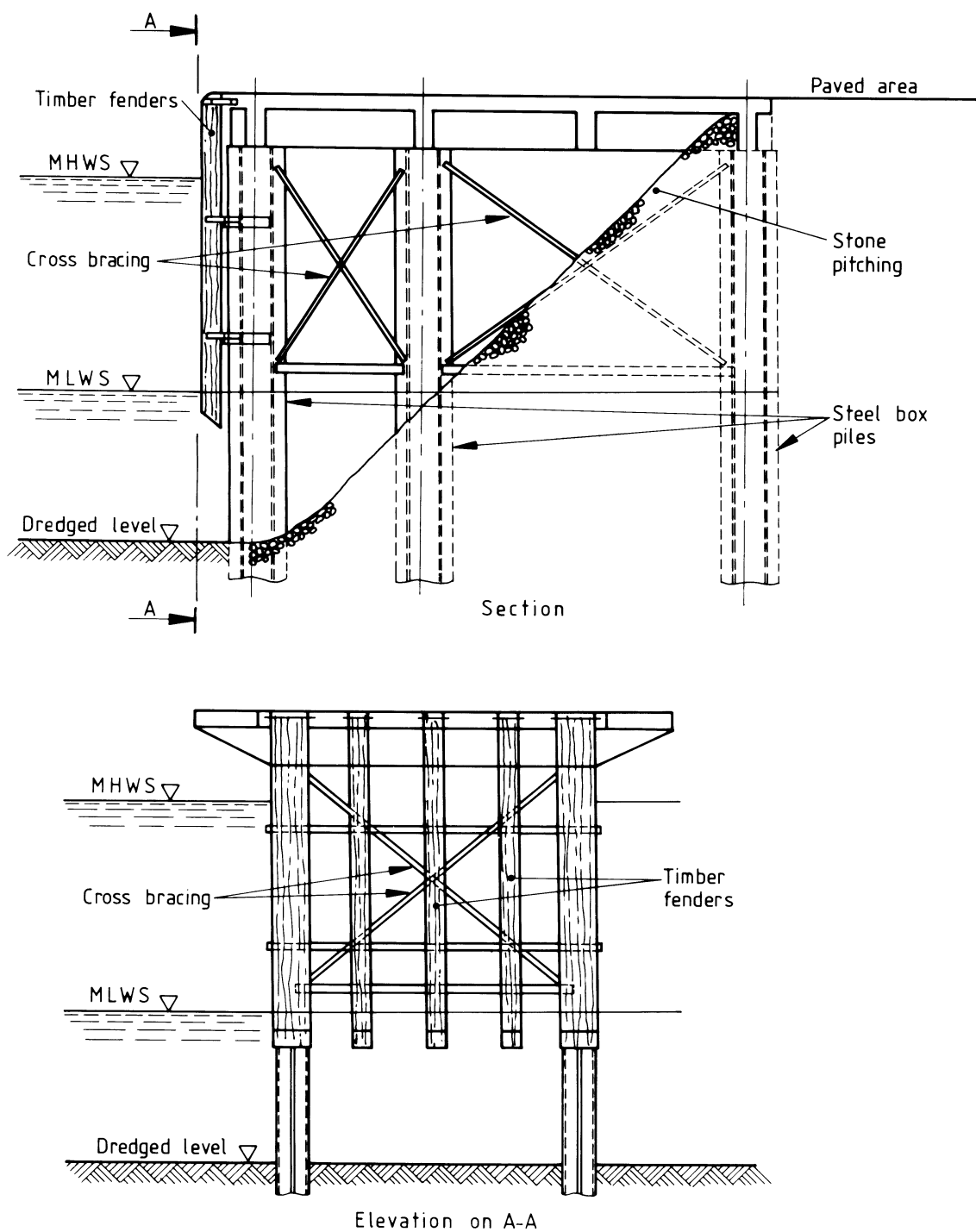
Net uplift on the structure may be reduced by providing a massive concrete pile cap, which also serves to disperse concentrated loads and to ensure rigidity. Such caps are commonly 1.5 m to 2.5 m thick.

7.5.1.3 Overall stability. Factors of safety for the overall stability of structures should be as given in the relevant Part of BS 6349. However, where overload conditions due to accidental impact or excessive line pulls are being analysed, the factors of safety may be reduced to 1.1. Factors of safety on material stresses under these conditions (except for those elements designed to fail) may be reduced in limit state design by applying a partial factor on applied loads of 1.0, and in working stress design by increasing allowable stresses to 90 % of yield stress.

Guidance on the design of members, grades of concrete and steel, allowable stresses and construction tolerances for open piled structures is given in section 6.

7.5.2 Other types of structure

Rigid dolphins other than those on raking piles may be solid structures or a group of vertical piles with a concrete cap and a strut/tie to shore.



(b) Braced frame supporting deck

Figure 47 — Examples of rigid dolphins (concluded)

Solid structures should be designed in accordance with section 5. Common types of structure for this purpose are concrete caissons, steel sheet-piled cells (circular or rectangular) and concrete blocks with or without vertical rock anchors.

Structures which incorporate a strut/tie to a strongpoint on the shore resist horizontal loads by axial forces in the strut/tie members, which should be designed to resist failure by buckling: for buried struts, the bracing effect of the ground may be taken into account. Longitudinal berthing and spring line loads will cause one of the members to be in compression and the other in tension.

Uplift due to mooring loads applied to strut/tie dolphins is resisted by the vertical piles of the dolphin.

Stability of the structure is determined in the same way as for a quay structure of the same type. Tie rods for strut/tie members should be designed as described in section 4. The factors of safety under overload conditions may be reduced as described in 7.5.1.3.

7.6 Mooring equipment

The mooring equipment to be provided on dolphins will depend on the sizes and types of vessel using the berth, the equipment provided on board the vessels, the environmental conditions and, in many cases, the past experience of the port operator and labour force.

Where large line loads are expected, consideration should be given to providing mooring line load measuring equipment linked to a display in the terminal operations room. This may be used to adjust the mooring lines during cargo handling operations, thereby allowing adjustment of line loads.

All mooring devices should have a safe working load of not less than the maximum breaking load of the largest line. Guidance on line pulls is given in clause 10 of BS 6349-4:1985. The factors of safety against yield of the material and against breakage of the device should be specified in relation to the safe working load of the mooring device. Factors of safety of 1.25 and 2.5 respectively are commonly adopted. Overloading of the dolphin structure may be prevented by a failure device such as break-off foundation bolts.

Provision should be made, if necessary, in the pile cap for cable ducts to each item of motorized equipment. Cables to the dolphin should be run in ducts fixed to the walkways where these exist, or otherwise laid on the seabed. If there is a risk of damage by anchors or material falling through the water, the cables should be laid in a trench.

7.7 Access and working space

7.7.1 Access

Access to dolphins for operating equipment and handling lines generally takes the form of a series of walkways from the jetty head. The walkways should be positioned so as to avoid the possibility of damage by vessels. A typical layout is shown in BS 6349-4.

All isolated dolphins and berthing beams should be equipped with ladders. For safety reasons and ease of line handling operations, the ladders should be installed on a face which is not used for berthing ships.

Where dolphins are built adjacent to land, pedestrian access should be provided.

Walkways and ladder details are covered in section 9.

7.7.2 Working space

Dolphins should be provided with a platform having adequate working space. Where an adequate area is not derived from structural considerations, a minimum clear space of 1.5 m should be provided around the working side of quick-release hooks, capstans and winches, and 1.0 m around bollards and fairleads.

A clear space should be provided at the ends of walkways and heads of ladders.

All working areas should be adequately drained and illuminated. Guidance on illumination levels is given elsewhere [37, 38].

7.7.3 Handrails and toe kerbs

Handrails and or toe kerbs should be provided on appropriate faces of all dolphins.

Section 8. Roll-on/Roll-off terminal ramps

8.1 General

For the purposes of this section, fixed structures include rigid shore ramps and adjustable shore ramps that are not supported by the water, together with their supports. For floating or semi-floating ramps reference should be made to BS 6349-6¹⁴⁾.

8.2 Layout

8.2.1 Typical layouts and structures

Ro-Ro terminals with shore ramps generally require the vessels to be berthed against fixed structures to hold them in position. These structures may comprise a single continuous berthing face, a series of dolphins or pontoons, or a combination of the two. Occasionally, vessels may be held off the ramp by buoys or anchors. Typical layouts are shown in Figure 48.

The orientation of berths is discussed in 2.1.5, and particular note should be made of the navigational constraints imposed by cross-winds and the availability of tug assistance.

Requirements for Ro-Ro ramps are given in BS MA 97. A typical fixed shore ramp and a typical adjustable ramp are shown in Figure 49 and Figure 50, respectively.

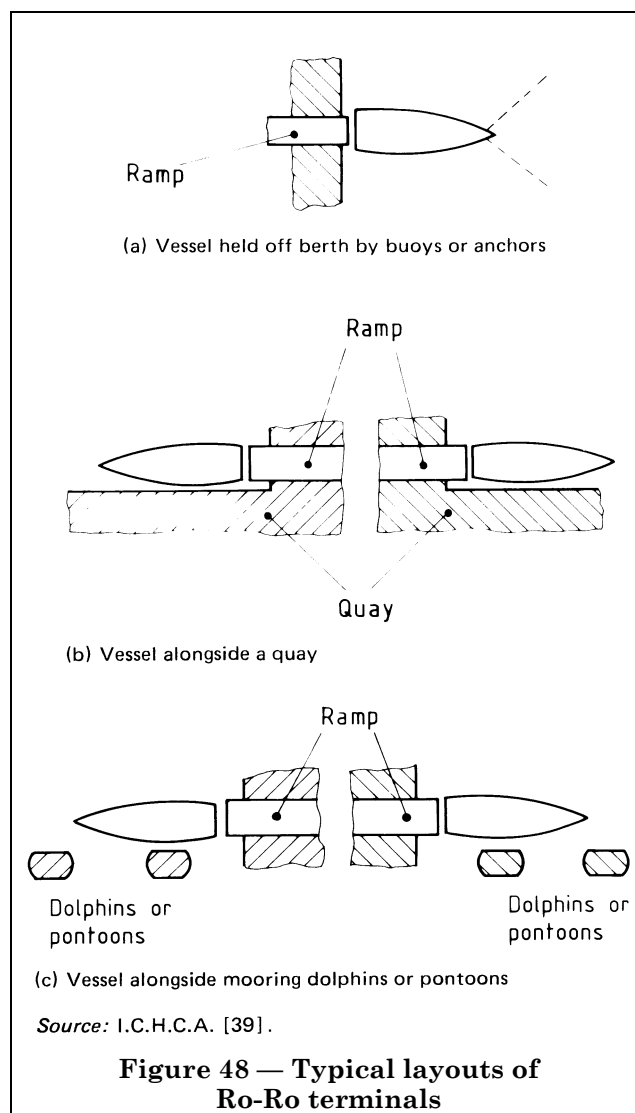
Dolphin layouts and berthing and mooring requirements generally are covered in BS 6349-4. For guidance on pontoons and buoy or stern moorings, reference should be made to BS 6349-6¹⁴⁾.

8.2.2 Vessels to be served

The range of vessels using the Ro-Ro terminal will govern the layout of the maritime shore structures. A terminal serving only one class of vessel (e.g. fixed route ferry terminals) may be designed to narrower geometrical limits than one serving all types of Ro-Ro vessel up to the limit of the available depth of water in front of the ramp.

Vessels operating at shore ramps have ramps which are parallel to the longitudinal axis of the vessel. They may have a single bow or stern ramp, twin ramps or a slewing ramp (see Figure 51). Vessels with side or quarter ramps do not usually require a shore ramp, unless the range of water levels and vessel draughts is large. Some vessels are built with direct access from the shore ramp to more than one deck: they may be served by a single adjustable shore ramp or by a two-level ramp.

For guidance on the interface between terminal and ships with straight stern/bow ramp reference should be made to BS MA 97.

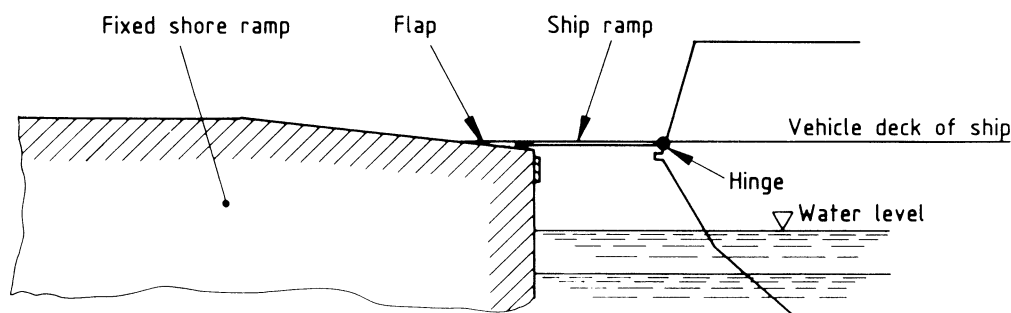


8.2.3 Support for adjustable shore ramps

An adjustable shore ramp is supported at its landward end on a fixed foundation. Adjustment of the level of the seaward end may be by hydraulic ramps or by cables, the system being motivated by electrical power. Hydraulic controls are generally restricted to ramps at locations where the tidal range does not exceed 6.0 m.

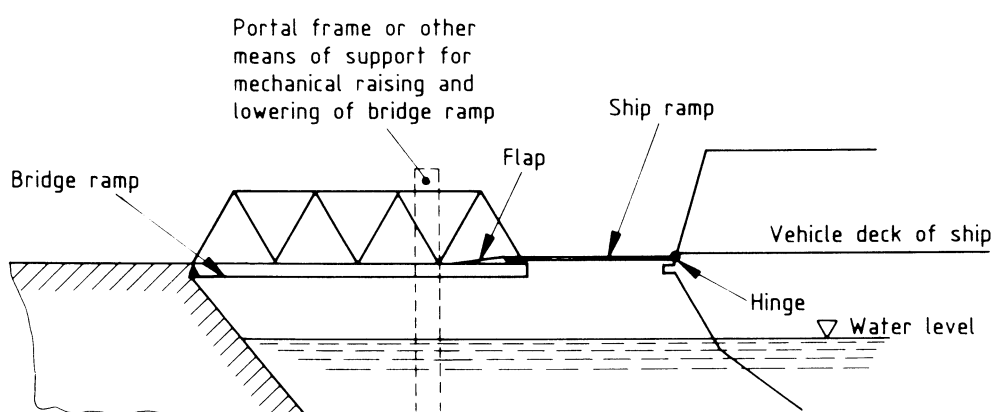
Adjustable ramps should have their safe working load clearly displayed.

¹⁴⁾ In preparation (see foreword).



Source: P.I.A.N.C. [40].

Figure 49 — Typical fixed Ro-Ro shore ramp



Source: P.I.A.N.C. [40].

Figure 50 — Typical adjustable Ro-Ro shore ramp

8.2.4 Heeling vessels

It is normal practice to compensate for heel of vessels during loading and unloading by ballasting the vessel. Any remaining heel or roll will then be taken up by the flexibility of the vessel's ramp. No provision for this is therefore required in the design of the shore ramp. A few Ro-Ro ships have a ramp with a single central support to accommodate heel.

8.2.5 Vessels without ramps

Where a ramp is to be provided for vessels without their own ramp, the short ramp should be fitted with a hinged flap end of sufficient length to provide adequate overlap. Compensation for heel in this case is made by dividing the flap end into segments parallel to the ramp centreline.

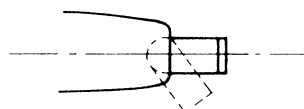
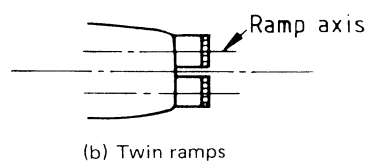
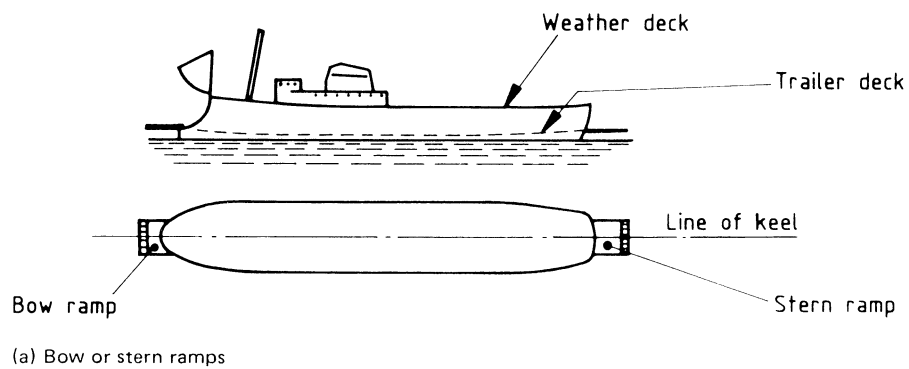
8.2.6 Ramp road surface

Many different road surfaces are used for shore ramps [39]: however, not all of these will be suitable for a particular terminal. The most important factors influencing the choice of surface are the durability required, cost and skid resistance. The possibility of ice formation should also be considered.

8.2.7 Safety barriers and road markings

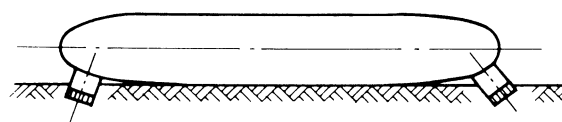
All ramps should be provided with safety barriers along each side.

All ramps should have a continuous white or yellow line painted along each side. Two-lane bridge ramps should also have a painted centre line.

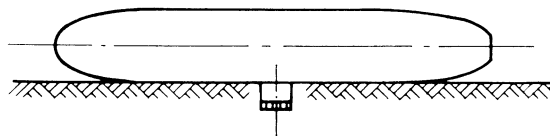


NOTE. Slewing ramps are included when axis of ramp is parallel to line of keel.

(c) Slewing ramps



(d) Quarter ramps



(e) Side ramps

Source: P.I.A.N.C. [40].

Figure 51 — Typical vessel ramp arrangements

8.2.8 Passenger access

At terminals for passenger Ro-Ro vessels, access for pedestrians should, where possible, be separate from the vehicle ramp. If no other means are available, the shore ramp should be of adequate width to include a walkway along one side, separated from the roadway by a continuous safety barrier or kerb. The walkway width will depend on the passenger throughput, but should be additional to the carriageway width.

8.2.9 Ladders

Consideration should be given to the provision of ladders at the sides of ramps to facilitate access by small craft for line-handling purposes and as a safety measure.

8.3 Loads

8.3.1 Vehicles

The loads for which a shore ramp is to be designed will depend upon the vehicular traffic which is expected to use the terminal. It is recommended that ramps should, as a minimum, support highway loading (as specified in section 6 of BS 5400-2:1978) and fork-lift trucks.

The effects of repetitive loading should be assessed (see clause 46 of BS 6349-1:1984).

Consideration should also be given to the imposition of concentrated loads by crane outriggers and front-end supports of fifth-wheel and other trailers, whether for normal or emergency operations.

8.3.2 Mooring and berthing loads

For details of mooring and berthing loads reference should be made to BS 6349-4.

8.3.3 Ramps

The load imposed by ship ramps on shore ramps will depend upon the vessel ramp size and capacity. If these are not known, they may be estimated using the data in [39] for the range of vessel sizes concerned and the vehicle loads given in 8.3.1. The ship ramp should normally be assumed to impose a line load, at right angles to the ship's centreline, on any part of the ship ramp landing area. The possibility of ships with a single central ramp support (see 8.2.4) using the port should be considered.

An allowance for impact should be made for axles travelling over the end of the ship ramp. A similar allowance should be made for the hinged flap ends of shore ramps where provided.

8.3.4 Waves and currents

The wave and current forces acting on floating supports to bridge ramps should be determined to assess the forces set up in the ramp and at the landward support. The cyclical nature of wave forces should be taken into account in assessing the fatigue resistance of the bridge ramp and its connections (see clause 47 of BS 6349-1:1984), where appropriate.

8.4 Bed scour

The effects of ships' propellers and bow-thrusters on the stability of the seabed and any nearby underwater slopes should be carefully assessed. This applies particularly to those Ro-Ro terminals where vessels are manoeuvred under their own power and always berth in the same position (see also 2.3.5).

Section 9. Pedestrian access

9.1 General

Reference should be made to BS 5395 for general recommendations on the durability and safety of walkways, stairways, ladders and handrails and to references [6, 7] for further guidance on health and safety aspects.

9.2 Walkways

Walkways are usually required for access by pedestrians to isolated structures such as dolphins, or between sections of open-decked jetty heads such as may occur at some bulk handling terminals. They should be situated so as to minimize the risk of damage by vessels and cargo handling equipment.

Walkways may be constructed in steel, aluminium, concrete or timber, and should be designed in accordance with the relevant codes of practice, and normally with the loading requirements given in 7.1 of BS 5400-2:1978. Open type metal flooring, complying with BS 4592 should be used to minimize the risk of ice formation where the general public has no access.

It is recommended that the minimum clear width of walkways be 1.0 m.

Where walkways are supported on flexible structures, due allowance should be made for all possible horizontal and rotational movement.

All supports designed to allow movement to take place should be adequately protected and, where relevant, lubricated in sealed units. Maintenance and replacement of bearings should be carefully considered during design.

Walkways should be adequately lit [37, 38].

9.3 Stairways

Stairways may be provided at berthing faces for access to and from small vessels such as ferries and harbour craft, particularly where the tidal range exceeds 1.0 m. At deep-water quays they should be located where they will not be blocked by berthed ships, preferably on return walls and otherwise at ends or changes in direction of queys. A typical arrangement is shown in Figure 52.

The type of construction will depend upon that used for the berth structure. In most cases, stairways will be of concrete, but steel may be more appropriate at skeleton-type jetty decks. Open type metal treads should comply with BS 4592.

Guidance on dimensions of steps is given in BS 5395. For maritime structures, a minimum width of 1.0 m is recommended where only port and vessel personnel will use the stairs, and 1.5 m where the general public has access.

Where the tidal range is 3 m or more, landings should ideally be provided at about 0.75 m above mean high, mean and mean low water levels and at height intervals not exceeding 3.0 m. For smaller tidal ranges, the number of landings may be reduced. Landings should be at least 1.5 m long.

Recessed mooring rings, or similar devices, should be provided at each landing level, at positions which will not cause mooring lines to obstruct pedestrians using the stairs.

Stairways should be adequately illuminated [37, 38].

Fendering should be provided at the berthing face next to and below stairways. It should be arranged to prevent vessel rubbing strips being caught during both rising and falling tides.

Handrailing and wearing surfaces are covered in 9.5 and 9.7 respectively.

9.4 Ladders

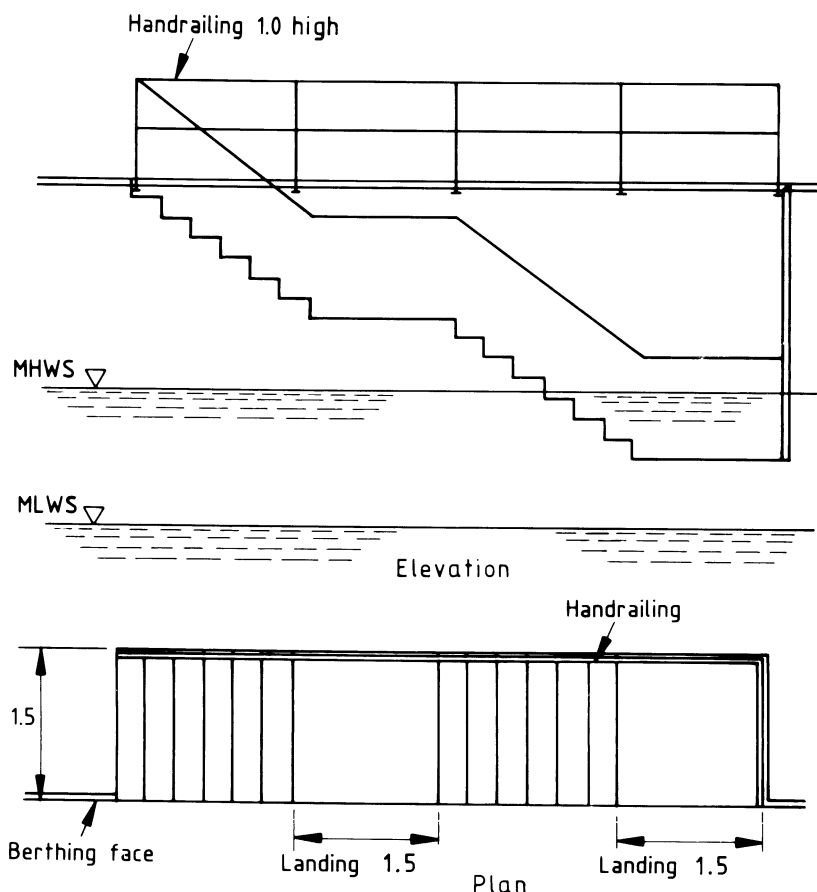
For safety, ladders should be provided at all berths and along access trestles and, in addition, at all structures where access by small craft is required. The recommended maximum spacing of ladders at berth faces is 30 m: a similar spacing may be adopted on access trestles. Ladders should be positioned where they will not be obstructed by mooring lines. Ladders should also be installed at return walls except where stairs are provided. Figure 53 gives typical ladder dimensions.

Ladders are usually fabricated in steel, although aluminium may also be used. Steel ladders should be in general accordance with BS 4211, except in the following respects:

- a) no safety hoop is required;
 - b) stringers should not extend above cope level in any area where they may cause obstruction, for instance to mooring lines;
 - c) no landings are required for life ladders;
 - d) minimum section sizes should be increased for durability in maritime conditions, as follows:
- | | |
|---------------------|---------------------------|
| stringer thickness: | 12 mm drilled and welded; |
| rungs: | 25 mm diameter. |

At the top of all ladders with stringers that do not extend above cope level, a handgrip of the same length and section as the rungs should be fixed with its top recessed in the cope: if a kerb is provided at the cope, the handgrip may be fixed at the level of the top of the kerb.

The top rung of ladders should be 0.15 m below cope level, and the bottom rung 1.0 m below MLWS.



NOTE Fenders and mooring fittings not shown.
All dimensions are in metres.

Figure 52 — Typical stairway arrangement for quay walls

Ladders on berth faces should preferably be placed in recesses to ensure a clear distance of at least 0.1 m between the cope line and the rungs. If the recess is deep enough to create a hazard, it should be surrounded with a kerb or covered with a removable lid.

If ladder recesses are not provided, fendering should be provided on or near each side of the ladder to prevent damage by small craft or ships.

At ladders which may serve small craft, moorings rings or similar devices should be installed on both sides of the ladder together with adequate fendering.

Ladders may also be required to provide access to various parts of a structure for inspection purposes.

9.5 Handrailing

Handrailing should generally be provided to both sides of walkways, to the shoreward side of stairways, to the edges of access trestles and around the landward edge of dolphins and hose-handling platforms. Where vehicle safety barriers are not provided, handrailing should also be installed at Ro-Ro ramps.

Handrailing is usually fabricated from mild steel, although aluminium may also be used. It should be designed for the loads given in 7.1.2 of BS 5400-2:1978.

Where members of the general public have access, mesh panels should be provided.

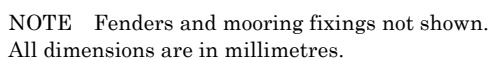


Figure 53 — Typical ladder dimensions

Where permanent handrailing would interfere with mooring lines, a demountable system should be employed.

Kerbs, which may be of timber or concrete, should be provided along the top perimeter of stairway recesses and, if required by the user, on all cope edges.

Kerbs should be at least 200 mm high. The front edge should be set back from the cope line by a distance equal to at least half the kerb height.

The ends of kerb runs and the top arrises of kerbs should be shaped smoothly to prevent snagging and abrasion of mooring lines. Drainage slots should be provided where required.

9.7 Wearing surfaces and durability

Steelwork used for the installations covered by this section should generally be hot-dip galvanized for durability. Items within the splash and intertidal zones may be coated by one of the methods described in clause 68 of BS 6349-1:1984.

Aluminium elements should be designed taking account of the recommendations in 59.3 of BS 6349-1:1984 and CP 118. Care should be taken to prevent bimetallic corrosion from occurring at junctions with other metals.

Concrete surfaces should have a brushed finish. Timber surfaces may require treatment to ensure adequate skid resistance.

9.8 Lifesaving equipment

Lifesaving equipment should be installed on all maritime structures. In particular, it should be provided at jetty heads and isolated dolphins. Lifebuoys (equipped with at least 27.5 m of buoyant line) or throwing lines should be fitted at regular intervals along handrails and at other convenient points which will not obstruct or be damaged by port operations. A further safety measure which may be adopted is to provide chains suspended at the seaward face, extending to 1 m below MLWS in between the ladders referred to in 9.4.

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- BS 4211, *Specification for steel ladders for permanent access*.
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- BS 4592, *Specification for industrial open type metal flooring and stair treads*.
- BS 5268, *Structural use of timber*.
- BS 5268-2, *Code of practice for permissible stress design, materials and workmanship*.
- BS 5337, *Code of practice for the structural use of concrete for retaining aqueous liquids*¹⁶⁾.
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¹⁶⁾ Under revision as BS 8007.

¹⁷⁾ In preparation.

¹⁸⁾ Referred to in the foreword only.

¹⁹⁾ Revision of Civil Engineering Code of Practice No. 2 Earth retaining structures [27].

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