Local actions applied to the wall during filling should be taken into account, and the size of material to be used and the rate of filling should be restricted accordingly.

NOTE 2 The seaward compartments can be filled with lean-mix concrete if conditions are such that the front wall could significantly deteriorate, or in order to provide increased resistance to vessel impacts. Otherwise the front cells may be left either empty or partially filled to adjust the overall centre of gravity and reduce bearing pressures.

NOTE 3 After placing, caissons are usually filled with sand, which may be pumped or tipped, to increase their stability. Compaction by vibration can be carried out to achieve a secure formation to the superstructure, but can also increase internal earth pressures on the walls.

6.1.7 Construction sequence

The likely sequence of construction should be taken into account in the design of embedded walls.

The design drawings should state or reference any constraints and assumptions made in the design that could depend on or restrict the construction sequence of the walls.

6.2 Concrete blockwork walls

*COMMENTARY ON **6.2***

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

6.2.1 Size of blocks

The size of blocks used should be chosen to suit the availability of plant.

NOTE Blocks of approximately 15 t are likely to be the smallest that will be used. Larger blocks are generally more appropriate on large projects or where heavy cranes are available.

6.2.2 Types of construction

When selecting the type of blockwork wall construction to be used, the following should be taken into account:

a) the possible effects of differential settlement on the wall;

b) the possible need to extend the wall in the future;

c) the availability of suitable casting areas and lifting equipment.

NOTE The different types of construction are described in Annex A.

6.2.3 Shape of blocks

*COMMENTARY ON **6.2.3***

Blocks are most commonly solid, but hollow blocks may be used to reduce handling weights in columns 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.

Blocks should be robustly proportioned and arrises should be chamfered to minimize spalling and damage during construction or as a result of high pressures in service. Where horizontal location keys and grooves are provided, the clearances should be adequate to permit placing to the required tolerances.

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.

NOTE Where high horizontal actions such as mooring loads and seismic actions are applied to the top of a wall, steel sections can be grouted into voids in the upper blocks to provide a dowel action.

6.2.4 Foundation

A rubble base foundation at least 1 m thick should normally be provided on top of the founding stratum.

NOTE 1 This thickness can 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.

NOTE 2 The base and bedding are sometimes laid at a slope to increase resistance to sliding.

NOTE 3 The tolerance in the level of the bedding layer surface depends upon the material being used.

The designer should assess whether settlement is expected, and, if so, this should be allowed for by constructing the bedding higher than the final required level, or at a greater slope.

The rubble base should extend on each side of the wall base to accommodate any construction tolerance and to spread the actions.

NOTE 4 This extension is typically 1 m on each side of the wall.

The bedding layer should be designed for the metocean conditions that are likely to occur for the period after placement and before the bedding layer is covered by the wall or scour protection.

If the seabed is rock, the designer should assess the benefits of replacing the rubble base with an in-situ concrete blinding layer. The thickness of this layer should be a minimum of 0.5 m, unless specialist techniques and materials are used (see also Note 5).

NOTE 5 If construction is undertaken in the dry then the layer thickness can be reduced to a minimum of 0.15 m.

*NOTE 6 The importance of keeping the surface of the bedding layer clean is discussed in **6.1.2.4**.*

6.2.5 Design of blockwork wall

6.2.5.1 General

The cross-section of the wall and the size of individual units should be selected such 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. Temporary environmental loading during construction should be assessed in accordance with BS 6349-1-2 and BS EN 1991-1-6.

Blocks should be connected together to resist high locally applied loads, where these exist.

NOTE Attention is drawn to the need to control concrete temperatures during casting. Further guidance is given in BS 6349-1-4.

6.2.5.2 Ground pressure

The wall should be configured in such a way as to spread actions as evenly as possible onto the foundation.

NOTE 1 This is normally achieved by maintaining the eccentricity of the resultant vertical serviceability action within the middle third of the base.

NOTE 2 A more even ground pressure can be achieved by extending some of the blocks on the landward side to counterbalance the disturbing earth actions or by projecting the toe of the wall beyond the seaward face. To achieve the necessary clearances, the maximum toe projection may be located within the thickness of an anti-scour apron.

Projecting blocks should generally be shaped to prevent formation of voids in the filling, although this might not be necessary where rubble backfill is used and where water movements or earthquakes are not expected.

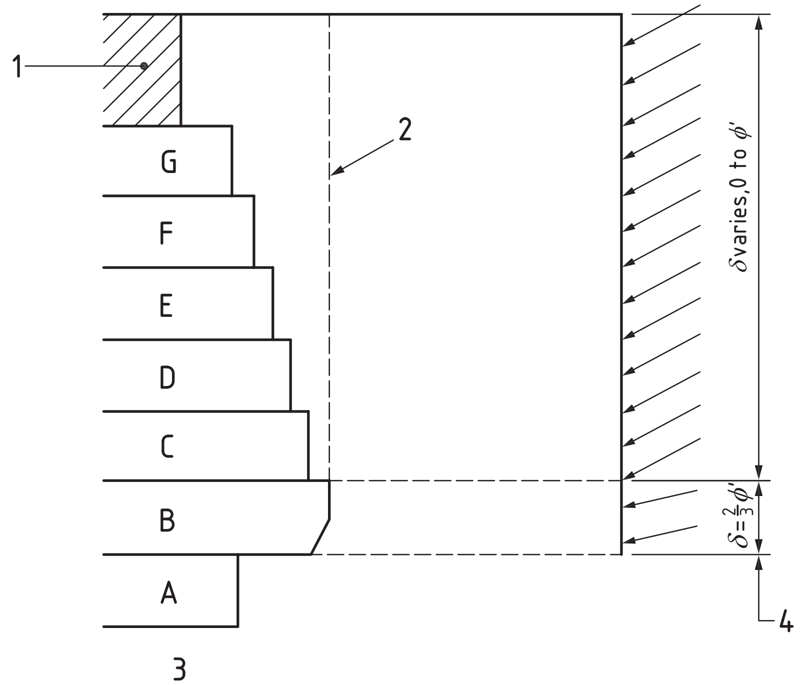
6.2.5.3 Virtual back

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.

NOTE The soil friction on the virtual back of the wall depends on the development of an active wedge in the retained soil and within the zone of soil above the projecting block. Where sufficient movement of the wall occurs and where there is sufficient space for an active wedge to develop in the soil above the projecting block, the friction on the virtual back is zero. As the width of this zone narrows, this friction increases.

The lateral pressure at failure on concrete elements which abut or intersect the virtual back should be calculated using an angle of friction up to $2/3\phi'$ (see Figure 16).

Figure 16 — *Angle of friction on virtual back of blockwork wall*



Key

1 Capping
2 Virtual back of wall for block B
3 Section through back of wall
4 Angle of friction on virtual back

6.2.5.4 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, the designer should assume partial relief of the lateral pressure over the upper part of the lower block. This partial relief is similar to that which occurs under a relieving platform [see Figure 9a)], and should 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 16, an analysis should be undertaken of the friction below the overhang, to determine the overall wall friction and horizontal actions.

6.2.5.5 Block interfaces

*COMMENTARY ON **6.2.5.5***

Blocks are laid with dry joints.

At the horizontal block interfaces, the eccentricity of the resultant of the vertical and horizontal actions should be limited such that the concentration of the local effects in the blockwork and any bedding material does not cause failure. The eccentricity should not exceed one third of the width of the contact surface, except under accidental actions. If the eccentricity under accidental loads exceeds one third, then special precautions should be taken as described in BS EN 1997-1.

NOTE Limiting coefficients of friction between two plain surfaces of precast concrete units lie typically within the range 0.4 to 0.6.

6.2.5.6 Hydrostatic uplift

The hydraulic pressure acting on horizontal joints, unless positive means are taken to restrict water access to the joints, should be assumed to be equal across the full width of the joint to the water pressure either on the back of the wall or in front of the wall, whichever is the greater.

6.3 Precast reinforced concrete walls

6.3.1 General

*COMMENTARY ON **6.3.1***

Precast reinforced concrete walls are used in maritime works, both as quay walls and as bulkhead walls adjacent to suspended deck quays.

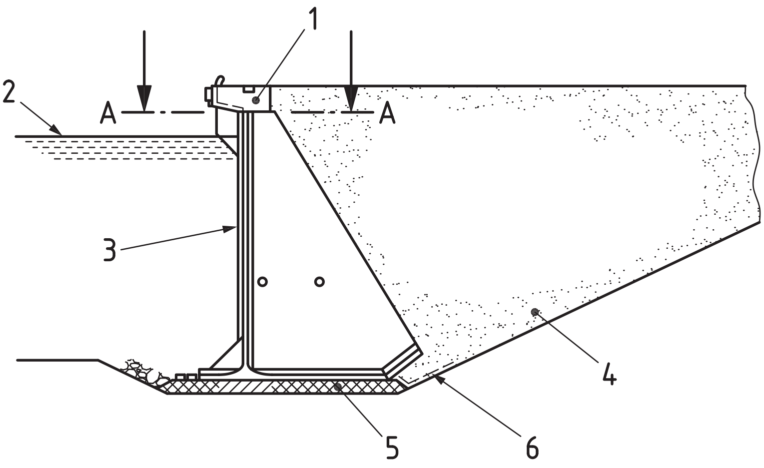
Plain cantilever walls are not generally used for heights in excess of 8 m, and for greater heights a counterfort wall may be used. Units more than 18 m high have been used (see Figure 17).

Precasting of the wall units is more difficult than for blocks and might require slip forming.

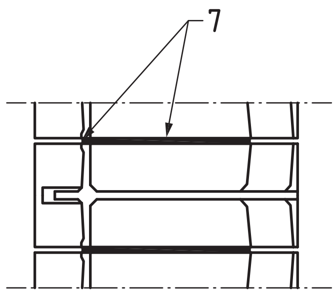
Precast reinforced concrete walls are unlikely to be suitable at sites subject to appreciable wave or current action because of the difficulty of placing the precast units with sufficient accuracy.

The feasibility of using this form of construction for deep-water berths depends on the availability of suitable lifting equipment, either floating or on a jack-up pontoon, and on there being a sufficient length of wall to justify mobilizing such plant and formwork.

Figure 17 — *Precast reinforced concrete wall*



a) Typical cross-section



b) Plan view A–A (showing counterfort units only)

Key

1	In-situ capping beam
2	Water level
3	Precast wall unit
4	Granular material
5	Crushed stone with screeded top surface
6	Filter matting
7	Vertical and horizontal grout joints

Precast reinforced concrete walls should be designed in accordance with BS EN 1992, BS EN 1997 and BS 8004. An in-situ concrete capping should be cast above the precast units after filling.

Less concrete is required than for a blockwork wall, but a significant weight of reinforcement has to be provided and the design should take account of the possibility of corrosion.

The tolerance between placed elements should be taken into account in the choice of fill material being retained and the detail of the joints.

6.3.2 Foundation

COMMENTARY ON ***6.3.2***

The recommendations given for the foundations for concrete blockwork walls (see ***6.2.4****) and concrete caissons (see* ***6.4.2****) apply equally to those for precast reinforced concrete walls.*

For high walls, the tolerance on the level of the top of the rubble foundation should be determined in relation to the design of the joints between units.

6.3.3 Precast units

The toe of the precast units should be arranged such that it does not project into the berthing envelope (see 4.1.5.5).

Crack widths in the walls of the precast units should be determined for quasi-permanent loads.

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

6.4 Concrete caissons

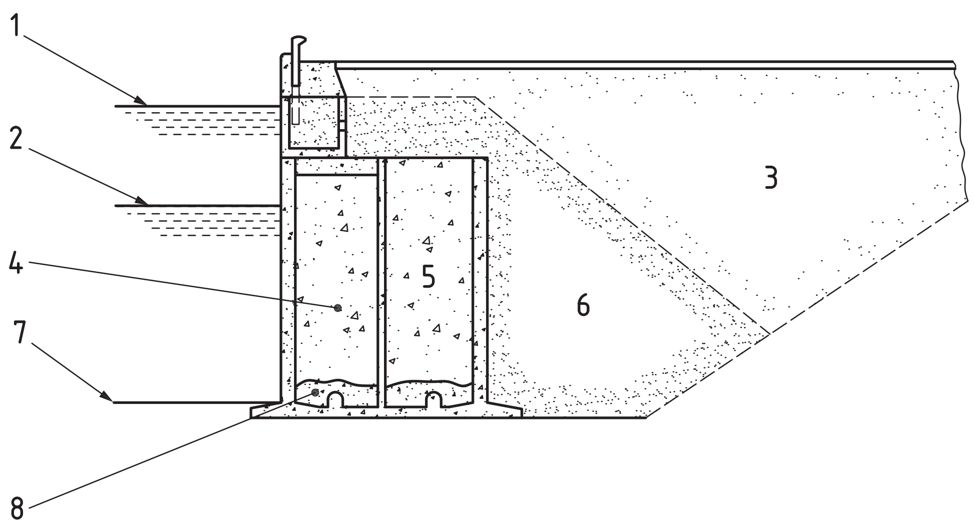
*COMMENTARY ON **6.4***

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 such that, after sinking, the top is above 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 18.

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

This method of construction is appropriate for constructing long lengths of quay, as the establishment of a caisson production and launching facility is difficult to justify for just a few caissons.

Figure 18 — *Example of capping arrangement for a concrete caisson*



Key

1	High water level	5	Gravel fill
2	Low water level	6	Selected fill
3	Hydraulic fill	7	Bed level
4	Front chamber (may be left unfilled)	8	Concrete ballast

6.4.1 Shape

*COMMENTARY ON **6.4.1***

Concrete caissons can be built in a wide variety of shapes on plan. The most common shape is rectangular, but circular, multi-lobe and rectangular with semi-circular ends 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.

An outer projection is often provided on the base slab to increase geotechnical stability, to provide support for wall formwork and for extended projections to improve hydraulic stability.

Caissons should be provided with splays on internal corners to reduce stresses, facilitate reinforcement and avoid local cracking. All external corners and arrises should be chamfered to minimize impact damage.

The length-to-width ratio of a caisson should be designed for floating stability.

6.4.2 Foundation

Caissons should be placed on a foundation that provides adequate bearing capacity and sliding resistance for the global stability of the caisson.

NOTE 1 Support over the full width of the caisson base is desirable to avoid excess bearing pressure, and is normally achieved by maintaining the eccentricity of the resultant vertical serviceability action within the middle third of the base.

NOTE 2 A more even bearing pressure can be achieved by extending the base of the caisson to the landward side to counterbalance the disturbing earth actions and/or by projecting the toe of the caisson beyond the seaward face. To achieve the necessary berthing clearances, the maximum toe projection may be located within the thickness of an anti-scour apron.

Foundation soils that are too weak should be removed and replaced by soils of adequate strength. Inadequate bearing capacity of the soils beneath the caisson should be addressed with methods such as grouting, vibro replacement, soil mixing or piling. The foundation should be designed to resist soil softening, liquefaction and shakedown under wave action and earthquakes if applicable.

Caissons should be placed on an even granular bedding layer placed over the foundation. The nominal size of the bedding should be sufficient to avoid erosion by the wave and current action expected during construction.

The design should prevent the loss of bedding due to erosion during installation and service.

6.4.3 Structural design of caissons

6.4.3.1 General

*COMMENTARY ON **6.4.3.1***

Large caissons generally need to be strengthened with internal walls. These permit economies to be made in the base and wall thickness. The walls are usually cast in situ. The compartments of cells may be used as ballast tanks while the caisson is floating.

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

Caissons should be designed to resist overturning, sliding and foundation bearing failure.

The caisson dimensions should take account of the potential facilities for fabrication and launching that can impose limits on width, height, and weight.

Where caissons retain fill or reclamation, the design should either be sufficiently stable to resist differential hydrostatic pressures behind the wall or be provided with drainage measures to relieve these pressures.

NOTE Greater stability can be achieved by increasing the base width, or by higher density caisson infill.

6.4.3.2 Ground pressure

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

6.4.3.3 Reinforced concrete

Caisson members should be designed for each stage of construction, installation and service. The internal and external walls should be designed for all conditions of lateral pressure due to unbalanced levels of water and granular or wet concrete or fill, taking into account any compaction applied to the fill. The structure should be designed in accordance with silo design practice in accordance with BS EN 1993-4 for steel structures or BS EN 1992-3 for concrete structures, and with actions in accordance with BS EN 1991-4, where appropriate.

A partial loss of ground support should be assumed in the design of caissons, which should be analysed as spanning in both directions. Design of base slabs should take into account the possibility of high points, and loss of ground support under filled cells and of higher than normal ground pressures under empty cells, where these occur. The slabs should be designed to span between the compartment walls.

6.4.4 Fabrication and installation

*COMMENTARY ON **6.4.4***

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

a) on land adjacent to water, then launched either by sliding the caissons down a prepared slope or slipway, lowering the caissons using a crane or lifting dock, or by transfer over a quay wall onto a submersible barge, e.g. using rails or pneumatic tubes;

b) in a casting basin formed behind a bund, and launched by removing the bund or gate, flooding the basin and floating the caissons out at high water;

c) in a dry dock;

d) in a floating dock.

It is also possible to construct caissons by casting the minimum height of caisson required for stable flotation on land or on a pontoon, and then launching. Fabrication is completed by casting the remaining height of the walls while afloat, the draught increasing progressively. Casting caissons afloat requires calm water conditions and deep water.

The caisson draught should be suitable for the available water depth at all stages of caisson launching, flotation transportation and placing. Caissons should have adequate freeboard to prevent accidental sinking due to waves and wash.

The influence on durability of early exposure of concrete to salt water should be taken into account in the choice of materials, design and fabrication of caissons, and additional protection provided if required.

NOTE 1 Further guidance is given in BS 6349-1-4.

Caissons should be designed for the actions occurring during construction, float-out, towing, transportation and installation.

When caissons are transferred from a quay onto a floating barge, the trim should be maintained and the barge should remain stable as the load moves across the deck.

The stability of a caisson should be checked for all conditions such as casting (if over water), launching, towing and placing. The effect of waves, especially those of long period, should be taken into account. In the static and placing conditions, the trim of a caisson should where necessary be adjusted by ballasting. For towed caissons the additional navigational, static and dynamic stability requirements should be taken into account.

The temporary works design should cover the ballasting, trim and control of the caisson during installation, with adequate provision of valves, pumps and standby equipment to prevent internal and external walls from becoming overstressed while achieving an even soft landing.

On first placing, the caissons should be ballasted with sufficient water to give temporary stability before full stability can be achieved by infilling with heavier material as necessary. At exposed locations, a large placing tolerance should be provided to allow rapid placement when sea conditions become suitable. The jointing system between caissons should be designed to allow the necessary tolerances.

Measures should be taken when backfilling behind caissons to prevent the earth pressure pushing the caissons seaward.

NOTE 2 In particular, end tipping of fill can impose significant force, especially as high pore pressures can effectively fluidize the fill, increase the pressure and destabilize the caisson.

The design should take into account wave events that can overtop the caisson wall that encloses a lagoon such that the resultant increase in water level pushes the caissons seaward.

The whole caisson launching, delivery and placing operation should be the subject of thorough investigation bringing together the expertise of fabricators, contractors, designers and naval architects. The response to accidental events should be pre-planned with all parties aware of their role and responsibilities.

6.4.5 Joints between caissons

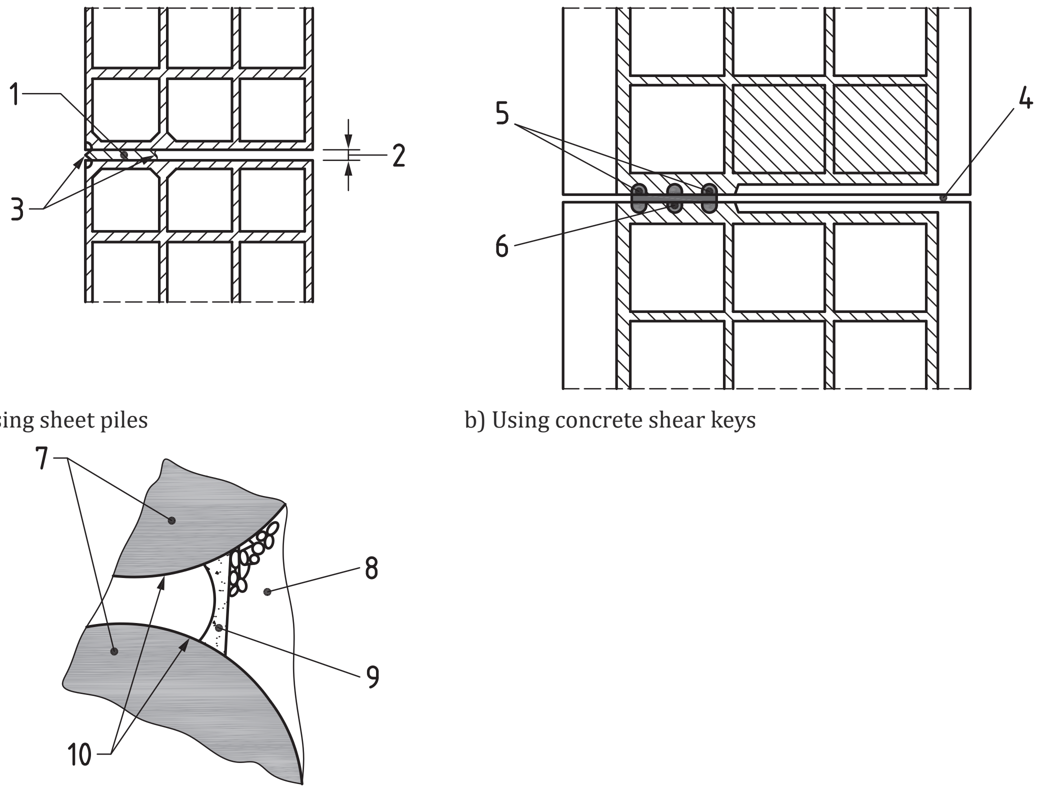
*COMMENTARY ON **6.4.5***

In-situ concrete keyed joints are often 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 19.

Joints between caissons, when used, should be designed to prevent waves from passing through the gaps, to retain reclamation material and to prevent the erosion of bed material.

Figure 19 — *Examples of joints between caissons*



a) Using sheet piles

b) Using concrete shear keys

c) Using precast concrete units between circular caissons

Key

1	Underwater concrete	6	Concrete fill
2	Typical dimension 400 mm to 500 mm	7	Sand fill
3	Sheet pile interlocks with closure piles	8	Rock fill
4	Tapered joint	9	Precast reinforced concrete retaining walls
5	Graded gravel fill	10	Caisson walls

6.4.6 In-situ capping

In-situ capping should be provided in accordance with 6.1.5.1.

6.5 Cellular sheet pile structures

6.5.1 General

COMMENTARY ON ***6.5.1***

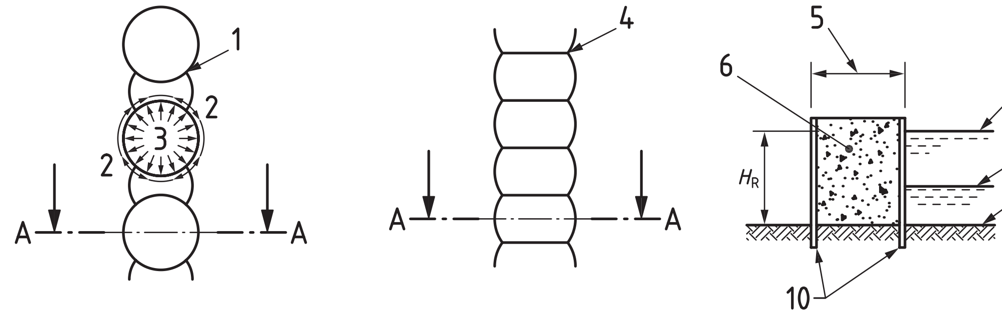
Cellular sheet pile structures consist of cells formed by interlocking straight-web steel sheet piles, driven or placed with their tops above water level. The cells are filled with granular material. The superstructure may be a solid in-situ concrete capping, or a reinforced concrete edge retaining wall which is backfilled and the top surfaced. Cellular walls are usually of either the circular or the diaphragm type. Figure 20 shows a typical plan of each. Junction sections are normally provided to form a continuous wall.

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

Sheet pile cells may be founded on soft rock, granular material or very stiff clay. Cellular structures may be used for both quay walls and jetties. They use less steel than double-wall sheet wall structures *because the steel resists the lateral soil actions in hoop tension rather than bending, and walings, tie rods and bulkheads are generally not required.*

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

Figure 20 — *Examples of cellular sheet pile structures*



a) Circular cells b) Diaphragm cells c) Typical cross-section A–A

Key

1	T-junction	6	Granular fill
2	Circumferential tensile forces	7	High water level
3	Outward pressure	8	Low water level
4	Y-junction	9	Bed level
5	Effective width ≥0.8H_R	10	Straight-web sheet piling

As a general rule, the effective width of the wall should be not less than 0.8 times the retained height.

Soft clays should be removed from cells before fill is placed. Where the layer of soft material is thick, the area in which the cells are to be constructed should be dredged prior to construction to avoid causing overloading of cell walls due to external earth pressure.

6.5.2 Materials and stresses

Sheet piles for cellular walls are of the straight-web type and should be selected on their ability to transmit horizontal tension.

NOTE Fabricated junction sections comprising T or Y shapes are generally available. Extruded sections are also manufactured for use where high tensile forces are to be resisted.

6.5.3 Constructability

COMMENTARY ON ***6.5.3***

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

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

Circular sections of wall should be driven using two gates, one near water level and another near the seabed, in order to achieve the required shape. The gates should be secured to prevent movement and damage to the piles by waves and currents.

Filling of cells in diaphragm-type structures should be carried out in such a way that the height of fill between adjacent cells does not differ by more than 1.5 m at any time unless otherwise allowed

