7 Design of closure works

1
2
3
4
5
6
7
8
9
10

CHAPTER 7 CONTENTS

7 Design of closure works

Chapter 7 explains **how to design** rock **closure works.**

Key inputs from other chapters

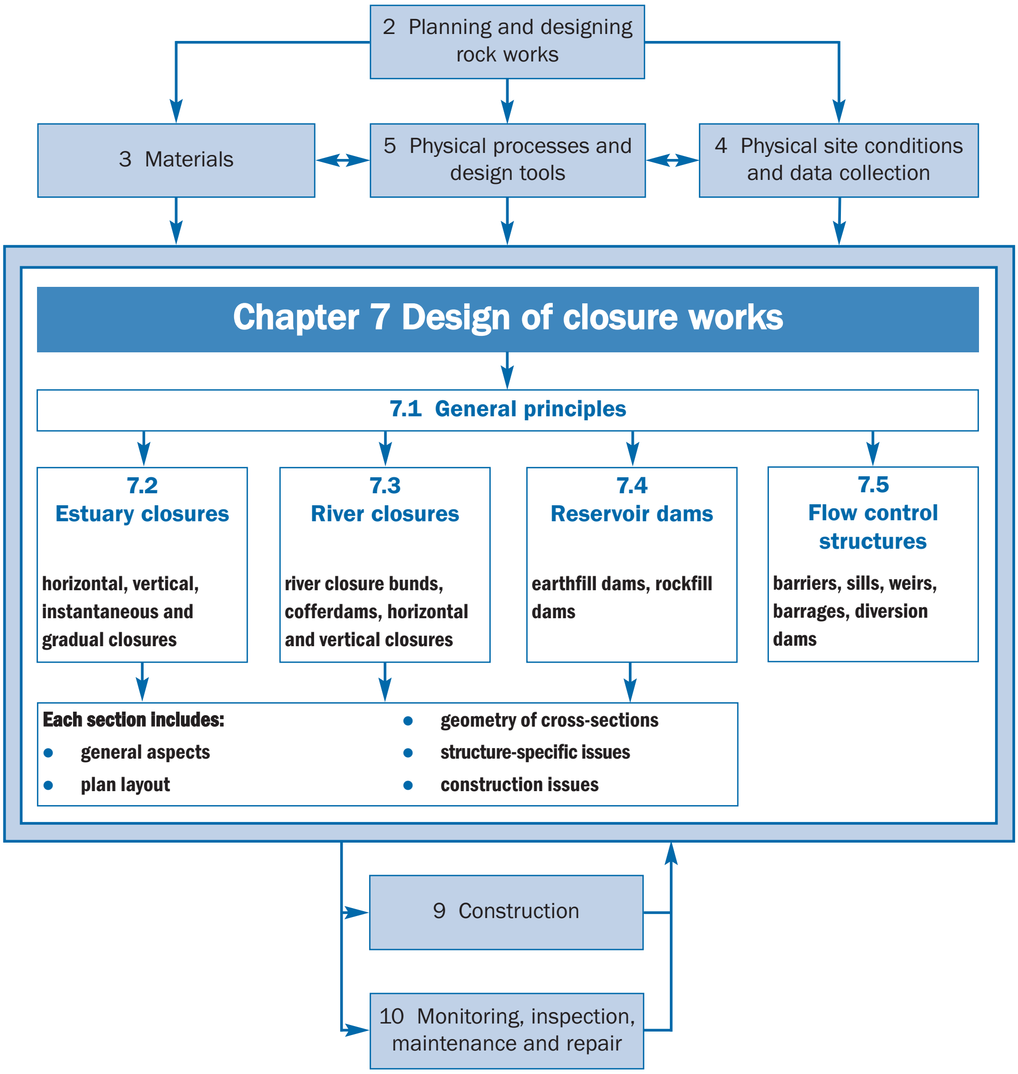
- Chapter 2 ⇨ **project requirements**
- Chapter 3 ⇨ **material properties**
- Chapter 4 ⇨ **hydraulic and geotechnical input conditions**
- Chapter 5 ⇨ **parameters for structure design**
- Chapter 9 ⇨ **construction methodology**
- Chapter 10 ⇨ **maintenance considerations.**

Key outputs to other chapters

- **structure design** (cross-section and plan layout) ⇨ Chapters 9, 10.

NOTE: The project process is **iterative**. The reader should **revisit Chapter 2** throughout the project life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it links to other chapters. Use it in combination with the contents page and the index to navigate the manual.



7.1 INTRODUCTION

This chapter discusses the design aspects of estuary and river closure works and those of reservoir dams and certain other hydraulic structures, as presented in Figure 7.1. The flow chart illustrates how the sections of this chapter relate to those sections elsewhere in the manual that are particularly relevant to the design aspects of closure works. **The focus of this chapter is on closures**, not on the situation after the closure has been completed. The stability of a completed closure dam in general and the stability of the armourstone cover layer (if used) makes a closure dam comparable to other hydraulic structures involving the use of rock. For stability issues affecting the rockfill closure dam after completion, the reader is referred to other sections of the manual.

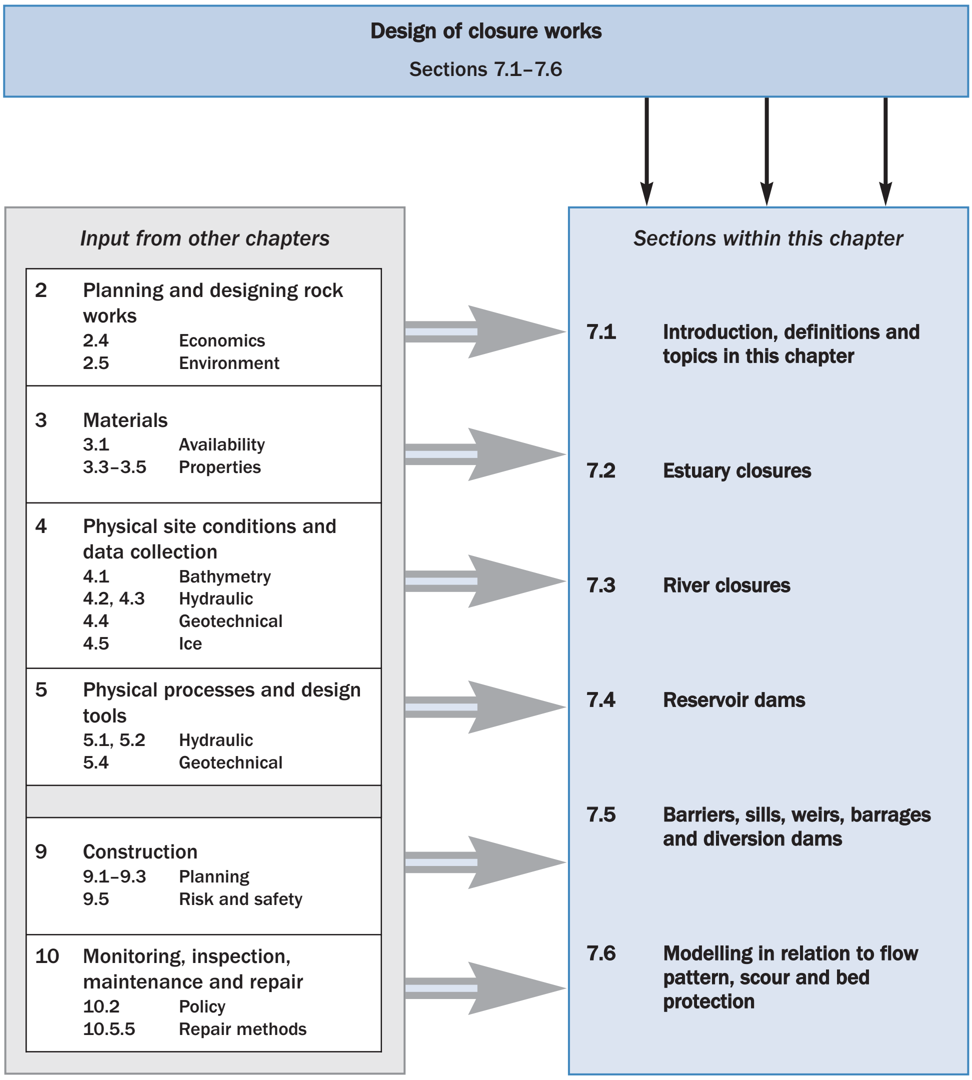


Figure 7.1 *Flow chart, indicating the relationship between sections of other chapters and this chapter*

7.1.1 Definitions and topics discussed

Closure works can generally be defined as structures required to accomplish and maintain the closure of a river channel, an estuary or other water basin. A closure can be permanent (reservoir dam, estuary dam), partial (weir) or temporary (cofferdam). A feature common to all closure works is that during closure the current velocities in the closure gaps increase gradually to a maximum just before the completion of the closure. Just after this moment, the velocities are reduced to zero. After the gap has been closed further work must be done to bring the closure dam up to full strength; this final strengthening of the dam is not discussed here.

Closure works demand proper planning and phasing of the construction process in relation to the hydraulic and physical boundary conditions.

From a hydraulic point of view, the closure of a river or estuary can be *gradual* or *instantaneous*. Instantaneous closure is usually achieved by closing gates in permanent or temporary concrete structures that are incorporated in the closure works.

Most dams in which closure works are necessary are designed and constructed to:

- retain water, eg reservoir, irrigation tank
- separate water bodies, eg an estuary from the sea
- divert part of the water from a river channel either permanently, eg for irrigation, or temporarily, eg by means of a cofferdam.

In closure works rockfill serves the following functions:

- as bed, bank and slope protection to prevent erosion by current or wave attack
- as part of a filter layer
- as part of a dam body, in a zoned or homogeneous fill dam
- in the actual act of closure of a river or estuary.

Rockfill structures such as protection and filter layers, dams, sills and weirs have various design and construction aspects in common. One might consider sills and weirs as intermediate stages in the construction of a dam. Nevertheless, there are basic differences in the nature of the structures, depending on whether they will have a permanent or temporary function as part of the final construction. A **cofferdam** may have a temporary function and be removed after completion of the project, but it can also form part of the permanent reservoir dam. The same is valid for an estuary closure, where in most cases the actual closure dam is incorporated in the final estuary dam profile.

This chapter concentrates on the design aspects of the rock works applied for the closing; the main focus is therefore on the hydraulics during construction of the dams and hence the hydraulic stability of the rockfill in the various stages (for guidance on geotechnical stability, see Section 5.4). The outer facings after construction may vary from clay and grass to asphalt or revetment blocks, armourstone or even concrete armour units, all depending on the design conditions during the lifetime of the structure.

The hydraulic aspects and rock works parts of such structures are discussed, not the structure as a whole. For the design of rockfill closure dams as a whole, as well as for the design of concrete-faced rockfill dams and tailing dams the reader is referred to various publications of ICOLD (see <www.icold-cigb.org> for the latest publications). In the case of rock armouring (as outer protection) of such dams, design guidance is given in this manual.

For the design of protection against wave action on reservoir embankments the reader is referred to Sections 6.3, 8.2 and 8.3. A specific design feature is the rock armouring of embankments where water levels vary significantly, which, although not specifically discussed in Section 8.2, should be included in the design process by the designer. The need to cover the entire area from below the lowest possible water level up to above the highest possible water level with armourstone depends mainly on the degree of variation of the water level in time. In the case of tidal water levels with waves, such effects should certainly be taken into account. This aspect is not specifically discussed in this chapter. The reader is referred to Section 5.2.2 for guidance on stability assessment and to Section 6.3 for general design guidance on slope protection against waves.

In this chapter structures that form closure works are divided into categories based on their function, the boundary conditions and planning of the construction phase.

Accordingly, the following types of closure works are distinguished:

- estuary closure dams
- river closure dams
- reservoir dams
- barriers, sills, weirs and diversion dams.

Filter and protection layers of armourstone have a distinctive function in most of the structures.

Definitions of structures used in this chapter are given below.

- **Closure dams** are structures with the primary purpose of stopping water flow. In some cases, there is a secondary purpose of acting as a temporary dam to protect a site where a dam or other major structure (sluice, barrage, drainage pumping station, navigation lock etc) is to be built in a construction dock.
- **Dams** are concrete, masonry, earth or rockfill structures designed to retain a body of water or to separate two bodies of water permanently.
- In the context of this manual, **barriers** are structures that are normally kept open, but are closed in periods of exceedance of specified high water levels, when they act as dams.
- **Sills** are defined as low dams or **bunds** that may occasionally be overtopped.
- **Weirs** are dams of moderate height (in most cases) that enable a certain specified discharge to pass either over the structure or, in gated structures – generally called barrages or gated dams – through the structure when the gates are open. In common with spillways and outlets (see below), weirs are designed to control discharge and/or water levels.
- **Diversion dams** are like weirs, but higher, shorter and without gates. A diversion dam is generally used to direct water round a dam site during construction.
- **Spillways** and **outlets** are structures over or through which flood flows are discharged.
- **Cofferdams** are temporary watertight structures enclosing all or part of the construction area so that construction can proceed in the dry.

7.1.2 Design methodology and criteria

For a general design methodology, which is also applicable to closure works, see Section 2.3.3. Figure 7.2 provides detail on this methodology with reference to specific design aspects of closure works. Cross-references to revelant sections of the manual are given.

Safety is the primary design criterion for closure works. The safety criterion is normally expressed as a probability of failure and is determined for (**a) the construction period** and **(b) the structural lifetime** of the project. Normally both (a) and (b) apply to closure works and their specific rock-based components. For cofferdams or overflow rockfill dams in estuary closures that are needed only during construction, only (a) applies. The failure probability can be defined:

- on the basis of economic criteria
- on the basis of site-specific or psychologically-acceptable safety criteria
- on the basis of national standards.

Generally, the two latter criteria are used for permanent works. Economic criteria play a major role in defining failure probabilities for temporary structures such as rockfill closure dams in estuaries, cofferdams and diversion tunnels. After the failure probability for the whole project has been established, failure probabilities are defined for each element for each construction stage by means of a fault tree and failure modes. Selection of appropriate design conditions is discussed in Section 2.3.3.2.

Risk should be assessed at the design stage for each structure to be built (see Section 2.3.3 for technical and Section 2.5 for environmental risks). The risk assessment should investigate everything that might go wrong, the possible consequences and the probability of such an event occurring at a particular site. Both the *ultimate limit states* and the *serviceability limit states* should be taken into account (see also Section 5.4.2).

The design process should consider at least the following aspects:

- reliability of design data used, including bathymetry, maximum water levels, type of hydrograph, river discharges and tidal currents, exceedance curves, waves and wind (see Section 4.2), geotechnical characteristics (see Sections 4.5 and 5.4) and expected scour depth (see Sections 5.2.2.9 and 5.2.3)
- reliability of the physical processes assumed to take place, on the basis of formulae applied and model tests carried out, including hydraulic and structural response, scour development, geotechnical interactions (see Sections 5.2.3, 5.3 and 5.4)
- design data and physical processes for intermediate construction stages, including currents, 3D flow pattern, seasonal influences
- possible constraints due to availability of labour, materials and equipment, seasonal variations in river discharges and tide levels (Section 9.3.6)
- possible modes of failure of completed or partly completed structures
- consequences of lack of maintenance (Chapter 10).

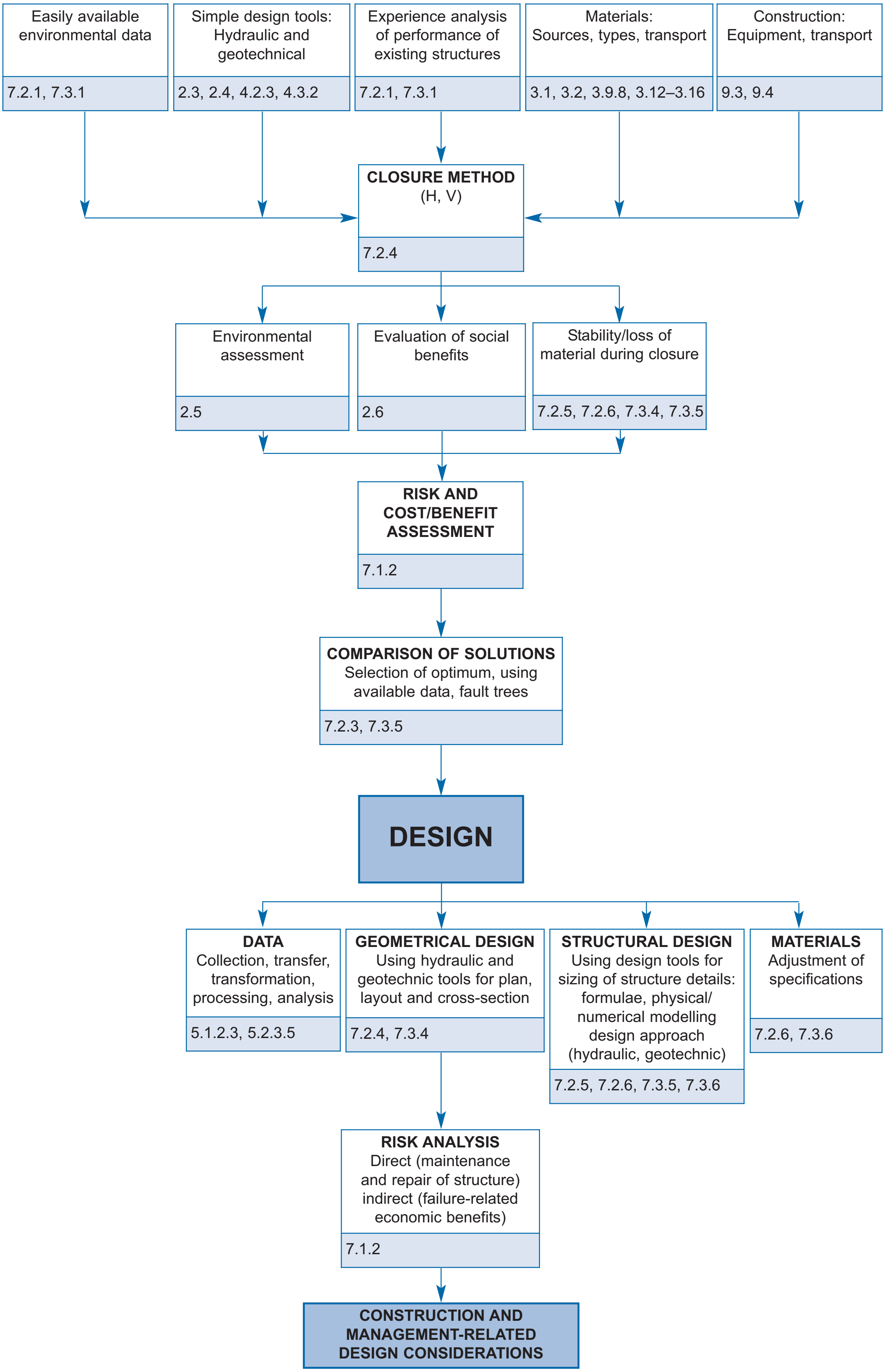


Figure 7.2 *Design diagram for closure works*

7.1.3 Repair, upgrading and maintenance

Closure works are in many cases temporary works. This is especially true for estuary closures where, after completion of the closure, the closure dam itself becomes the starting point for the construction of a new sea dike. This implies that maintenance of the closure dam itself will not be needed although the rock revetment of the final sea dike will have to be maintained, and if needed upgraded (see Section 6.3.3 for revetment design, Sections 6.3.7, 6.3.8 and 10.5 for maintenance, repair and upgrading).

Closure dams in rivers are often transformed into reservoir dams. For reservoir dams made of rockfill maintenance/upgrade of the rockfill itself may not be relevant. Maintenance has to be undertaken on the dam cover layers, especially around the waterline. This is comparable to maintenance on river structures (see Section 8.2).

In designing a structure an appropriate balance should be achieved between the ongoing costs of inspection and maintenance versus the initial capital costs of the project. For this purpose an inspection and maintenance plan is required (see also Chapter 2 and Chapter 10). The plan should be available at the design stage, so that the design can be adjusted to suit the inspection and maintenance procedures or vice versa. For example, after construction of the structure is complete, inspection and maintenance procedures should be feasible. If a structure is designed such that inspection and maintenance procedures, post-construction, are not possible then the design must have a low probability of failure.

To develop an inspection and maintenance plan it is necessary to consider how closures may fail after construction (see Section 2.3.1):

- sudden collapse of an embankment of which the rock-based structure forms a part:
  - sliding of embankment along failure planes in the subsoil, for example due to rapid decrease in water level in rivers or reservoirs or due to scour at the toe of a slope
  - liquefaction due to earthquakes or rapid lowering of water levels
  - failure of transitions
- sudden collapse of bed protection layer:
  - damage by ships eg negative keel clearance, dragging anchors, propeller jets
  - attack by high waves or currents
  - scour at the edge, followed by liquefaction
  - vandalism eg removal of elements
- sudden local failure of bed protection:
  - transition failure
  - vandalism: removal of individual elements of protection layer
- rapid degradation of bed protection:
  - grossly undersized protection layer
- gradual degradation of embankment and bed protection:
  - deterioration of individual elements of protection layer due to overloading, climate and solar effects
  - deterioration of exposed geotextile due to ultra-violet radiation
  - clogging of underlying filter.

Many of the considerations listed above are also applicable during the construction phase. Most of these failure mechanisms can be monitored by regular surface inspection (eg using echo-sounding equipment). However, for gradual degradation processes, such inspection may not be adequate and periodic detailed underwater inspection by divers or using special remote-controlled TV cameras may be appropriate.

7.1.4 General features

Rockfill closure dams can be temporary or may ultimately be incorporated into permanent works ie a reservoir dam, estuary dam or a weir.

Although closure dams on rivers and estuaries have much in common there are a number of differences that are important when determining design and construction methods. These differences are summarised in Table 7.1.

Table 7.1 *Possible differences between closures of rivers and estuaries*

Aspect	Estuaries	Rivers
Wave attack	May be an important factor	Negligible in most cases
Current velocities • Direction • Variation	Depends on tidal prism Direction alternates Varies within the tidal cycle and also from day to day and seasonally	Related to percentage of cross-sectional area closed and related to river discharge Always directed downstream Varies with the season
Subsoil	Nearly always easily erodible alluvium	In most cases rock at closure site
Availability of armourstone	Usually stones have to be transported over considerable distance to reach closure site	In most cases stones of various sizes available near closure site
Type of closure operation	Usually a combination of waterborne and land based	Mainly land-based
Nature of closure	In most cases permanent (ie part of permanent closures)	In many cases temporary (cofferdam)
Differential head during closure	Up to 8 m during spring tides (as typical extreme value)	Up to 4 m at end of closure (as typical extreme value)

In estuaries the tidal volume resulting in current velocities, and the nature of the subsoil, are the predominant factors governing the design and the construction method for a closure, whereas in rivers the discharge, and consequently the current velocity, is the principal parameter to be considered. Estuary closures are discussed in Section 7.2 and river closures are discussed in Section 7.3.

7.2 ESTUARY CLOSURES

7.2.1 Purpose and required studies

Purpose of estuary closures

Estuaries and other coastal areas are closed for one or more of the following reasons:

- flood control and land reclamation in low-lying coastal areas
- creation of a freshwater reservoir fed by upland river discharge
- tidal power plant
- prevention of saltwater intrusion into a river
- protection of the estuary environment against possible pollution by oil or other contaminants.

In many cases a discharge sluice (also called a regulator) is part of the closure works. The function of the discharge sluice is to discharge river floodwater, especially flood waves, to the sea and generally to control the water level of the man-made reservoir behind the estuary dam.

In this manual the expression *estuary closure*, or *tidal closure*, is used to distinguish the above types of closure from river closures. However, the element to be closed is not always an estuary in the geographical sense – it can be a small tidal creek on the coast, a lagoon, a bay, an inlet or a true estuary. Common to all these areas are water levels and currents that are mainly determined by the tide*. During one tidal cycle currents and water levels vary constantly and currents also change direction. The magnitude of the currents during the closure operation depends on:

- the tidal water levels at sea in the undisturbed situation
- the wet surface area to be closed off from the sea by the project.

Data collection, estuary studies and surveys

The extent of the area and duration of the period over which data are collected and studied, and surveys are carried out, depends on the magnitude of the closure. These activities will at least comprise the following:

- bathymetry of the estuary area and the closure site
- wave climate
- tidal characteristics and their variation over the year
- subsoil conditions: peat, clay, silt, fine or coarse sand, rock, including their erodibility
- availability of construction materials: clay, silt, fine or coarse sand, gravel, rock
- locally available labour: skills and numbers
- possibility (or impossibility) of using large waterborne equipment
- possibility (or impossibility) of using land-based equipment
- accessibility of the site.

For a detailed description of data collection and surveys see Huis in 't Veld *et al* (1984) or Section 4.2.3 (for estuary closures) and Section 4.3.2 (for river closures). For estuary closures the **storage area** of the basin to be closed, the **cross-sectional area** at the closure site and the **tidal difference** are the most essential parameters to consider. When these three parameters are known, a first evaluation of the closure can be made.

Model testing

After the first evaluation, the design should be tested in more detail, for which a 2D or 3D mathematical model is often used. Physical model testing of the entire closure operation is rarely necessary, although a physical model may be needed to test details such as scouring at the end of a bed protection.

7.2.2 Types of rock-based structure in estuary closures and their functions

In estuary closures armourstone is used to:

- prevent formation of scour holes at and near the axis of closure by means of **bed protection**
- construct rock bunds, as part of a **gradual closure**, to stop the tidal flows.

Bed protection

Closures in areas with alluvial soil, especially where it consists of sand or silt, can be accomplished only when the bed of the river, estuary or sea in the area of concern has been protected prior to the increase in current velocities. The protection should be a filter layer of sufficient weight to resist strong currents, for example up to 4 m/s or more. These currents

* When the mouths of tidal rivers are closed the river discharge must also be taken into account.

have to be calculated accurately to determine the required mass of the filter material. The filter may comprise a geotextile with concrete blocks fixed to the fabric to form blockmats, a geotextile filter mattress with fascines or a classical fascine mattress of about 20 × 50 m^2. Special equipment is used to roll out blockmats, of approximately 50 × 200 m^2, while fascine mattresses and geotextile filter mattresses are towed to the site by tugs as floating rafts and then sunk by ballasting with one or more layers of stones. Details of the sinking operation are described in Section 9.7.5.2. In countries with high labour costs, geotextiles with fixed concrete blocks are used only for very large projects.

Gradual closure using stones

After the bed protection has been installed and the cross-section in the area to be closed off has been narrowed with dams of sand and/or by other means, the remaining gap in the tidal closure is gradually closed. This is done by constructing a bund, using heavy elements like stones, gabions, concrete cubes or, in less extreme circumstances, even using clay- or sand-filled bags.

7.2.3 Plan layout and concept selection

When selecting the alignment of the closure works it should be noted that the magnitude of the closure operation is directly related to the area to be closed. The closure works may comprise such elements as:

- discharge sluices
- navigation locks
- power plant, if there is a tidal power plant
- estuary dams constructed as hydraulic dams.

The actual design and planning of the closure depends on site-specific conditions and on the results of model testing. Estuary closures are intended to stop the flow between the tidal environment and the reservoir to be created. In most cases the closure is part of the future estuary dam. During the closure operation the discharge sluices (if part of the scheme) can also play a role, since they can be opened during the actual closure operation in order to lower the head difference over the closure gap. For more information on plan layout and concept selection for the closure refer to Huis in 't Veld *et al* (1984) and other publications on the subject. For ease of reference some tidal closure concepts are given below:

- gradual closure by pumping sand in such huge quantities that the tidal currents cannot carry it all away and the closure is achieved in a number of days
- instantaneous closure by placing or dumping sand- or clay-filled bags along the axis of closure within one or two tidal cycles
- shallow closure, gradual or instantaneous, by placing relatively small caissons on a sill across the shallows in the estuary
- instantaneous closure by placing sluice caissons, which will be closed during slack tide, when all elements have been placed and ballasted
- gradual closure by constructing a bund of rockfill along the axis of the closure
- combinations of two or more of the closure concepts above.

This chapter provides guidance for mainly gradual closure by means of rockfill, and the possible use of armourstone as a caisson foundation. The bed protection forms the foundation, or sill for the closure. When it has been agreed to make an armourstone closure, the next step is to decide whether to make a horizontal or vertical closure* or a combination

* In this manual the terms "horizontal closure" and "vertical closure" are always used as illustrated in Figure 7.3 and defined in the subsequent two paragraphs. It should be noted that in some other literature the use of these terms is reversed.

of these. Schematic cross-sections are given in Figure 7.3. Differences between these types of closure are clearly defined in Huis in 't Veld *et al* (1984) and in Sections 5.1.2.3 and 5.2.3.5 of this manual. Here only some of the main features are highlighted.

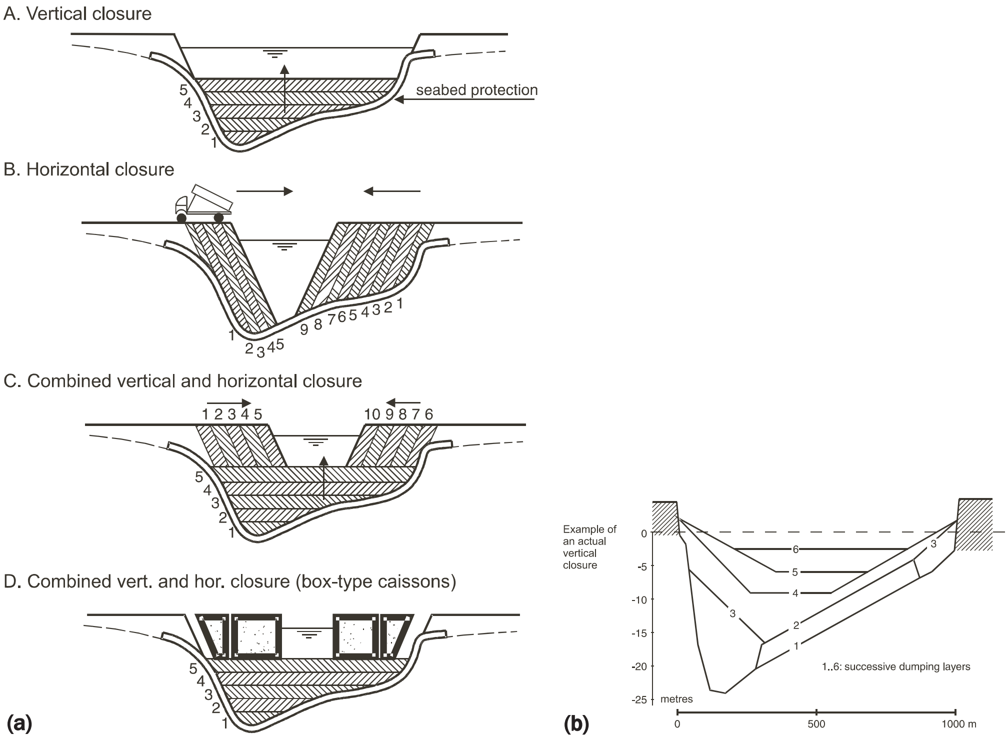


Figure 7.3 *Gradual closure methods (schematically) showing horizontal and vertical closure methods*

A **horizontal closure** is based on the end-dumping of stones (see Figure 7.3a – method B). Advantages are that end-dumping is a relatively simple system and it may be possible to use smaller stone sizes at the start of the operation. Against this is the need to provide a rock bund that is wide enough for a two-way haulage road and, in the case of a long estuary closure, the distance dump trucks have to travel. It is important to remember the increase in current velocity is a function of the reduction in length of the closure gap; only in quite small tidal basins will these velocities not exceed the approximate 4 m/s limit at which the bed protection starts to collapse.

A **vertical closure** is based on the principle of gradually raising the sill over the full length of the closure gap (see Figure 7.3a – method A). This operation may allow the use of waterborne equipment to carry out the dumping in the initial stages. A further advantage is the limitation of the maximum current velocity to that of a broad-crested weir. Disadvantages are the need to use large stone sizes over the full length of the closure gap and, the consequential quarrying requirements (see Section 3.9.3 and Section 6.1.8.1) and the associated porosity of the rock bund during the period of full closure. Moreover, after the bund has reached a certain elevation in relation to tidal water levels, it is no longer possible to utilise waterborne equipment, so other means must be employed to transport and dump the stones. Various options are available for this purpose, of which the two most convenient are:

- dumping from a jetty or a bridge erected across the closure gap
- transport and dumping by means of cableways erected across the closure gap.

A vertical closure creates less turbulence downstream of the gap, so lighter bed protection material can be used. Moreover, a smaller maximum stone size is needed in the last stage of the closure.

All methods require substantial investment in equipment. In addition, designers tend to concentrate on the use of concrete cubes when using cableways rather than on the use of rock, for which it is difficult to design a simple and efficient loading and dumping operation. Stones could be used if packed in 3–4 t gabions, or even larger ones, as it is usually difficult to quarry large blocks of equal size. Note that there is no hydraulic reason to use stones of the same size, but variations in the mass of the stones have undesirable effects on the stability of the cable car, so their use is not advisable.

Finally, it should be emphasised that the grading of available armourstone determines not only the percentage of loss, and consequently the cost and duration of the closure operation, but also the degree of porosity of the closure bund after completion. This porosity leads to through-flow, which in turn may jeopardise the stability of the slopes of the closure bund.

7.2.4 General considerations for cross-section design

The estuary closure has a temporary function, ie to stop the flow, whereas the permanent functions of the final estuary dam profile are (a) retaining high water levels at sea, (b) preventing saltwater seepage and (c) resisting wave attack. Because the flow has been blocked by the closure, it is not necessary to make the higher parts of the dam from heavy stone. Usually cheap material such as sand is used. To prevent erosion of the dam by waves and other forces, a cover layer is applied, usually consisting of grass on clay, above the normal high water level, with a rock revetment structure in the intertidal zone. Revetments are not covered in this chapter because their requirements are similar to those discussed in Section 6.3 for marine structures and Section 8.2 for river structures.

After the decision on the closure design has been taken and, for gradual closures, the construction method for the closure bund selected, the cross-sections of the various rock-based structures can be designed. Similar to other hydraulic engineering structures, the cross-section will be determined by hydraulic and geotechnical boundary conditions, availability and delivery of materials, available equipment and construction considerations including local experience of comparable construction. These site-specific considerations are discussed below.

7.2.4.1 Hydraulic boundary conditions

A general discussion on the type and extent of hydraulic boundary conditions required for the closure of estuaries can be found in Chapter 4. More specifically, a distinction can be made between:

1. **Overall hydraulic boundary conditions** (see Sections 4.2.2, 4.2.4, 4.3.2 and 4.3.3): these are conditions that do not change as a consequence of closure works and include the astronomical tide (vertical tide at sea), wind set-up, wave climate, and, if applicable, also river flow and discharge.
2. **Geometry of the closure area** (see Sections 4.2.3.3 and 5.1.2.3): this has a direct impact on the changes in, and values reached by, local hydraulic conditions, ie the hydraulic response and the way in which the data are collected. The geometry of the basin is characterised by the intertidal area, the bathymetry and the closure dam alignment. The mathematical modelling of the estuary will be based on the prevailing geometry of the basin and the planned geometry of the closure gap(s).
3. **Local hydraulic boundary conditions** (see Section 5.1.2.3): these determine the hydraulic loads on the cross-sections of the rockfill-based structures as they are gradually created in the course of construction. Because of the gradual change in the cross-sectional area of the closure gap, as construction progresses, the local hydraulic boundary conditions will change. Moreover, these hydraulic conditions also fluctuate as a consequence of the variations in tide, wind and wave climate, and river discharge (see point 1 above).

The local hydraulic boundary conditions concerned are:

- water levels at both sides of closure alignment
- head differences, discharges and current velocities at the closure gap
- current velocities near the closure alignment.

These boundary conditions can be determined by using:

- a flow model (see Sections 4.3.5 and 5.3)
- a more schematised basin model (see Section 4.2.3.3)
- stage-discharge relationships (see Sections 4.2.3.3 and 5.1.2.3). These, however, can only be applied when water levels are known.

As the water levels both inside and outside the closure gap vary with time, it is important to know in advance how long the construction period will be in relation to the seasons and how the closure will progress over the course of these seasons. The water levels determine the magnitude of the head differences, discharges and currents in the gap. In their turn, discharges determine the magnitude of the currents near the closure alignment. Any change in the cross-sectional area of the closure gap has an impact on the tidal prism and thus on the water levels at both sides of the closure gap and, through these, on the head difference, discharges and current velocities.

For design purposes the most important local boundary condition is the maximum current velocity, $\hat{U}_g$ (m/s), that can be reached in a given situation or stage of closure and at a given location, eg the axis of closure. $\hat{U}_g$ is used to determine:

- the extent of bed protection at and near the closure alignment
- the stone size for bed protection
- the stone size for each construction stage of the gradual closure.

The method to assess the required stone size for a stable dam face in the various stages of closure for the vertical and the horizontal closure method is illustrated in Box 7.1 for one specific case. The relationship between the maximum current velocity and the required *(relative) stone size*, expressed as ΔD_n (m), is discussed in Section 5.2.3.5 for both vertical and horizontal closure methods. ΔD_n is defined as the characteristic relative nominal stone size (m), with Δ being the relative buoyant density of the stones, = $(\rho_{app} - \rho_w)/\rho_w$. Note that for an armourstone grading for D_n to read: D_{n50}, the median nominal stone diameter (ie the equivalent cube size).

From Figure 7.4 it is apparent that if both the variations of the tidal level, H (m) – defined as the upstream water level relative to crest level in the case of a vertical closure (see Section 5.1.2.3, Figure 5.21), or h_1 (m) in the case of horizontal closure (see Figure 5.23) – and the area, A_b (m²), of the estuary to be closed are given, the decision on a method of closure automatically results in a maximum value of $\hat{U}_g$ and ΔD_n at some stage of closure. The graph gives the calculated maximum values of ΔD_n (m), as a function of the constriction of the gap:

- **vertically** by increasing the sill level, d (m)
- **horizontally** by increasing the relative gap width, A_b/b (m), by reducing gap width, b (m).

For the selected closure strategy, the maximum velocity, $\hat{U}_g$ (m/s), occurring at any moment during the closure and hence the critical stone size, ΔD_n (m), can be found. The latter is directly given in Figure 7.4. In a **purely horizontal closure** (H), $\hat{U}_g$ and ΔD_n can be found by proceeding to the right (reducing the gap width, b) along a horizontal line for a given value of the water depth, $h - d$ (m), where h is the water depth relative to bed level (m) and d is the sill height (m).

Vertical closures (V) are represented by vertical lines at relatively large gap width – in the case given in Box 7.1 at the left (lower) side of the graph, A_b/b, proceeding upwards when the sill crest, d, is raised.

Combined closures (H/V, V/H) are schematised by two lines – a horizontal part and a vertical part. In the first case (H/V), an initial horizontal constriction is made from one or both sides, narrowing the gap. Subsequently, the sill is raised. This is illustrated in Figure 7.4. In the second case (V/H), first the sill is constructed and on top of this a horizontal closure is carried out.

Box 7.1 *Example of the calculation of the required stone size*

To demonstrate the interrelationship between the various parameters mentioned, a graph is presented in Figure 7.4. This graph is based on the simple basin model described in Section 4.2.3.3. For the overall situation the reader is referred to Figure 4.18. Figure 7.4 shows contours of constant values of ΔD_n (m), where ΔD_n is a function of the velocity, $\hat{U}_g$ (m/s). The relationships between ΔD_n and $\hat{U}_g$ for horizontal and vertical closure methods are given in Section 5.2.3.5. The other relevant parameters are the width of closure gap, b (m), the original depth of the gap, h (m), the height of the sill, d (m), and the area of the basin, A_b (m²), to be closed. Figure 7.4 has been constructed for a tidal difference of 5 m, ie a tidal amplitude of 2.5 m. Suppose a closure gap is 15 m deep, the gap width is 300 m and the basin area is 30 km².

The relative gap width, A_b/b, is 0.1 × 10⁶ m. The initial condition is indicated in the figure. From this point, horizontal closure and vertical closures are indicated. For a vertical closure the maximum required relative stone size, ΔD_n, indicated in Figure 7.4 is approximately 1.1 m. A combined closure is also indicated, with an initial horizontal closure to a width of 100 m and then the closure continued vertically. The maximum required relative stone size is approximately 1.7 m in that case.

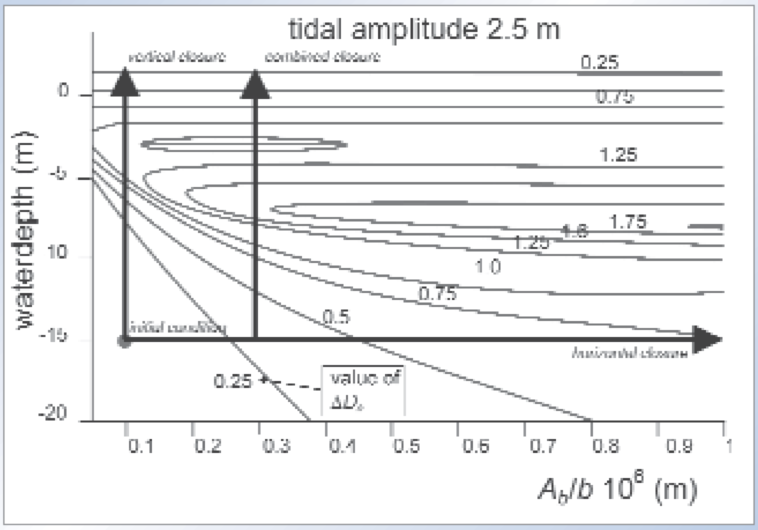


Figure 7.4 *Comparison of closure methods*

Apart from the boundary conditions imposed by the tide, ie the range of H or h_1, and the basin area, A_b, for a specific closure, the actual current velocity diagram will mainly depend on secondary effects such as slightly varying actual discharge coefficients (see Section 5.1.2.3) and/ or variations in A_b with the water level in the basin due to sloping underwater embankments.

General conclusions from Figure 7.4 are:

- high maximum velocities can be **avoided** by a vertical closure or by a combined (H/V) closure with a small relative gap width, A_b/b (m)
- high maximum velocities can be **expected** mainly in the final stage of a horizontal closure, ie with a large relative gap width, A_b/b (m)
- **extreme velocities** can be expected for intermediate relative sill levels, d/h (-), and large relative gap widths, A_b/b (m). This may for example, be encountered with a combined (V/H) closure at a sill level, d/h, chosen too high
- velocities are very sensitive to sill levels in the range of intermediate relative sill levels.

NOTE: the above statements concern the **basic principle only**; for any specific case the critical range of d/h (-) and/or A_b/b(m) or, preferably, a sufficient set of isolines for both $\hat{U}_g$ and ΔD_n should be determined.

7.2.4.2 *Geotechnical boundary conditions and interactions*

Geotechnical boundary conditions play an important role in estuary closures as these almost always take place in areas that have alluvial subsoil. From a geotechnical point of view, the following boundary conditions/interactions govern cross-sectional design:

- the maximum scour depth at the end of the bed protection, and/or behind the sill, as the scour depth will increase over time as a consequence of increasing current velocities
- the grading and size of subsoil and bed materials in view of filter design
- vulnerability of the subsoil to liquefaction
- geotechnical response of subsoil to rapid loading during rockfill closure dam construction
- seepage through subsoil, possibly developing into piping, due to head differences over a completed closure
- rate of sedimentation on bed protection of sill and intermediate stages of a rockfill closure bund.

An analysis of these boundary conditions/interactions in relation to the gradual construction of the closure, eg materials applied, method used, cross-section, construction stages, rate of build-up, will show whether failure mechanisms will develop and what actions have to be taken.

Some of the interactions that are most commonly known to endanger structures are:

- **scour** which may lead to slides and/or liquefaction, which in turn may endanger the stability of the rockfill closure dam (see Section 5.4.3)
- **migration** of materials through filters or by means of seepage/piping, which may lead to local slides and/or settlement (see Sections 5.4.3.6 and 5.4.3.7)
- **sedimentation** during intermediate stages of closures may weaken the structure through subsequent migration of the materials followed by settlement or loss of stability of the closure structure
- **weak subsoil** – if present in the subsoil under the hydraulic structure, and to avoid major slides and/or settlement, materials such as peat or certain clay fractions may have to be removed by dredging and replaced with more suitable sand
- **fine loosely packed sand** – if found in the subsoil, this material may have to be compacted prior to loading by hydraulic structures.

7.2.4.3 *Materials availability*

Contrary to the situation prevailing in most river closures, where rock is generally quarried at or near the construction site, for estuary closures armourstone has to be brought in from further afield. Accordingly, the designer tends to determine desired armourstone properties – for example mass, density, shape of individual pieces as well as the grading – on the basis of an economical design and then look for a supplier. During closure the time when the material has to withstand full load is relatively short. From this point of view, a closure is often a *temporary structure*, so the material requirements may be less strict than for permanent structures. For most material properties, therefore, the requirement NR, as defined in the European armourstone specification, can be set (see Section 3.7). Apart from the grading requirement (light grading class B, LM_B , or heavy grading class B, HM_B ; for definitions, see Section 3.4.3), only the length-to-thickness ratio class A, LT_A , may be needed. The need for compressive strength requirement CS_{80} depends on the method of dumping (see Section 3.7).

1 2 3 4 5 6 7 8 9 10

Concrete blocks or gabions provide an alternative to stones, but their production is only economical if the quantities required are large or the size is greater than stipulated minimum dimensions (say $0.5 \times 0.5 \times 0.5$ m^3) and where logistics such as concrete plant, supply of cement, sand and aggregates can be organised. A disadvantage of concrete cubes is the absence of gradation, which results in through-flow of the rockfill closure bund, the prevention of which requires further stones or coarse gravel to fill the voids. This can be a time-consuming operation.

7.2.4.4 *Materials supply*

As estuary closures take place in a marine environment, in most cases the logistics of transporting the stone and other materials to the site, local storage of the materials, transhipment and loading are all important factors in the preparation works for the closure works. An estuary closure may have to take place within a few weeks or even days. However, the production and supply of materials may take many months or even years, and this supply has to be ensured before closure starts. To accomplish a tight construction schedule a well-organised supply system and storage areas should be developed. The site needs to be accessible to barges and ships for the supply of all materials, which may imply dredging and maintenance of access channels. The storage area has to be able to carry heavy loads. The materials should be stored in an orderly manner so that retrieval for the closure operations can take place uninterrupted. The distance from the closure site should preferably be small. All these conditions may require the development of man-made islands in the area (see Section 9.7) prior to construction of permanent works, comprising construction harbours, construction camps, workshops, storage areas etc.

It is important to calculate the extra volume of materials required to accommodate potential problems in the closure work. This requires a study of potential problems and their consequences in terms of the extent of repair work. As stated earlier in this chapter, the study of failure mechanisms and the drawing up of fault trees should provide answers to these questions and should be used to establish the extra quantities of materials needed.

7.2.4.5 *Construction considerations*

The hydraulic and geotechnical boundary conditions and the selected closure design determine the construction method, the timing and sequence of the various operations and the overall period of construction. The method of construction determines whether and to what extent land-based or waterborne equipment should be used as well as the type, capacity and quantity of equipment required. This is not a simple exercise, as special equipment, such as split-bottom dump barges or side-dumping vessels (see Section 9.3), may only be available in limited numbers. For an estuary closure in the Netherlands (Tholense Gat; see Section 7.2.6) an inventory was made of all available stone-dump barges as well as their planned input on other projects under construction to find any possible conflicts of interest well in advance of closure operations. With regard to materials supply, it is necessary to make a risk analysis of the consequences of equipment breakdown and the need for additional equipment in emergency situations.

Finally, the construction schedule should be reviewed for any navigation requirements or constraints. For example, the top level of a sill must be low enough to enable ships to pass before closure. If ships are to be used on both sides, a navigation lock in an estuary closure will need to be in operation before closure.

7.2.5 Structure-specific design aspects of instantaneous closures

In this section, structure-specific design aspects for instantaneous tidal closures are discussed by considering the small-scale tidal closure of the Amtali Creek in Bangladesh in 1982. Similar examples exist elsewhere, but this closure has been closely monitored and various aspects relevant to this manual are included in the works for this closure.

7.2.5.1 General features of the tidal closure of Amtali Creek (Bangladesh, 1982)

A 130 m-wide creek was closed by the limited use of sophisticated closure equipment, such as tugs, dump barges, cranes and dredging plant, combined with abundant use of manual labour. In Bangladesh in the 1980s the use of local materials, such as bamboo, palm leaves, reed and jute bags, filled with clay instead of stones, were part of the adaptation to local conditions. This, and related tidal closures, have been described in Huis in 't Veld *et al* (1984), van Duivendijk and Te Slaa (1987) and Yoon (2003). Here, only the structure-specific design aspects are highlighted.

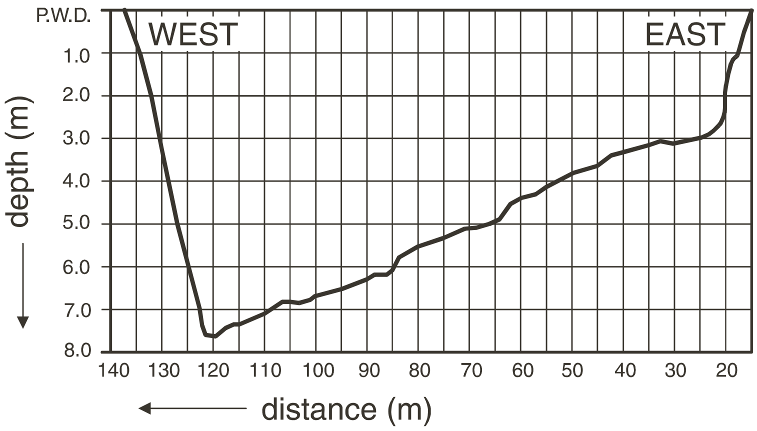


Figure 7.5 *Cross-section of the Amtali Khal*

7.2.5.2 Data and overall boundary conditions

The tidal basin to be closed was a creek. It had a tidal volume of 6 × 10^6 m^3 during maximum spring tide, ie dh = 2.7 m. At the site of closure, the creek was b = 123 m wide at MSL. The cross-section shows a channel with a maximum depth of PWD -8 m (see Figure 7.5, with PWD = local reference level, ≅ MSL -0.4 m). The subsoil consists of fine sands with silt. It is normal in Bangladesh to close such creeks during the winter months when both MSL and tidal differences are at their lowest. At that time there is no danger of cyclones or high wind speeds; also (after the harvest) manual labour is abundantly available in rural areas. Because of the locally available materials, the available skills, the desire not to use merely stones for closures and the labour surplus, the closure concept described below was developed during subsequent experiments (on prototype scale) for such creeks.

7.2.5.3 Bed protection

First, bed protection mattresses were placed on the bed of the creek on the envisaged axis of closure over the full width of the creek and a length of 90 m in flow direction. The bed protection mattresses were ballasted by bags (M ≅ 50 kg) filled with clay (ρ = 1500 kg/m^3). This ballasting required 5500 bags per mattress over an area of 700 m^2, or 390 kg/m^2.

7.2.5.4 Sill, hydraulic interactions and construction

Subsequently, a sill was formed by dumping successive 0.60 m thick layers of clay-filled bags on this bed protection. Calculations were made by using a simple tidal basin model (see Section 4.2.3.3). The tidal range at spring tide in January (during which closure was supposed to take place) is 1.75 m. The tidal basin covers an area of A_b = 230 ha and the width of the sill (the initial gap) is b_0 = 123 m. Current velocities were calculated for various sill levels, d (m), and tidal differences (see Figure 7.6).

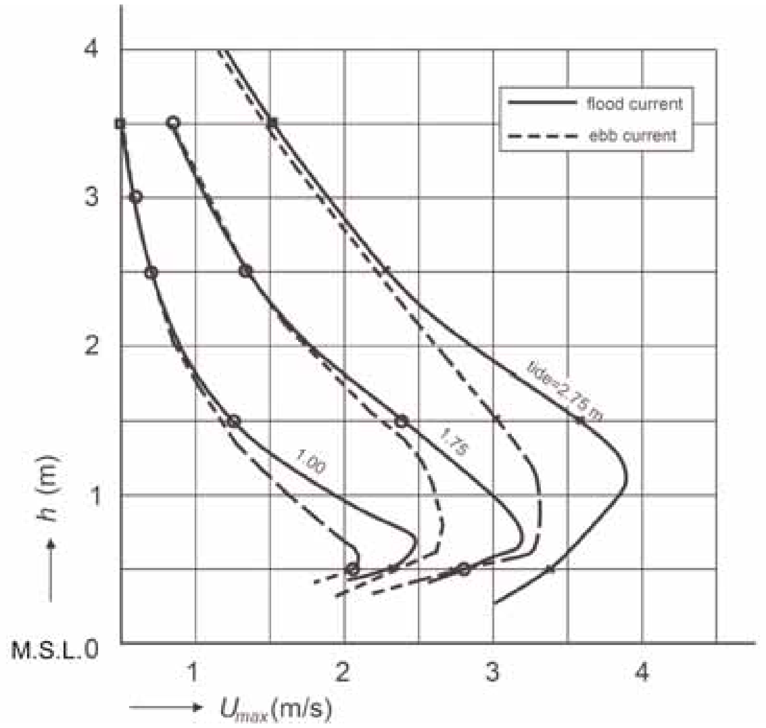


Figure 7.6 *Computed maximum current velocities for various sill levels and tidal differences during closure of Amtali Creek; h is the water depth on top of the sill, MSL = PWD + 0.38 m*

It was known that in the simple basin model, maximum velocities on the sill, when using dumped clay-filled bags as described, should not exceed U = 2.5 m/s and exceptionally, during short periods, U = 3 m/s. It was also known that in prototype, current velocities would be 20 per cent less because of schematisations used in the model (in prototype this means that bags should stay in place with U = 2 m/s while under exceptional conditions U = 2.4 m/s may be temporarily accepted).

The restriction to U = 2.5 m/s **in the model** indicated that the sill level must be at MSL -1.4 m at a tidal difference of 1.75 m (Figure 7.6). As MSL is at PWD +0.38 m during the month of January, for practical reasons the crest level of the sill was designed at PWD -1.2 m (see Figure 7.7, lower drawing).

After completion a bed protection mattress covered the sill. A total of 478 000 bags was dumped on the sill and to ballast the mattresses. On a day of 10 hours (daylight) an average of five barges, representing overall 17 500 bags, could be dumped. Because of delays in the early stages, the sill was only completed during the course of February. Bags were washed away from the crest and the creek side-slope during the flood currents of a spring tide on 10 February. Approximately 50 per cent of the bags dumped during that period were lost. After the bags had had an opportunity to settle during a neap tide period, it emerged that their resistance against removal had greatly increased. The maximum height of the sill was d = 6.3 m at the deepest point in the channel.

7.2.5.5 *Final closure*

Instantaneous closure was subsequently made by successively:

- constructing a jetty across the creek on top of the sill (see Figure 7.7)
- using bamboo poles to transform the jetty into a *cage* during slack tide
- dumping of 200 000 clay-filled bags by manual labour from the jetty into the cage over a period of two days to create a cofferdam.

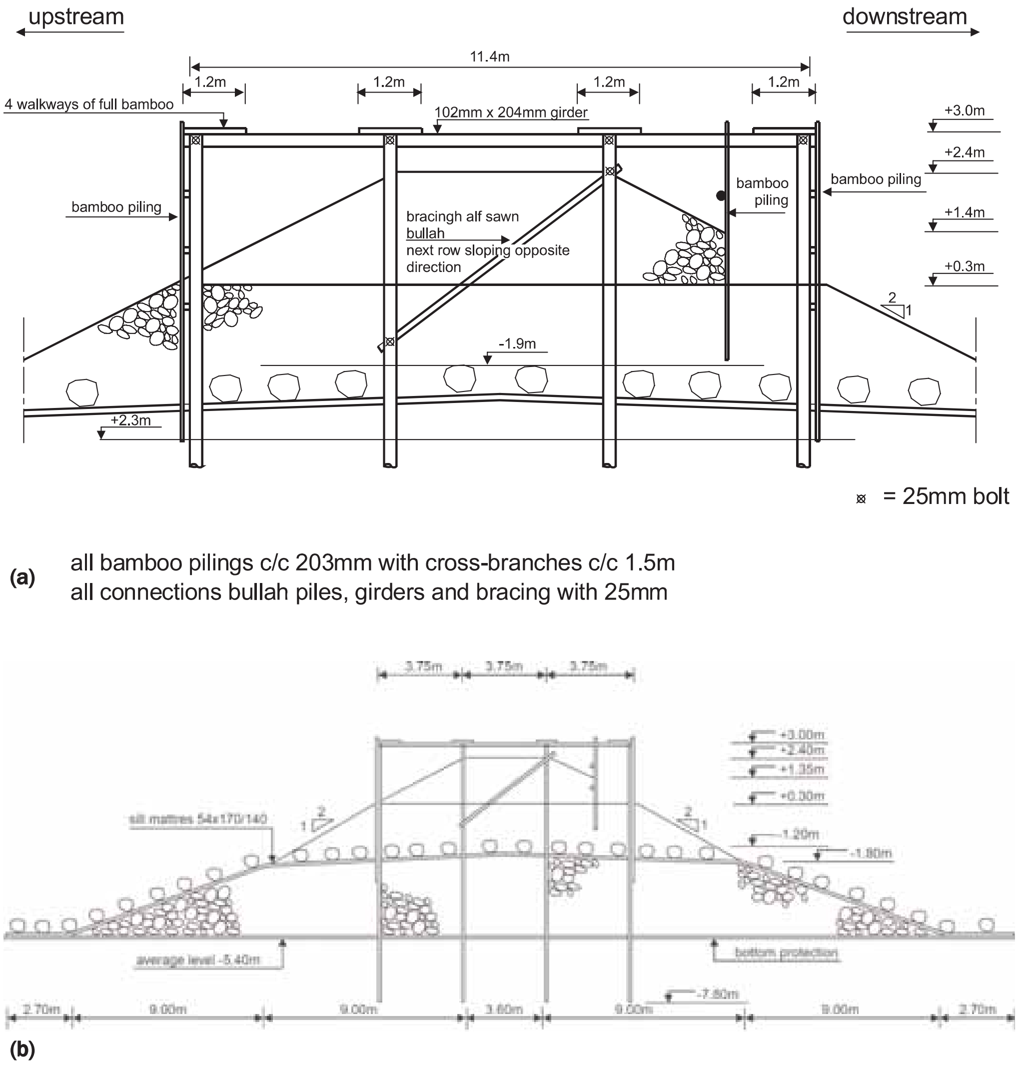


Figure 7.7 *Cross-section over jetty/cofferdam and sill for instantaneous closure of Amtali Creek*

The stability of clay-filled bags in tidal currents can be checked. Since $\rho = 1500$ kg/m³ and $M = 50$ kg, the nominal diameter of the layer of bags is: $D_n = (M_{50}/\rho)^{1/3}$ (see Section 3.4.2) = 0.32 m, which gives: $\Delta D_n = 0.16$ m. Now, for a broad-crested dam, the critical velocity according to *Izbash* (Equations 5.120 and 5.121 in Box 5.10) becomes $U = 1.5$ m/s for exposed stones and $U \cong 2$ m/s for embedded stones. When they touch the river bed, the clay-filled bags will deform and thus be well embedded, during the dumping. Therefore, $U = 2$ m/s and, after further settling down, $U = 2.4$ to 3.0 m/s are reasonable figures for these bags, depending on the remaining degree of exposure to currents.

7.2.6 Structure-specific design aspects of gradual closures

The medium-scale tidal closure of the Tholense Gat (the Netherlands) in 1986 illustrates the structure-specific design aspects of gradual tidal closures. Many similar examples exist, but this closure has been closely monitored and various typical aspects relevant to this manual are included in the works for this closure, carried out by means of a rockfill overflow dam.

7.2.6.1 General features and the closure concept of the gradual tidal closure of the Tholense Gat

The closure of the Tholense Gat tidal channel was part of the Delta Project. The closure was effected between a man-made island built in the Eastern Scheldt and the island of Tholen (see Figure 7.8). The channel to be closed was 370 m wide at mean sea level (MSL), while its maximum depth was at MSL -21 m (see Figure 7.9).

The closure plan and schedule was as follows:

- construct slope protection at both sides of the closure gap in the vicinity of the axis of closure (2.5 months)
- construct a sill in the channel by means of dumping sand, protect sill simultaneously with bed protection mattresses (total three months)
- intermediate winter period, during which no activities take place (eight months)
- gradual vertical closure (two months).

The bank protection was implemented before the start of the sill construction (see Figure 7.8) to prevent erosion and landslides caused by scour of the banks during closure operations. Slope protection works extended over a distance of 300 m on both sides of the closure axis.

The sill level was determined on the basis of:

- construction costs (ie loss of sand during construction)
- minimising impact on navigation during intermediate period
- impact on hydraulic boundary conditions of a tidal closure in another channel (Marollegat) in the same tidal basin
- extent of scour at both toes of the sill.

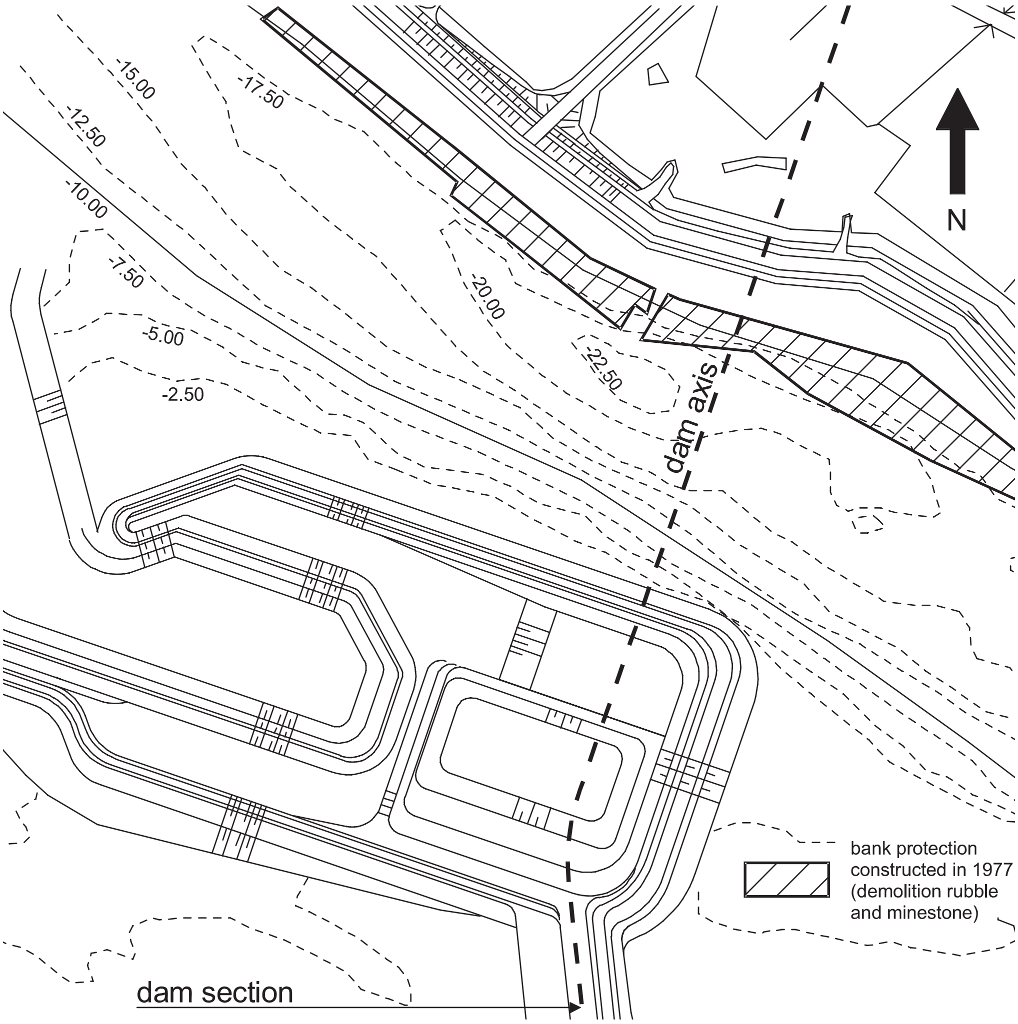


Figure 7.8 *Situation of closure of Tholense Gat, the Netherlands (depths in m)*

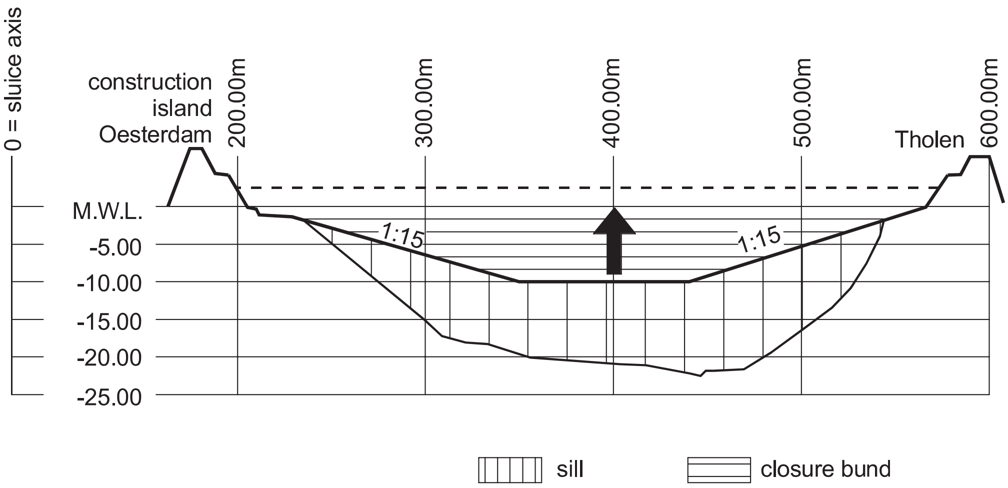


Figure 7.9 *Cross-section Tholense Gat at axis of closure after sill construction (depths in m)*

7.2.6.2 *Sill*

Ultimately, the optimum sill crest was assessed to be at a level of MSL -9.5 m, predominantly governed by the loss of sand by currents. This implied a maximum height of the sill of d = 11.5 m at the deepest point in the channel. Because of the chosen construction method (dumping sand in water) the density of the sand in the sill would be low, so the sand would be prone to liquefaction as and when scour holes developed at both sides of the sill. Accordingly, the upper parts of the sill were protected against erosion. To limit the cost, this protection was applied only to a point where a scouring hole cannot cause harm to the closure bund in the case of sliding or liquefaction. The principles of the assessment of the length of scour protection are presented in Box 7.2.

Box 7.2 *Length of bed protection*

Downstream of a closure a scour hole will be formed. To guarantee the safety of the closure structure this scour hole should be an adequate distance away from the structure. This is illustrated in Figure 7.10. Should a slide occur at the edge of the bed protection, the resulting slope after sliding will not reach the structure itself. With the depth of the scour hole denoted as y_{max} (m), a safe length of bed protection, L (m), is given by Equation 7.1:

$$L \geq n_s y_{max} \tag{7.1}$$

where n_s is cotα, with α being the slope angle of the soil after failure (see Figure 7.10). For normal sand the value of n_s is in the order of 6; for sand sensitive to liquefaction, a value of 15 should be used. The calculation of the depth of the scour hole is outside the scope of this manual and the reader is referred to appropriate literature, eg the *Scour manual* (Hoffmans and Verheij, 1997).

Figure 7.10 *The relation between scour depth and protection length*

7.2.6.3 Bed protection

To limit the risk of liquefaction as much as possible, a layer (0.5 m thick) of phosphorous slag was placed on the channel bed before sill construction began. This was done near the future edge of the bed protection, being the most likely place where scour holes would develop. This layer was 28 m wide over the full width of the channel and was placed at both sides of the sill (see Figure 7.11).

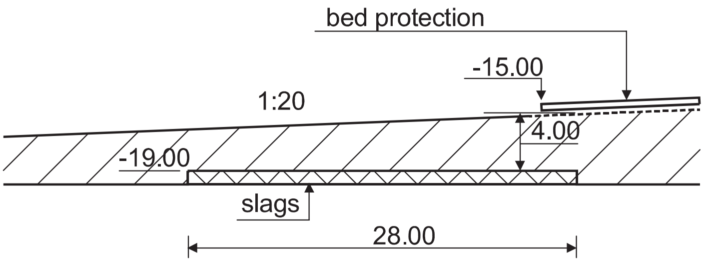


Figure 7.11 *Layer of phosphorous slag placed at toe and heel of sill prior to construction (dimensions in m)*

The top layers of slope and bed protection had to be designed to resist the maximum currents that occur when vertical closure has decreased the area of the closure gap to 40 or 50 per cent. The design of the downstream bed protection (ie the assessment of required stone size and extent of protection) is dependent on the stage of the closure: for vertical closures defined by: d/h (-); for horizontal closures: b/b_0 (-). The stone sizes required are smaller at increasing distance from the closure alignment.

For the design of the bed protection downstream of a sill and/or closure bund, two methods can be used:

- physical (or scale) model tests (see Section 5.3)
- use of a stability criterion, for example Equation 5.129 (in Section 5.2.1.9), including velocity profile factor, Λ_h (-), and turbulence amplification factor, k_t (-), discussed in Sections 5.2.1.8 and 5.2.1.3 respectively.

The problem of determining the stone size for the bed protection downstream of a dam (in fact, a bund or sill) is basically a matter of the principal hydraulic interactions with the dam: velocities, U (m/s) and turbulence, ie the relative intensity of turbulence, r, expressed as a percentage (see Figure 7.12). Laboratory tests gave an indication of the stability of the stones of the bed protection relative to that of the closure dam stones. Based on the situation with the same stone size used for both the bed and the dam, a discharge factor, F_q (see Section 5.2.3.5, Equation 5.240), was defined. This factor, F_q (-), may be interpreted as a relative safety factor. The tests showed scattered values in the range of F_q = 1 to 2, with a tendency towards higher values for increasing height of dam or sill (d/h). When using a discharge criterion, the critical diameter D is a function of $q^{2/3}$ where q is the specific discharge (discharge in m³/s per m width). This implies that at a sufficiently large distance from the sill the stone size can be reduced to 60 per cent of the size near the sill. However, because the given length of the bed protection was only 100 m, no reduction was applied.

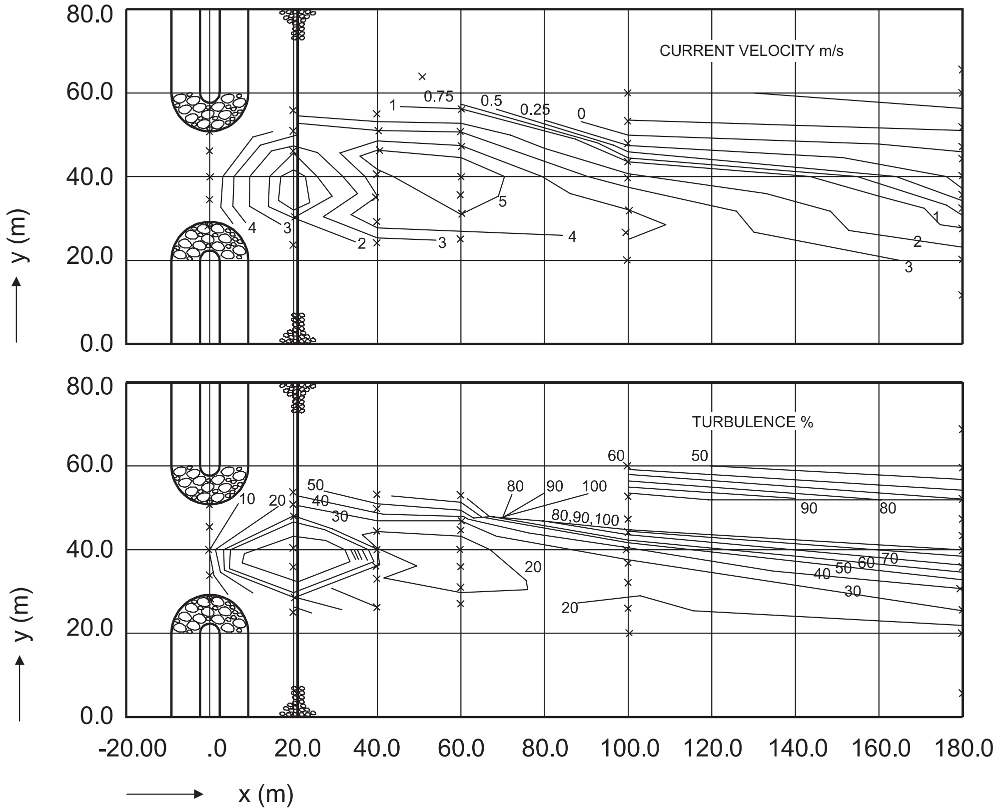


Figure 7.12 *Flow and turbulence intensity pattern downstream of a combined closure. Note: the flow is directed at an angle of 10° ith the line perpendicular to the dam axis*

At the closure stage that is most critical for the bed protection, the vertical closure had reached a level of MSL -4 to MSL -5 m. At that time 10–300 kg stones were used for the vertical closure. For a relatively thin layer of stone to be used as a bed protection, the grading should preferably be less wide than for stones in a closure bund and therefore a cover layer of 60–300 kg was chosen.

Near the banks, the bed and bank protection suffered heavier current attack than in the middle of the channel as the vertical distance between sill level and bed protection was smaller than in the centre of the gap. This heavier attack took place when the closure bund has reached a level of MSL -3.5 m. On the closure bund, the armourstone grading 300–1000 kg was then applied. The same grading was applied on the bed protection upward from a depth of MSL -7 m. If h = 8 m, this stone size should be applied over a distance of 7 × 8 ≅ 60 m from toe and heel of closure bund and then be reduced to 60–300 kg. At 110 m distance from toe and heel, it was possible to use armourstone grading 10–60 kg.

7.2.6.4 Final closure

For the actual closure by means of a rockfill bund across the channel on top of the sill, a combined H/V closure was considered (see Section 7.2.4). The horizontal closure would have started as soon as the crest level reached by a vertical closure would enable a free overflow weir to function. Such a horizontal closure, however, would have had to proceed simultaneously from both banks. As no stockpile area for stone was available at the Tholen side, this idea had to be abandoned. A horizontal closure from one side only is undesirable in view of current attack on and turbulent flow along the bank. This also ruled out a full horizontal closure. In view of all these considerations, it was decided that a vertical closure should be made. The size of stone to be used during various stages of closure was determined in accordance with the principles and formulae discussed in Sections 5.1.2.3 and 5.2.3.5.

Apart from the four flow regimes – ie low, intermediate and high dam flow and through-flow, as discussed in Section 5.1.2.3 (see Figure 5.20) and in Section 5.2.3.5 (see eg Table 5.57) – the designers realised that during the construction various types of dams would develop, such as broad-crested, sharp-crested and multi-crested (see also Table 5.57 for design values of stability numbers etc). Finally, the completed vertical closure bund had to be able to resist wave attack as well as overtopping before it was incorporated into the final dam profile. In practice, this was achieved by adding one-third of the significant wave height, H_s (m), to the upstream water level, H (m), to obtain the equivalent overtopping height, H_{eq} (m). This is illustrated in Figure 7.13. The model tests indicated that a dam geometry with a 5 m-wide crest and slopes of 1:2 would result in the related stability numbers, $H_{eq}/(\Delta D_{n50})$ (-), as given in Table 7.2 for three typical degrees of damage.

Table 7.2 *Stability numbers for final closure Tholense Gat*

Damage	Stability number $H_{eq}/(\Delta D_{n50})$
Almost none	≤ 1.0
1 stone per m run	1.1
Failure	1.3–1.4

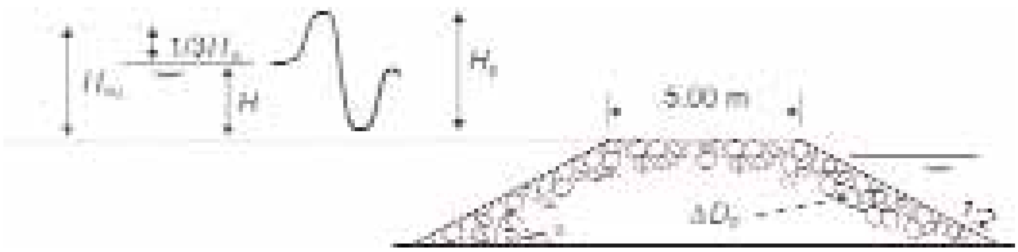


Figure 7.13 *Wave and current load during overtopping of completed closure bund*

Within the context of this manual it is impossible to summarise all variations in hydraulic boundary conditions, possible failure mechanisms and probabilistic optimisation procedures to be evaluated in the design of the various construction stages. The general principle followed has been to use a certain stone size until a failure probability of P_F = 2 to 3 per cent has been reached, because up to this point if damage occurs it is cheaper to carry out repairs than to use larger stones.

7.2.6.5 Closure bund

To prevent typical contraction effects around the dam heads, the closure bund was built up in layers with a maximum thickness of 1.5 m (see Figures 7.9 and 7.14). The materials used for closure were phosphorous slag and armourstone in various gradings (10–300, 60–300 and 300–1000 kg). The use of different materials also necessitated a study of internal erosion (filter rules, see Section 5.4.3.6) between layers of different composition. Other aspects studied were:

- micro-stability during high-dam flow, ie characterised by $H > 0$ and $h_b/(\Delta D_{n50}) < 0$, where H = upstream water level (m) and h_b = downstream water level (m), both relative to dam crest level (see Figure 5.20 in Section 5.1.2.3)
- settlement of subsoil and dam body (see Section 5.4.3.7 for further guidance on design)
- the transition between closure bund and the banks of the channel (also in view of geotechnical features of these banks).

From MSL -9.5 m up to MSL -5 m the closure bund was built up with phosphorous slag. Slopes were then covered by a 1 m thick layer of stones (grading 10–300 kg). The closure bund was subsequently built up by placing 1.5 m-thick layers of stones up to MSL -3.5 m

(0.75 m of 10–300 kg followed by 0.75 m of 60–300 kg). The subsequent layers were placed as shown in Figure 7.14.

Up to a level of MSL -1 to -1.5 m, the stones were dumped from special controlled side-dumping vessels. At higher levels dump trucks were used as well as backhoes. Care was taken always to close a stone layer at some distance from the banks. This led to the complicated structure shown in Figure 7.14. A special feature was the construction of runways for the dump trucks by using smaller stones on top of the 300–1000 kg grading.

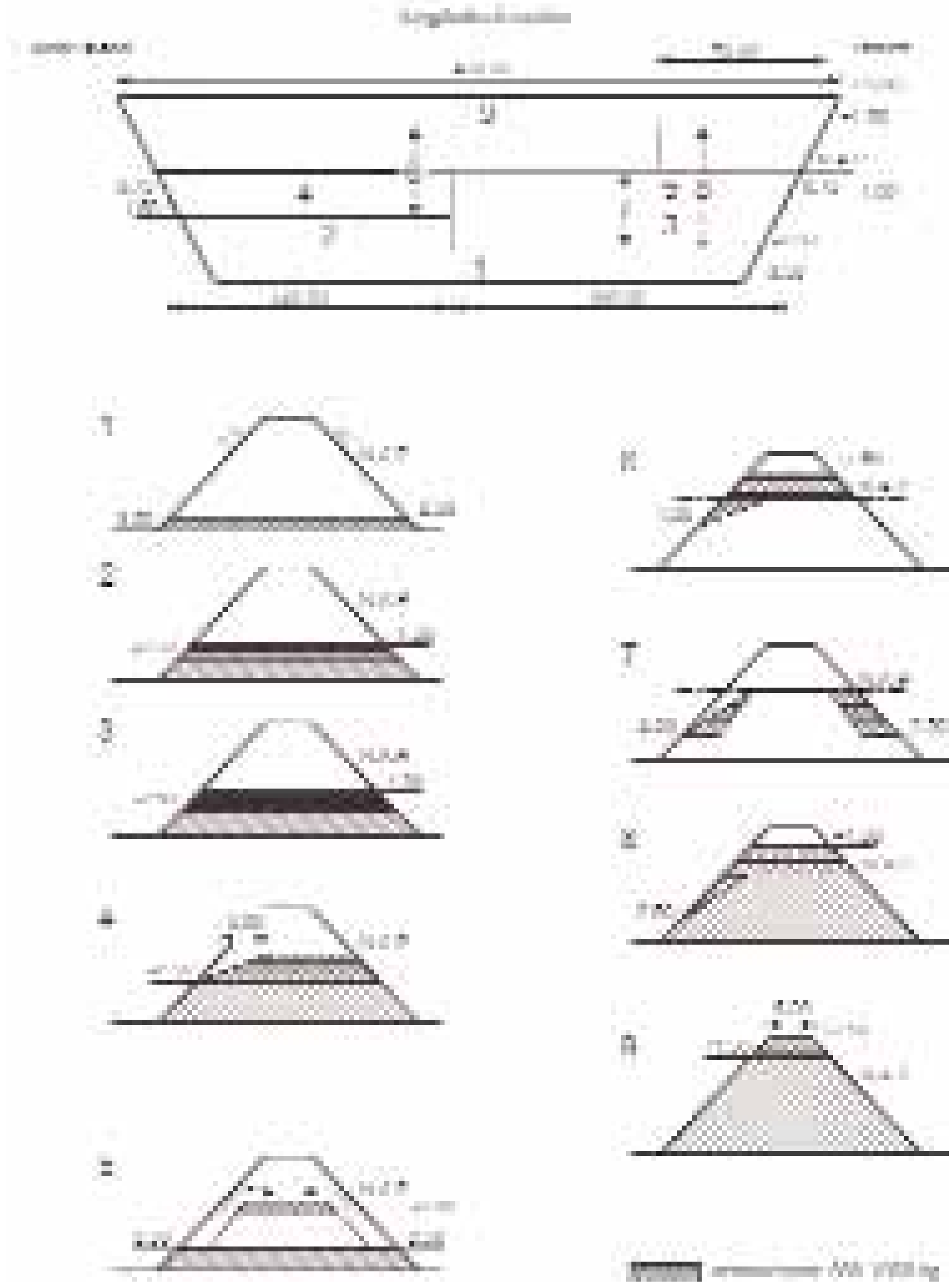


Figure 7.14 *Build-up of closure bund at Tholense Gat, from MSL -3.5 m to MSL +3 m. NAP = local reference level, approximately equal to mean sea level (dimensions in m)*

The average tidal movements at both sides of the closure bund as well as the current velocities are shown in Figures 7.15a for crest at MSL -1 m and in Figure 7.15b for crest at MSL. In this range, armourstone with grading 300–1000 kg was applied, corresponding to M_{50} = 630 to 800 kg (see Table 3.6 in 3.4.3.7. The corresponding value of D_{n50} for a mass density of 2650 kg/m³ is 0.65 m (see Section 3.4.2).

For a crest level of MSL -1 m and average tidal movement, the upstream and downstream water levels are H = 2.4 m and h_b = 2.0 m respectively, so $h/(\Delta D_{n50})$= 1.9, which corresponds to intermediate flow (see Figure 5.20 in Section 5.1.2.3 and Table 5.57 in Section 5.2.3.5), and therefore Figure 5.97 (Section 5.2.3.5) applies. Now, since $h_b/(\Delta D_{n50})$ = 1.9 and $H/(\Delta D_{n50})$ = 2.3, it can be seen from Figure 5.97 that this stone is stable. Moreover, it should be noted that the maximum velocity ($\hat{U}_0$ = 4 m/s) appears at t = 15 hrs (see Figure 7.15a) with H = 2.3 m and h_b = 1.3 m. Under these conditions, $h_b/(\Delta D_{n50})$ = 1.25 and $H/(\Delta D_{n50})$ = 2.2 and from the same Figure 5.97 follows that this is still a stable situation.

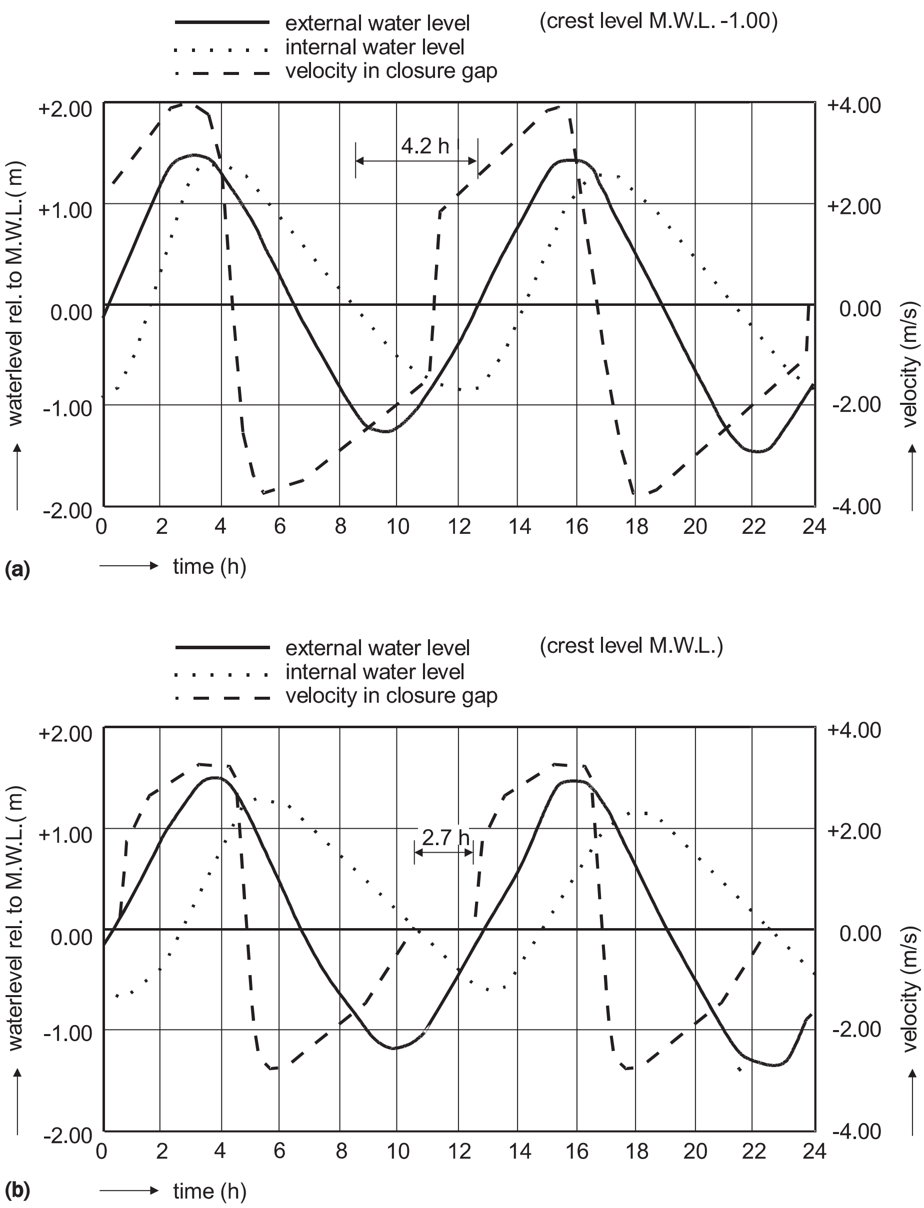


Figure 7.15 *Water levels at both sides of closure bund and current velocities across the sill for two different crest levels of sill during an average tide*

7.2.7 Construction issues that influence design

As discussed in Section 7.2.4, horizontal or vertical closure methods may be used. Hydraulically, vertical closure is preferred because the flow pattern is less turbulent, so the required stone size is smaller and scour is less. Vertical closures can be achieved by:

- **using dumping vessels, for example side stone-dumping vessels**: this method is only possible in sufficiently deep water, eg approximately 4 m

- **using a cableway with cable cars**: this method is only possible for relatively large closures because of the high overhead costs
- **using a bridge and dumping the stones directly from trucks on the bridge**: this method is only possible for relatively shallow closures with a narrow closure gap.

All three methods are relatively expensive, so it is often decided to perform a horizontal closure. Horizontal closures can be executed using dumper trucks only. This means that no special equipment or skills are needed. The disadvantage is that usually larger stones are needed, the scouring processes are more severe and larger losses of stone have to be anticipated.

7.3 RIVER CLOSURES

7.3.1 Purpose of river closures

The expression *river closure* is misleading because a river cannot be closed completely or even temporarily. A river is a vital part of the drainage system of a river basin and it cannot be *closed* unless the river discharge is:

- diverted through a tunnel and subsequently discharged further downstream into the same river or into a different river basin
- stored temporarily in a reservoir and subsequently discharged in a controlled manner by means of spillway, powerhouse or irrigation inlet.

It follows from the above that river closures are required:

- to close one or more channels of a river as part of river control works or to create temporary river diversions by means of cofferdams, to establish a safe and dry environment for the construction of the permanent reservoir dam and its ancillary works (see Figure 7.16)
- as part of the closure of diversion passages through an unfinished reservoir dam at the time of its completion or as part of the separate diversion works. This is generally called *closure of diversion works*
- to establish diversion dams that, by set-up of the water level against the dam, will divert water in all seasons. Diversion may be to an irrigation intake or to a trans-basin canal or tunnel. Such a diversion dam can be overtopped
- to create a more or less permanently overtopped weir, without gates, which has a function for river navigation and/or run-of-river hydropower generation.

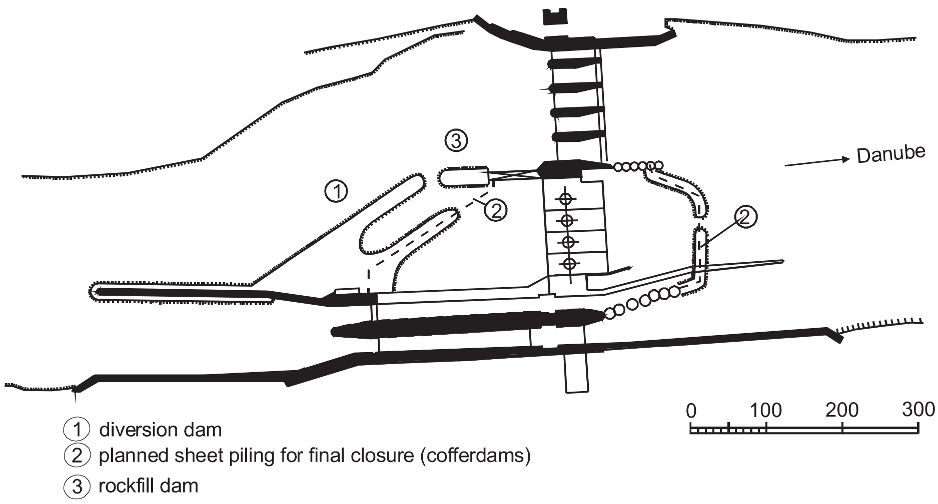


Figure 7.16 *River diversion for the final stage of construction of the Aschach hydropower plant in the River Danube, Austria (dimensions in m)*

7.3.2 Typical characteristics of partial river closures and cofferdams

A partial river closure might be required as part of river control works for navigation.

The diversion of a river during dam construction involves the design and construction of river closures for areas surrounded by a cofferdam. This creates working areas free from water and safe from floods inside which the permanent works can be built in the dry. Such river control works should be included within the overall project design, since the solution adopted will have a major impact on the overall design, construction cost and construction programme of the closure works, ie the reservoir dam and its ancillaries. Generally, such river control works consist of a series of cofferdams made, and/or extended or removed, during subsequent construction stages and related diversion works, eg tunnels, channels, culverts and openings in the partly completed dam.

A typical example of such river control works was those carried out from 1967 to 1968 at Bajibo Rapids, 55 km downstream of Kainji Dam, Nigeria, for the improvement of navigation (see Figure 7.31). The work comprised two main parts: (1) complete closure of one river channel and (2) construction of a weir in another channel. These improvements made the third remaining river channel navigable throughout the year. Figures 7.17 and 7.18 show works for the Kainji stage II and III diversions.

Figure 7.17 *Diversion stage II of the River Niger at Kainji Dam will take place through unfinished spillway structures (courtesy J van Duivendijk)*

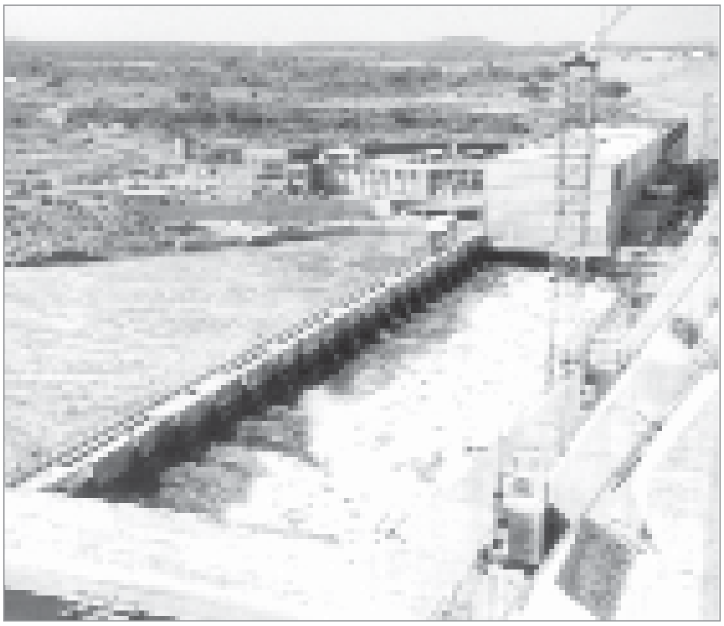


Figure 7.18 *Diversion stage III of River Niger at Kainji Dam takes place through six skeleton units of the powerhouse (courtesy J van Duivendijk)*

The implementation of river control works is always a critical operation because construction can only take place in the limited periods when the river is low and carries only low flows. Delay, failure or less successful construction can be very costly and is nearly always detrimental to the overall planning of the project. This is especially the case for rivers with a medium or high discharge or for rivers subject to sudden and significant floods. Further consideration of combined design for diversion and permanent works is beyond the scope of this manual and readers are referred to ICOLD (1986).

A diversion scheme can be single-stage or multi-stage. The general sequence of works is as follows.

1 Build a small cofferdam to allow for point 2 below (if required).

2 Build diversion tunnels, culverts, channels and control works.

3 Build cofferdam(s) across the river channel during the low-flow season, thus forcing the river to flow through diversion passage(s).

4 Build permanent works inside the area protected by a cofferdam, including outlet works.

Plus for multi-stage only:

5 Demolish cofferdam(s) and let river flow through outlets in the reservoir dam (see Figure 7.18).

6 Build second-stage cofferdam(s).

7 Build permanent works inside the area protected by the second stage cofferdam(s).

8 Close outlet works, or diversion passages, and start impounding water in the reservoir, ie the river closure has been achieved.

7.3.3 Plan layout and overall concept selection

As mentioned in Table 7.1, one of the basic differences between river closures and estuary closures is the nature of the subsoil. In most cases closures of rivers are made on rocky river beds and between shorelines. While the quality of such rock is often too poor for permanent works, it may be good enough to form the foundation, and possibly the construction material, for a closure.

As with estuary closures, a distinction should be made between the actual river closure, ie the *stoppage of flow* through a certain cross-section, and the subsequent dam constructed along the axis of the river closure or adjacent to it.

The closure can be either part of a reservoir dam or part of river regulation works or bridgeworks to close off only a specific river channel. Both cases result in a permanent structure. However, if the river closure is constructed as the initial part of a cofferdam, the situation differs in that:

- the structure is temporary in nature so failure probabilities can be higher and design criteria and construction specifications can be less stringent
- on completion of the permanent works, the structure must either be removed or left in place, for example when it is submerged in the reservoir behind the dam.

River closures rarely have a single objective. Their location, layout and boundary conditions are highly dependent on the nature of the permanent works, their planning (including the diversion scheme) and the construction schedule. It is therefore impossible to discuss the layout and selection of designs for river closures without considering river regulation and control. As the latter is not the purpose of this manual, the list below is included for ease of reference only. ICOLD (1986) described the factors listed as the “current trends in river control facilities”:

- it is preferable to build complex structures such as power plant, spillways and guide bunds on the banks of the river or at a broader site in the river valley, thus avoiding the construction of expensive cofferdams
- there is a tendency towards reduction of the diverted maximum flows and sometimes to accept overtopping of cofferdams during construction. The latter option may be particularly attractive if diversion of extreme floods through tunnels is expensive
- over the past 25 years river closures have been constructed on the natural river bed as vertical closures. Sometimes two parallel embankments to limit the individual differential heads have been constructed simultaneously
- model tests are extremely useful tools for the analysis of most problems related to river control and especially diversion structures, including interaction with the river system and closures on the natural river bed. However, it is essential that appropriate parameters, including the boundary conditions, density and shape of materials, and the relationship between water levels and river flows, are correctly represented. It is also important to note that certain problems are difficult to reproduce in model tests. These include seepage through embankments, vibration of steel units or thin concrete slabs, internal stresses within materials, junctions with the banks, and the ultimate fracture of large blocks. For these reasons, model tests may not show possible failure accurately, particularly for overtopping rockfill or earthfill cofferdams.

7.3.4 General considerations for cross-section design

River closures made of stone are usually covered by earthfill layers. These embankments may function as an access ramp to a bridge, a roadway to an island or as a reservoir dam. After it has fulfilled its function, the actual closure bund is covered, so no specific functional requirements can be listed for these bunds.

The same applies to cofferdams. In most cases these are temporary structures, built by a contractor to his own design. Their function is purely utilitarian and the kind of general functional requirements, as listed in Section 8.1.2.3 for river training, will not apply. A cofferdam can also form part of permanent works, in which case a few general functional requirements may apply, but usually the structure is covered by the permanent structure (see Figure 7.19).

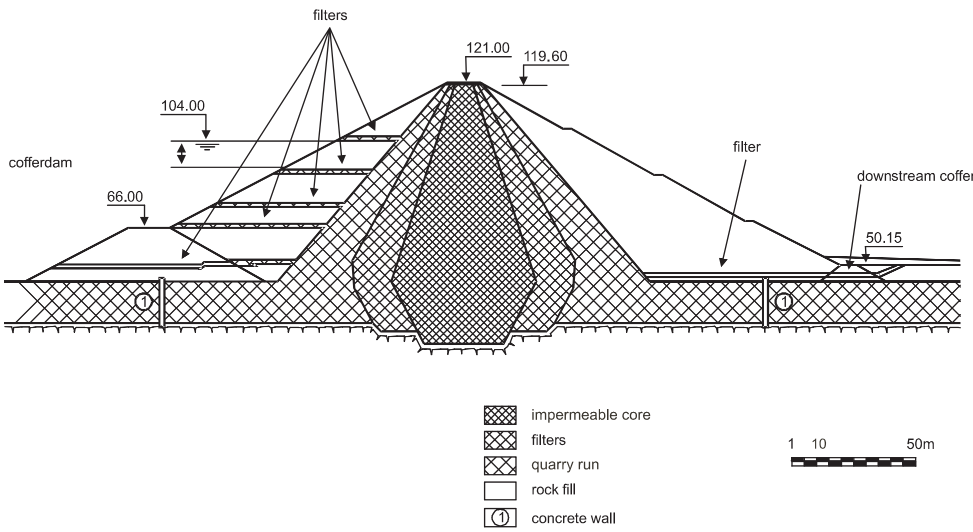


Figure 7.19 *Cross-section of a rockfill dam in Spain showing the incorporated cofferdams (dimensions in m)*

When a watertight core is installed inside a rockfill closure dam, special attention should be paid to the connection between the core and the subsoil. The occurrence of piping needs to be prevented (see Section 5.4.3.6).

The dam cross-section is determined by hydraulic and geotechnical boundary conditions, the availability and supply of materials and by construction considerations, including local experience of similar structures. These site-specific requirements are discussed below.

Hydraulic and geotechnical boundary conditions

Chapter 4 provides definitions of hydraulic boundary conditions in terms of (i) winds and waves, (ii) water depths as a consequence of discharge and scour, (iii) currents and water levels as a function of discharge and local bathymetric conditions, (iv) sediment processes and (v) seismic activity, together with geotechnical boundary conditions.

Especially important for river closures are the seasonal variations in discharge and water level as normally presented in discharge and stage hydrographs (see Section 4.3.3). Several years' discharge and water level data will enable corresponding exceedance curves to be compiled (see Figure 7.20), which are indispensable for the calculation of stone size and crest level in relation to time planning of the construction and the available construction window.

In addition to the hydrographs, the main hydraulic boundary condition is the differential head across the closure as it gradually develops over the course of the closure operation as a function of (i) flow distribution over the closure gap and diversion passage and (ii) river flow conditions upstream and downstream of the closure site. For overtopping cofferdams the distribution of river flow over the cofferdam crest and diversion passage (eg a tunnel) for flood waves of different size and frequency is of interest. In this case, *frequency* relates not only to the frequency of maximum discharge, but also to the frequency of specific durations and volumes of flood wave (see Section 4.3.3).

In general, wind waves are not a significant boundary condition for river closures. The same applies to changes in water levels and currents induced by navigation. Few closures have been attempted in alluvial rivers and therefore river morphology is usually less significant for the design of river closures (see, however, Section 7.3.5 for special aspects). For geotechnical boundary conditions, see Sections 4.4 and 5.4 for general and specific mechanisms respectively.

Whether or not failure mechanisms can develop largely depends on the grading and characteristics of the material used for the closure. The migration of material as a consequence of through-flow may lead to local sliding and/or settlement of the closure structure. Earthquakes may also induce settlement, but, because coarse materials are normally used in river closures, they rarely result in liquefaction.

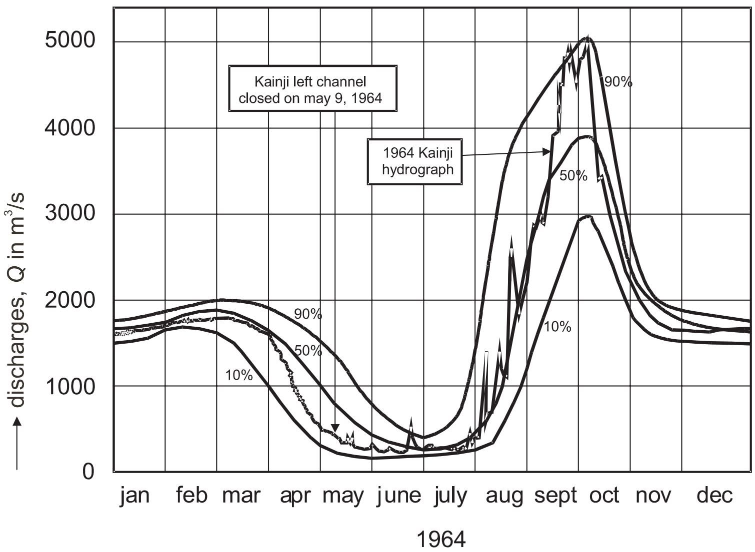


Figure 7.20 *Discharge hydrograph of River Niger at Kainji Dam site showing exceedance curves, 1964*

Materials availability

The most frequently used material for closures is quarry run (see Section 3.4.4) or selected rockfill that includes elements between 1 t and 5 t. Some quarries cannot supply stone of such large sizes, or cannot provide it in sufficient quantities, whereas other quarries can produce blocks with a mass of up to 15–20 t. If quarries at or near the site cannot supply the desired material the design of the closure will need to be matched to the quarry yield and materials available. This will mean either adapting the closure design or introducing concrete blocks with a mass betwen 5 t and 30 t. Sometimes very large blocks are linked by cables. Another option is to use smaller stones packed in gabion-type nets (see Section 3.14).

The density of material, though important, is usually a given point of departure for the design. The same applies to shape. Quarry run is relatively cheap, while large-size, sorted rockfill is costly and may necessitate the use of prefabricated concrete blocks. Grading of the material may influence the porosity of the closure during and immediately after a closure, but usually this factor is less important.

River closures differ from estuary closures because of:

- the sediment transport of the river, which tends to clog up the voids
- the unchanging direction of the current.

This current direction and the relatively high differential head, compared with that of estuary closures, also makes the sealing of river closures quite easy. By dumping coarse materials, ie gravel and sand, first and then possibly clay along the steep upstream slope of the closure bund it is possible to arrive at nearly perfect sealing within a few days.

Materials supply

The considerations presented in Section 8.2.8 for river training works also apply here. Generally, however, on a reservoir dam site in a hilly or mountainous area where a cofferdam is a relatively small part of the works to be constructed, the supply of materials is not a problem.

Construction considerations

The design and construction of river closures is normally in the hands of one party, ie the contractor. The nature of river closures means that the design is left to the contractor, who takes into account all the construction considerations. For a more detailed discussion, see below in Section 7.3.5.

7.3.5 Structure-specific design aspects

As follows from Section 7.3.1 a distinction can be made between:

- design and construction of river closure bunds
- design of cofferdams, including those that can be overtopped during construction.

Design aspects differ for the two types of closure and so are discussed separately below.

7.3.5.1 River closure bunds

Even when a theoretical analysis of river closures has been made and many model tests have been carried out (see Section 5.2.3.5), it is still difficult to determine the size and characteristics of closure materials. Evidence has shown (ICOLD, 1986) that the following aspects are important when designing and performing a river closure:

- the **river discharge** during closure
- the **speed of closure**, ie dumping capacity of materials in tonnes per hour (t/h)
- the **density, grading** and **maximum size** of available closure material
- the closure or **dumping method**, ie horizontal or vertical
- the maximum **differential head** when current velocities through the gap become critical
- whether or not **loss of material** is acceptable
- the **thickness of soft or small material**, eg peat, sand, gravel, covering natural stones in the river bed.

River discharge

Preferably, a river closure should be made during the annual period of low discharge. Obviously, this is a relative measure. Even with low discharges one may have a high head difference over the closure dam. In the case of the River Paraná (Argentina, see figure 7.21) the design team had to cope with a discharge of 1200 m³/s. It was for example important to check the probability of exceedance of this discharge, its consequences and/or the measures to be taken in those circumstances. By constructing two dams simultaneously, the head difference for each dam was decreased, making the closure feasible. The "upstream cofferdam" - built by means of the end-dumping method (see Figure 7.21 top part) - would become part of the eventual dam, whereas further downstream a temporay dam was built using the vertical closure method, with the aid of a service bridge.

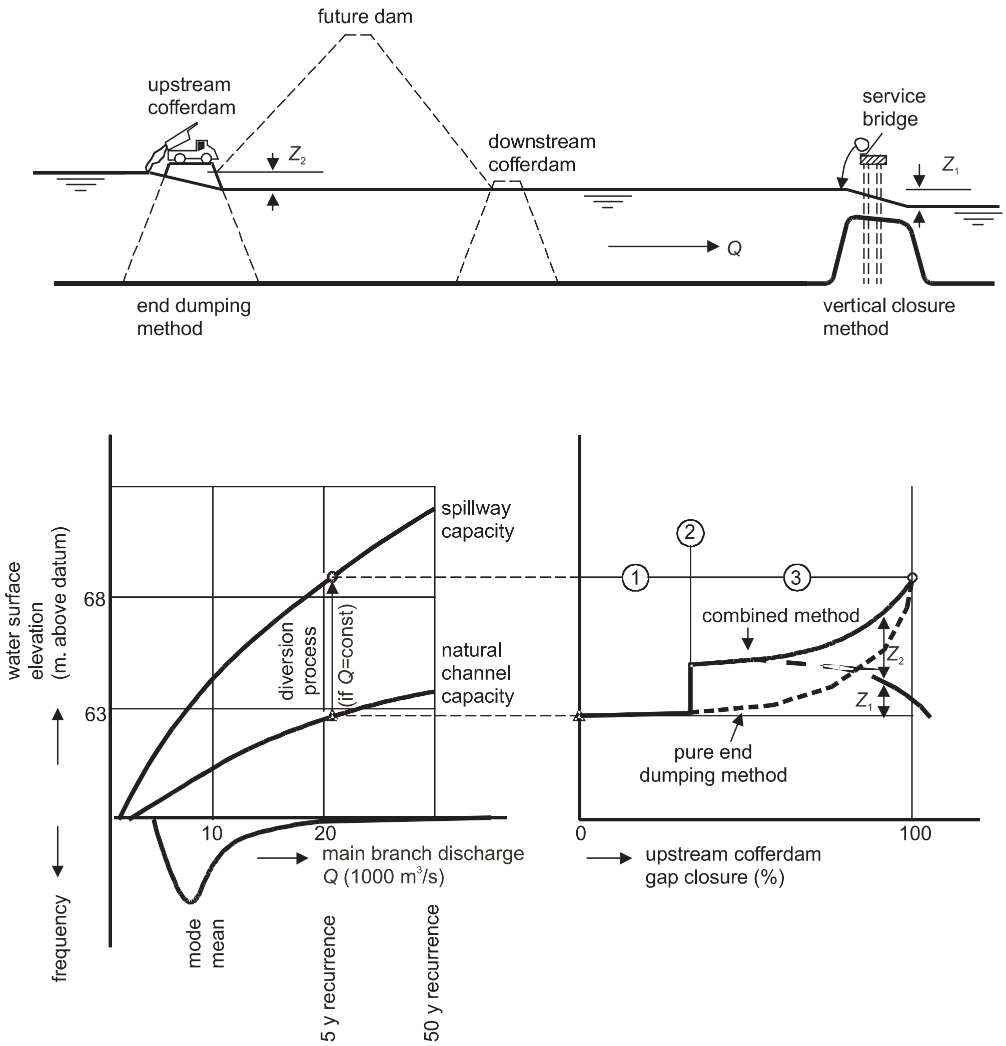


Figure 7.21 *Yacycreta diversion scheme, River Paraná, Argentina*

Speed of closure

Speed of closure depends on:

- the capacity of transport equipment and access facilities. End-dumping can reach 1000 t/h of materials, requiring the use of large rear dumpers, which in turn demand wide (15 m) haulage roads. Land-based operations, ie horizontal closure, are safer than waterborne operations because a hold-up in the closure process is less likely to occur in the former case if many items of equipment are used. If a closure requires 50 trucks and one breaks down, closing capacity is still running at 98 per cent, whereas if the closing is effected by two side stone-dumping vessels and one breaks down, capacity decreases immediately to only 50 per cent
- the experience of the project contractors. Those engaged on dam projects are often not experienced in waterborne operations
- the availability of waterborne equipment
- the time during which parts of the unfinished/incomplete closure are exposed to the current. In a horizontal closure only the dam head is exposed and as long as dumping is done more or less continuously stones that are at the end of the structure may remain in place because they are only exposed to the current for 5–10 minutes.

Dumping method

End dumping method or horizontal closure method is most efficient for river closing works, using heavy earthmoving plant (up to 65 t per payload). In the past also vertical or frontal dumping was used, because large dump trucks were not available. The advantages of the vertical method for estuary closure are less obvious in river closures.

Closure material

For comments on materials used for closures refer to Section 7.2.3.

Differential head

There is a close relationship between differential head across the closure gap, average velocities in the closure gap, loss of material and size of material. During a vertical closure, four subsequent flow regimes are distinguished for tailwater depth, h_b (m): low dam flow, intermediate dam flow, high dam flow and through-flow (see Figure 5.20 in Section 5.1.2.3). In a horizontal closure, the three successive flow regimes distinguished are: *subcritical*, *supercritical* and *through-flow* (for definitions see Figures 5.22–5.24 and the Equations 5.92–5.94 in Section 5.1.2.3 under "**Horizontal closure method**").

Most river closures are **horizontal closures**. During the first stage of subcritical flow the differential head, h_1 - h_3 (m), where h_1 is the upstream and h_3 the downstream water level (m), is still moderate and velocities close to the stones dumped at the sides of the gap are often lower than the average velocity in the gap. In practice, as long as $h_1 - h_3 < 2$ m (approximately), this may imply that quarry run materials with, eg $M_{50} < 0.5$ t, are effective in deep water.

During the second stage, supercritical flow develops and heavier blocks should be used. This situation can be aggravated by smooth bedrock on which the dumped blocks slide and are displaced by the current. During this stage, differential heads, h_1 - h_3, increase to 3–4 m and the designer should either use relatively small units, up to 8 t, and accept large losses of material, or use very large units, 20–50 t units of rock or concrete, without loss of material. Closure is easier if gabions are used.

Alluvial river bed

Few river closures have been attempted in alluvial rivers. If the layer of alluvium is thin, up to 3 m thick, designers tend to leave it unprotected and accept scour during closure. Bed protection, if required, will usually consist of a 1–2 m-thick layer of quarry run. This finer part of the quarry run (eg 0–500 kg) will be washed away. However it is cheaper to use raw quarry run and to allow for some amount of material loss rather than process the material prior to placement. If a closure has to be performed on a wide river flowing in an alluvial bed, and armourstone is not available at or near to the site it may be possible to use a sand closure (as discussed for estuary closures in Section 7.2.2). In addition to the extent of closure achieved, ie percentage of wet cross-section of river closed, whether this method of closure is technically feasible is determined by the current velocities, dredger(s) capacity and median sieve size of the sand particles, D_{50}.

7.3.5.2 Cofferdams

Although it appears logical to incorporate closure embankments in the profile of the main cofferdams, it is not always the best solution.

The purpose of a cofferdam is essentially the same as a reservoir dam or estuary dam, ie to retain water. There are some basic differences, however:

- construction takes place in a shorter time and in a given season
- there is a higher risk of overtopping
- construction or operation occurs in rapid flow.

The regulations governing construction of cofferdams are usually less strict than those applied to permanent dams because of their temporary nature – for example, greater settlements and seepage may be permitted than would be allowed for a permanent dam.

A cofferdam can be made of any suitable material, but the discussion in this manual is limited to the use of rockfill.

Cofferdams are usually built in a short time and partly under water, so it is not possible to use a design similar to that for a permanent rockfill closure dam incorporating a well-compacted clay core or an upstream membrane. The river closure rock bund should be combined with an upstream section, placed after closure, consisting of a clay core and/or a sheet-pile curtain in a sand body.

If rockfill dams are designed to be overtopped the following aspects should be considered:

- for preference, overtopping should occur in the central part, away from vulnerable banks
- floating matter may damage the crest and downstream slope
- the crest and downstream slopes should be protected by means of selected armourstone, gabions, prefabricated concrete blocks, a concrete lining or reinforced rockfill.

Selected rockfill and concrete blocks

Various formulae are available (see Sections 5.1.2.3 and 5.2.3.5) to select and design an armourstone protection for the crest and downstream slope. However, no formula can accurately represent the combined effects of local turbulence, air entrainment and block packing, although the empirical formulae, presented in Section 5.2.3.5, may give a satisfactory result and should be used. On relatively long steep slopes, say 10 m high, with $\cot\alpha = 1.3$, low specific discharges, say up to $q = 1$ m³/s per m, should be allowed if large stones or concrete blocks are used. However, the results of doing this are not always sufficient and in such cases it is more effective to create a mild slope, ie $\cot\alpha = 2$ to 3. This is an economical solution for low heads. At Cabora Bassa, the differential head was only 3–4 m and this enabled q = 50 and 74 m³/s per metre width to be passed over the upstream and downstream cofferdams (D), respectively (see Figure 7.22).

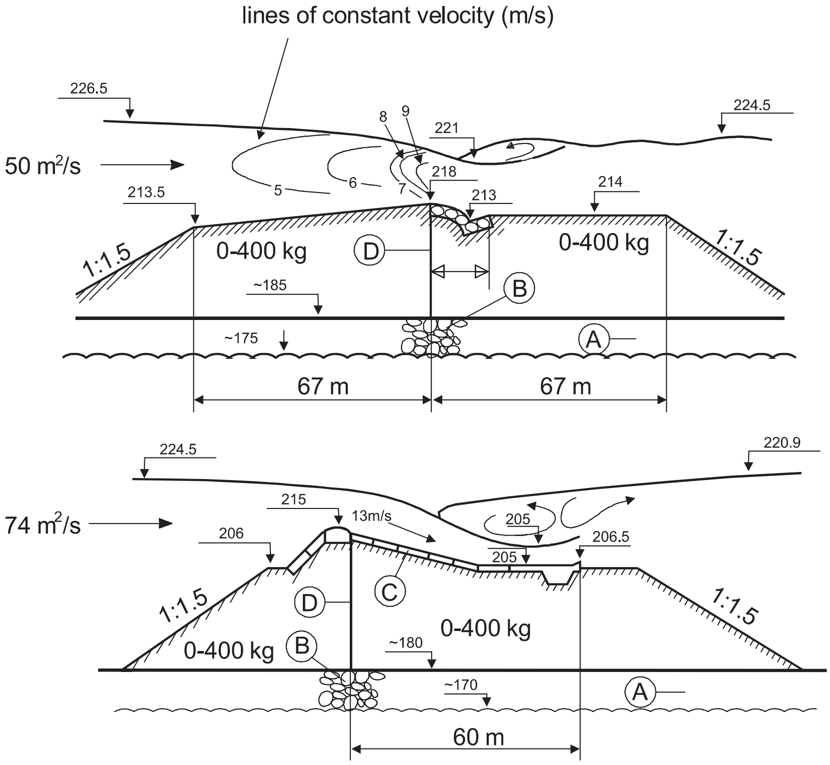


Figure 7.22 *Cross-section over upstream and downstream cofferdams (D) at Cabora Bassa, Mozambique (dimensions in m) – units per metre width*

Reinforced rockfill

Reinforced rockfill is rockfill on the downstream slope that has been firmly fixed to the interior of the stone mass by means of a steel mesh on top and anchoring bars (see Figure 7.23).

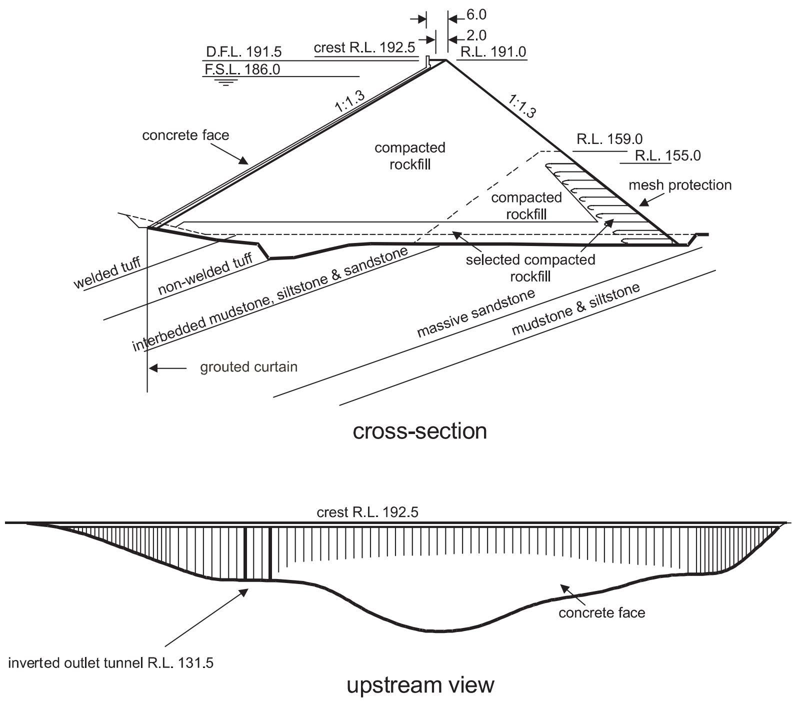


Figure 7.23 *Mesh protection of rockfill on cofferdam that was later incorporated into main dam (dimensions in m); RL = reference level*

It has proven difficult to calculate the size of the mesh and anchoring bars that are required. In Australia, empirical rules based on experience with 22 overtopped structures have been developed. In most cases a steep slope, $\cot\alpha$ = 1.5 to 1.3, has been adopted with about 50 kg/m² of steel (see also Stephenson (1979) and ICOLD (1986)).

Specific flows of q = 10 to 15 m²/s for overall differential heads up to 20 m and discharge depths of up to 3 m are possible. This also implies that higher specific flows are feasible for lower heads and lower specific flows for higher heads. Failure can occur at velocities of U = 15 m/s and, with overflows of long duration, at U = 10 m/s.

Mesh protection is employed on cofferdams to reduce costs and to reduce the magnitude of diversion works. Reduction of the magnitude of diversion works can be an important factor when the working season is short.

One of the parameters in the design of mesh protection is the mesh protection level, which indicates the upper level at which the mesh is terminated on the downstream slope of the dam. This level coincides with a 1 in n years flood that can still pass through the diversion works without overtopping the dam under construction. Termination of mesh protection at a lower level would imply that any flood in excess of 1 in n years flood would have to spill over a partly unprotected downstream slope. The major considerations in deciding on the mesh protection level are:

- reliability of hydrological data
- the estimated time and the season of construction of the embankment from the level at which the mesh is terminated to a higher level and during which there is a very low probability of the structure being overtopped
- the probability of overtopping during construction of the dam above the mesh protection level
- the incremental cost of mesh protection
- the cost of damage and delay if overtopping occurs during building above the mesh protection level
- the reliability of construction programmes for construction above the mesh level during the dry season.

For further details on design, model tests and stability calculations of reinforced rockfill, see ICOLD (1993a).

7.3.6 Design features of horizontal river closures

Horizontal methods are most frequently used for river closures. During the first stage (I) of **subcritical flow** (see Figure 7.24a), the upstream water depth, h_1 (m), is large in comparison with the differential head, $h_1 - h_3$ (m), and quarry run material should be used – see Section 5.2.3.5 for design guidance. During the second stage (II) the differential head becomes so large, up to 3 or 4 m, that **supercritical flow** develops (see Figure 7.24b). The relatively light material applied in the first stage is now insufficient and much heavier elements must be used. The relationship between the overall discharge, Q, as a function of upstream water depth, h_1 , and control, h_{con} , or tailwater depth, h_3 (m), mean gap width, b (m), and discharge coefficients, μ (-), are given in Equations 5.92 and 5.93 in Section 5.1.2.3.

The hydraulic parameters for an example horizontal river closure are discussed in Box 7.3, together with the evaluation of the required armourstone size required for stability of the advancing dam face.

Box 7.3 *Hydraulic parameters for horizontal river closure*

The data used for this example, h_1, h_3, Q, and the results are summarised in Table 7.3. General data used in this example are (i) a slope of $\cot\alpha$ = 1.25 and (ii) a relative buoyant density of the stones of Δ = 1.65.

This example shows the difference between the first and second stage with regard to the required armourstone sizes. For stability in accordance with the Pilarczyk formula (Equation 5.219 and Table 5.53) the following applies:

- characteristic size of stones: D_{n50} for rip-rap and armourstone
- the velocity or depth factor: k_h = 0.6 (for a not fully developed velocity profile – Equation 5.222)
- the slope factor: k_{sl} = 1 (assumed because in the case of front end dumping $\beta = \phi$, the angle of repose of the stones; hence Equation 5.116 cannot be used)
- the stability factor: ϕ_{sc} = 1.5 (exposed edges during and after placement)
- the turbulence factor: k_t^2 = 1.5 (the larger stones will induce strong turbulence).

The results are listed in Table 7.3 (definitions, see Figures 5.23 and 5.24).

Table 7.3 *Boundary conditions and hydraulic response for Stages I and II of a horizontal closure*

				Stage of closure	
Parameter	Notation	Equation	Figure	I	II
Upstream water depth (m)	h_1		7.24	7.0	8.0
Downstream water depth (m)	h_3		7.24	6.6	5.0
Discharge through gap (m³/s)	Q			450	400
Gap width (bottom) (m)	b_t		5.24	15.0	5.0
Discharge coefficient (-)	μ	5.92	5.29	0.86	0.85
Gap width factor (-), = $b_t/(2h_1 \cot\alpha)$	p	5.93			0.25
Control depth (m)	h_{con}	5.93			6.1
Average gap width (m), = $b_t + h_2 \cot\alpha$	b	5.92	5.24	28.3	12.6
Velocity in gap (m/s)	U_g	5.94		2.4	5.2
Shear parameter (Shields number)	ψ_{cr}	5.105; 5.106	5.32	0.03	0.03
Stone size (m)	D_{n50}	5.219		0.27	1.26
Median stone mass (kg)	M_{50}	3.6		52	5300

Note: Stage I is the situation with subcritical flow and Stage II is the situation with supercritical flow (see Figure 7.24).

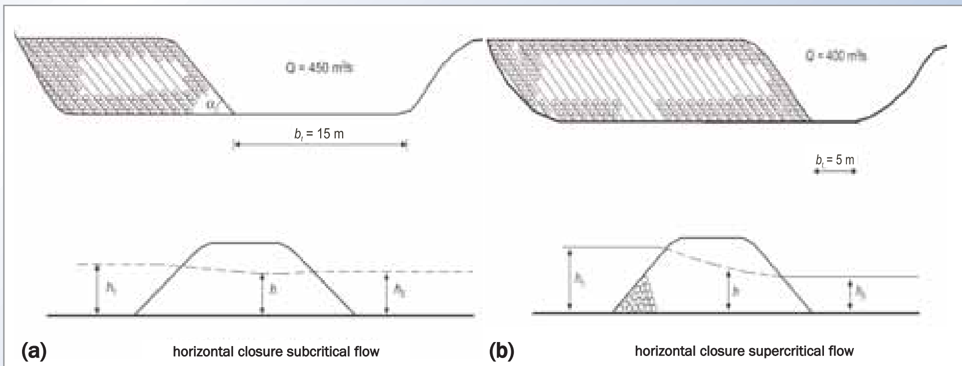


Figure 7.24 *Example of horizontal closure during (a) subcritical flow and (b) supercritical flow*

It should be emphasised that the armourstone grading used depends on the instantaneous differential head. This head is usually less than the final head over the completed watertight closure, since, at the same river discharge, the greater initial porosity of the closure bund as well as 3D effects of the flow tend to decrease the head, for example by 5–10 per cent compared with the final situation.

Because of the cost of producing, transporting and dumping very large armourstone or concrete units the overall differential head should be split over two or even three embankments. This is recommended because in most cases it is necessary to construct two parallel cofferdams across the river to create the desired work site. These multiple closures are used for final differential heads over 2–3 m. If two embankments are constructed the differential head at each embankment may reach 60 per cent of the head at the critical time for a single embankment. In wide rivers the embankment should be split into two parts only at the last stage of closure. Care should be taken to leave sufficient distance, ie 100 m in most cases, between the two embankments to allow for decreases in current velocity and turbulence. Progress of the embankments should be co-ordinated to avoid uneven distribution of the differential head over the embankments.

7.3.7 Design features of vertical river closures

When a vertical closure is made it is possible to reduce maximum current velocities by introducing a specific geometry for successive dam stages. Such dam geometries are, for example, broad-crested, sharp-crested and multi-crested (see Table 5.57 for design values of stability numbers for these types of dam geometries). As demonstrated in Figure 5.25 in Box 5.8 (in Section 5.1.2.3), a vertical closure will always result in less extreme current velocities than those experienced in a horizontal closure of the same river cross-section. However, restricted availability of heavy closure materials might still make a horizontal closure attractive because in the latter case part of the closure can always be constructed from gravel or similar materials or even with sand. Moreover, a combined closure may be designed such that it matches the stock of available closure materials with the maximum current velocities appearing in certain stages.

As previously mentioned and shown for vertical river closures in Figure 5.20 in Section 5.1.2.3 four typical flow regimes can be distinguished.

- Stage I: low dam flow, with $h_b/(\Delta D_{n50}) \geq 4$, where h_b = tailwater depth relative to dam crest level (m)
- Stage II: intermediate dam flow, with $-1 < h_b/(\Delta D_{n50}) < 4$
- Stage III: high dam flow, with $h_b/(\Delta D_{n50}) < -1$ and $H > 0$, where H = upstream water level relative to dam crest level (m)
- Stage IV: through-flow, with $H < 0$

The four stages of closure (I to IV) are described in Box 7.4 for one typical example comparable with the example discussed for a horizontal closure in Box 7.3 in Section 7.3.6. Subsequently, the required stone sizes are discussed in Box 7.5.

Box 7.4 *Overview of the various stages of a vertical closure*

Stages I and II (low and intermediate dam flow)

For Stage I, the data used in this example are the same as that for the horizontal Stage I (see Table 7.3). The characteristics of the hydraulic boundary conditions and responses for Stages I and II are given in Table 7.4, containing also the results for the major hydraulic response parameters, which for Stages I and II use the same equations (numbers are listed). The values for the discharge coefficient, μ, are taken from Table 5.15 (Section 5.1.2.3). In Stage II, h_b > 2/3 H, the flow is still subcritical (see Equation 5.82). The structural responses and the consequential stone sizes are discussed in Box 7.5.

Table 7.4 *Comparison of the first two stages of a vertical closure*

Parameter	Notation		Equation	I	II
Upstream water depth (m)	h_1			7.0	7.3
Downstream water depth (m)	h_3			6.6	6.3
Discharge through gap (m³/s)	Q			450	425
Height of sill/dam (m)	d	$= h_3 - h_b$		2.95	3.9
Width of gap (m)	b			40	40
Discharge coefficient	μ			1.1	1
Specific discharge (m²/s)	q	$= Q/b$		11.3	10.6
Head difference (m)	$H - h_b$	$= h_1 - h_3$		0.40	1.0
Upstream water depth (m)	H	$= h_1 - d$		4.04	3.4
Tailwater depth (m)	h_b		5.81–5.83	3.65	2.4
Velocity on crest (m/s)	U_0		5.90; 5.91	3.08	4.4

Stage III (high dam flow)

Assume that the dam has reached a level of d = 7 m, while the upstream and tail water depths are H = 1 and h_b = -1 m respectively. Since h_b < 2/3 H now holds, the flow is supercritical, so the **overtopping** discharge, q_{ov} (m³/s per m), and velocity, U_0 (m/s), are calculated according to Equations 5.85 and 5.91 respectively, while μ = 1 (see Table 5.15). For the overtopping discharge over the crest is thus found: q_{ov} = 1.7 m²/s and for the velocity: U_0 = 2.6 m/s.

In addition to overflow, in this situation **through-flow** will also occur. The basic approach by Darcy (see Equation 5.288) can in many cases not be applied, because of the occurrence of turbulent flow. The appropriate approach is to use the Forchheimer equations (see Equations 5.289 to 5.291). With the above data the gradient, i (-), can be approximated by $(h_1 - h_3)/L_s$ or $i \cong$ ¼. With a stone size D_{n50} = 1 m (see Box 7.5), a bulk porosity n_v = 0.4, the kinematic viscosity of water, $\nu_w = 10^{-6}$ m²/s and the coefficients α_{For} = 1000 (-) and β_{For} = 1.0 (-), the velocity through the voids can be determined: U_v = 0.5 m/s. With the general expression for the specific discharge through the dam, this is: $q_{tf} = U_v n_v d$ (m²/s), $\cong$ 1.3 m²/s. The overall discharge $(q_{ov} + q_{tf})$ is: $q \cong$ 3 m²/s.

Use can also be made of Equations 5.86 to 5.88. Taking a crest width B = 5 m and dam height of d = 6 m (d also determines the water level in the dam at the downstream side, see Figure 5.22), the through-flow discharge, q_{tf} , should be calculated according to Equation 5.86. Using h_1 = 8 m and h_3 = 6 m and C = 0.5 (see Table 5.15) and assuming a porosity n_v = 0.4 and a stone size of D_{n50} = 1 m (see below), for the length of through-flow and the resistance factor are found L = 8.3 m and C' = 0.8 × 10^{-3} respectively. Substitution in Equation 5.86 results in q_{tf} = 2.2 m²/s, which is in the same range as when using the first method; this higher value is caused by the fact that the slope is very steep. In total q = 4 m²/s.

Another alternative is to use Equation 5.76 (Martins and Escarameia, 1989): with $C_u = D_{60}/D_{10}$ assumed to be 1.5 (-), and a void ratio, e = 0.67, the velocity is: U_v = 0.9 m/s. This gives a through-flow discharge of $q_{tf} \cong$ 2 m²/s. Also this method gives slightly higher values than the first, but in the same region.

Stage IV (through-flow only)

In this situation, d = 9 m, while H = -0.7 m and h_b = -3.3 m and accordingly, h_1 = 8.3 m and h_3 = 5.7 m. Because h_1 and h_3 have changed, the gradient (i) has increased to approximately i = 1/3. As a consequence, the velocity U_v and the discharge q_{tf} increase slightly: U_v = 0.6 m/s and $q_{tf} \cong$ 1.8 m²/s.

Box 7.5 *Armourstone size calculation for a river closure*

Guidance is given in Section 5.2.3.5 on the sizing of armourstone. Instead of using Equation 5.219, the use of which was demonstrated for a horizontal closure, here the use of Figures 5.97 to 5.102 (in Section 5.2.3.5) are demonstrated for the four flow regimes. The input parameters for the calculations are as defined in Box 7.4.

Low-dam flow

For this situation of Stage I, Figure 5.99 can be applied, because in this case, $H - h_b$ = 0.4 m and since Δ = 1.65 for the rock source selected and the crest is wide. According to Table 5.57, the relative difference in head $(H - h_b)/(\Delta D_{n50})$ must be between 1.5 and 2, or D_{n50} > 0.16 m. (Note that the criterion for low-dam flow, $h/(\Delta D_{n50})$ > 4 is satisfied as $h_b/(\Delta D_{n50})$ = 13.8.)

Intermediate flow

This is the situation of Stage II for which both Figures 5.97 and 5.98 can be applied. Figure 5.97: D_{n50} = 1.0 m satisfies the criterion because $h_b/(\Delta D_{n50})$ = 1.5 m, while $H/(\Delta D_{n50})$ = 2.1, together defining a point (x, y) = (1.5, 2.1) just right of the curve, which area guarantees stability. Figure 5.98: for the same values of D_{n50} and $h_b/(\Delta D_{n50})$ but now for the non-dimensional discharge the value is $q/\sqrt{(g(\Delta D_{n50})^3)}$ = 1.6, so in this graph the point (x, y) = (1.5, 1.6) is situated at the stable side (right) of the curve.

High-dam flow

Bearing in mind the complexity of the situation described in Section 5.2.3.5, for this type of flow one may use Figure 5.100. Taking the medium estimate for q_{tf} (from Box 7.4) and the corresponding overall discharge of q = 4 m²/s, with the same stones as above, $h_b/(\Delta D_{n50})$ = -0.6 and $q/\sqrt{(g(\Delta D_{n50})^3)}$ = 0.6. Again with D_{n50} = 1 m, this defines a point (x, y) = (-0.6, 0.6) in the graph that is still stable. (Note that an extremely high value of the through-flow, if ever occurring (see Box 7.4), of q_{tf} = 7 to 8 m²/s would be close to the critical condition.) However, a dam of d = 7 m does not represent the most critical situation. The critical situation occurs at the transition from subcritical to supercritical flow, ie when $h_b = 2/3H$. This will happen when d = 4 m, H = 3.3 m and h_b = 2.2 m. Using Equations 5.91 and 5.89 in Section 5.1.2.3, the corresponding discharge and velocity are: q = 11.3 m²/s and U_0 = 4.7 m/s; because these values do not significantly exceed those found for Stage II, stability is guaranteed.

Through-flow

For the above q_{tf} = 2.2 m²/s and again using D_{n50} = 1 m, $q/\sqrt{(g(\Delta D_{n50})^3)}$ = 0.3 but now $h_3/(\Delta D_{n50})$ = 3.5 and $h_b/(\Delta D_{n50})$ = -2.0 and $H/(\Delta D_{n50})$ = -0.4. Using Figure 5.101, the point (x, y) = (3.5, 0.3) remains below the critical condition. Again, when any possible higher values of q_{tf} are considered, q_{tf} would still (just) be a stable condition. An extra check is carried out by using the H-criterion of Figure 5.102. Here the point (x, y) = (-2.0, -0.4) clearly confirms that the chosen stone size is stable.

Grading selection

For selection of the appropriate grading, in particular gradation, see Section 3.4.3.

7.3.8 Construction issues that influence design

Vertical closures are preferred for both estuarine and river closures. However for river closures, the advantages over horizontal closure are less distinct than for estuarine closures, especially where there is also a diversion. Floating equipment is rarely used in river closures because of the restricted water depth. Cable-car closures should be considered, especially when the river is between two mountains.

7.4 RESERVOIR DAMS

7.4.1 Functions of reservoirs

Reservoirs are created to store water and to release it subsequently in a controlled manner. This controlled release functions as either discharge control or water level control, or a combination of both. Flood control by way of *damping* the height of the flood wave when it passes the reservoir, and **water management** are typical examples of **discharge control**:

- **water level control** (or **stage control**) is required for **navigation** and **recreation**
- **hydropower** and **irrigation** require a combination of both types of control.

When a reservoir is created for more than one of the above, it should be referred to as a multi-purpose reservoir. Reservoir operation is vital for effective control of the release of water. Often there are conflicts of interest regarding release of water and the reservoir operator tries to find the optimum balance. For example, for agriculture a full reservoir at the start of the irrigation season is desirable, whereas for flood control it is important to have a partly empty reservoir during periods when flood waves might enter the reservoir. Where the primary use of the dam is for hydropower, the optimal solution is to have the water level as high as possible at all times.

7.4.2 Outline and planning of a reservoir dam project

A reservoir is usually constructed in the upper or middle reaches of a river, preferably, by closing off a river valley by means of a dam. The **location** (siting) of the reservoir dam is determined by the topography, hydrology, sedimentology and geology of the site. Favourable **topography** is characterised by a contour plot that permits the use of the natural side slopes of the river valley with a minimum dam length. The topography also determines the reservoir capacity by means of the capacity-stage curve and its vulnerability to evaporation losses through the area-stage curve. The **hydrology** determines how much water is available for storage in the reservoir. The **sedimentology** should indicate sediment load and its composition and whether or not sediment will be deposited in the reservoir. Finally, the geology must indicate the types of foundation that are possible for the reservoir dam and the construction materials that are available.

As well as the above considerations, the size of a reservoir is also determined by its operation.

Knowledge of the five disciplines of topography, hydrology, reservoir operation, sedimentology and engineering geology, as well as the known requirements for discharge and/or stage control, should enable the designer to outline and plan a reservoir dam project. Obviously, other considerations like cost and benefits, environmental impact of the project and socio-economics also play a role in determining the feasibility of the project and the siting and dimensions of the reservoir dam (see Chapter 2).

7.4.3 Types of dam and construction materials used

In countries such as India and China dams are often built using local stone quarried with local technology or brick masonry. In most other countries all reservoir dams are built of concrete, earthfill, rockfill or a combination of these. The various types of concrete dam are outside the scope of this manual, although they may function as regulating structures.

The majority of reservoir dams are either earthfill or rockfill, although other materials such as steel, bitumen, geotextile and concrete can have an important role in structures built as parts of these dams or for slope protection works. Types of fill dams are illustrated in Figure 7.25. Additional information can be found in Degoute (2002) and in various *Bulletins* of the International Commission On Large Dams (ICOLD).

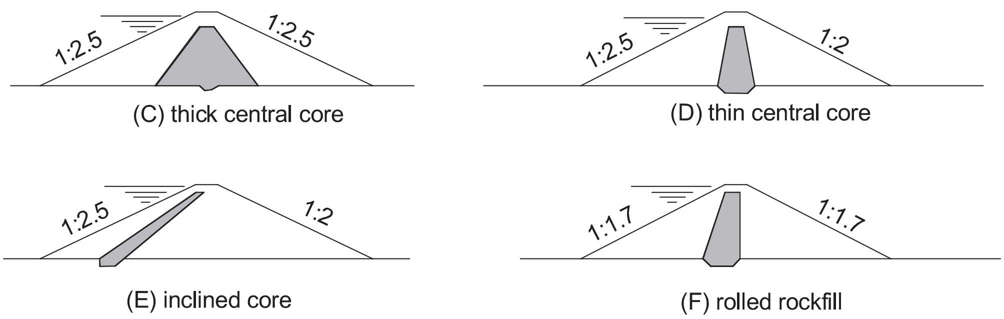
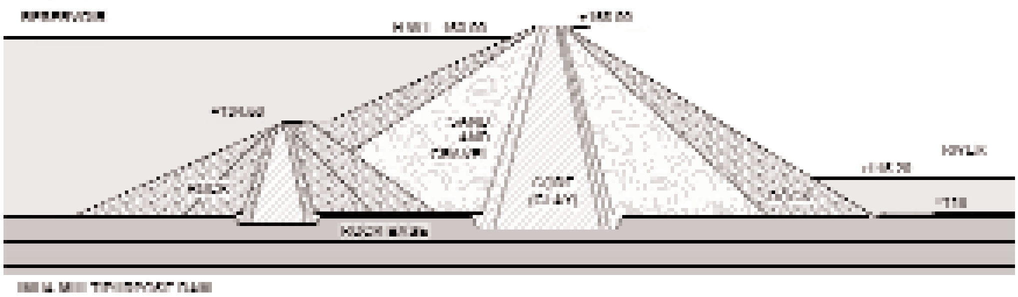


Figure 7.25 *Types of fill dam*

In many cases combination types are used, for example the Imha Dam in the River Banbyeoncheon in South Korea (1984–1993) (see also Figure 7.26).



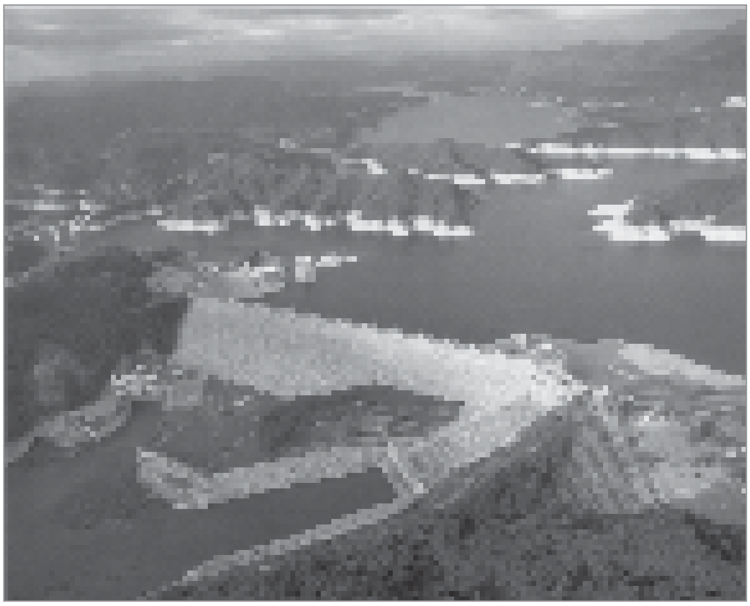


Figure 7.26 *The Imha Dam in South Korea (courtesy KOWACO)*

7.4.4 Earthfill dams

Earthfill dams can be homogeneous, zoned or homogeneous with a relatively thin core. This chapter only covers the use of rock in earthfill dams. For these armourstone is placed:

- on the upstream slope together with an underlying filter as a protection against wave and current attack (near spillways and outlets)
- on the downstream slope with an underlying filter, as a protection against rainfall runoff
- in filter drains near the downstream heel of the dam.

For the design of protective armourstone layers refer to Sections 5.2.2 and 5.2.3 (depending on the type of load). In some cases it is possible to combine the upstream slope protection with the watertight *membrane*, generally called *facing*, that functions as the *core* of the dam.

Before designing the armourstone layers, the designer should determine the extent and magnitude of hydraulic loads. Seasonal climatic variations such as wind speed and direction, reservoir operation, spillway and powerhouse operations can all cause considerable variation in these hydraulic loads over the course of the year and at different locations. 3D effects should also be considered. For a discussion of the geotechnical response to hydraulic and other loads refer to Sections 5.4.3 and 5.4.5.

There are some significant differences for reservoir dams when compared with coastal situations. For rockfill dam design there are no wave conditions or records available, simply because the reservoir still has to be made. The only option is to calculate wave heights from wind speeds (see Section 4.2.1). Often the site is located in a remote area. For example, such projects have been undertaken in the north Rocky Mountains, Canada, and in the middle of desert in Argentina, where limited wind records are available. In mountainous areas, wind velocities may change completely within a few kilometres, because of the influence of the mountains on wind patterns. For rockfill dams, water level variation may be very large, up to 20–30 m, and is often well known over the year. Rockfill dams may be very high, for example up to 200 m
(eg WAC Bennet Dam, Canada), and require a large extent of slope protection to protect against wave attack. The WAC Bennet Dam has a slope of 1:2 and 30 m water level fluctuation, resulting in a slope length of around 80 m that is protected against waves.

When designing a rip-rap protection layer on a dam slope the human factor should be considered.

1 Loose pieces of stone can be very attractive to the local population. The slope protection can be made vandal-proof by means of grouting or by using concrete blocks. In remote areas the available rock may not be sufficient. Concrete units or larger stones from elsewhere are likely to increase the dam cost significantly. In remote areas vandalism is less likely to be a problem because of the absence of people.

2 For reservoirs intended to have a recreational purpose a clear distinction should be made between the dam and the slope protection around the reservoir. Rip-rap slope protection around the reservoir may be difficult to access for recreational users. The designer should question whether or not the dam slope should be made accessible to the general public (see Section 2.6). Also a clear decision should be made regarding the accessibility of the crest to the public – for example, should there be a public road on the crest or not? Where there is a public road, it might be difficult to avoid public access to the dam slopes.

For more information refer to ICOLD (1993b).

7.4.5 Rockfill dams

Dams should be constructed in rockfill where earthfill is not available in sufficient quantities or does not have the required quality. In rockfill dams the slopes can be steep (V/H = 1:1.5 to 1:1.75) and the dam should be designed for overtopping, ie to function as a spillway. A rockfill dam with a water-retaining function should have a wide clay core inside the dam or a thin membrane on the upstream side. A membrane is sometimes also placed inside the dam. It may consist of concrete, bitumen, steel or geosynthetics. When concrete, bitumen or steel are placed on the slope this layer is called *facing*.

Rockfill with a wide grading and a high proportion of fines is normally used for the mass of the dam body and should satisfy requirements for geotechnical stability (see Section 5.4.3).

The use of rockfill in dams exposed to flowing water either over, past or through the dam is of interest. Such rockfill should consist of heavy armourstone with wide gradation but with little fines. In Section 9.7 design and construction features are discussed for stones used on slopes and crests of dams for various hydraulic loads:

- **upstream slopes**: wave attack and currents – see Sections 5.1, 5.1.2.3 and 5.2.2.2
- **crest and downstream slopes**: overtopping by waves – see Sections 5.1.1.3 and 5.2.2.11.

For overtopping as part of spillway operations, the statements made in Section 7.2.3 regarding the relationship between the steepness of the downstream slope and the size of the stones or concrete elements are also valid here. A combination of through-flow and overflow is worse than overflow alone. In a reservoir rockfill dam acting as a spillway, such through-flow will not enter through the upstream slope (as happens with a river closure under construction, see Section 7.2.3), but it can still enter from the submerged crest of the overflow section into the armourstone "layers" unless special measures are taken. This is also valid for a weir or diversion dam.

Another point to be considered is the frequency of overtopping and its duration. In some cases (Escarameia, 1998), reinforced grass spillways may well satisfy the requirements. The acceptable overtopping rates for dams are by definition much lower than for breakwaters. In case of failure of an inner breakwater slope as a result of overtopping, the breakwater itself has to be repaired, but there are only limited subsequent costs and usually no loss of life. Failure of an inner dam slope will give catastrophic damage; a flood wave will go though the thalweg and cause significant damage and possibly deaths.

Section 7.2.3 gives information on reinforced rockfill with regard to the overtopping of cofferdams. Principal differences are explained below.

Most rockfill dams are constructed in the dry and although cofferdams can be incorporated in the final dam profile (see Figure 7.29) such rockfill dams will normally have a steep (V/H = 1:1.8 to 1:1.3) downstream slope. Particularly in the case of high dams, ie with a height exceeding 10 m, designers should not divert from this principle because of the higher costs involved. This automatically leads to the need to use **grouted** armourstone for the downstream slope of overtopping rockfill dams.

Progressive downward failure of mesh protection systems should be avoided by use of suitable anchorages such as crank-shaped anchors, anchors fixed to grouted dowels in fill, and inclined anchors.

Reservoir dams should have a design lifetime of up to 100 years, whereas cofferdams are normally already obsolete after 3–10 years. The long lifetime of reservoir dams may result in partial or complete corrosion of the mesh and anchor bars in the course of the design lifetime, unless appropriate measures are taken, such as:

- use of large diameters for mesh and anchoring bars
- use of special low-corrosion steel.

For more information refer to ICOLD (1993a).

7.5 BARRIERS, SILLS, WEIRS, BARRAGES AND DIVERSION DAMS

7.5.1 General

Section 7.1.1 provided general definitions for barriers, sills, weirs and diversion dams. The common features of these structures include:

- they have been designed for either through-flow or overflow
- the through-flow or overflow is present continuously or most of the time
- usually they are low structures with a height of less than 10 m
- they have been built for a single specific purpose.

1 2 3 4 5 6 7 8 9 10

7.5.2 Barriers

Barriers are structures that have a discharge or stage control function and are normally kept open (see Section 7.1.1).

These barriers are closed by gates:

- when water levels are expected to exceed a certain elevation (storm surge barriers)
- when saltwater intrusion is imminent because of low river discharges
- to fight oil pollution.

In most cases such barriers are built in coastal areas. Well-known examples are:

- Eastern Scheldt Storm Surge Barrier, the Netherlands
- Thames Barrier, London
- St Petersburg barrier (under construction), Russia
- Maeslantkering, Rotterdam, the Netherlands
- Nakdong Estuary Barrier, Korea

Common to all these barriers is the continuous presence of significant current velocities (up to 7 m/s) and an erodible river or estuary bed immediately upstream and downstream of the structure.

If for any reason one or more gates have to stay open when all other gates are closed, very high currents may induce an asymmetrical flow pattern on the downstream bed protection during periods up to 50 hours. Consequently, extensive permanent bed protection works are required upstream and downstream of the barriers. Moreover, these protection works have to be made in flowing water. For this reason high-density rock is preferred to concrete blocks. The protection works are built up like a filter layer.

More details on the design of bed protection works can be found in:

- Sections 5.2.3.5 and 6.3.1.2 scour downstream of protected area
- Sections 5.2.2.10 and 5.4.3.6 filter design
- Sections 5.2.3.1 and 5.2.2.5 dimensioning of top layer of bed protection.

Figure 7.27 shows the bed protection works carried out in the Eastern Scheldt.

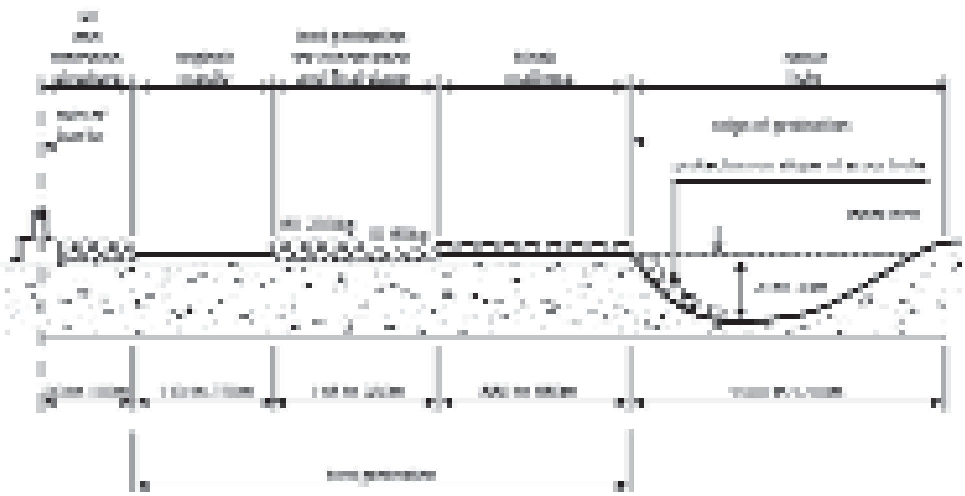


Figure 7.27 *Cross-section of bed protection works for Eastern Scheldt Storm Surge Barrier, the Netherlands*

7.5.3 Sills

In Section 7.1.1 sills are defined as low dams that will be frequently overtopped. Such sills will be constructed for the following purposes:

- controlled overflows from rivers into the floodplain if controlled flooding of the floodplain is desired (example: Danube barrier Austria, see Figure 7.28)
- overflow sections in riverside embankments at the entrances to flood retention reservoirs or river bypass channels
- prevention of erosion of river beds or mountain streams by means of concrete sills or low stone masonry retaining walls
- low dams across channels in estuaries or across tidal creeks, to function as the first stage of tidal closures (example: Feni Closure Dam, Bangladesh, see Figure 7.29)
- strips of bed protection mattresses in tidal closures, to stop regressive erosion of ebb channels and to function as foundations for sluice caissons (for example: Schelphoek shallow emergency closure 1953, the Netherlands, see Figure 7.30)
- overflow crests that are part of fuse-plug spillways or service/auxiliary spillways on abutments of reservoir dams.

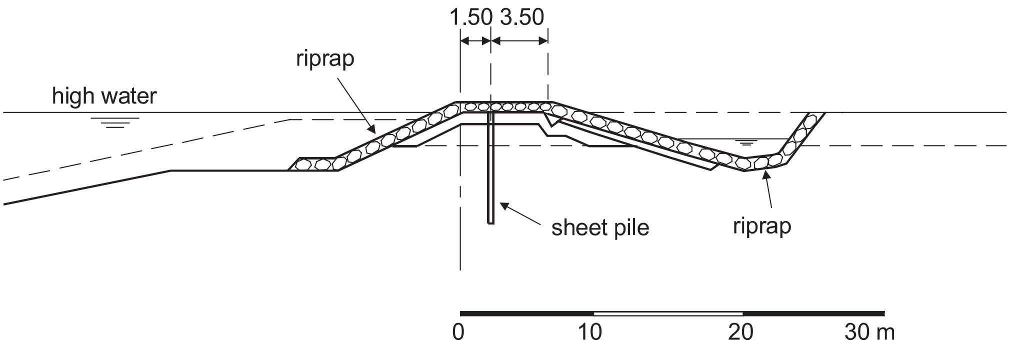


Figure 7.28 *Overflow section in riverside embankment upstream of barrier, River Danube, Austria*

A sill may consist of:

- a filter construction comprising filter layers of different gradings
- a bed protection mattress comprising geotextile(s), fascines and stone ballast
- a low broad-crested dam of stones placed on a bed protection mattress
- a concrete slab or slab made of stone masonry.

As far as hydraulic loads are concerned, these are normally linked to current velocities.

The current velocities are low in the case of a riverbed sill or controlled overflow from river into flood plains. In the case of a mountain stream, they are high but harmless, while for a fuse-plug spillway they are high but very infrequent. This leaves only the sills made as part of tidal closures for further consideration.

Figure 7.29 *Sill in estuary closure, Feni River, Bangladesh (courtesy J van Duivendijk)*

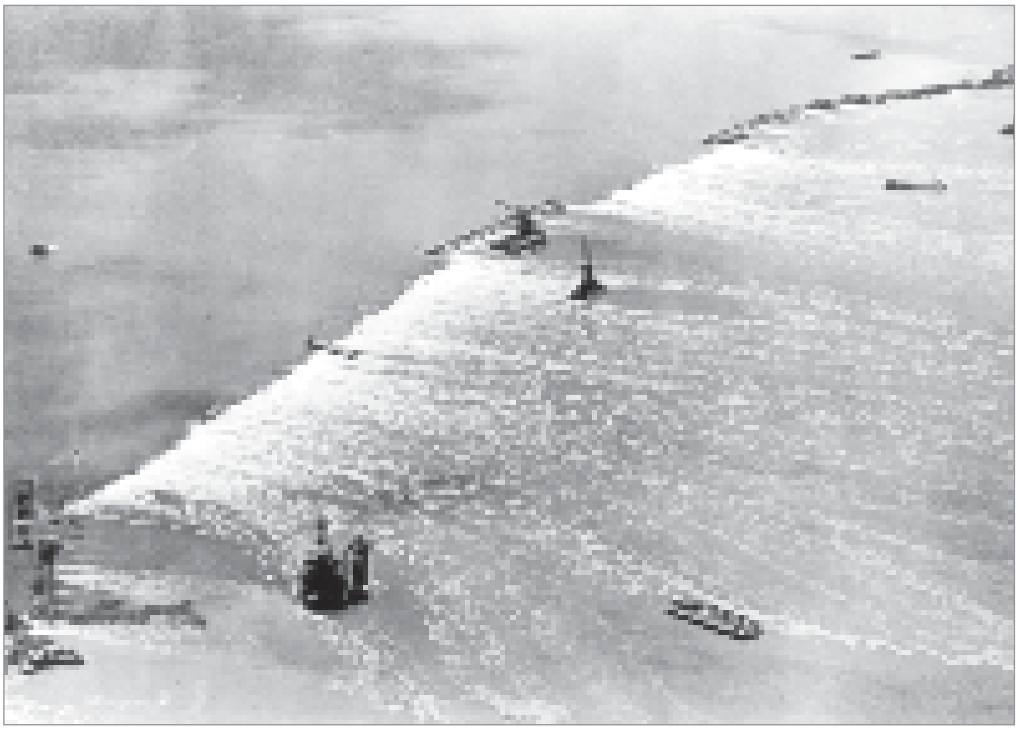


Figure 7.30 *Bed protection in Schelphoek shallow closure, the Netherlands (1953) (courtesy KLM)*

7.5.4 Weirs and barrages

A weir or barrage is normally built in a river for the purpose of stage regulation upstream of the weir. Stage regulation can be required immediately upstream of the weir for local run-of-river hydropower generation or to divert water to an irrigation intake or conveyance channel leading in turn to a hydropower plant. Stage regulation can also be required to make a river navigable upstream of the weir or for recreation purposes. Depending on its function, a weir will have to be combined with navigation locks and/or a hydropower plant.

The extent (duration, height) of overtopping will depend on the hydrography of the river (Section 4.3.2), the length of the weir and the water flow diverted for irrigation, hydropower or domestic purposes.

A weir can be a fixed-crest weir or movable weir. In the latter case it is called a barrage (or a gated dam) and consists of a concrete sill with gates on top. A fixed-crest weir normally has a limited height (say up to 5 m), but this will depend on the desired differential head. Such a weir may have the shape of a small gravity dam built of concrete or stone masonry. However, it can also have the same shape as a broad-crested dam built up of selected rockfill. A crucial factor is the manner in which this rockfill has been placed. If the weir has been constructed in the dry and suitable stone is available, an open-type revetment structure in which stones are interlocked can be built. Such a stable situation can also be reached by packing stones in gabions or by introducing reinforced rockfill (see Section 7.2.3).

More critical are weirs that are made of rip-rap. Here a very gentle downstream slope will be required as well as armourstone of narrow gradation. Such weirs will be built in the dry when regularly shaped stones are lacking.

When a weir has to be built in the water, the only option is a low dam of dumped rockfill. It may be difficult to close such a dam unless the water can be temporarily or partly diverted. Usually, after construction, such a weir of dumped rockfill needs some reshaping. Care must be taken to use stones of more or less equal size, all of a certain minimum weight. The current will remove smaller stones and larger stones will roll down the slope if they are projecting too far.

In most cases a weir of dumped rockfill will be built on a rock foundation. If the river bed consists of alluvium, either a bed protection must to be placed downstream of the weir or a completely different weir must be designed and subsequently built on a dry construction site.

Figure 7.31 *Weir at Awuru, River Niger (courtesy J van Duivendijk)*

The crest height of the weir is determined by (i) the requirements for stage regulation and (ii) the part of the overall discharge, which should pass over it when the river is flooding. A weir built for navigation purposes in the River Niger at Awuru (downstream of Kainji Dam) is shown in Figure 7.31. This weir was built largely by mechanical means (bulldozers, trucks etc) and it required considerable reshaping and repair after its completion when the first flood had passed. The reshaping involved increasing the elevation (required because of the local differences in water levels upstream of the weir, caused by asymmetrical flow towards the weir) and also the downstream slope, which was locally too steep.

The repair works were needed because of the damage due to crest levels that were locally too low, or downstream slopes that were too steep, and also to compensate for the washing out of fines as a consequence of through-flow.

Figure 7.32 *Weir at Bajibo, River Niger (courtesy J van Duivendijk)*

It was interesting to compare this weir, built by mechanical means, with the one built further downstream at Bajibo Rapids (see Figure 7.32), which was built completely by manual labour and suffered no damage whatsoever after completion. In the first case rip-rap was used, while in the second case stone pitching was used.

7.5.5 Diversion dams

The difference between a diversion dam and a weir as described above is arbitrary. Usually a diversion dam is higher (say up to 20 m), shorter in length because it is built in a narrow valley or canyon, and it has only one function: the diversion of water to an intake. It never has gates. Like the weir, the diversion dam will be frequently overtopped. Consequently, if built of rockfill it must have a gentle downstream slope. Points made earlier about the use of rockfill in cofferdams and rockfill dams are also valid here.

Figure 7.33 *Diversion dam in China (downstream face) (courtesy J van Duivendijk)*

Sometimes, a diversion dam is built of stone masonry. In this case it is possible for internal build-up of water pressure through cracks to occur and the possible development of weep holes through the downstream facing must be considered in the design stage. Figure 7.33 shows a diversion dam in China constructed as part of an irrigation project.

7.6 MODELLING IN RELATION TO FLOW PATTERN, SCOUR AND BED PROTECTION

Where possible, physical and mathematical models should be used to determine flow pattern, scour and the armourstone grading in bed protection works upstream and downstream of diversion passages and regulating structures. It should be emphasised, however, that the results of any model in respect of scour and armourstone grading are not so easily transferable to the prototype.

Various methods, usually empirical, are available to predict scour (Section 5.2.3.5 and Box 7.2) and to calculate armourstone stability (Section 5.2.3) in relation to current velocities that have been determined in models (Section 4.3.5) or by hydraulic calculations (Sections 4.3.2 and 5.1.2.3).

Scour prediction methods are not discussed in this manual, as there are ample references available, eg the *Scour manual* (Hoffmans and Verheij, 1997).

Whether physical hydraulic scale models are made to obtain the design scour or to verify the preliminary predictions and ultimate observations for the final design and verification during construction will depend on the magnitude of the works concerned. Depending on the size of the river and scale relationships associated with the hydraulics of the diversion scheme and relating to the size of the bed and bank material downstream of the diversion, the physical hydraulic models are made on an undistorted scale of 1:50 to 1:120.

The physical model usually has a mobile (erodible) bed. The model should encompass the entire width of the river and extend upstream and downstream of the diversion site beyond the influence of the diversion. As stated earlier, a physical model can also be used to evaluate the river closure as well as investigating the hydraulics of the diversion structure.

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