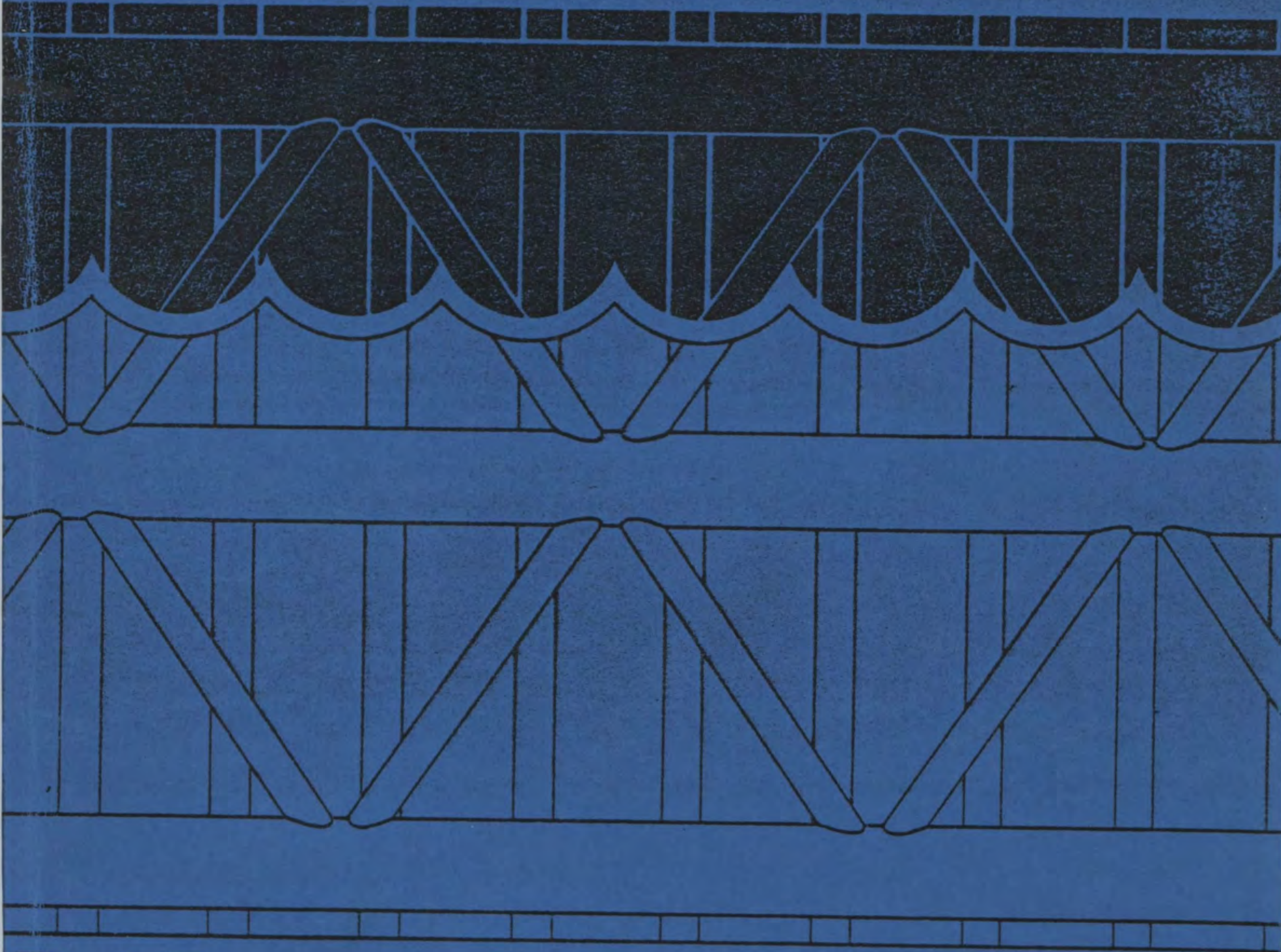
rijkswaterstaat

closing tidal basins

lecture notes: hans huis in 't veld



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DESIGN AND OPERATIONS OF CLOSURE WORKS

1 APPROACH

1.1 General

From a technical point of view, the closing of dike breaches is not essentially different from the construction of dams in existing tidal waters.
In general, dike breaches must be closed as quickly as possible to avoid damage in the inundated regions. As a consequence, such a closure will be more provisional than the dam construction, since the latter can be extensively studied ahead of time by computations and model tests.

Before the decision is taken to close off a tidal basin, it is necessary to carry out a feasibility study of the project, including a forecast of future requirements, and an economic study of the plan which will give an insight into the viability of the project.
There are many different purposes for closing a tidal basin (see table 3.1). Whatever the purpose may be, there will be impacts on other aspects. For instance the closure of a tidal inlet for security reasons may have (positive or negative) impacts on ecology, commercial fishing, recreation etc.
All these impacts have to be studied during the (feasibility) study phase.
A scheme of the stages of analysis is given in fig. 3.1. During the feasibility study also a preliminary layout has to be made.

After te decision fo close off a tidal basin the planning phase follows.
During the planning phase preliminary designs and alternatives will be prepared for the closure dam, discharge sluice(s), lock(s) and various auxiliary facilities. A Design Interim Report will contain all information on the preliminary designs and alternatives, the design criteria and the materials and methods to be used for construction. Additional site investigations will be initiated and hydraulic model tests will be executed to obtain the required data for the design. The environmental, ecological and social impact of the project will be studied (see table 3.2)
Also choices must be made about boundary conditions, which will influence the design, such as:

- choosing a water level and permissible fluctuations
- choosing for fresh or salt water
- choosing a method for closing off the estuary and, in case of a fresh water body in future, for replacing salt with fresh water.

These decisions are of great influence on the size of the discharge sluices. Considerations as to the nature of the landscape involved may influence the design of the dam as well.

The objective of the design phase is to work out the best and most economical design for the closure dike, discharge sluice, lock and gates and various auxiliary facilities, including preparation of the technical specifications, bills of quantity and tender documents for bidding. The preliminary cost estimates and the required period for construction will be determined by the designers. Plans will be prepared for material testing, for construction supervision services and for procurement of major construction equipment.
An operation and management plan will be prepared, including the organization, personnel, required budget to assure effective operation and management of the project after completion.
If required, countermeasures will have to be determined against the adverse effects on environment and ecology in the area.

table 3.1.

Purposes of closing a tidal basin.

* Land reclamation.
  - Zuiderzee closing (Neth. 1932)

* Protecting against floods from the sea.
  - Primary dams of the Delta Project (Neth. 1961-1985)
  - Eider (Germany 1970)
  - Keta Lagoon (Ghana 1969).
  - Easter part of Finnish Gulf to protect te city of Leningrad (U.S.S.R.1990).
  - Thames Barrier (Londen, England 1982).

* Creating of fresh water reservoirs.
  - Zuiderzee closing (Neth. 1932)
  - Haringvlietdam (Neth. 1970)
  - Philips- and Oysterdam (Neth. 1985)
  - Plover Cove (Hong Kong)

* Production of tidal energy.
  - Rance (France 60'th)
  - Severn estuary (Wales/Eng. 1995?)

* Closed sea harbours (behind locks)
  - Asan Bay (Rep. of Korea 1990 ?)

* Creating a construction pit for locks, sluices etc. (closure of the ring embankment) or water supply basins.
  - Haringvliet (Neth. 1957)
  - Philipsdam (Neth. 1978)

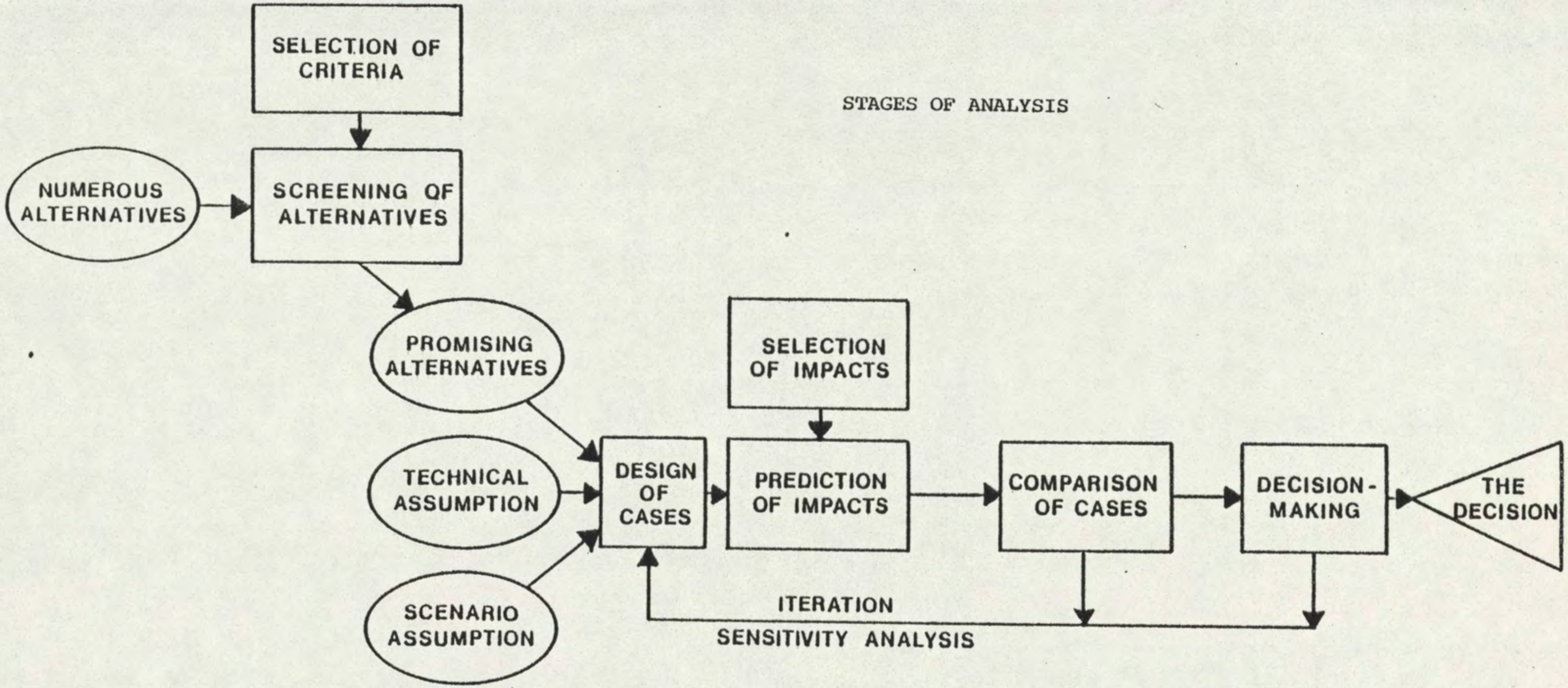

STAGES OF ANALYSIS
SELECTION OF CRITERIA
NUMEROUS ALTERNATIVES
SCREENING OF ALTERNATIVES
PROMISING ALTERNATIVES
TECHNICAL ASSUMPTION
SCENARIO ASSUMPTION
DESIGN OF CASES
SELECTION OF IMPACTS
PREDICTION OF IMPACTS
COMPARISON OF CASES
DECISION-MAKING
THE DECISION
ITERATION
SENSITIVITY ANALYSIS

Fig. 3.1

1.2. Design phase

The design phase started with a review and analysis of existing data. Depending on the purposes and circumstances data in the following fields are necessary :

- meteorology
- oceanography
- hydrography and hydrology
- geotechnics
- topography
- geology
- construction materials and equipment
- labour
- environment
- land and fishery rights

Additional data will be searched for in the initial stages of the project. All available data will be reviewed and analysed; special attention will be paid to the quality and reliability of the data.
The data will serve as starting points for the layout and design of the structural elements of the project. As soon as possible after the project a programme for additional field investigations will be prepared.

During the design phase the broad outlines of the possible alternatives have to be worked out in more detail in order to evaluate the advantages and disadvantages of each alternative from a technical point of view as well as from an economical point of view.

For each alternative the following has to be prepared:

- a general plan and layout for all structural elements and marine works
- a longitudinal profile and cross sections
- major details
- a rough cost estimate

When preparing the closing of a dike breach the decision how to realise the closing can be influenced by the positioning of the dam:

- reconstructing the damaged dike on its original course often involves the closing of gullies
- reconstructing a so called "horse-shoe" dike round the gap on the land side, generally involves a wide gap on groundlevel.

After the evaluation of the site investigations the following main subjects will have to be studied:

- workability during the construction
- the availability of construction materials in quantity and quality
- the availability of labour and equipment
- the use of model investigations
- the location and dimensions of temporary works, like building pits, work harbours, storage areas, cofferdams, etc.
- the cross section of the dam in the ultimate situation, which may have impacts on the materials to be used for the closure and for the final dam. The stability of the slopes and the seepage through the dam have to be considered.
- the wet cross-sectional area of the channel and the shape of the closure gap during the several construction stages. The closure gap is narrowed by building out construction pits for the locks and sluices, etc. The effects of this narrowing on the tidal velocities in the gap have to be considered.
- navigation requirements during execution
- the method of closing

The influence of available materials, labour, equipment, areas , etc. are not seldom underestimated by the designer .
It often happens that the material required for marine works is either not available locally, or only in limited quantities.
For instance: the quarry stone, often required for protection works in Holland, has to be imported from abroad. Therefore alternative constructions have been developed in recent years.
In many countries there is only limited experience in the execution of closure works and consequently it may be a problem to find the necessary skilled labour. The employment of expatriate labour demands special attention with regard to housing, travelling and leave periods. The possibility of training local labour therefore has to be thoroughly investigated. Also the utilisation of local equipment, eventually adapted, has to be investigated and should be compared costwise with equipment to be mobilised from abroad. The provision of spareparts always asks for attention - especially if projects have to be realised in remote places.
Site conditions may impose considerable influence on the realisation costs of the works.
All these considerations should play a vital role in the selection of the appropriate design.

Investigations

Both the preliminary and final designs of the ultimate cross-section of the closure dam will be based on desk studies. As regards the closure operations, desk studies during the initial planning period will provide data to select the most appropriate method of closure. The type of model investigation required to support the definite design depends largely on the chosen method of closure.

When gradual horizontal or vertical closure is decided upon, the model studies will mainly deal with the stability of rubble and boulders, used to build up the dam, at various closure stages. Such studies can be performed in a discharge flume. A wider flume is required in the case of gradual horizontal closure than in the case of gradual vertical closure. The required width depends on the model scale, which in turn depends on the hydraulic boundary conditions and the size of rubble. The hydraulic boundary conditions (head difference, discharge) will be obtained from the mathematical tidal model.

A scale model of the closure gap is required in the case of a sudden closure (caissons). This model, which will have a scale of 1:50 to 1:100, will provide detailed information on the current pattern for the various execution phases. Also here, the boundary conditions to operate the model will be obtained from the mathematical tidal model.

If the thickness of the silt/sand layer, overlying the rockbed, is significant, local scour tests will become necessary to enable optimal protection of the area or bed.

A scale model of the closure gap is required for these studies. Obviously it is advantageous to combine these and the above studies in one model.

Important aspects which can be investigated in a (hydraulic) tidal model are the direction and the sequence of construction of the dam-sections. In connection with the extent of the sandloss as well as possible scouring of the bed, the criterion for the selection is determined by the way in which the flow-pattern develops right in front of the sand-fill and in the remaining parts of the mouth.

If a hydraulic model is not available, for instance in case of small closures, the designer must be provided with sufficient data about the local tidal currents. In addition, more insight will be required into the manner in which they will adjust to the changing geometry caused by the construction of the dams.

Model type in relation to type of estuary

model / estuary	mathematical model			hydraulical scale model
	comb.	1 dim.	2 dim.	
short basin	*			
long basin		*	*	*
tidal river		*		

Model type in relation to data needed

Data needed	mathematical model		scale model	
	1 dim.	2 dim.	horizontal scale different from vert.scale	steady flow hor.scale = vert. scale
vertical tide	*	*	*	
discharges	*	*	*	
current pattern large scale		*	*	
current pattern small scale			*	*
phenomenons, strongly influenced by vertical velocity distribution (3 dim. current pattern)				*

Table 3.2.

Environmental aspects of the design fase

In the following table the "design fase" is taken to include not only the primary dam (A) but also the discharge sluices (B), locks for ships (C) reinforcement of the shoreline (D), secondary dams (E) and obtaining sand (F) or modifications of the shipping channels.

Environmental aspects \ design aspects	A	B	C	D	E	F
1. Areas which will become dry						
1.1. Purpose and function	-	-	-	x	o	x
1.2. Vegetation	-	-	-	x	-	-
1.3. Morfology (water level)	o	x	-	x	-	o
1.4. Natural landscape	x	-	-	x	o	-
1.5. Birds	-	-	-	-	-	-
2. Water quality						
2.1. Choice of fresh/salt water	-	x	o	-	o	-
2.2. Stratification	-	o	-	-	x	x
2.3. Toxic substance	-	-	-	-	-	-
2.4. replacing salt water by fresh water	-	x	-	-	o	-
2.5. sedimentation	-	x	-	-	o	x
3. Fisheries						
3.1. Present and future situation	-	-	-	-	-	-
3.2. Raised levels of production, fish farming	-	-	-	-	x	-
4. Surrounding area						
4.1. Discharge to the sea	-	x	-	-	-	-
4.2. River banks	-	-	-	x	-	-
5. Water supply and control (water level)	-	x	o	-	o	-

x = very much interrelated during design fase

o = moderately interrelated during design fase

- = not interrelated during design fase.

design philosophy

Up to now, and most probably for some years to come, a deterministic approach has been utilized for the design of marine works. The design criterion is based on the prevention of instabilities under extreme load conditions.
Usually an extreme current and an extreme wave height are separately considered.
An obvious disadvantage of the deterministic design approach is the lack of criteria for the choice of the "extreme loading".
Such a choice remains in fact a subjective one.

In the last years the conviction grows that the marine works need not to be designed to fully withstand the most extreme currents and waves possible.
In economic design some damage can be accepted, especially in those cases that repair is not too costly. New studies must give better insight in the frequencies of occurance of extreme currents and waves and the probabilities of coincidence of load combinations.
Apart from these studies the practical approach should not be forgotten, such as the construction of test sections in models and in situ.
The proposed design approach is a probabilistic one. To apply this approach not only information about the frequencies of occurrence of loads should be available, but also the response of the construction.
The response function is obtained either from hydraulic model tests or by applying known "transport" relationships. If more than one type of loading is acting, the summation of the damage should be computed by integration over the various load combinations.
The resulting total damage is a measure for the expected maintenance of the marine works for a given dimension.
A process of economical optimization, based on the costs of construction and maintenance, can further be carried out, leading to the selection of the optimum dimensions.
Besides this minimum integral costs criterion, one should also consider the "expected total damage" and the risk of progressive damage, if repair betimes is impossible for technical, organizational or financial reasons.

LAYOUT OF AN ENCLOSURE DAM

2.1. Positioning the dam

The factors contributing to the selection of the location of the dam-alignment can be devided into two groups, viz. factors, that are generally indicative and factors which determine in detail where the alignment should be.

The first group comprises the factors:
- purpose of the dam to be constructed
- hydraulic conditions during construction and after the dam has been completed
- watermanagement conditions after the dam has been completed
- environmental aspects

These are already mentioned before

The second group comprises factors such as:

a. the conficugration of the bed in situ

b. the composition of the bed

c. the connection with the shores

d. the closure method

a. The configuration of the bed in situ

The position of channels and shoals in the area involved, and, in particular their evolution over time, are important. It is evident, that for the location of both the dam and the closure-gap, the existing pattern of channels and shallows must be utilized optimally.

Some of the postulates for the first provisional assessment of the respective positions are:
- that the deepest parts of the cannels should be avoided for both economic and technical reasons;
- that the centre lines of the gaps to be closed should cross the channels as nearly perpendicularly as possible, i.e. they should be as short as possible and be virtually perpendicular to the stream lines of both incoming and outgoing tides;

- that confluence or division of channels in the vicinity of the closure gap must be avoided as much as possible, due to a stable current pattern and bed configuration, which gives a better prediction for several construction stages.

If the location of a closure-gap near a confluence or division proves to be necessary, then a choice must be made between the construction of one large closure-gap, and two smaller ones, with an artificial work-island in between. Placing the sections of the dam in the shallows would in general not seriously alter the current pattern, but can affect the distribution of the discharge over the various channels.
The boundaries of these dam sections on shallow parts are chosen in such a manner, that natural development is considered (fig. 2.1.a.).

Generally it can be assumed that the oulet-area of an estuary is in a dynamic equilibrium (particularly where it concerns the conluences and divisions). However, the construction of dam-sections or working-islands usually disturb this equilibrium. Of course, the consequences of this disturbance for the bed-configuration depend strongly on their extent and duration.

b. The composition of the bed.

The aspects relating to the composition of the bed are, inter alia, the bearing capacity in connection with the stability of the damsections to be constructed, or the structures within the actual dam and also the availability of building materials (sand) for the dam.
The choice of the location should, if possible, thus be taken that areas with a weak bottomcomposition are avoided. If this proves to be impossible, then a soil improvement will be a solution.
This implies, that a trench must be dredged, that new material should be supplied and that, if necesaary, the soil should be compacted to increase the bearing capacity.
Also, if the available bed-material is not suitable as building material for the damsections, or for final sand-closure, then additional costs will occur for the supply of suitable sand.

c. The connections with the shores.

The connecting points of the dam with the shores may cause problems, either they are located "ashore" or "offshore".
The problems ashore often concern the existing infrastructure in the environs of the planned damshore connection. Problems may arise then forming the connection between the carriage way over the newly constructed dam with the

existing infrastructure.

The problems offshore could be:

- trying to avoid locations where the channel runs close to the shore;
- trying to avoid places where the slopes of the shores are less stable (sensitive to liquefaction)
- trying to avoid, in general, connections in an outer-bend;
- trying to avoid as much as possible the disturbance of areas which are interesting from an environmental point of view (this also applies to the inner-dike sections).

Due to the prevailing boundaries of the closure-gap and (during construction) the building of the slocure-dam, the velcity of the current and equally the intensity of the turbulence increase. In addition, the current will, due to the directional influence of the vertically narrowed gap, tend to perpendicularly cross the crest of the closure-dam. As a result of this phenomenon, the higher current velocities will enforce the attack on the boundaries of the channel in an outside bend.

d. The closure method.

The planned method of closure will play an important role in the actual design of the location of the dam and the closure gaps. The most conventional closure-methods are those, which "follow" the shape of the gap in situ, e.g.:

- in case of a gradual closure, a wide and shallow closure-gap is preferred, as, in a wide cloruse-gap the current will be more diffused during the final closing-procedure; so a shallow gap restricts the amount of filling material required. With this closure-method, the boundaries are less important as no dredging will be required.
- an attractive feature of the sand-closure method is, that from the sand-loss point of view, the execution of the final closure procedure is carried out at a relatively high ground level (the time to complete the sand-fill will be shorter and the loss of sand less). This means that one boundary of the gap must be at some distance from the channel's edge (fig. 2.1.b.).
- in case of a caisson-closure (sluice caissons), a relatively narrow and deep gap is preferred, as, on the one hand the number of placing-manoeuvres will be limited (which are complicated an risky), while on the other hand, the waterdepth above bottom and sill remains relatively deep in order to maintain an adequate wet cross-section; the boundaries of the slocure-gap must be placed in relatively deep water in order to limit dredging near the shores.

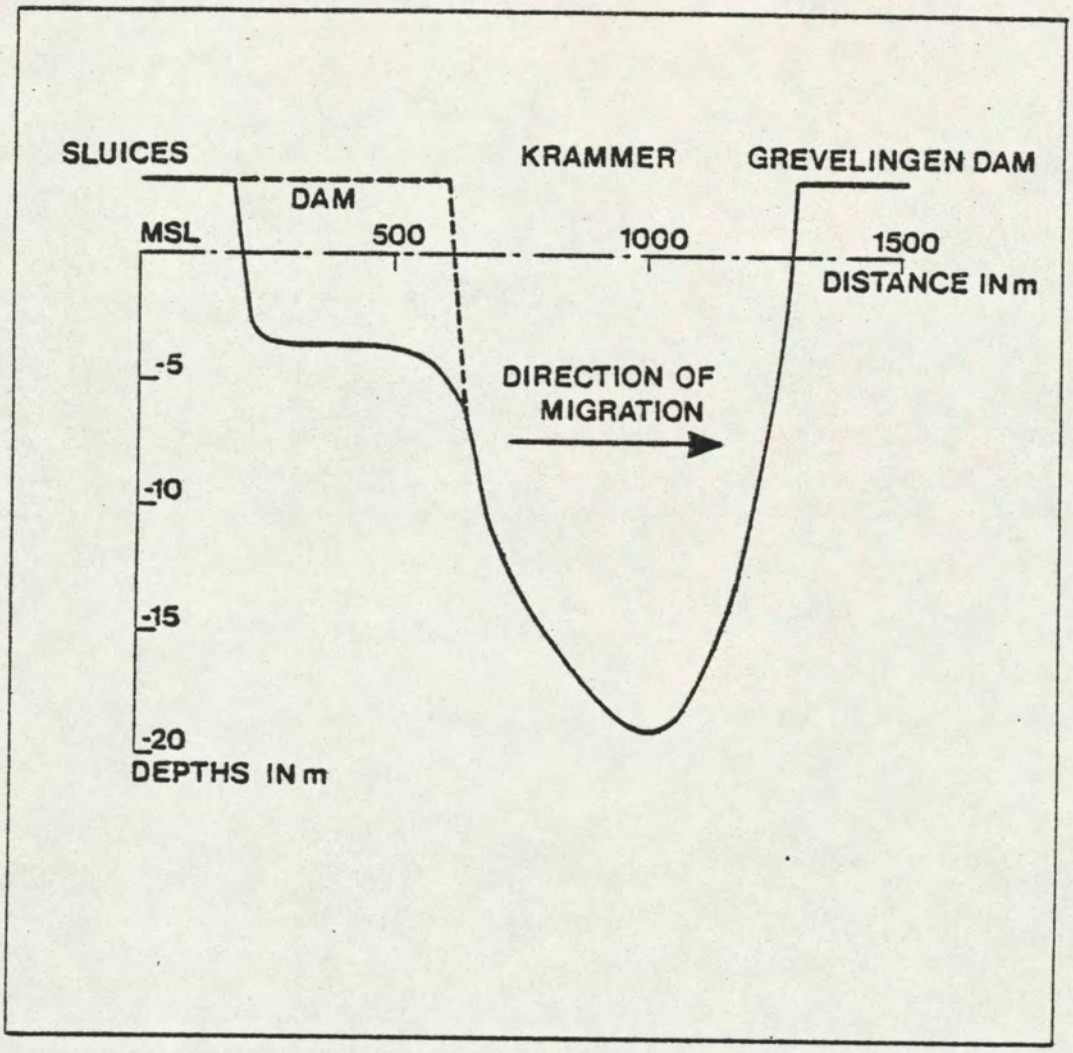


Fig. 2.1.a.

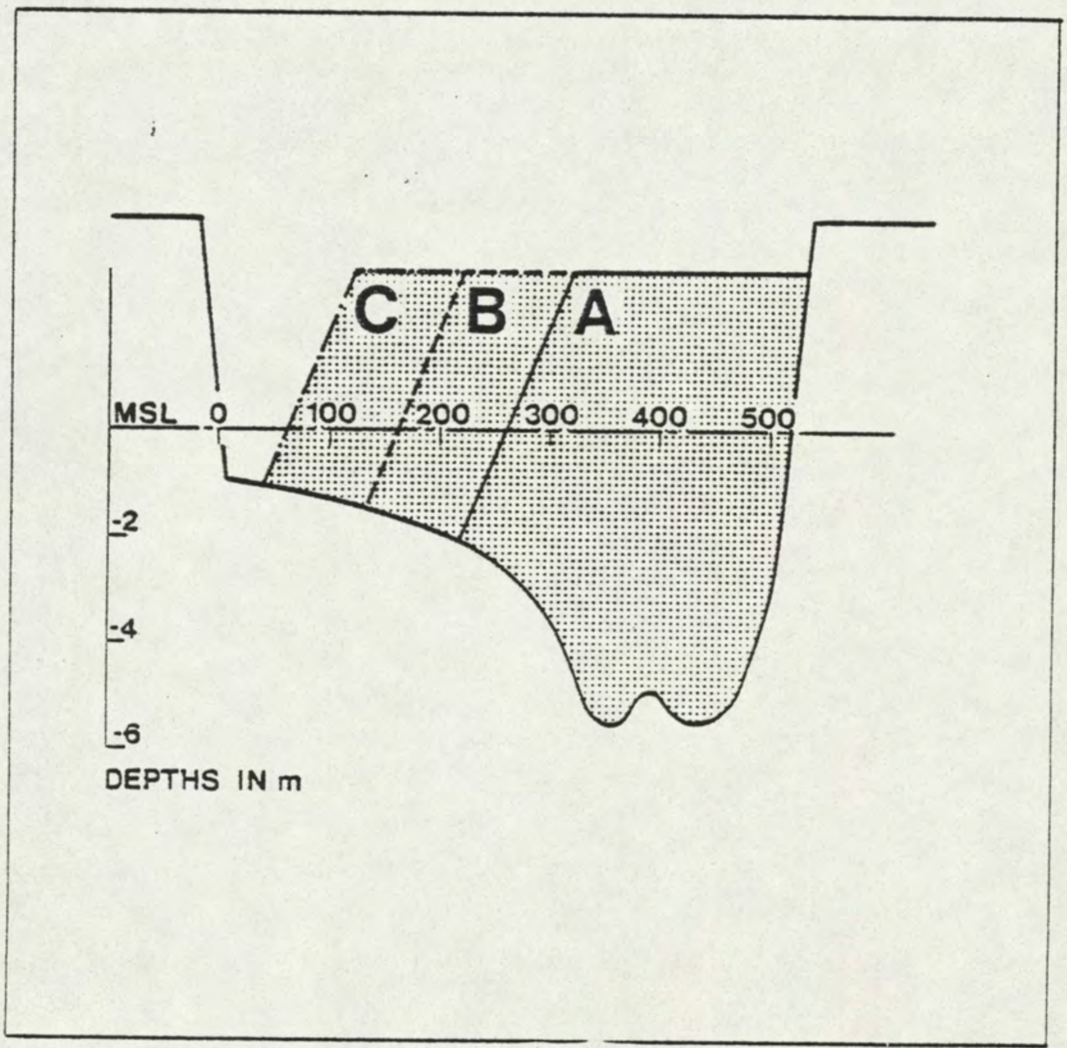


Fig. 2.1.b. Sand closure

2.2. The location of structures in the dam.

The questions relating to this aspect are, inter alia: how will the structure function after it has been completed, and also: how can the structure be built without causing too much disturbance of the tidal action?
In the final stage nautical aspects and preservation of the water quality will influence the choice of the location of a lock or a discharge sluice.
In the construction stage of a lock or a sluice in general a building pit will be necessary.
By construction of an earth embankment, built with local materials, a building pit will be formed. This building pit will have to be kept dry by means of a drainage system. Investigations and study of hydrological and geological data could necessitate a review of the location and call for special construction methods and ways of draining the building pit.
From a constructional point of view, building on shallows will always be preferable as the costs will be relatively low and there will be little disturbance of the tidal action. However, for the final stage additional costs will generally incur by the dredging of approach channels.
Conversely, for the final stage, construction in the channels could be more favourable although this would mean that during the construction additional problems could arise due to erosion (construction of the cross-section in the closure-gap resulting in an increase of the current-velocities); moreover the construction costs could be extremely high.

Discharge sluices

The main function of a discharge sluice is to pass excess water from the basin into the sea after the completion of an enclosure dam.
Not only will it be necessary to maintain the waterlevel in the basin between certain limits, but also irrigation schemes upstream of the sluices will require proper regulation of the waterlevels in the basin.
In view of possible damage due to flooding fields critical maximum levels in the basin will have to be determined.

The discharge capacity of the sluice will have to be determined based on the maximum floods from the rivers debouching into the area; taking into account permissible waterlevels in the reservoir, the storage capacity of the reservoir, local rainfall and the tidal waterlevels. The above will be determined by calculations.
From the required discharge capacity the main dimensions of the sluice can be determined. Depending on the aperture of the discharge sluices a part of the tidal motion can be allowed to pass through them during the

final stage of the closing works. In this way it may be possible te reduce the drop in head over the closure embankment.

Navigation requirements

The shipping sector, for instance, can have special requirements. One of these will be that, during the construction of the dam, navigation will be hampered as little as possible. During the dam construction certain demands can come from shipping which normally use the route across, and/or others from shipping normally using a route running more or less parallel with the dam. If a dam is provided with a lock, this lock will allow the passage of floating equipment during the works.

Modern lock systems are more and more equipped with centralized operation-systems. Such a system is costly and requires highly qualified personnel. Depending on the operation system, the location and layout of a central control building and satellite control buildings will be determined. In all cases the basic requirement will be an unobstructed view of and over the total lock complex.

Workshops and stores will be required for maintenance of the various mechanical and electrical components. Storage will have to be provided for the sparegates, the larger parts of the beacon system, etc.

For the shore-based part of the navigational aid system, an adequate building will have to be provided in certain situations.

To monitor the water level fluctuation, within the basin additional hydrometric stations will have to be established.

2.3. The shape of the closure gap.

Concerning the shape of the closure gap the following factors are important:

- the location and orientation of the closure-gap.
- the dimensions of the closure-gap,
- the shape of the boundaries (heads)
- the absence or presence of a sill and the dimensions of that sill.

The impacts of the above-mentioned factors are interrelated and cannot be designed separately. The resulting flow-pattern in or near a closure-gap will be determined by a combination of all these factors. In practice, initial designs will be made in which all factors are included, and subsequent selections of the most appropriate design will be carried out by means of desk studies.
The application of desk studies has become possible on the basis of the large experience which has been gained in the last three decades during the execution of closure-work. To enable this, data should be available on the tidal motion before, during (various stages of) and in the ultimate stage of the construction. Such information can be obtained by means of a mathematical tidal model. The necessity and type of model investigation required to support the definite design depend largely on the chosen method of closure.

Location and orientation of the closure gap.

The factors determining the location of the dam-position and, consequently also that of the closure-gaps have already been discussed. In order not to change the tidal action too much during the construction of the dam-sections, a closure-gap will be made in each (main) channel. Therefore, the number of closure-gaps will equal the number of channels feeding the estuary. If any sizable, secondary channels exist in the alignment-area, and, assuming they are included in the dam-sections long before the closure-operation, then the current velocities in the closure-gaps will increase (see fig.2.1) In the case that the bottom consists of loosely packed materials, as a consequence some scouring will occur. In that case, it is recommended that a bottom protection in the closure-gaps is constructed as early as possible. If, however, the scourings form a critical part in the design, then the secondary channels have to be kept open, until just before the final closure of the main channel(s), or until the scouring process is completed.

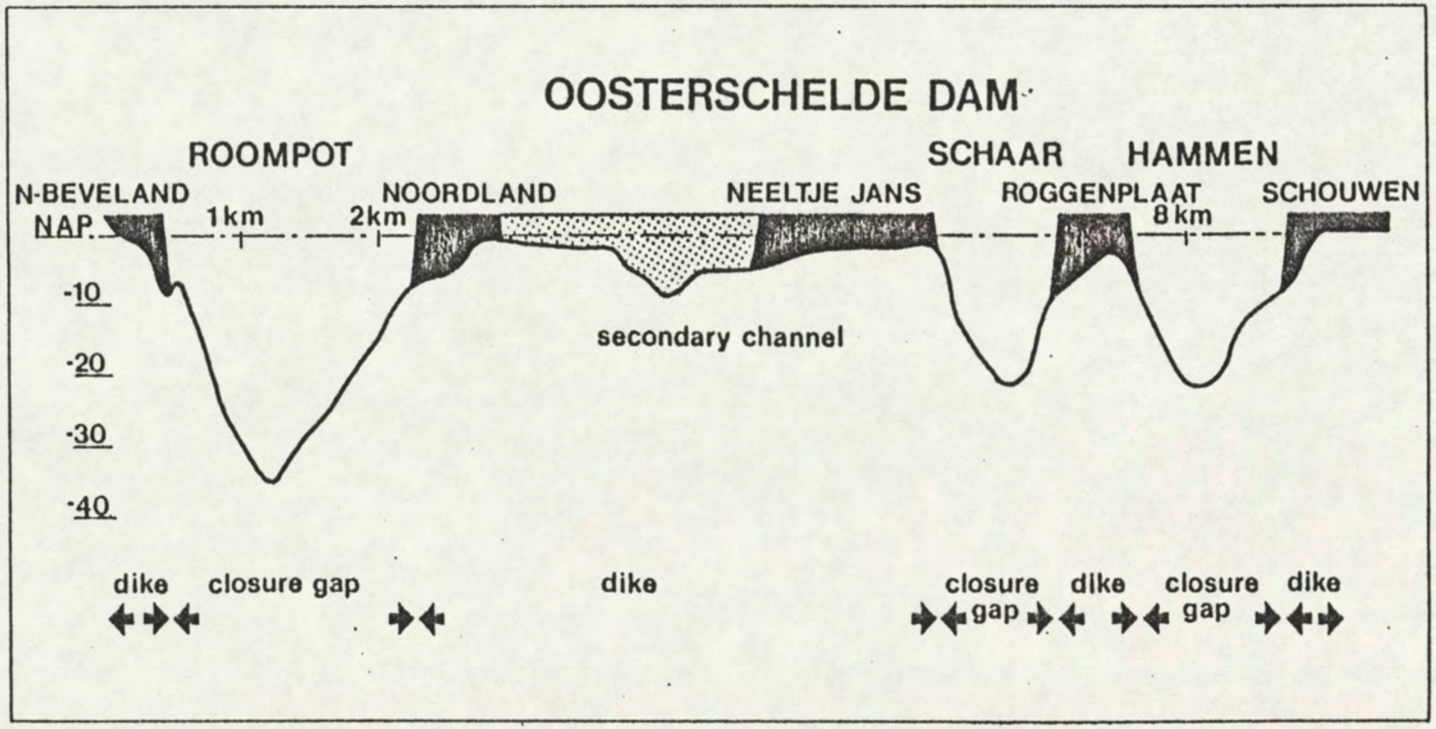


Fig. 2.2. Principle of closure works in tidal area.

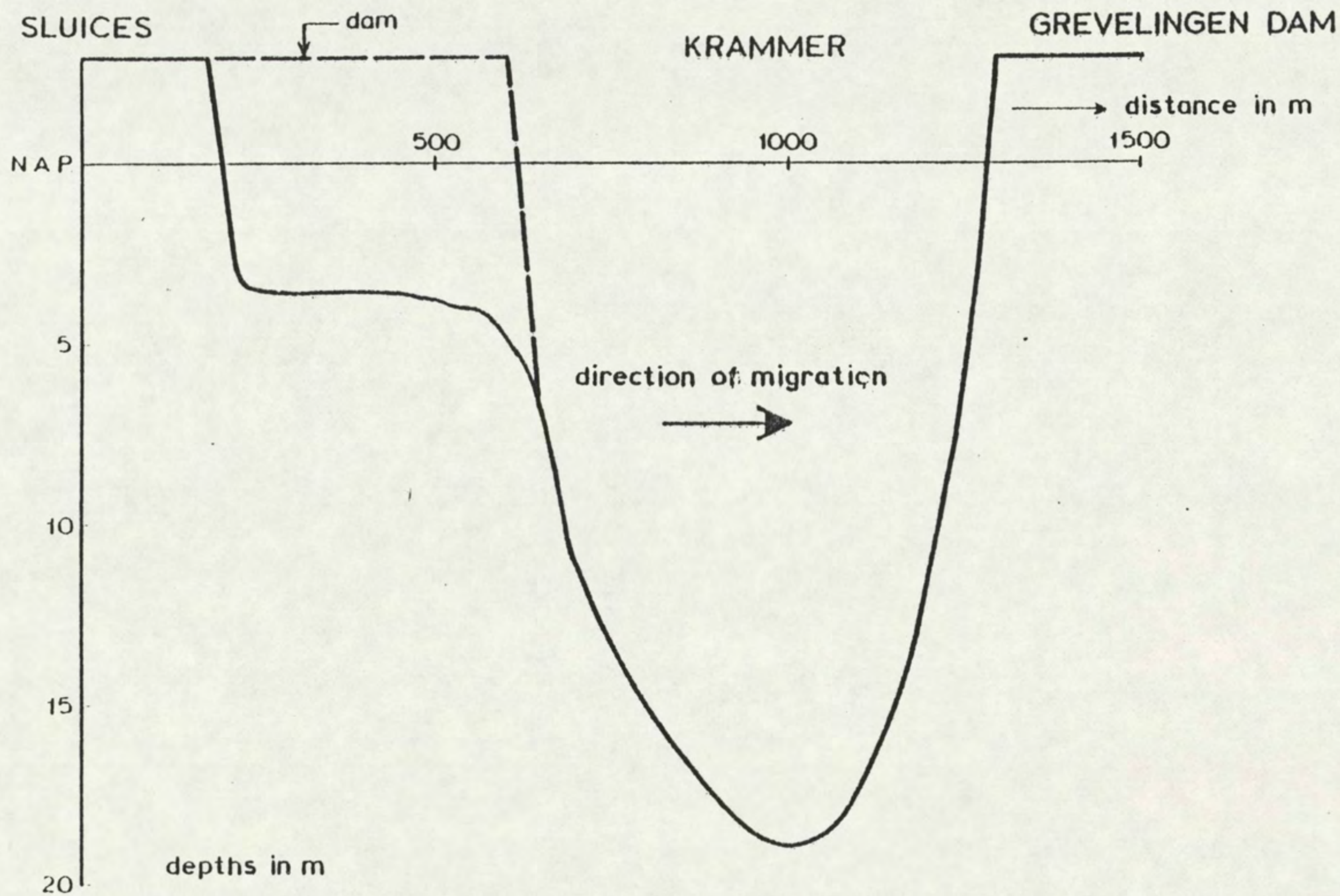

SLUICES
dam
KRAMMER
GREVELINGEN DAM
distance in m
500
1000
1500
NAP
5
10
15
20
direction of migration
depths in m

Fig. 2.2. a.

It is mentioned before that the closure-gap should be designed perpendicular to the flow-direction (in particular during and around the maximum flow-velocity). A current obliquely approaching the closure-gap will cause turbulence and consequently severely attack the bottom downstreams.
In addition, these peak-velocities, that are coupled with increased turbulence will form an additional threat to the dam under construction. Due to oblique currents, sedimentation problems may occur in those areas where eddies are formed near the edges of the closure-gaps. If an oblique approach to the closure-gap is unavoidable, then the consequences may - partially - be alleviated by the choice of an appropriate shape of the boundaries of a closure-gap.

Dimensions of a closure-gap.

The tidal volume flowing in or out of the estuary will hardly be affected by the construction of the dam-sections. A possible (limited) discharge-function of the shoals (for instance at HW), will be taken over by the closure-gap, just as is the case with the closure of secondary channels already described above. The average current velocity through the closure-gap will, apart from the affect of tidal and estuarine characteristics (tidal volume), be determined by the size of the wet cross-section.
In addition, the (horizontal) velocity-distribution in the closure-gap will be important. This factor will be determined by the location and the current approach to the closure-gap and the position and shape of the dam-heads.
The occurring current-velocities are important for the stability of the dumping operations, excavations and the use of floating equipment. It should be noted that the increase in velocity of the current is important for the scouring and erosion as this factor will determine the extent by which the capacity of the current to absorb, exceeds the sediment-supply.

In contrast, as far as the stability of the dumped material and the current-loads on floating equipment are concerned, the absolute magnitude of the velocity is important.

In estuaries adjoining seas with high tides, the current velocities will be relatively high even in an undisturbed situation. In this case, the closure-gap will have to be designed as wide as possible with regard to the work-ability (floating equipment/caissons). This implies that the amount of the material required for the closure will increase.
Therefore, a dam-section projected too far into the channel (relatively low current velocities in the undisturbed situation) will result in an

increase in velocity in the remaining part of the closure-gap. In addition, the adjoining part of the closure-gap will be obliquely approached by the current resulting in separation of the current and generation of vortex streets, which phenomenon occurred in the Hammen-closure-gap in the mouth of the Oosterschelde (fig.2.3.).

An important aspect is, that in the design of the boundaries of the closure-gap, the natural development of the sea-bed is taken into account. This means that in case a channel should migrate towards a dam-section, measures will have to be taken to ensure that during the construction period, this dam-section will not be engulfed by the channels, as this would require additional defence structure.
If a channel does move away from a dam-section, then this dam-section will have to be constructed on the boundary between channel and shoal. If the pace of the migration of the channel is high, then it may prove to be necessary to extend the actual dam-section (a relatively cheap operation) shortly before the final closing procedures, in order to obtain an adequate boundary of the closure-gap.

Shape of the boundaries.

The current-pattern near and in the closure-gap will, to a great extent, be affected by the shape of the boundaries of the closure-gap (or the heads of the dam-sections). With relatively long dam-sections, in particular, the current-velocities along the toe of these dams will, to a great extent, be determined by the shape and size of the dam-heads. In these cases, it is often necessary to build (an) extensive construction(s), which will ensure that the current is more or less directed straight towards the closure-gap. There are plenty of examples of directional constructions, such as: the dam-head "Noordland" and the "Neeltje Jans" in the mouth of the Oosterschelde (see fig.2.2). These constructions are so enormous that they provide ample room for forwking- or refuge-harbours to be built within them.

However, it is not always necessary to build such extensive directional dams. The position of the closure-gap can be such, that a simple dam-head construction will suffice. The designer can compensate for its effects (the effective width of the closure-gap is diminished) and extend the bottom-protection locally.

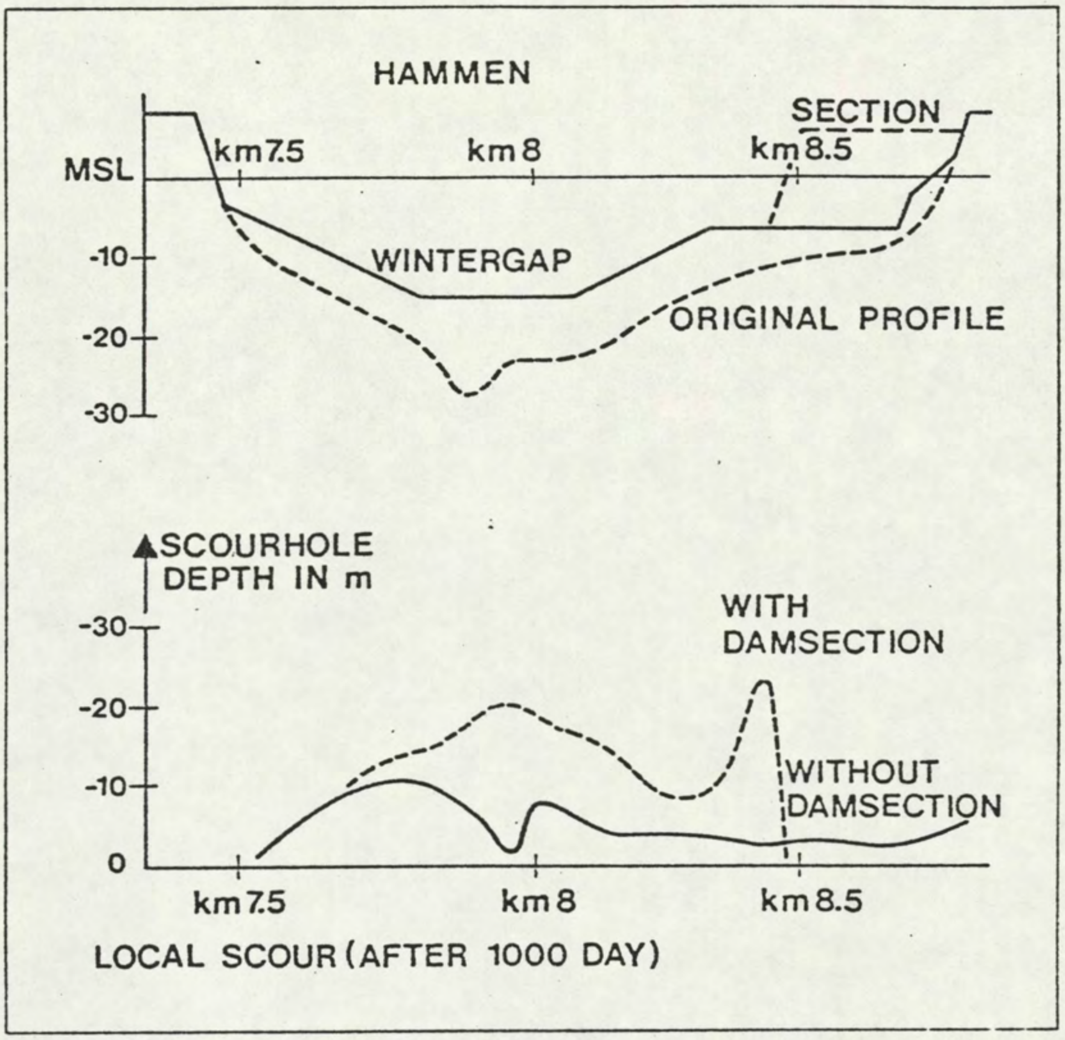

HAMMEN
SECTION
km 7.5
km 8
km 8.5
MSL
-10
-20
-30
WINTERGAP
ORIGINAL PROFILE
SCOURHOLE
DEPTH IN m
-30
-20
-10
0
WITH
DAMSECTION
WITHOUT
DAMSECTION
km 7.5
km 8
km 8.5
LOCAL SCOUR (AFTER 1000 DAY)

Fig. 2.3.

A sill in the closure-gap.

A sill is in fact part of the closure-construction of the gap. From the work-planning aspect it is, however, often necessary to build the sill during a previous season.
A caisson-closure, for instance, requires that the crest of the sill be absolutely level. As the realisation of such a project requires a long period of time, endeavours will be made to build the sill at least one season prior to the actual final closure. In case of a gradual closure, this advance sill-construction means, that a large part of the material will already have been dumped in the closure-gap (largest volume, uniformity in height). However the necessity for an early construction is less evident in this case. It depends, on the length of the "working season" and the available dumping capacity.
It should be noted however, that completion activities must take place in the same working-season. In particular, with a relatively shallow closure-gap, the prior construction of the sill can or even should be avoided. In this case, the presence of the sill would constitute an important constriction of the closure-gap, which would require large(r) grained dumping material (in order to survive the winter season) and a longer bottom-protection (because of larger excavations).

A sill in the closure-gap is usually favourable for the current-pattern.
The applied obstruction promotes egalisation of the current-distribution and, as such, an efficient use of the wet cross-section.
The higher the sill, the higher the egalisation-effect. On the other hand, the scouring will increase in proportion to the height of the sill (see fig.24)
Efforts must be made to find an optimal solution which will combine the advantages of both effects.

Finally, some more information about the shape of the sill.
The longitudinal cross-section (transversing the channel) strongly depends on the closure-method to be applied. For the caisson-closure the crest of the sill has, in general, to be horizontal (and flat).
For a gradual closure, the cross-section of the crest may follow the original channel-profile, which may prove advantageous for the current-dustribution on the down-stream side of the closure-gap.

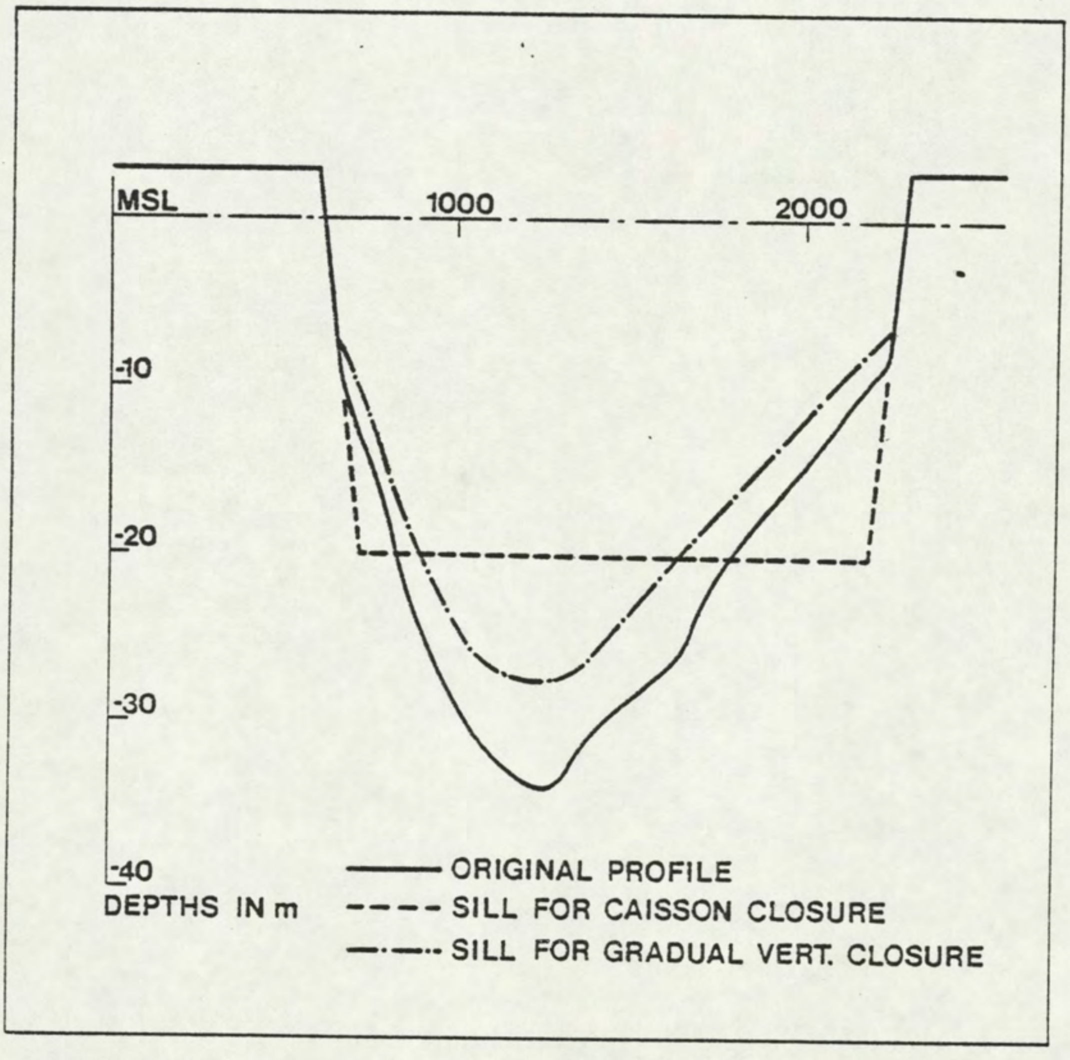

MSL
1000
2000
-10
-20
-30
-40
DEPTHS IN m
ORIGINAL PROFILE
SILL FOR CAISSON CLOSURE
SILL FOR GRADUAL VERT. CLOSURE

Fig. 2.4.

2.4. Cross section of the dam

The cross section of the final dam depends on several factors.

- the hydraulic boundary conditions (waterlevels, waves, ice)
- the necessity of a road over the dam and the intensity of the traffic
- requirements about the sealing of the dam (a division of salt and fresh water)
- environmental conditions (landscape, recreation etc.)
- available construction materials, methods and period
- the used materials and method for the closure operation

The cross section of the final dam would have to be designed to withstand the tidal difference over the dam and the wave or ice attack.

For a given stormsurge level and wave height, the wave run-up and hence the crest height of a dyke are determined by the slope gradient and the width of the wave breaking berm (if a berm is built - see fig. 2.4.a).
Other conditions being equal, the necessary crest height of a dyke with a faint slope on the seaward side, broken by a berm will be less than for a dyke with an uninterrupted or sleep slope.
This follows from the wellknown formula:

$z = 8 \; Hs \; tg \, \alpha$, where z = wave run-up against the slope (m)
Hs= significant wave height (m)
α = slope gradient

A faint slope is favourable for asphalt revetment and better compaction results can be achieved (see literature "Asphalt revetments of dyke slopes" - Rijkswaterstaat Communications no.27).
It is most economical to make the inward slope as sleep as possible.

The cross section of the final dam may have impacts on the used materials for the closure.
If the closure embankment is a temporary structure it may be interesting to check the possibility of recycling the used materials. Recycling the materials will be easier when applying rubble, rock or concrete blocks, than when caissons are used.
If the sand closure method will be applied an interesting alternative will be to create a dune and beach profile (see fig. 2.4. c).
When using "sluice" caissons, which are virtually floating sluices, it is possible to consider them as temporary structures that stay in the core of the final dam, but it is also possible to design them as the definite sluices, storm surge barriers or turbine cassons for tidal power energy.

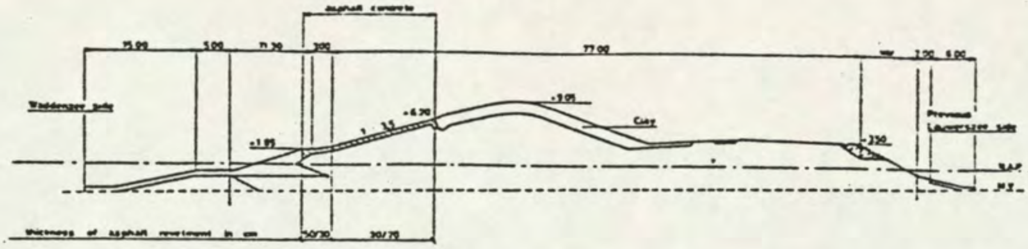


Figure a Cross-section of Lauwerszee closure dyke.

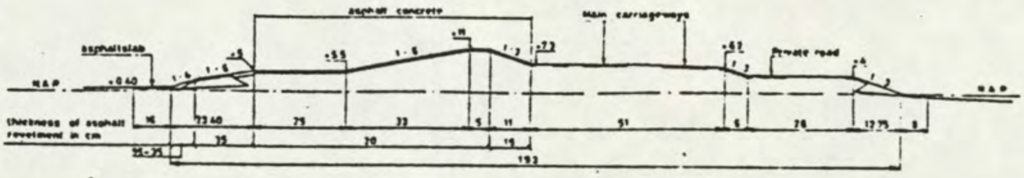
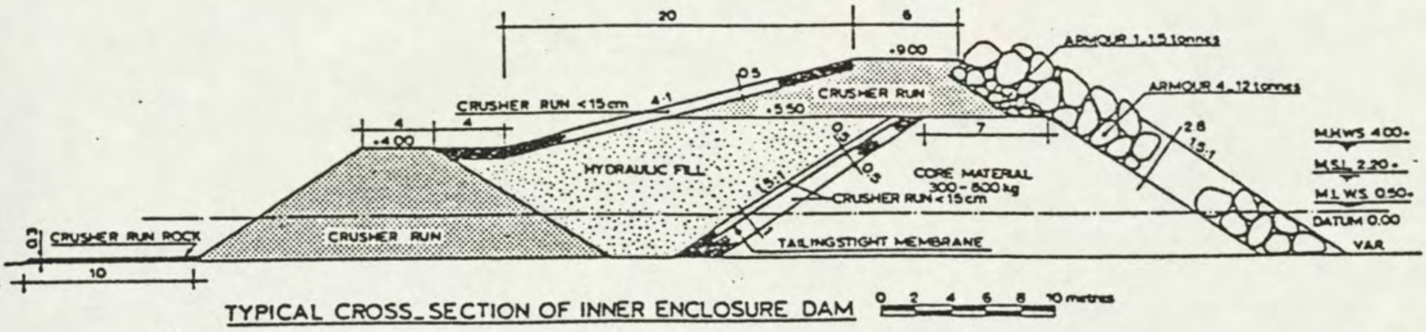
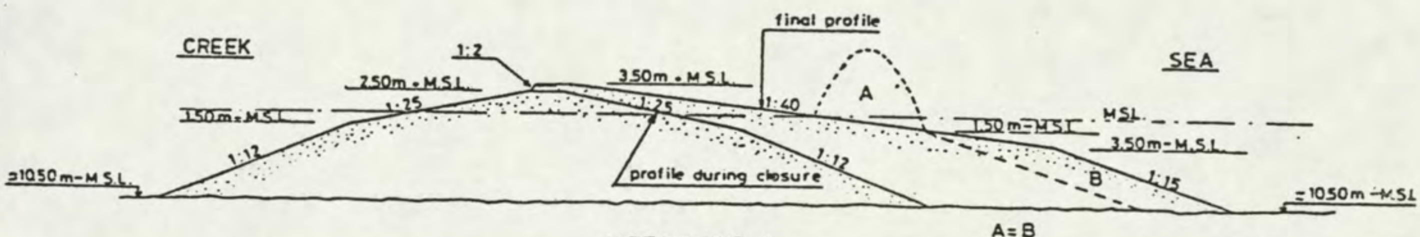


Figure b. Brouwersdam cross-section.



TYPICAL CROSS_SECTION OF INNER ENCLOSURE DAM

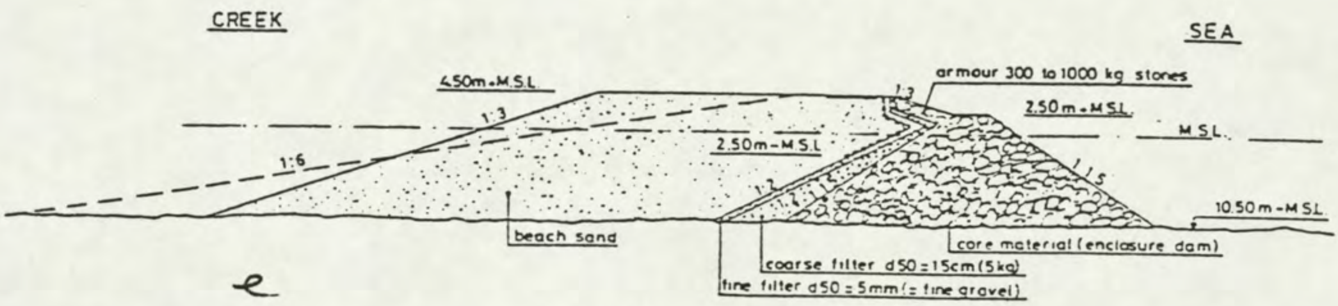
c



ALTERNATIVE 1

creation of dune and beach
(hydraulic fill method)
use of two high capacity suction dredgers(ø65) is required

d



e

ALTERNATIVE 2

rockfill dam made watertight
by means of sandbody on creekside

Fig. 2.5

3 CONSTRUCTION METHODS

3.1. Closure methods

When building a dam to close off a tidal basin, the inflow and outflow through the narrowed gap will be hampered, causing a decrease in the tidal range in the basin and higher current velocities through the remaining opening. As a consequence, the scouring effect on the bottom near the dam will be increased which endangers the stability of the riverbottom and thus the foundation of the new seawall to be built on it. This implies that the bottom, when consisting of easily erodable sand, must be protected by current resistant material. Naturally, these problems will not be encountered where the bottom is rocky.

A special problem arises from the method of closure. In the beginning the velocities will usually be low; therefore, relatively cheap materials initially may be used to narrow the gap - if possible (and if available) sand, and otherwise small sized rubble (gravel) or clay. The gap is narrowed by building out from the sides and by heightening the sill. If the velocities become higher than 2 to 3 m/s, heavier material must be used. The difficulties in effecting the closure will increase for still higher velocities, because navigation in the gap will not be possible during the whole tide, and the scouring effect will become more serious. To close the final gap there are three groups of closure methods: (see fig. 3.1.)

I: the gradual closure
II: the sand closure
III: the "sudden" closure

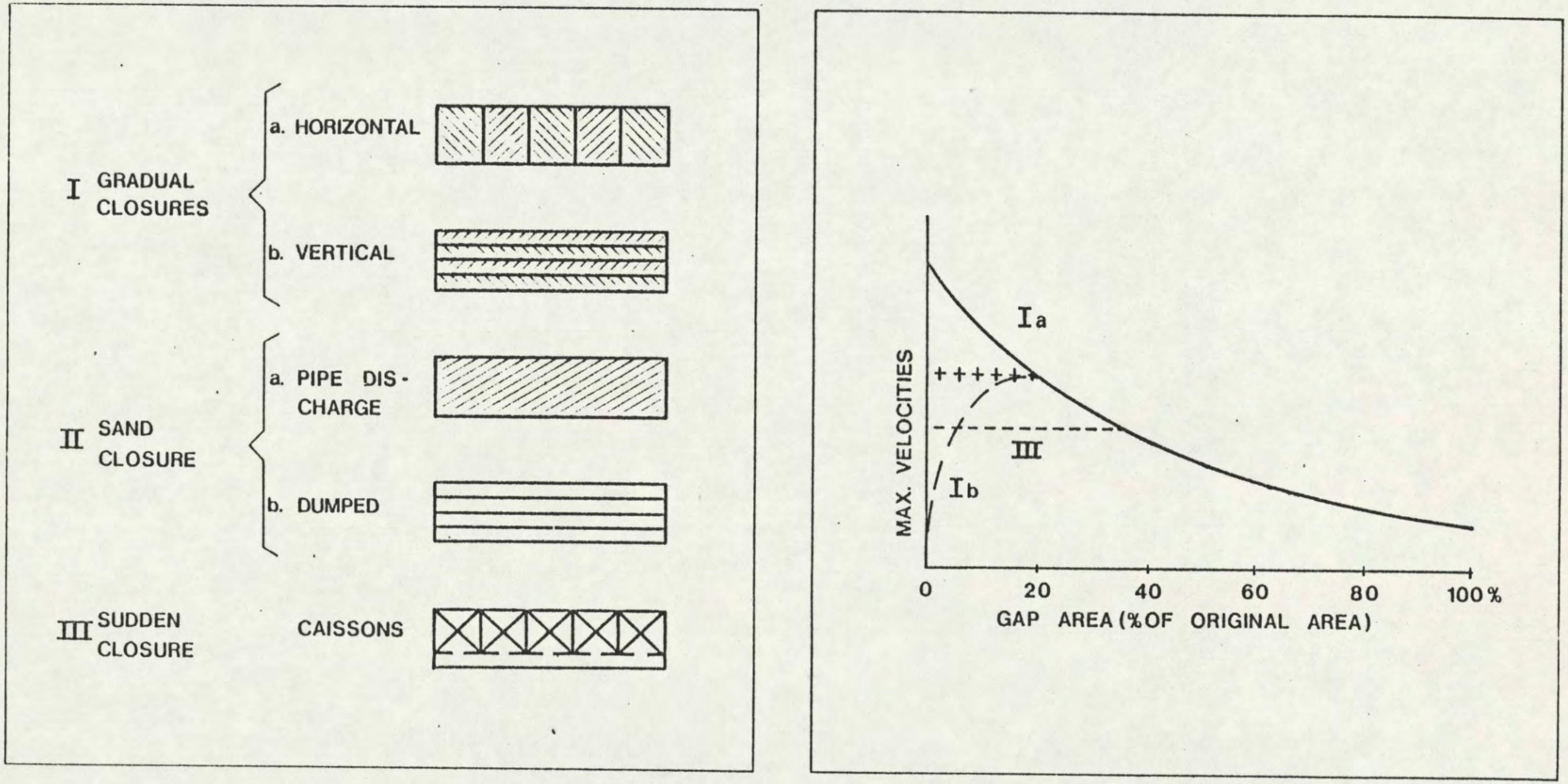

I GRADUAL CLOSURES
a. HORIZONTAL
b. VERTICAL
II SAND CLOSURE
a. PIPE DIS-CHARGE
b. DUMPED
III SUDDEN CLOSURE
CAISSONS
MAX. VELOCITIES
Ia
Ib
III
0
20
40
60
80
100%
GAP AREA (% OF ORIGINAL AREA)

Fig. 3.1.

I. a. Gradual horizontal closure

If a closure gap with a low sill is horizontally constricted the current velocities increase in proportion to the gradual decrease of the cross-section area. Therefore very heavy material will be needed to close the last gap, whereas, at the same time, the bed protections will be heavily attacked.
This method of closure is not applied on a large scale for the closing of tidal basins in areas with a large tidal range.
When closing a gap with a high sill, the sill is raised to such a height that a situation of a clear overfall is reached, this is to say, that the current velocity over the sill will be determined by the height of the sill and not be influenced by the head difference over the barrier.
The horizontal constriction of a closure gap with high sill can be done by dumping glacial clay, bags filled with either sand or clay and rubble or by placing box-type caissons.

I.b. Gradual vertical closure

A gradual vertical closure is usually applied as follows.
First, after the bottom has been protected, the dam is built out from one of the banks. Generally this becomes a damsection on relatively shallow parts. This proces can be continued until the velocity of the current in the remaining gap is 2 to 3 m/s, depending on the materials used in these damsections.
Then, the closure gap is built up in horizontal layers. As a consequence, the velocity increases and, therefore the weight of the stones to be used must also increase. There are various methods of dumping stones, such as floating equipment, a cable way, a bridge or helicopters. During the closure period, the velocities may greatly increase.
By heightening the dam, the total quantity of water flowing over the crest, however, decreases and the velocities on the river bottom may become relatively lower and the scouring effect is weakened.
Usually the larges velocities are found in the upper-layers of the vertical velocity distribution (immediately below the water surface).
The increase in the maximum velocity will stop when the flow over the crest of the embankment becomes critical during maximum flow. The situation of a clear overfall has been reached. Further heightening the embankment will result in a decrease of the maximum velocity until the dam has attained its full height.

II. Sand closure

A requirement for a gradual or sudden closure is that the material to be used should have sufficient weight to resist erosion, whereas for a sand closure the material (sand) has to be supplied in such large quantities, that the main portion is not carried away by the current. If current velocities in the closing gap reach high values (order 3 m/s or more) large scale methods are necessary. Due to the high velocities, expensive bottom protection works are required. If the current velocities are of moderate magnitude (order 2,5 m/s or less) local sand may be used. A part of this material will be lost during the closure operation.
However, the costs of the expensive bottom protection are saved.
Therefore, the losses of sand may well be accepted in the overall economics of the works. Recently, a number of tidal channels, both in the Netherlands and in Germany, were closed succesfully by pumping sand into the gap.
This was mainly possible because of the increased capacity of the modern suction dredgers.

III. Sudden closures.

This method deals with structures which allow the whole gap to be closed suddenly. Normally, these structures are caissons which are placed in the gap during a slack water period and thus close the whole gap at once.
Another solution is the use of sluice caissons which are placed during several subsequent slack water periods, and are kept open during the period over which all the sluice caissons are placed.
After the placing of the final caisson, they are all closed at the same slack water period by means of gates. Sluice caissons should be used when the gap to be closed is a large one and the tidal motion is considerable.
Closed caissons are useful in small gaps, say nog larger than 1 to 3 caissons or in areas where only a very small tidal motion occurs. Characteristic for a sudden closure by means of sluice caissons is to maintain an as wide as possible effective wet area of the closure gap until all the caissons are sunk in position on the sill.
By using open caissons, a great increase of the velocity in the gap is avoided, so that strong scouring does not occur, provided the flow pattern is not too much influenced by the walls of the caissons.

Closure dikes Name	year of design	length km	max.water depth.m.	tidal level[1)] highest m	tidal level[1)] lowest m	max. current velocity m/s	Tidal[1)] Prism 10^6 m3	Closure method gradual horizontal	Closure method gradual vertical	Closure method sudden caissons
Zuider Zee closure dike	1930	32	7	+ 0.5	- 0.5	3.5		x		
Veerse Gat dam	1960	3	17	+ 1.5	- 1.4	4	70			x
Grevelingendam	1962	5	10	+ 1.5	- 1.5	3.5	nil[3)]	x	x	
Lauwerszee closure dike	1967									x
Volkerakdam	1967	5	12	+ 1.7	- 1.7	3.5	nil[3)]			x
Haringvlietdam	1968	4	12	+ 1.3	- 1.3	4	260		x	
Brouwersdam	1969	5	30	+ 1.3	- 1.2	4	300		x	x
Eastern Scheldt	1974	9	35	+ 1.4	- 1.4	4	1100		x[2)]	
Philipsdam	current	8	20	+ 1.8	- 1.8	4.5	200		x	
Oesterdam	current	12	20	+ 2.0	- 2.0	4.5	70		x	

1) at mean tide

2) original design, before the decision for a storm surge barrier

3) secondary dams, not closing off a tidal inlet

3.2. Bottom protection works

The scour which will take place in the vicinity of the enclosure works is the time-integrated resultant of the progressively increasing current velocity. If the sea bed is composed of loose sediments, a certain part of it under and on both sides of the enclosure works will have to be protected in order to ensure their stability. This aspect can be disregarded in the case of a rocky bed or when the silt/sand cover on the rock base is in the order of only a few metres.
Initial designs and subsequent selection of the most appropriate type of revetment have to be carried out.

The main distinction between permeable and impermeable revetments is not always clear.
The distinction depends not only on the construction itself, but also on the characteristics of the load variation and the properties of the subsoil.
The consequence is that one and the same construction can be considered permeable for slow changing water pressures and act as impermeable for rapidly changing water pressures.
Under this proviso, the following tabulation may give a general indication of the various options open to the designer.
The technical realisation of bank and bed protection works will be determined by a number of factors. The most important of these can be formulated as follows:
- availability of construction materials
- availability of labour and equipment
- external boundary conditions at the site.
- interrelationship between labour, equipment and material cost.

The present possibilities of laboratory investigations are such, that new concepts of bottom protections, can be designed and tested in a relatively short period.

- using permeable materials
  - prefabricated construction
    - with loose ballast
      - Fascine mattrasses with quarry stone
      - combined fascine/polypropylene mattrasses with quarry stone
    - with fixed ballast
      - mattrasses with concrete blocks
      - mattrasses with gravel tubes
      - mattrasses with stone asphalt
  - construction in situ
    - cohesive materials
      - sand asphalt
      - stone asphalt
    - non cohesive materials
      - filter layer systems

- using impermeable materials
  - construction in situ
    - natural stone
      - loan
      - clay loan
    - with asphalt
      - asphalt concrete
      - mastic asphalt
      - mastic grouted quarry stone
    - with concrete
      - concrete slabs
      - injected concrete (stabilisation)
    - impermeable cloth
      - plastic sheet with protectionlayer and ballast
      - bituminous membrane

3.3. Temporary works

The temporary works envisaged for an enclosure project consist of:

- access roads
- labor camps
- construction harbour(s) with storage areas
- construction jetty
- building pit for a lock, a discharge sluice or caissons

An adequately designed access road must be provided to facilitate the transport of men, equipment and materials to the construction site: a carefull assessment must be made of maximum size of equipment and maximum loads to be transported to arrive at an economical design of the access road.

The size of the labor camps is determined by the number of men to be housed. Special attention must be paid to the lay-out, so that accessibility, drainage, fire-protection, sanitation, power- and watersupply are realized at minimum cost.
The catering service and recreational facilities must be located in the camps.

A construction harbour is envisaged to serve as bases for the storage and delivery of construction equipment and materials. This equipment can be required for the construction of a lock, a discharge sluice and the enclosure dam. In many cases construction harbours are necessary on artificial islands at the end of damsections on the shallows. The construction sites where elements of the dam are prefabricated (f.i. bottom protection mattrasses) have to be situated in the close neighbourhood of a construction harbour.
A construction jetty may be required in addition to the construction harbour. The location of both the construction harbour and the jetty must be subject of an intensive study taking into consideration the requirements of the enclosure dam, the construction of a lock and a discharge sluice.

The building pit for the lock is a major construction job. In conjunction with the possible alternative locations of the lock a careful economic evaluation must be made to arrive at the most economical solution. Special care must be taken to ensure a proper drainage under all circumstances. Besides the influence of the building pit on the circumstances in the remaining enclosure gap has to be studied. The same applies to the building pit of a discharge sluice or caissons (sudden closure).

4 GRADUAL CLOSURES

4.1. Hydraulic conditions

The gradual closures can be subdivided into three methods, such as:

- horizontal constrictions with a low sill
- horizontal constrictions with a high sill
- vertical constrictions

The most significant difference between these methods is the current velocity in the closure gap, when during the closure the size of the wet cross-section decreases (see fig. 4.1). This figure shows that, particularly during a gradual horizontal closure, on a low will, the current-velocity can be increase considerably. Therefore, this method can only be applied to provided the head over the closure-gap is not too great.
With the gradual horizontal constriction on a high sill, the current-velocity follows from the formula for sub critical flow: $V = m = \sqrt{2g(H-h)}$
where: V - average current velocity in the closure gap (m/s)

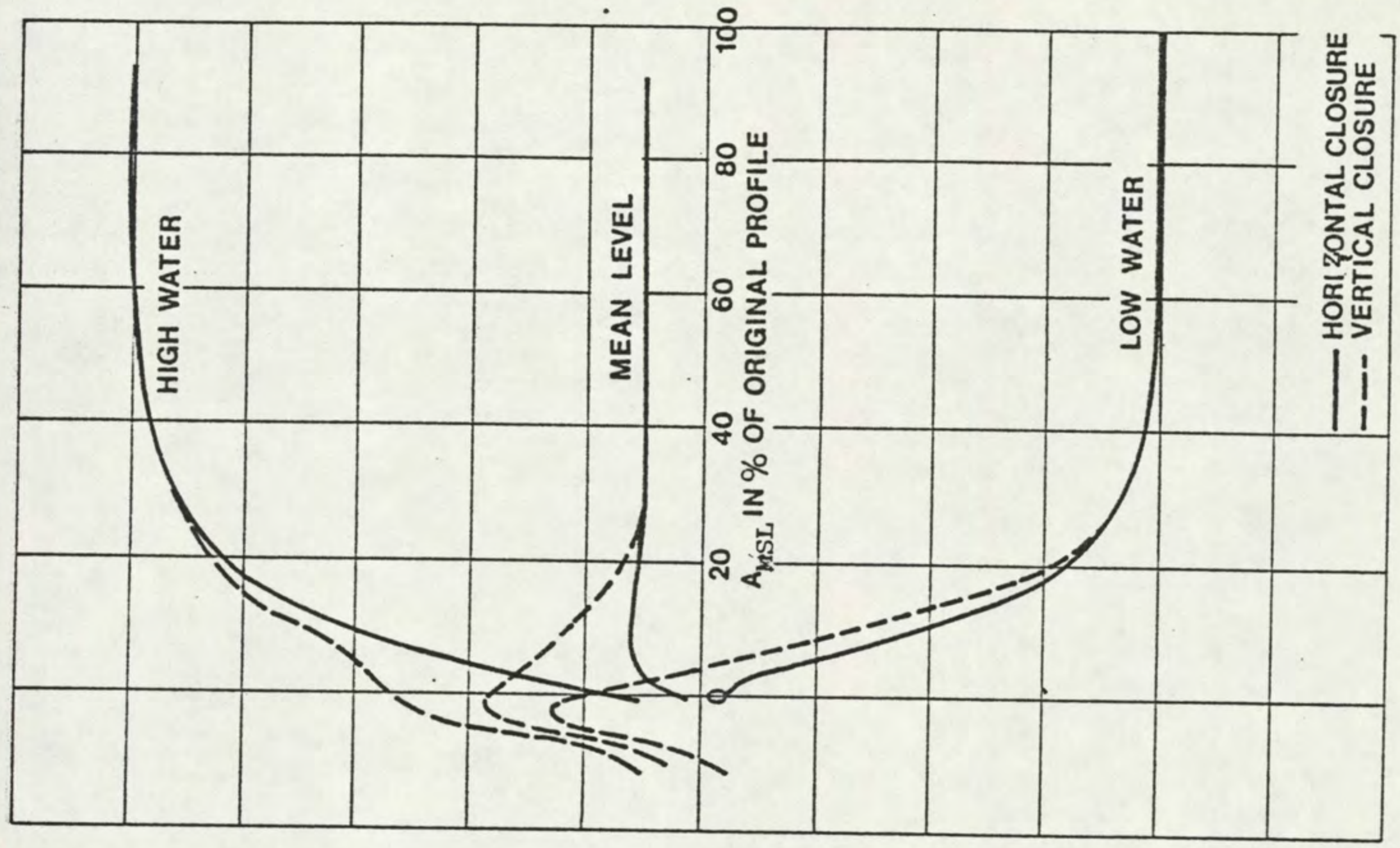
	m - discharge coëfficient	(-)
	g - acceleration of gravity	(m/s^2)
	H - waterlevel upstreams	(m)
	h - waterlevel downstreams	(m)

With a gradual horizontal constriction on a high sill, the sill is raised to such a height, that a situation of a clear overfall is reached. This means that the current velocity over the sill will be determined by the height of the sill, and not be influenced by the head difference over the barrier.
This follows from the formula for super critical flow:

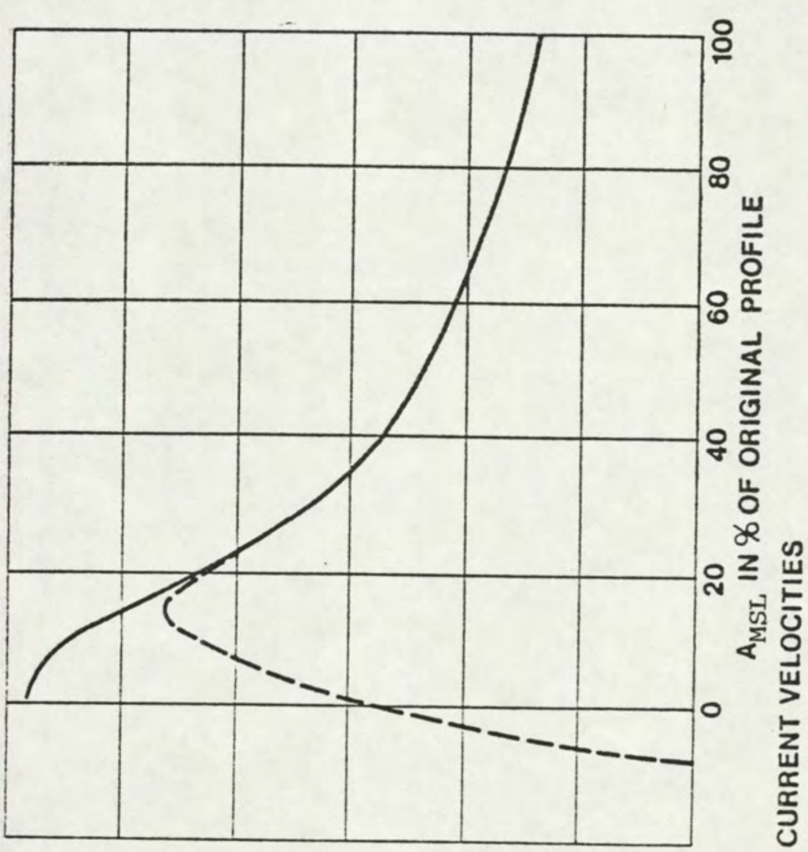
$V = m\sqrt{\frac{2}{3}gH}$, where H - height of the waterlevel upstreams above the crest of the sill (m)

Characteristic for a gradual vertical closure is the building up of the embankment over its whole length. It is possible to build up in horizontal layers, but also to follow the shape of the cross section of the channel to be closed.
During the closure period, the velocities may greatly increase. As a consequence therefore the weight of the materials to be used must also increase.
The increase in the maximum velocity will stop when the flow over the crest of the embankment becomes critical during maximum flow. The situation of a clear overfall has been reached. Further heightening the embankment

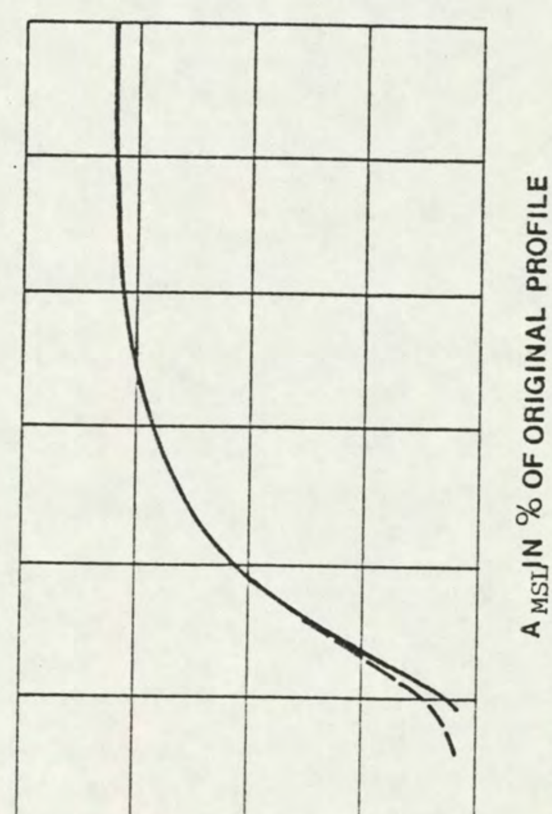
will result in a decrease of the maximum velocity until the dam has attained its full height.
An advantage of this method of closing is the decreasing of the velocities at the riverbottom during the heightening of the embankment; usually the largest velocities are found in the upper-layers of the vertical velocity-distribution, depending on the height of the sill and the waterdepth.



WATERLEVELS



CURRENT VELOCITIES



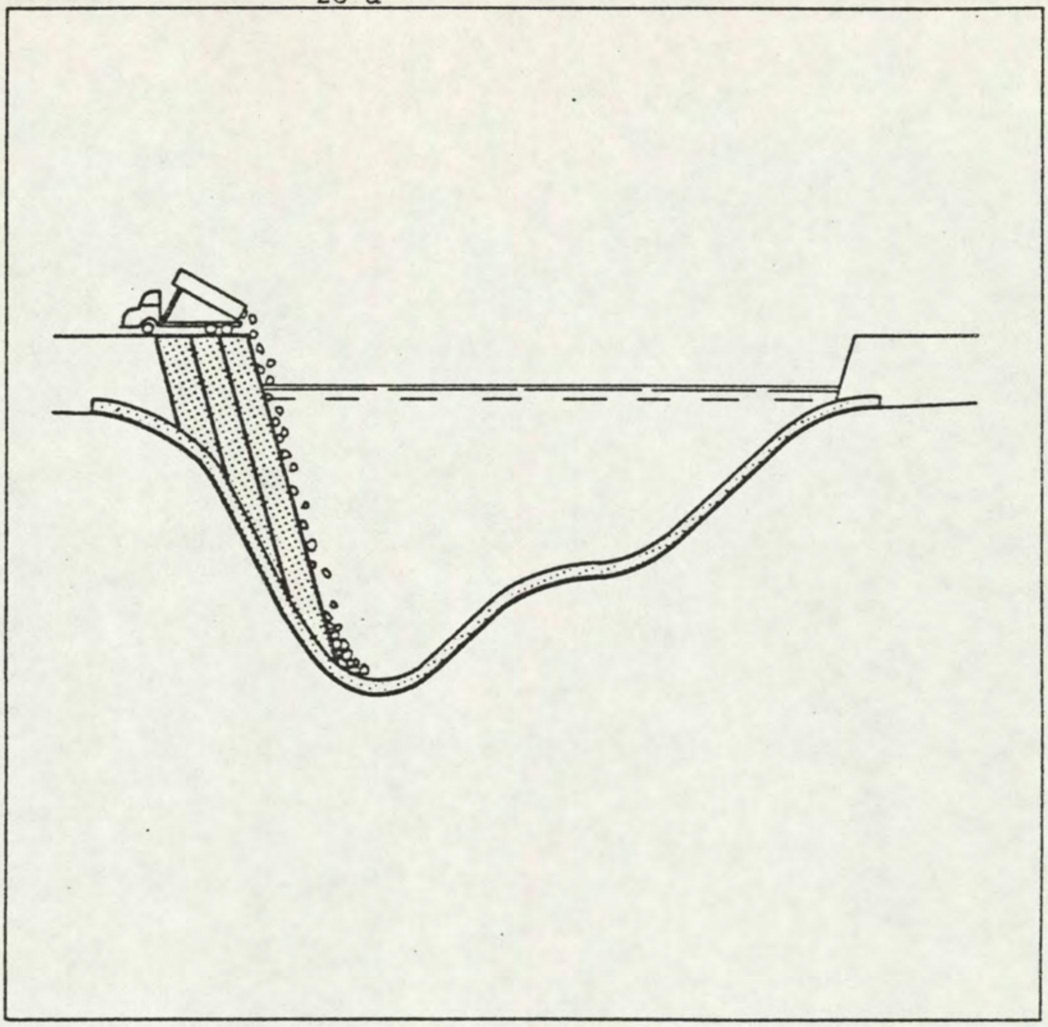
DISCHARGES

4.2. Construction methods

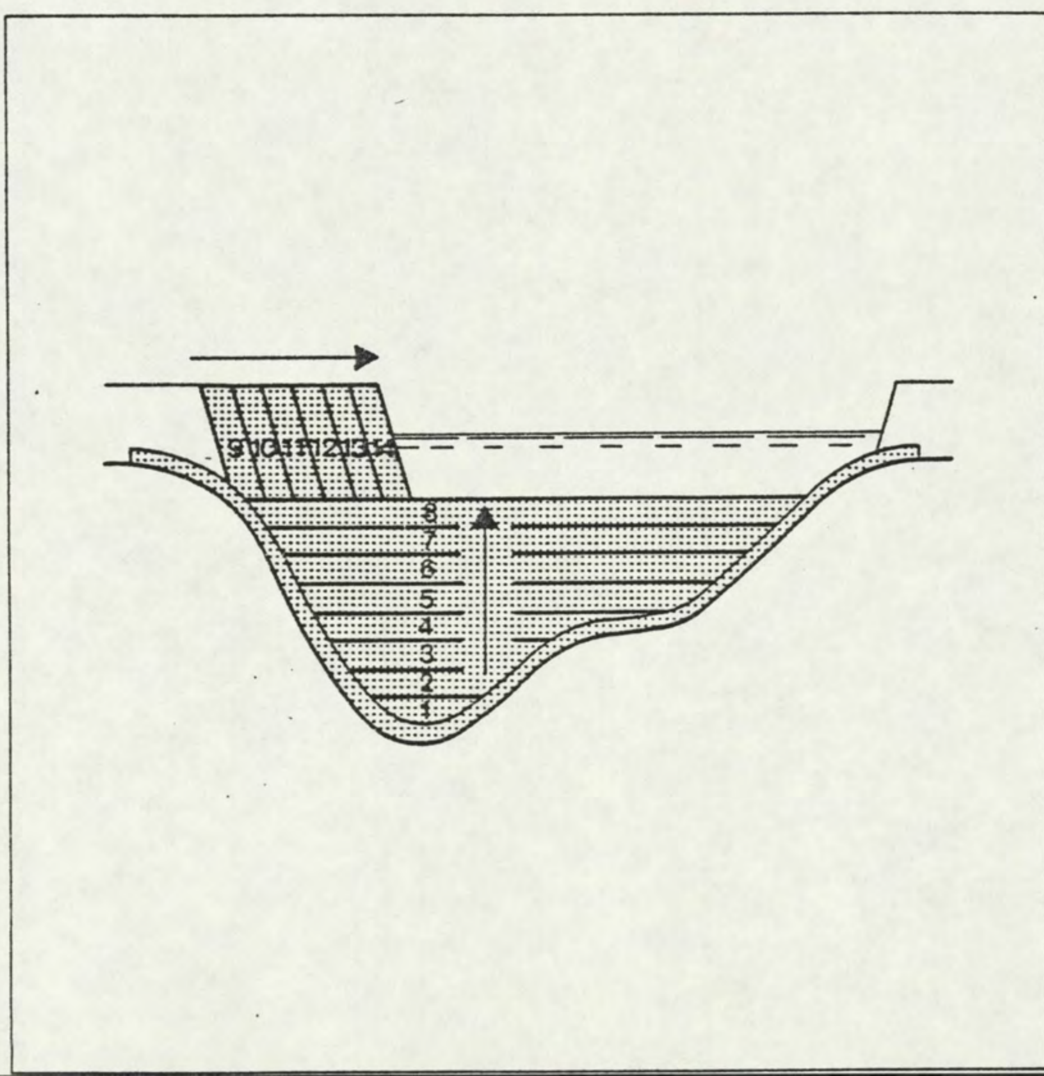
The materials and equipment that can be applied at gradual closures, depend strongly on the scale and local circumstances.
As much as possible local methods and materials have to be used for closing tidal basins.

The following methods can be applied:

- horizontal constriction (narrowing the gap from the sides):
  - dumping gravel, clay, rubble or bags filled with either sand or clay , over the head of damsections by manual labour, trucks or cranes
  - placing tight box-type caissons
- vertical constriction:
  - dumping rocky materials (gravel, stones, concrete blocks or rock units), clay or sandbags by
    - . manual labour or trucks over a causeway
    - . floating equipment, such as stone dumping vessels or floating cranes
    - . cableway
    - . helicopters



horizontal construction dumping over the dam heads



combined vertical and horizontal construction

Horizontal constriction by means of manual labour

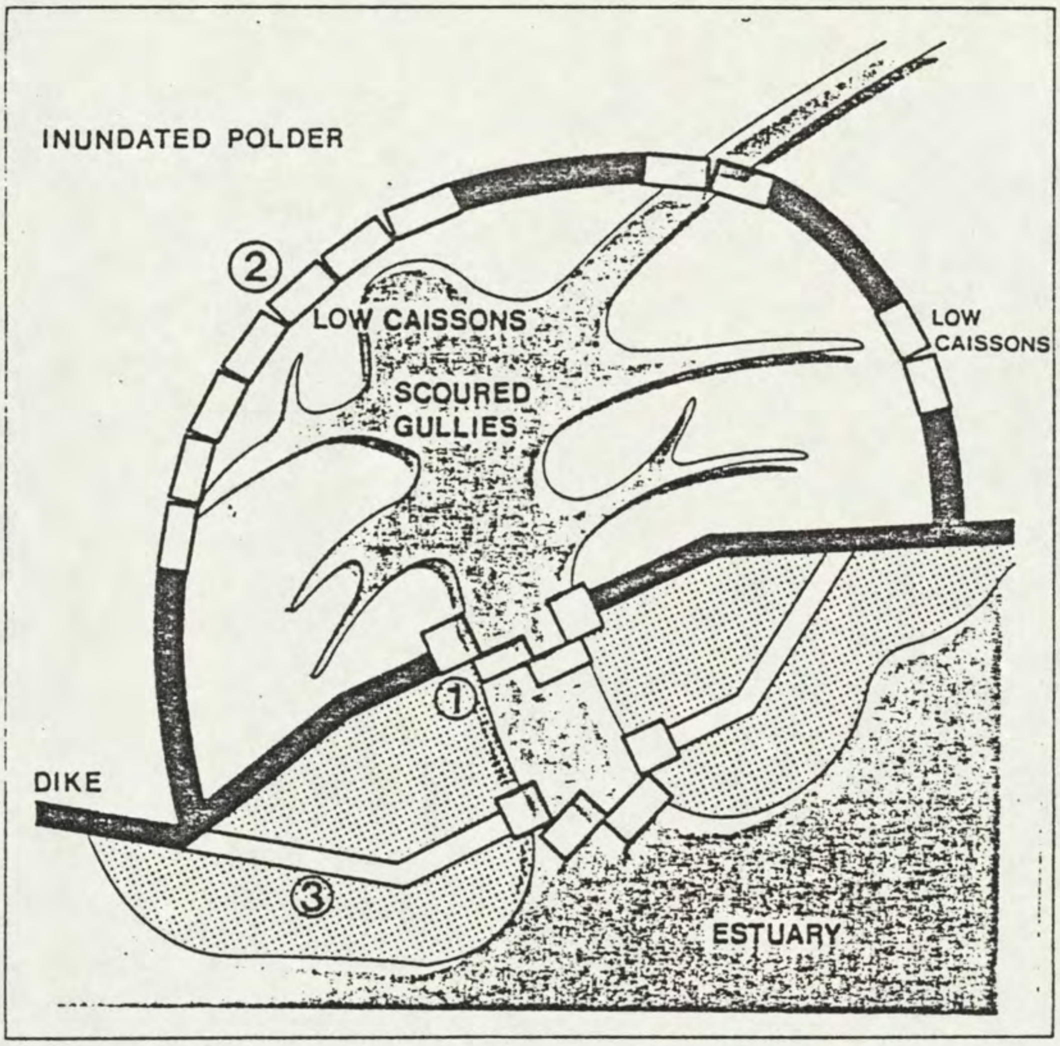
For instance, to repair the dike on the isle of Tholen (Neth.) in February and March 1953 after the flood disaster, bags filled with sand and clay were used as filling material. About 3.400 people were required to carry the sand-bags manually. A high capacity was obtained by circuit walking. Within a fortnight 1,75 million sand-bags were transported and in total 2,35 million bags were used in this operation. As the normal methods of supplying these bags would have required too much time, they were dropped at various places by aeroplanes.

Another example is the method developped in Bangla Desh (Bay of Bengal) to close small channels.
The local method involves the horizontal construction of dams from both banks. The dams are composed of compartments of widely spaced timber piles filled with large rolls of straw, palmleaves and clay.
The rolls generally do not disintegrate in the current and the clay is retained. This method is suitable provided that the ultimate closure-gap is sufficiently small to be closed during one low water period, while still being sufficiently large to avoid very high watervelocities which would cause scouring of the channel bottom and the dam heads.

Horizontal constriction using tight caissons

This method has been used for the secondary dams (S.W.Netherlands) in the Zandkreek (1960) and the southern channel of the Grevelingen Dam (1962). In this way a gradual horizontal constriction was achieved on a sill with a depth of M.S.L. - 5 m, by placing compact caisson-units (concrete, box-like elements approx. 11 m long, approx. 7.5 m wide and approx.6 m high). As no tidal basins were closed here, the differences in water level over the retaining dams were limited to a few decimeters only. Consequently, the maximum current velocities did not increase by more than approx. 2 m/s in the final closure gap.
In 1953 caisson-units were built for the first time as a means of closing the dike breaches caused by the flood disaster earlier that year.
At that time it was impossible to determine at short notice the dimensions and the necessary number of caissons.

When a dike breach occurs, the closure can be done either at the flow-gap or behind the flow gap at ground level (see fig.4.3). The reconstruction of the demaged dike along its original course, involves the closing of gullies. The construction of a "horse-shoe" dike round the gap on the land side involves a wide gap on ground level.
A striking example of the application of caisson-units on a high "sill" is the closure at ground level at Schelphoek in August 1953.
At Schelphoek, the breach in the coastal dike formed a flow gap 525 m long, with a maximum depth of 37 m. This gap was closed by a "hors shoe" shaped (an inter-section) dike 4 kilometers long for which, in total, 235 caisson-units were used.
To prevent seepage under the generally narrow bottom protections (40 to 50 m) the closure of a ground level gap had to take place at high speed. When closing the gap at Schelphoek, 462 meters of caissons (42 units, 11 m long) were placed within 24 hours. While closing the gap at ground level, it appeared that the soil in situ, sandy clay, needed a bed protection of fascines, what in its turn caused the current velocity at groundlevel to increase up to 1,5 to 2 m/s. It was intended to select a sill with such a height that, at the peak low water level, current velocities of 3 m/s would not be exceeded. However, because of the altitude of the ground level and the thickness of the fascines, these demans could not always be met; the velocities exceeded by far the 3 m/s standard. However, the bed protection was not severely damaged.

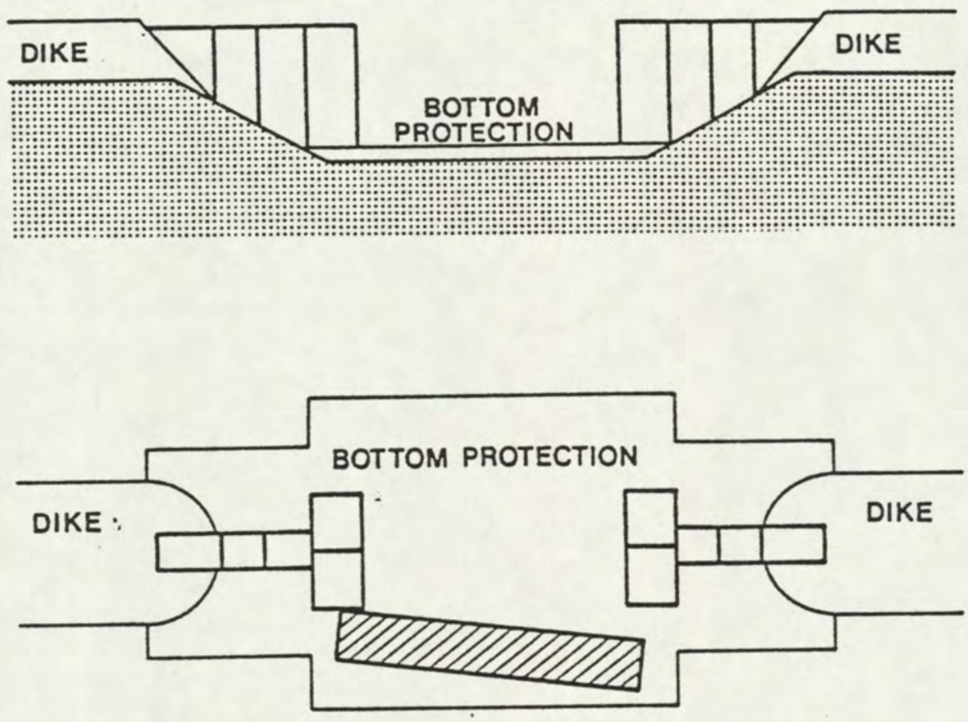


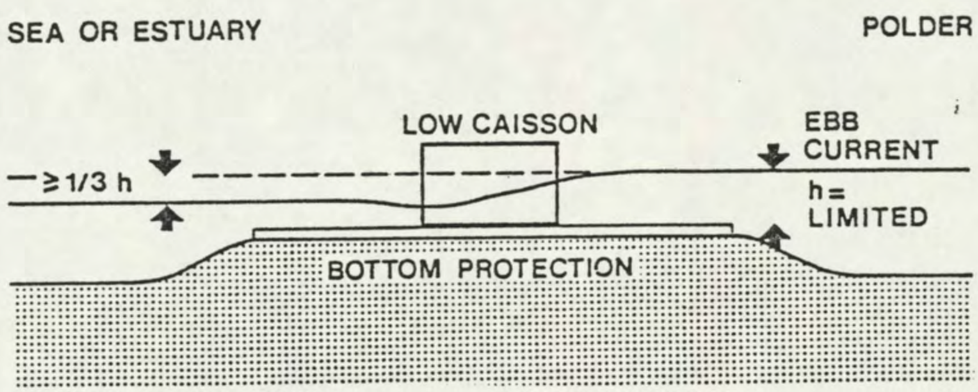
1. By reconstructing the damaged dike along its original course, which involves the closing of gullies.

2. By constructing a "horse-shoe" dike round the gap on the land side, which involves a wide gap on ground level.

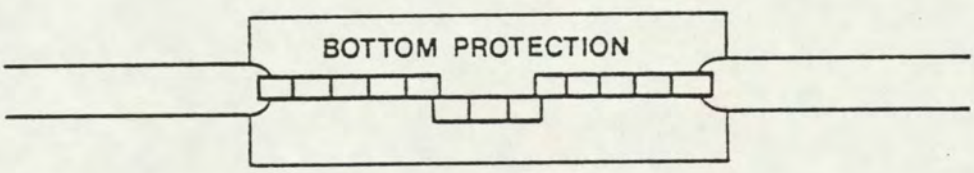
3. By constructing a "horse shoe" dike along the shallows outside the original dike, which involves the closing of gullies.

CLOSURE GAP WITH LOW SILL





CLOSURE GAP WITH HIGH SILL AT GROUND LEVEL; SITUATION OF CLEAR OVERFALL



Vertical constrictions

A gradual vertical closure is generally divided into the next main construction-stages:

1. a stable, scour resistant sea bed is necessary at the gap and protection of the sea bottom by a revetment is the first stage, if the bed consists of loosely packed sediment.
2. most times building up a sill with the function of pressure distribution in the subsoil (if necessary) or the function of a filter between the bottom revetment and the materials of the closure embankment.
3. closure materials are dumped into the sea, thus gradually forming an embankment along the length of the gap. The embankment is heightened evenly along its length, until it emerges above the H.W.level.
4. when the closure embankment has been built up enough to stop the tide, the permanent dam is built around it.

Equipment for a gradual vertical closure

1. Manual labour or trucks over a causeway.

advantages:

- building up in horizontal layers, depending on the construction of the causeway
- relatively cheap

disadvantage:

- only applicable at relatively shallow channels

2. Stone dumping vessels or floating cranes

advantages:

- building up the embankment in horizontal layers;
- less materials needed in the last critical closing stage
- relatively cheap

disadvange:

- low capacity (decreasing until zero when using vessels) in the last stage (from about 2 m below L.W.), because the vessels can only cross the embankment at H.W. and when the current velocities are sufficiently low.

3. Cableway, helicopter, trucks over a bridge
   advantage :
   - high capacity during all stages

   disadvantages:
   - expensive
   - many stones or concrete blocks to be dropped in the final stage
   - more scouring

4. First: stone dumping vessels
   Later: cranes, trucks, cableway or helicopters
   advantage :
   - high capacity during all stages

   disadvantage :
   - two types of closure equipment

Floating equipment

On a smaller scale a vertical constriction of the closure gap can be achieved by dumping clay, sand-bags or small grained rubble. In 1870 already the Sloe (the Netherlands) was closed off by the classical method of sinking matresses above each other. A disadvantage of this method is, however, that at the final closings the slack water period is too short for the sinking manoeuvres.
However, dumping-sites of glacial clay with a height from 5 up to 8 meters or more will show a tendency to slump.
To transport and dump huge quantities of rubble and concrete blocks, stone-dumping vessels and floating cranes can be used as equipment.
An advantge of using this equipment is, that the retaining dam can be built up in horizontal layers, thus avoiding that at the last critical closing stage large quantities of rubble must be dumped in the closure gap. The side slopes of a retaining dam, consisting of rubble, has an angle of broadly 1:1,5. This implies that with a low-lying sill already approx. 80% of the materials has been dumped before a situation of a clear overfall is generated.
On the other hand, however, it is a fact that the capacity of the floating equipment is hampered at the final stages of closing and thus reduced.
Stone dumping vessels are only able to float over the retaining dam during the high turn of tide, due to the draught and the highly increased current velocities. Floating cranes have a relative low dumping capacity and are unable to anchor behind the closure gap due to the high current velocities.
The use of floating equipment, such as split barges, elevator barges or stone dumping vessels to close a channel vertical, is quite an easy and cheap solution until the crest of the embankment reaches about 2 to 3 m below L.W.
The use of floating equipment during the final stage of a gradual closure, requires adaptation of the execution method with regard to the increasing current velocities.

Example:

Stone dumping vessel , data:

carrying capacity	700 to 800 ton
load draught	2,5 m
length	55 m
beam	11 m

Flow resistance $= \frac{1}{2} C_w \rho_w V^2 . A \approx 80$ to $110^{x)}$ V^2 (KN)

Skin friction $= \frac{1}{2} C_f \rho_w V^2 . S \approx 3$ to 4 V^2 (KN)

Energy gradient force $\approx G \sin \alpha \approx \dfrac{1 \text{ to } 10 \quad V^2 \text{ (KN)}}{85 \text{ to } 115^{x)} \quad V^2 \text{ (KN)}}$

x) data for transverse flow

V = current velocity (m/s)

Depending on the anchoring system stone dumping from the vessel is possible up to current velocities of:

* 1,5 to 1,8 m/s under transverse current attack
* 2,5 to 3 m/s under head current attack

If the distance from the dumping site to the depot is about 1 km, the average cycle period of 1 stone dumping vessel of 700 ton carrying capacity will be approximately 4,5 hours.

The capacity of one vessel will be about 2700 ton/day provided that dumping is possible during the whole tidal cycle.

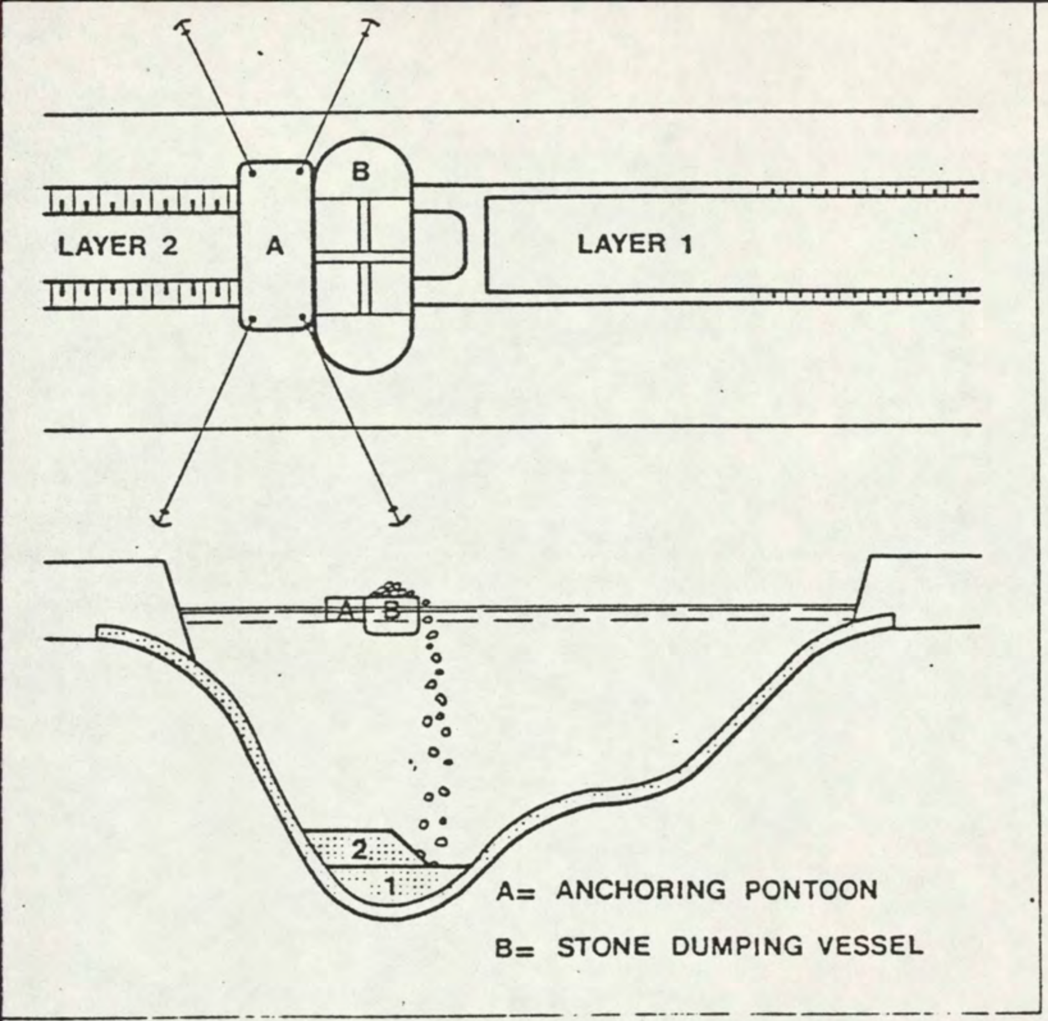
If, however, dumping is only possible, due to the draught and the current velocities, during slack H.W. the capacity of one vessel decreases to 1400 ton/day (diurnal tide).

If the tidal range is 3,5 m, the maximum height of the crest constructed by using stone dumping vessels is approximately 1,5 m to 2 m below M.S.L. The critical state for maximum ebb and flood has been reached (clear overfall).

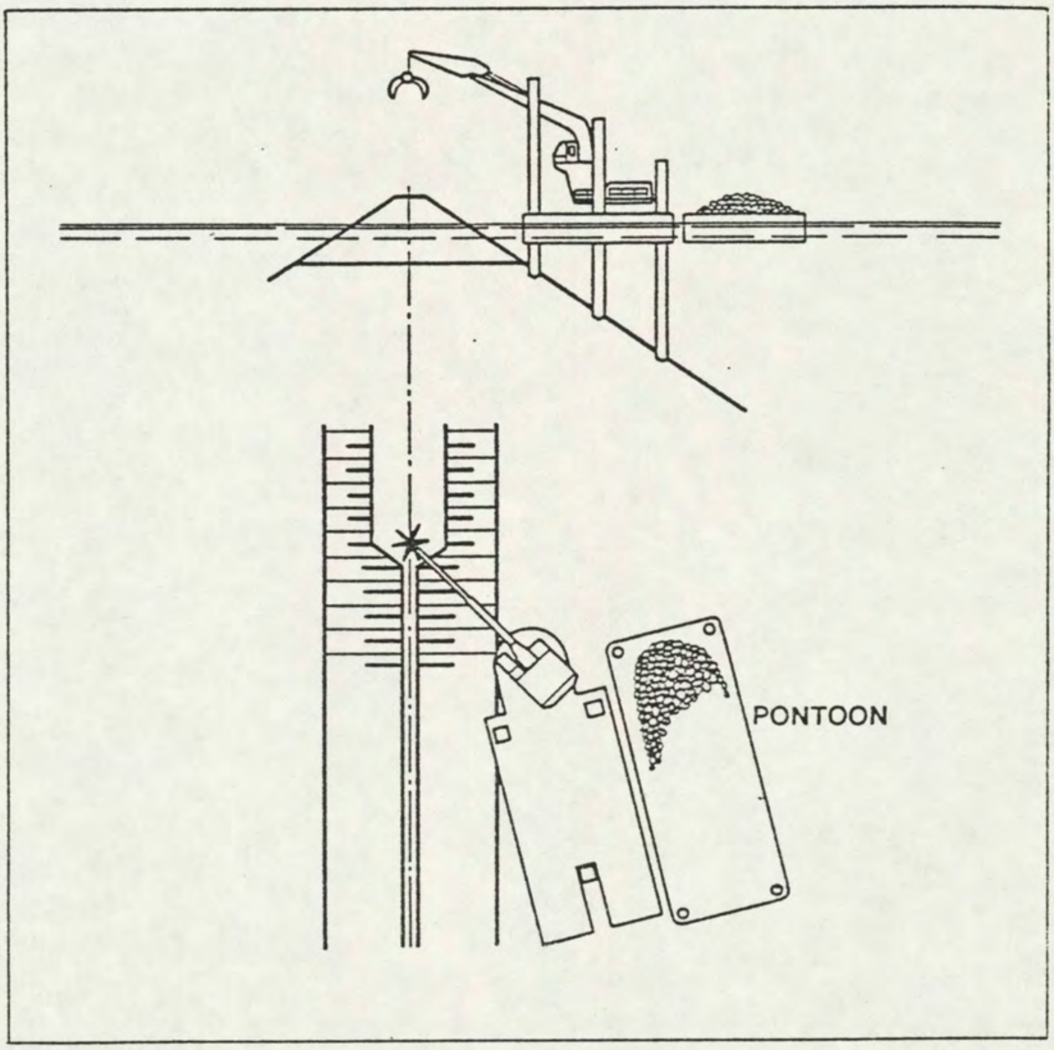
Then the final closure takes place by narrowing the gap from the sides, e.g. by placing caissons on the sill or by using stones. In that case the maximum velocities will remain constant for similar tides. If stones are used to narrow the gap horizontally floating cranes or backhoe dredgers can be used as equipment.

This combined vertical and horizontal closure is, in fact, a horizontal closure on a high sill.

An other possibility is to close the final stage vertical with stones by using hydraulic cranes on the crest of the closure embankment during low water. Experience was gaines with this method during the closure of an eastern part in the Eastern Scheldt (near Bergen op Zoom) in 1980 (see photograph).



Floating equipment



Gambins toplayer with floating equipment

Cable way

In view of the disadvantages of the floating equipment cable-ways have been applied for closing the Grevelingendam (1963), the Haringvlietdam (1970) and the southern channel of the Brouwersdam (1971) in the Netherlands. During the gradual closings in the Haringvliet and the Grevelingen the tidal differences were still rather small: respectively 60 and 90 cm. In relation to the occurring tidal differences in these waters the drop in head over the embankments was comparatively small; in the Grevelingen this was due to the fact that the area at the rear was not closed off, whereas in the Haringvliet the tidal range could be limited by opening the large discharge sluices.

In the Brouwershavense Gat the tidal range and consequently the current-velocity was considerably higher as here a closing took place in an area sealed off at the rear; therefore there was a close relation between the tidal difference and the tidal range. Many model tests were carried out to determine the best dumping material.Because rubble is hardly available in the Netherlands, most of the tests are carried out using concrete blocks; the cube appears to be the most appropriate shape for these blocks.

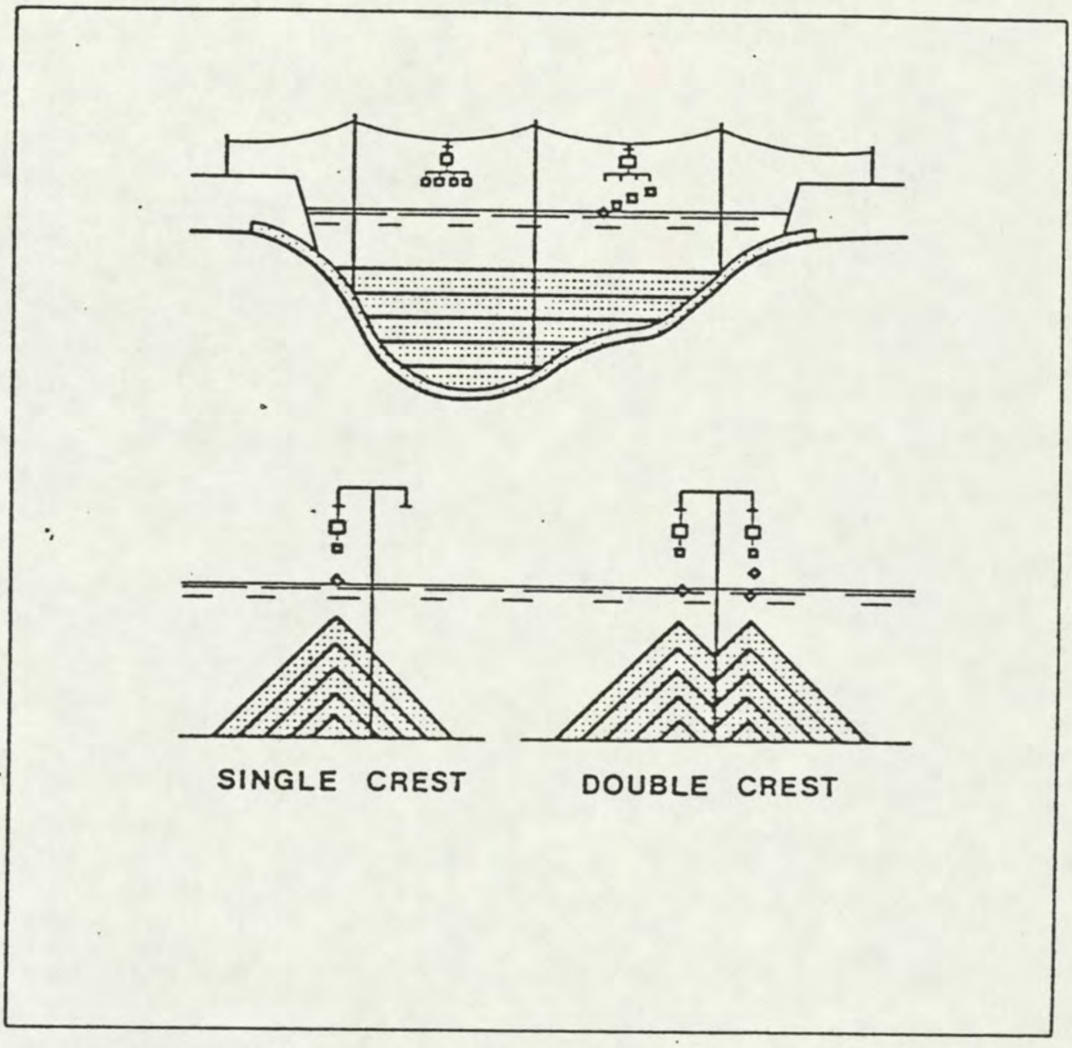
As can be seen from table 1, the dumping capacity could still be further increased in three gaps closed by means of cable-ways.

Table 1. Closings in the south western Netherlands by means of cable-ways.

	material	dumping capacity
Northern channel Grevelingendam 1963	rubble 60 - 300 kg in loading-nets	120 ton/h.
Rak of Scheelhoek Haringvlietdam 1970	4 concrete blocks of 2,5 ton per telpher (own weight 20 ton)	300 ton/h.
Southern channel Brouwersdam 1971	6 concrete blocks of 2,5 ton per telpher (own weight 17 ton)	1.000 ton/h.

A cableway consists of a fixed rail-structure on each side of the gap to be closed.

Between these structures tow cables (Haringvliet closure: approx. 92 mm thick and 6 m apart) are stretched. At the Harinvlietdam and Brouwersdam cableways the distance between the two rail structures was divided into four by three intermediate supports, with resultant spans of 80, 560, 580 and 110 m (Haringvliet).



cross section closure embankment
using a cableway.

The cables were independent of eachother, and were anchored at one end and tensioned at the other end by a tensioning weight or counterweight.
They were anchored by means of a heavy concrete block, pretensioning cables and a cast steel anchorage seating.
The average working tension of 300 tons in a cable was maintained by a 300 ton weight, that was able to move vertically in a shaft on one of the shores.
The cables slid over supports fitted with bronze linings through the fixed rail structures and towers.
Depending on the size of the load and the place of the load in the span, the cable more or less sagged, so that it had to be able to move to and fro over the points of support. As a result of the ever-changing cable length required in the spans through conveing one or more loads, the counter weight moved up- and downwards in the shaft.

The construction of a cableway is in general such, that only one telpher could be travelling along one span at any time. To get a high dumping capacity loading points are required on each shore.
These loading points are installed in the fixed rail structures, where the telphers passed along the flange of a rail section.
The telphers could travel a circuit, i.e. making the outward trip along one cable, describing a half-circle turn at the other end and returning on the other cable.
The vehicles that transported the concrete blocks, measuring 1,04 m3 and weighing 2,5 tons each, from the depot to the loading points are normal trucks, whose body have to be replaced by a special frame in which the blocks are placed.
If a single crested closure embankment will be stable enough and the telphers follow a circuit and load on two shores, it will not of course be possible to make a tontinuous crest. Around the half-way point a connection section (6 to 7 m) has then to be made, this being the distance between the cables.
Due to the capacity the circuit operation has to maintain as long as possible.

For determining the exact position at which the telhper has to drop its load use can be made of a metre counter in the telpher and a specially developped guide light system. The guide light system is favourable for the telpher-operations, except for the odd occasion when for make this impossible.

Analysis of the vertical closure methods for the Philipsdams main channel "Krammer"

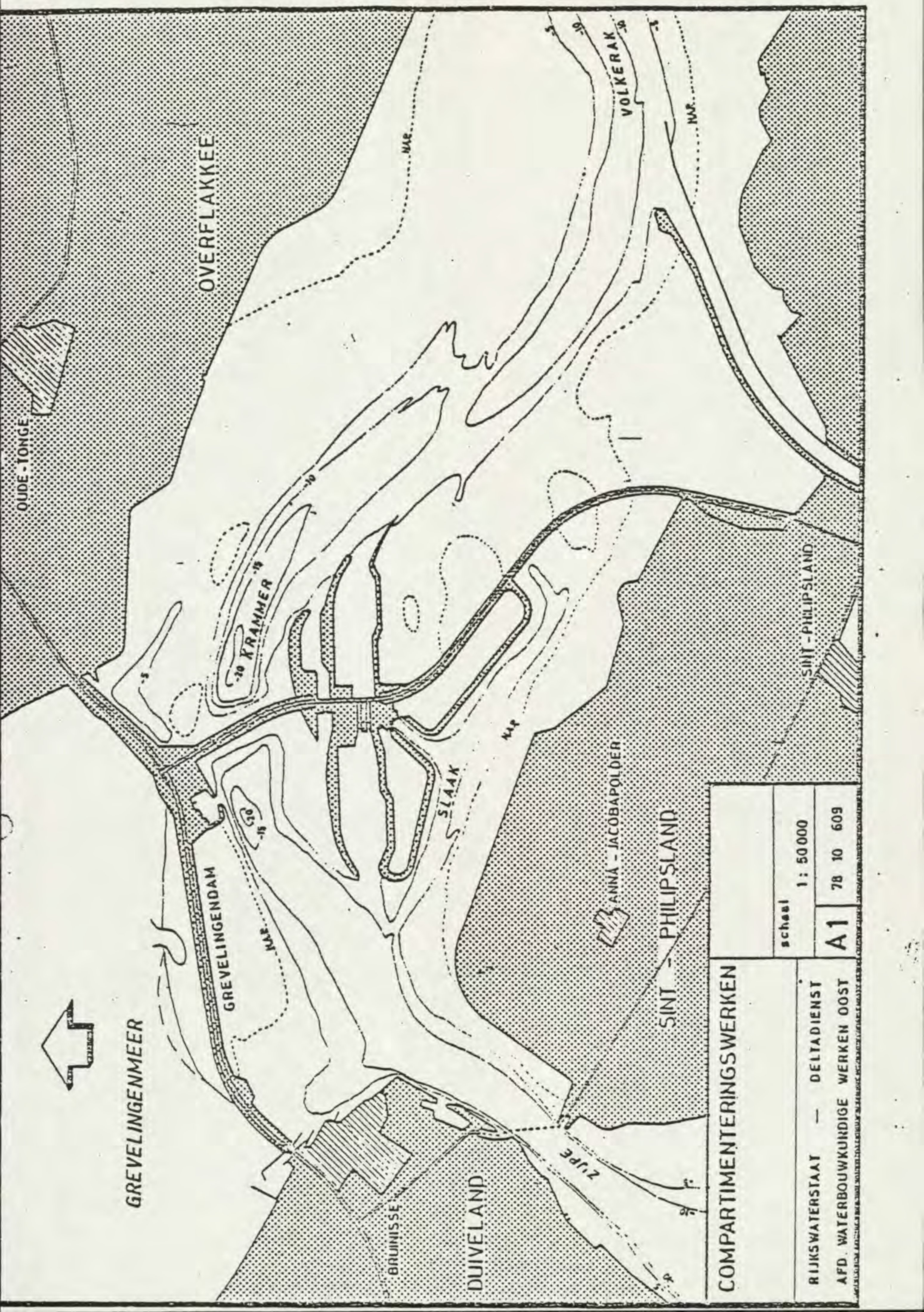
The channel Krammer is to be closed in 1986. A cross section of the channel is given in fig.

At the end of 1984 - after building dam sections and locks on the shallow parts and closing with sand the smaller channel Slaak - there are the following main data:

breadth on M.S.L.	~ 900 m.
max. depth	19 m - M.S.L.
Wet cross section	~ 12.000 m2
max. flood discharge	13.500 m3/s
max.ebb discharge	13.500 m3/s
max.flood velocity	1,8 m/s
max. ebb velocity	2,0 m/s

The next alternatives are studied:
Gradual vertical closures by means of:

a. cable way
b. working bridge
c. helicopters


OUDE-TONGE
OVERFLAKKEE
VOLKERAK
KRAMMER
GREVELINGENMEER
GREVELINGENDAM
SLAAK
ANNA-JACOBAPOLDER
SINT-PHILIPSLAND
SINT - PHILIPSLAND
BRUINISSE
DUIVELAND
ZIJPE
COMPARTIMENTERINGSWERKEN
schaal 1: 50000
A1
78 10 609
RIJKSWATERSTAAT — DELTADIENST
AFD. WATERBOUWKUNDIGE WERKEN OOST

Gradual vertical closure

Material: concrete blocks 1 x 1 x 1 m3 (2,5 t.)

Single crested embankment.

Height of the crest M.S.L. + 3 m.

Required number of blocks 125.000.

a. Cable way

filling station
with fast way

cable way : 3 pillars, 2 filling stations with fast way, circuit service, 2 ropes

8 telphers : self propelled, capacity 6 blocks (15 t.)

Capacity

telpher velocity over the ropes:	3,5 m/s	driving time rope 275 s
length of ropes	900 m	
telpher velocity over fast way:	1,5 m/s	driving time fast way 123 s
length of fast way :	185 m	
loading	70 s	
turning	70 s	

Theoretical circuit period: 2(257 + 123 + 70 + 70) = 1040 s

Working coëfficient 0,7

Practical circuit period: $\frac{1040}{0,7} = 1485$ s

Practical capacity: $\frac{3600}{1485} \times \underbrace{8}_{\text{telphers}} \times \underbrace{6}_{\text{blocks/telpher}} \times \underbrace{2}_{\text{times loading}} = 230$ blocks/h.

Closure duration $\frac{125.000 \text{ bl.}}{230 \text{ bl./h.}} = 540$ hours 6 weeks

b. <u>Working bridge over the channel</u>

- Single way
- 10 trucks in each direction
- 6 blocks (15 t.) on a truck: side dumper

Bridge sections l = 100 m

→ 11 sections → 10 pillars

Capacity

- = truck

o = loading place

Loading capacity: hydraulic cranes:

$$\left.\begin{array}{r}\text{40 blocks/hour}\\ \text{6 blocks/truck}\end{array}\right\}\quad\begin{array}{l}\text{loading time}\\ \text{9 min./truck}\end{array}$$

2 sets of 10 trucks: 1 set driving + 1 set loading

10 hydraulic cranes on each side

Capacity $10 \times 6 \times \frac{60}{9} = 400$ blocks/hour

Closure duration $\frac{125.000 \text{ bl.}}{400 \text{ bl/h}} = 310$ hours $\sim 3\frac{1}{2}$ week

c. Helicopters

1. Carrying capacity 1 helicopter " 4 blocks (10 ton)

   Capacity (see diagram page 42)

   $$\text{flights/hour} : \frac{450}{0{,}01\ d + 6}$$

   depots of blocks on both sides 100 m from the banks.

   From both sides dumping $\rightarrow$ average flight distance d $=$ 230 m.

   Capacity:

   $$1 \text{ helicopter net flights/hour} = \frac{450}{0{,}01.230+6} = 55$$

   capacity 4 x 55 = 220 bl/hour.

   in summer about 60 to 70 flight hours/week

   $$\text{Closure duration } \frac{125.000}{220} = 570 \text{ hours} \sim 9 \text{ weeks with one helicopter}$$

   If 2 helicopters are applied: closure duration: 4,5 weeks

2. First: stone dumping vessels until crest of embankment 5 m - M.S.L.

   $\sim$ 90.000 blocks

   Later: helicopters $\sim$ 35.000 blocks

   - elevator barges 400 blocks/barge

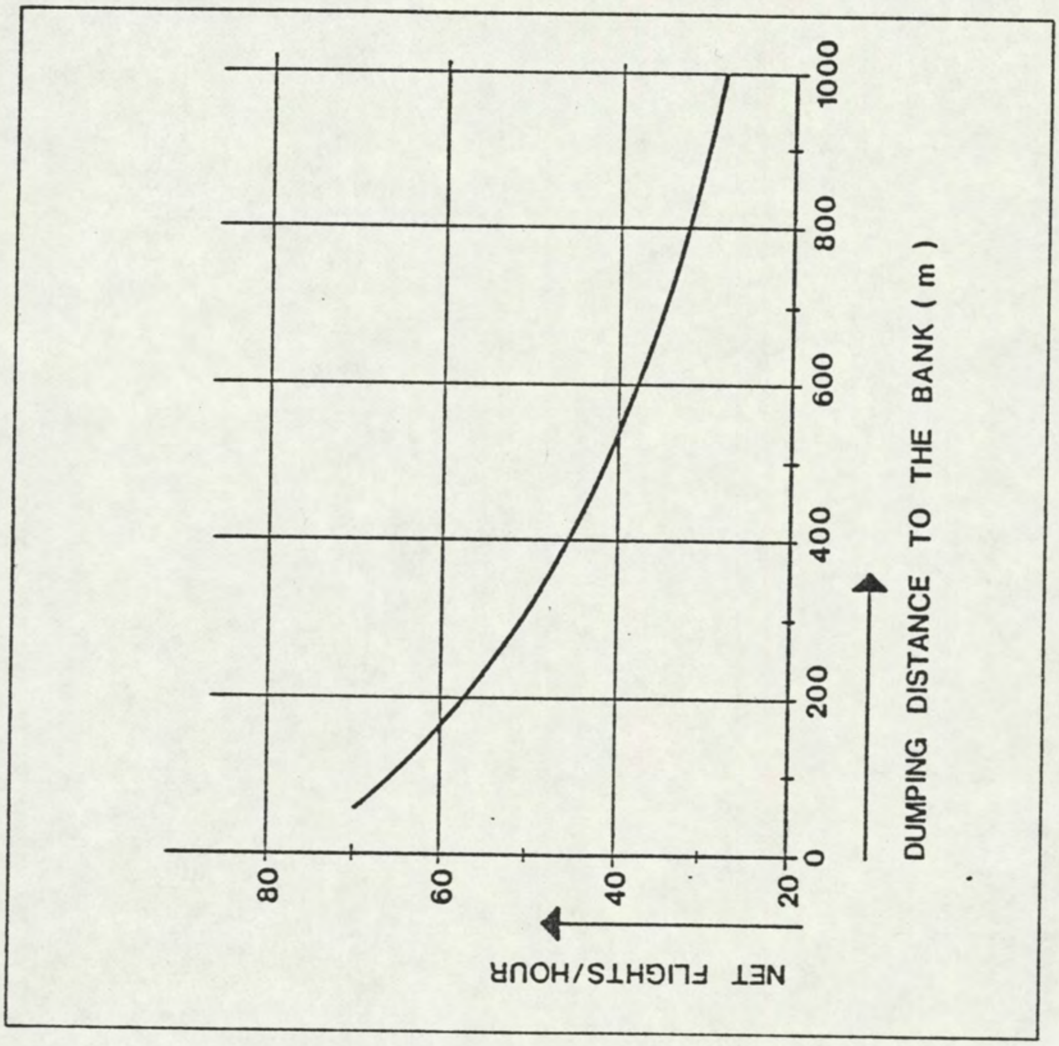
     1 barge $\longrightarrow$ 4 dumpings/week

     4 barges from each side (work-harbours): 32 dumpings/week $\longrightarrow$ 12.800 blocks/week

     $$\text{duration } \frac{90.000}{12.800} \sim 7 \text{ weeks}$$

   - 1 helicopter 220 bl./hour

     $$\text{duration } \frac{35.000}{220} = 160 \text{ hours} \sim 2{,}5 \text{ weeks}$$


NET FLIGHTS/HOUR
80
60
40
20
0
200
400
600
800
1000
DUMPING DISTANCE TO THE BANK (m)

III. 5 SAND CLOSURE

Sand closure general

The sandclosure is carried out by supplying sand either by a boat from which the sand is dumped, or by a pipeline that takes a watersediment suspension to the head of the dam.
The suspension runs off over the fill, where the sand deposits due to decelerating watervelocities and will build out the dam in that way.
Due to narrowing the closure gap, which causes increasing tidal velocities, and due to the fact that a part of the sediment supplied by the pipe will reach the closure gap in suspension, a loss of sand will appear.
A distinction is made between gross and nett loss of sand. The gross loss is the sane that deposits outside of the closure dam profile and is of importance for the time involved in the closure operation. The nett loss is the sand that is taken beyond the profile of the final dam and will determine the actual loss.
In case of an upper-discharge the ebb flow will be stronger than the flood below and will hold longer during a tidal cycle. By shifting the ax of the closure dam against the direction of the upper-discharge, the nett loss can be minimized (fig.5.1.).

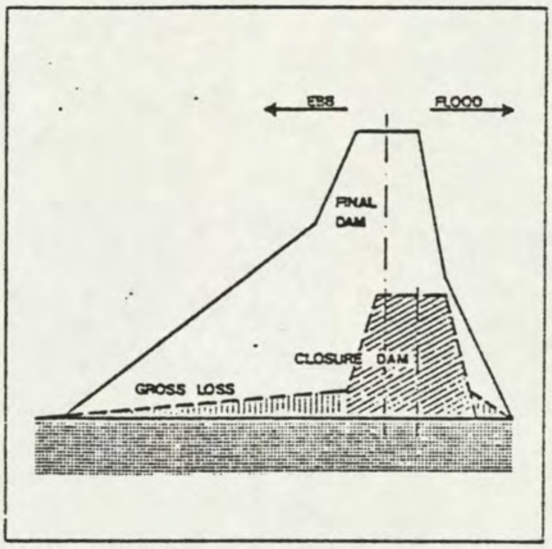


Fig. 5.1.

The crest-elevation of the closure dam depends on astronomical tide, wind set-up and the run up.
When preparing a sand closing the determination of the crest width of the closure dam is very important for the following reasons.
A minimum crest width is fabourable with respect to:

1. Reducing the nett loss. The larger the difference in cross section between the final dam and the closure dam the smaller the nett loss will be.

2. Reducing the gross loss. A closure dam with a small crest needs a smaller sand-volume during the closing operation. For this reason the time involved during the critical stage of the operation will be shorter and so the gross loss of sand will be reduced.

A wider crest however is more favourable with respect to the available working space for equipment at the fill.

In case sand is supplied only by pipe (not dumped) the pipes have to be lengthened during the production. The pipes will be used in pairs: one for produktion while the other can be lengthened by connection a new pipe section.

Pipes with a diameter in order of 0,7 m are still easy to handle. The sand discharge capacity of such a pipe is about 2000 m3/hour. The slopes of the dam in the tidal range will vary from 1:30 till 1:100, depending on the wave action. The slopes under low-water will become in order of 1:15. The side slopes of the dam above M.S.L. are mostly created by bulldozers and can be taken equal to 1:5.

5.1. Sand fill by horizontal pipe discharges

Based on the experience gained with the first sand closures of tidal channels a rough calculation method was developed for sand closings. The developed calculation method for losses of sand could be tested in practice with later sand closings.

The applied calculation method is based on the calculated sand transport in suspension. As for all completed sand closings relatively fine sane (diameter up to 250 micron) has been used only sandtransport in suspension has been taken into account.

In that case the bed transport can be neglected. If a coarsely grained type of sand is used, for instance with a diameter of over 300 micron, then the bed transport must be taken into account.

The suspended sediment loss can be calculated using the Kalinske-Kirkham formula, which was introduced in the Netherlands by Morra.

The equation, describing the suspended sediment load apply to steady flow in a condition of equilibrium between erosion and sedimentation.

The condition of steady flow is not satisfied and the current velocity curve during the tide is therefore schematized in time-intervals (0,5 or 1 hour).

Also the equilibrium of erosion and sedimentation is not statified, the flow needs distance to adapt to its new conditions.

The total sediment transport can be calculated by integration of the sediment concentration over the water depth and unity of width and time as:

$$T_c = \int_0^h c(z)\ u(z)\ dz$$

where: T_c -suspended load transport per unit width (capacity), c-sediment concentration, u-velocity, h-water depth and z-vertical coördinate.

For calculating of the vertical sediment concentration Morra (2) has adapted the Kalinkske formula with Rouse vertical distribution.
Because of the complicated form these equations have been solved numerically and presented as a diagram (fig.5.2).
Some numerical values of T_c integrated for three different depths and velocities (under assumption that u(z)=u=const.) are presented in fig.5.2.
The above mentioned deviations in respect to the tehoretical assumption (steady flow and equilibrium-stage of sediment transport) are in the calculation method compensated by the overall experience factors A en B as it will be explained further.

The closure gap may be divided in two areas: the sloping head of the fill and the part of the channel outsided this head.
The increase in sand transport capacity (q_{cap}) along the streamlines of the gap will result in a certain amount of erosion.
If q_{LD} represents the transport on the head of the fill in tidal direction and q_{LE} the transport outside this zone, Svasek et.al. assumed the following proportionality factors:

$q_{LD} = B\ q_{cap}$ where in B = 2
$q_{LE} = A\ q_{cap}$ where in A = 0,25

If the transport capacity on the head of the fill exceeds the supply a reduction factor of 0,5 or even 0,25 may be applied for the part of exceedance.
The actual sand losses are depending on the tidal conditions during a certain stage of the closure.
If a good estimation can be made of the progress of the closure and thus the tidal condition during a certain stage, the time of closure can be estimated with the formula $T = \dfrac{V(1 + {}^{p}/100)}{Q}$

where T is time of closure, V is volume of closuredam, p is percentage of loss and Q is the dredgerproduction.
A large production which involves faster work reduces the overall loss due to the reducing number of tides.

The last fase of the operation, where the actual closure is realised, must be carried out under neap condition. The maximum waterlevel will be limited and will reduce the chance of a break-through.
In order to reduce the sand loss in the final stage of the closure, the remaining gap should be situated in a shallow part of the channel.
This is due to the fact that less sand should be brought in to make the same progress as in the case of a deeper gap.
In the final gap bulldozers and draglines operate on the fill to control the sanddeposition above the waterlevel. A special problem is to control the supplied water eroding channels while running down the fill.
The duration and cost of the operation depend on the location where the sand for the dam is borrowed. Travel distances should be kept as short as possible. Borings are needed of both the dredging area and the channel.
The hydraulic and soil mechanical boundary conditions for the calculation of sand loss (q_{cap}) are:

V_m - average current velocity over the vertical (m/s)

h - waterdepth (m)

D_{50} - mean diameter of both the sand that will be supplied and that of the bed material in the gap (mm)

n - Manning coëfficient (-) for the bottom roughness or C - Chézy coefficient; average value of n for Dutch coast: 0.024.

Some examples of influence of the parameters:

1. Influence of grain size (D 50)

Given: V_m = 1,00 m/s
h = 6,00 m
n = 0,024

D_{50} mm	q_{cap} m3/m1/h	loss of sand m3/m1/h: head of fill B = 2	loss of sand m3/m1/h: bottom A = 0,25
0,150	6,29	12,58	1,57
0,175	3,96	7,92	0,99
0,200	2,58	5,16	0,65

2. Influence of bottomroughness (n):

Given : V_m = 1,00 m/s

h = 6,00 m

D_{50} = 0,175 mm

n	q_{cap} m3/m1/h	loss of sand m3/m1/h: head of fill B = 2	loss of sand m3/m1/h: bottom A = 0,25
0.026	5,06	10,12	1,27
0.024	3,96	7,92	0,99
0.022	3,00	6,00	0,75

3. Influence of dredger produktions

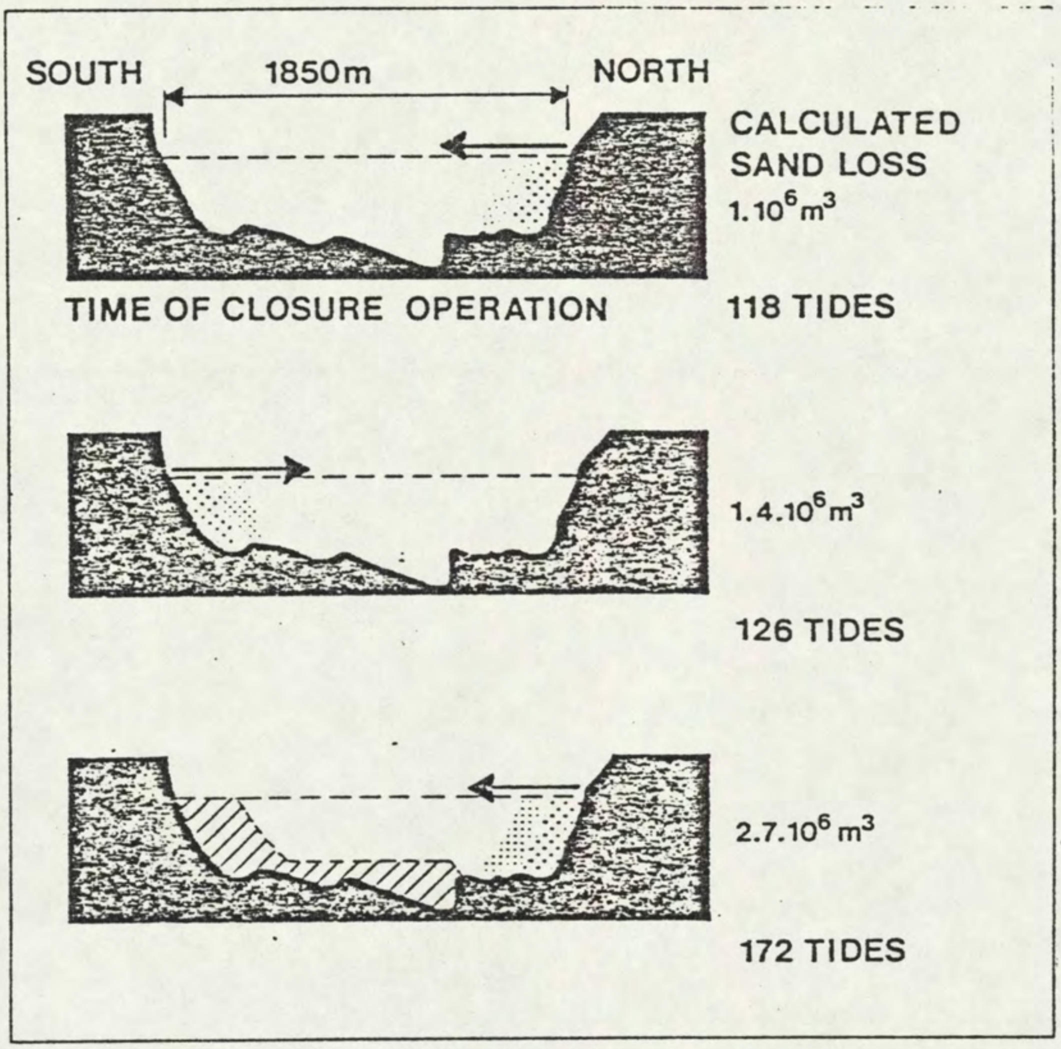
Dam through Springerdiep-Brouwersdam (1970)

Tidal range : 2,6 m; length of the gap: 1000 m; max.depth 6 m -MSL; volume of closure dam 1,6 10^6 m3, D_{50} = 0,20 mm.

Assumed dredger productions m3/week	Time of closure operations (weeks)	Calculated loss of sand (10^6 m3)	Observed loss of sand (10^6 m3)
230.000	5	0,50	
300.000	3,5	0,35	0,25
350.000	2,5	0,25	

4. Influence of closing direction Channel Geul - mouth of the Eastern Scheldt (closed in 1972).

Tidal range: 3,1 m; length of the gap: 1850 m; max.depth 11m -MSL; volume of closure dam 3,3 10^6 m3 and D_{50} = 0,15 mm (fig. 5.3)



dredger production in all these calculations 500.000 m³/week

Fig. 5.3.

5.2. Sand fill by dumping

Last years experience was gained in the execution method of very accurate sand fill by dumping. This was a part of the soil improvement, that was necessary for the pier foundation of the Eastern Scheldt Barrier (fig.5.4).

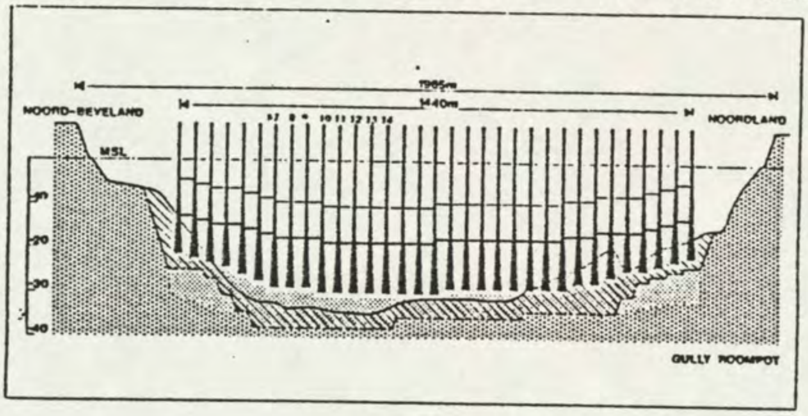


Fig. 5.4.

The refill of dredged trenches was carried out by using hopper dredgers with a capacity of approx. 3000 m3.
The refill consists of sand with a mean grain size of approx. 0.2 mm (apart 0.35 mm). The maximum layer thickness to be replaced amounts to 12 metres, the minimum thickness 6 metres. The waterdepth varies between 18 and 39 m below MSL.
In the middle of the flow channels, where the waterdepth is greater than the foundation level of the piers to be placed the bottom has to be brought up to a level which is higher than the original bottom level. Because of the high current velocities (up to 2.0 m/s springtide) these artificial sills are subject to erosion during the execution.
After having finished the placing these sills are covered with current resistant material (gravel).

Application of hopper doors.

At the start of the execution of the refill only one placing method was used, viz. the dumping of sand through the doors in the bottom of the hopper dredger.
Due to the waterdepth (30 to 40 metres) the sandlosses were very great (50% or more). In the optimalization process other placing methods have been considered as well especially the dumping of sand through the suction-pipe in inverse direction to the bottom.
The placing takes place alongside an anchored pontoon. The positioning of the dredger during the dumping is relative to the position of this pontoon. From the measurements results the following conclusions about the physical process can be derived. (fig. 5.5).

On its way from the doors to the bottom the sand-water-stream accelerates. In the meantime the sandconcentration reduces due to entrainment of surrounding water. Close to the bottom the sand-water-jet has a downward front speed of approx. 3 to 5 m/s (estimated, not measured).
This jet causes high impact forces on the sandy bottom and consequently a crater is formed (depth approx. 1 m.). The impact of the fluid jet generates a surge which spreads outwards; due to the irregular shape of the bottom and the presence of a tidal velocity, this surge does not spread radially. The front speed of the surge is initially very high; at approx. 20 metres from the hopper a front speed of 3 m/s has been measured.
The sediment concentration at a distance of approx.20 m varies between 2000 and approx. 5000 p.p.m.
At a distance of approx. 150 m. from the point of impact the energy and the sand contents of the surge are so low, that the phenomenon ceases to exist. Eddies reach the upper waterlayers from the impact area and the high turbulent surge. Deposit of sand from the surge starts at approx.20 m. from the centre of the impact area and reaches a maximum at a distance of approx.50 m.

Application of the suction pipe

The dumping time using the suctionpipe is approx. 1 hour, while the former using the hopper doors only takes 10 minutes.
Moreover, the workability under heavy water conditions using the suction pipe is much less than when the hopper doors are applied.
This workability depends strongly on the anchoring and manoeuvring capabilities of the pontoon. In the beginning dumping through the suction pipe could only take place at slack water, this in contrast with the hopper doors. (current limitation of 1 - 1,2 m/s due to the efficiency).
On the other hand the echo soundings for the suction pipe method showed with this method higher net efficiencies can be obtained in comparison with the hopper doors.
In general the sandlosses using the suction pipe method are dependent on:

- distance between the mouth of the suction pipe and the bottom
- sand discharges per unit of time
- tidal velocities in relation to the anchored pontoon.

In a later stage successful trials were done using the suction pipe alongside a large pontoon moored on 6 anchors.
In this matter it was possible to apply the suction pipe up to current velocities of 1,5 m/s.

Summary of the hopper experience

Subject	Dumping through hopper doors	Dumping through the suction pipe
Net efficiency	30 - 60%	60 - 80%
Slopes under water	1:10 - 1:15	1:4 - 1:6
Dumping time	10 minutes	1 hour
Restriction	to avoid craters and unflat surface a special execution procedure is necessary	normal execution at slack water; net efficiency highly depend on the navigation of the track; with an adapted anchoring pontoon execution possible during the whole tidal cycle
sphere of sedimentation	fine sand: 100 - 150 m coarse sand: 70- 120 m	10 - 30 m

It is also possible to apply this method in combination with a gradual closure. If the foundation of a closure embankment of rock-units can be brought up to a level which is higher than the original bottom level with sand, the quantity of rock-units during the closure will decrease.
Especially when many rock-units (rubble or concrete blocks) have to be dropped during the last closing stage, this stage will cause more scouring at the end of the bottom protection. So it is possible that the application of a sand sill reduces the total scouring during a gradual (horizontal or vertical) closure operation.
Especially when the toe of the sand sill falls inside the cross section of the final dam, the losses of sand may will be accepted in the overall economics of the works.
For the closure of the Oesterdam (compartimentationdam in the Eastern Scheldt) this alternative is in consideration now (figs. 5.6 and 5.7).

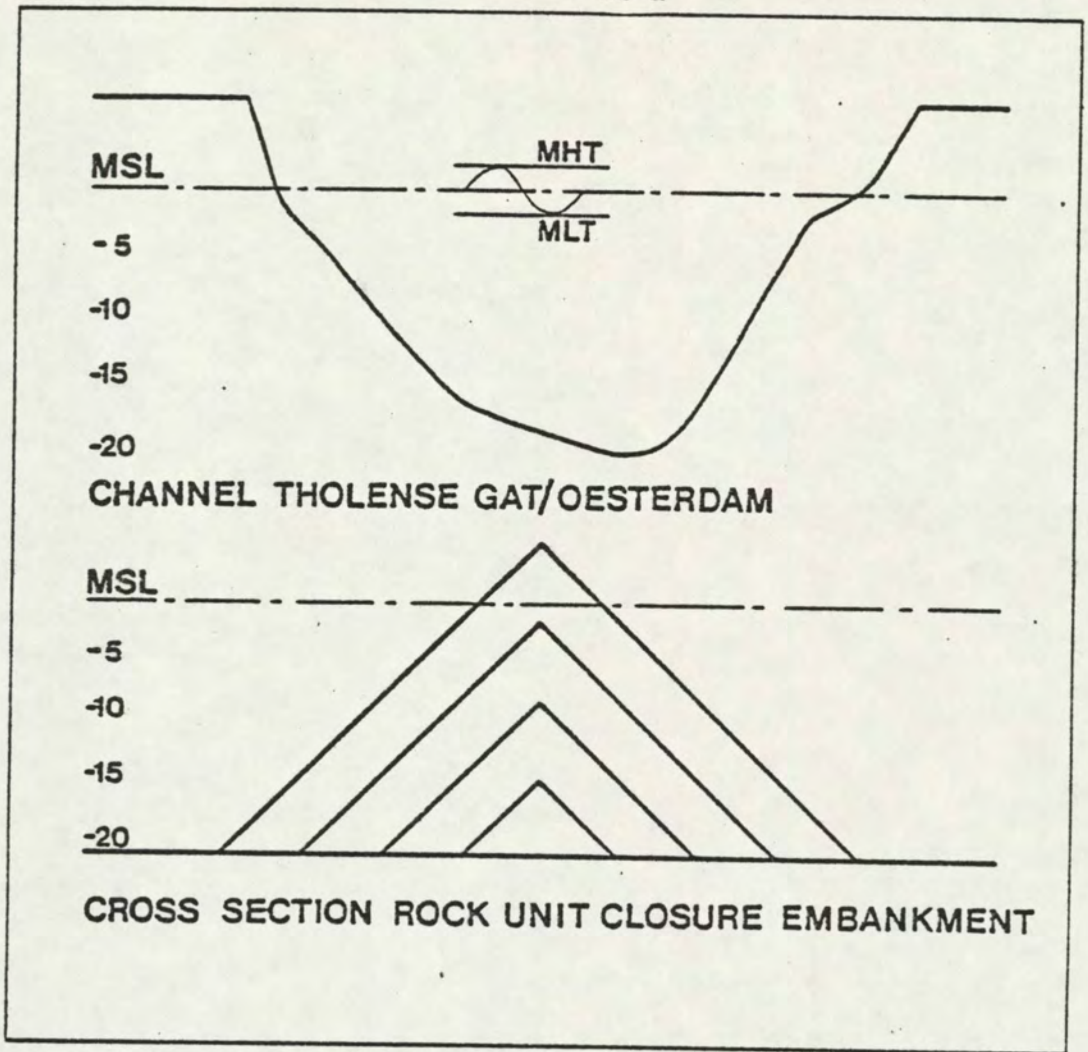

MHT
MSL
MLT
-5
-10
-15
-20
CHANNEL THOLENSE GAT/OESTERDAM
MSL
-5
-10
-15
-20
CROSS SECTION ROCK UNIT CLOSURE EMBANKMENT

Fig. 5.6

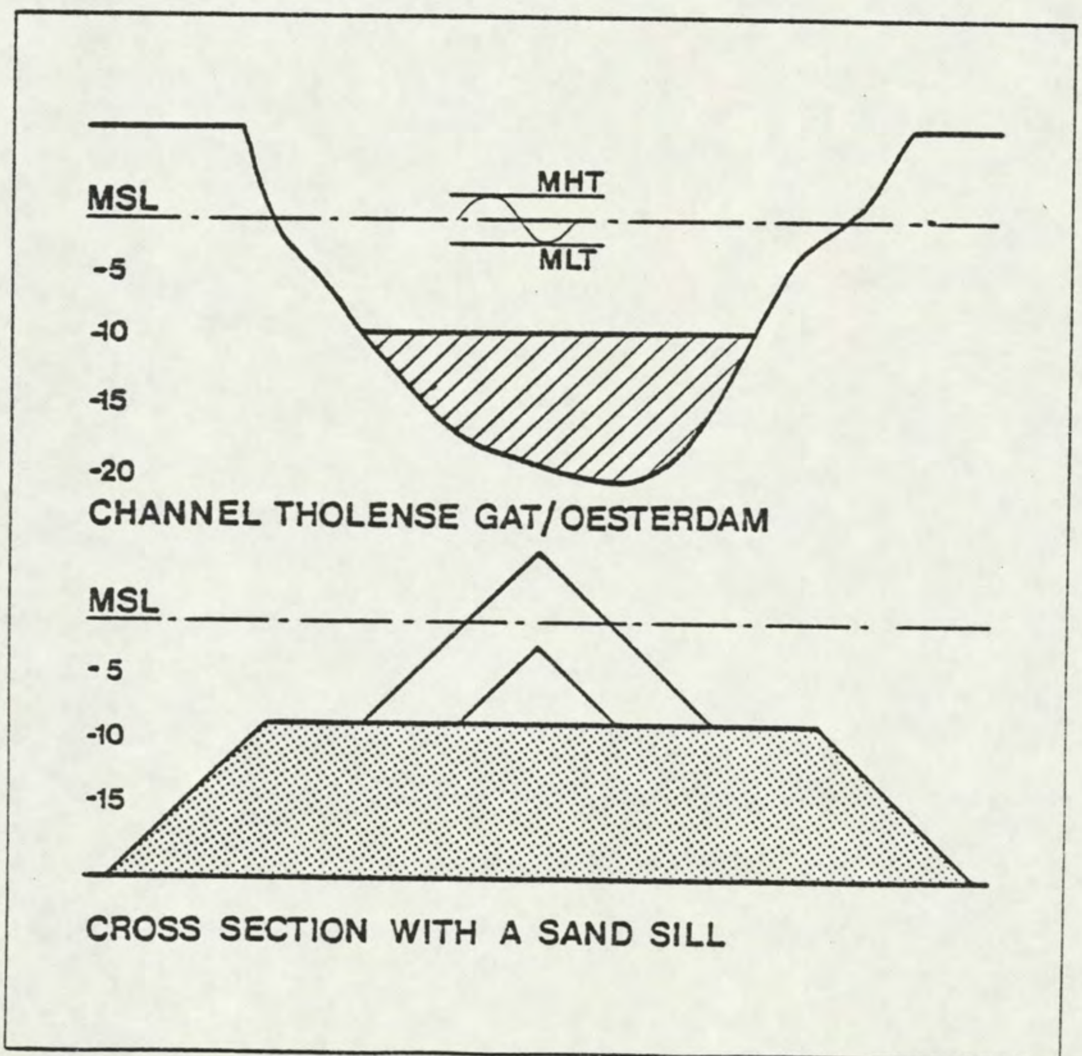

MHT
MSL
MLT
-5
-10
-15
-20
CHANNEL THOLENSE GAT/OESTERDAM
MSL
-5
-10
-15
CROSS SECTION WITH A SAND SILL

Fig. 5.7

6 SUDDEN CLOSURES

6.1. By means of (sluice) caissons

General

This method deals with structures which allow the whole gap to be closed suddenly. Normally these structures consist of caissons which are placed into the gap during a slack water period and thus close the whole gap at once. Another solution is the use of sluice caissons which are placed during several subsequent slack water periods, and are kept open during the period in which all the sluice caissons are placed. After the placing of the final caisson, they are all closed at the same slack water period by means of gates. Sluice-caissons should be used when the gap to be closed is a large one and the tidal motion is considerable.
Closed caissons are useful in small gaps, say not larger than 1 to 3 caissons, or in areas where only a very small tidal motion occurs.
Characteristic for a sudden closure by means of sluice caissons is to maintain as wide as possible an effective wet area of the closure gap until all the caissons are sunk in position on the sill. By using open caissons, a great increase of the velocity in the gap is avoided so that strong scouring does not occur, provided the flow pattern is not too much influenced by the walls of the caissons.

Advantages:

- The sluice caissons have the advantage that they do not strongly influence the tide, and that the velocities in the gap can be kept relatively small before being changed in a short time to zero.
- If the closure gap is situated under strong wave or ice attack during a great part of the year, it may be a solution to pre-fabricate these large concrete structures in milder climatological zones in a year round operation. These structures can be towed to their final destination whenever circumstances permit.

Disadvantages:

- The placing operation is a critical one, highly sensitive to weather conditions
- The use of caissons needs a slack water period which is long enough to place at least one of them(the final caisson during a neap tide).

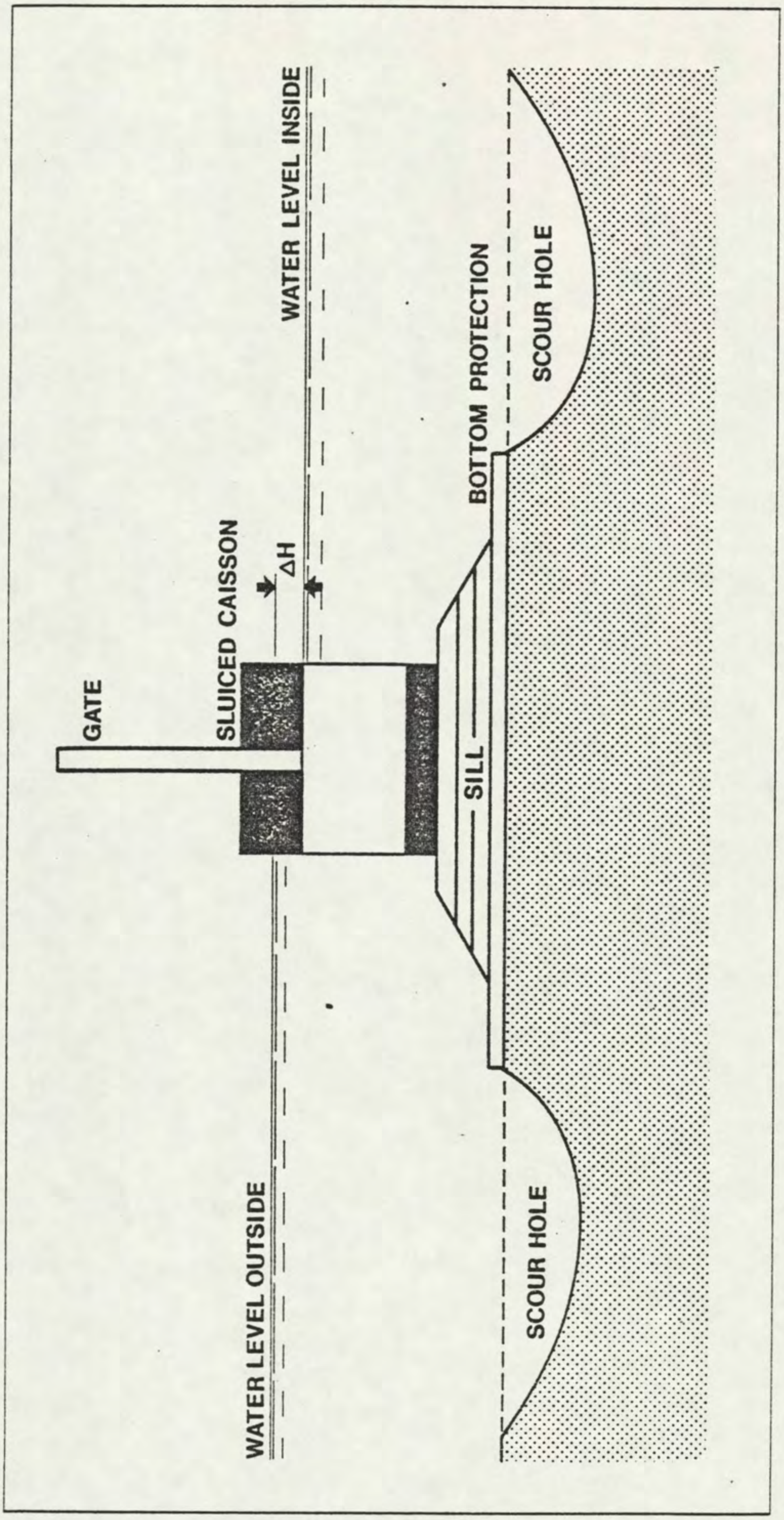
A sudden closure by means of sluice-caissons is generally divided into the next construction stages (figure 6.1).

1. A flat stable sill structure with vertical or oblique abutments, is constructed on a stable, scour-resistant sea bed, protected by covering it with a bottom-revetment.
2. a) Rigid concrete caissons are floated into position, successively, across the gap.
   b) When a caisson has been sunk into position on the sill, wooden boards along the sides of it are opened, in fact totally removed, so that the tide can flow through it.
3. When all the caissons are in position, their gates are closed simultaneuosly at slackwater, thus effecting the closure.
4. The permanent dam is then built around the caissons, if they have only a temporary function.

There are two ways in which to use sluice caissons. First, they can be used as temporay structures: after closing they are incorporated in a dike-body of sand. This structure is impermeable but is very large and expensive. The other possibility is to use the caissons as definite elements of the barrage, without covering them with other material. These elements have to be stronger than the temporary ones but, in some cases, the absence of large amounts of sand and revetments makes the total structure cheaper.
On the other hand, the permeability causes problems.
The side-connections of the caissons cause minor problems; the permeability of the sill is a more difficult problem and has to be solved, for example, by undergrouting.

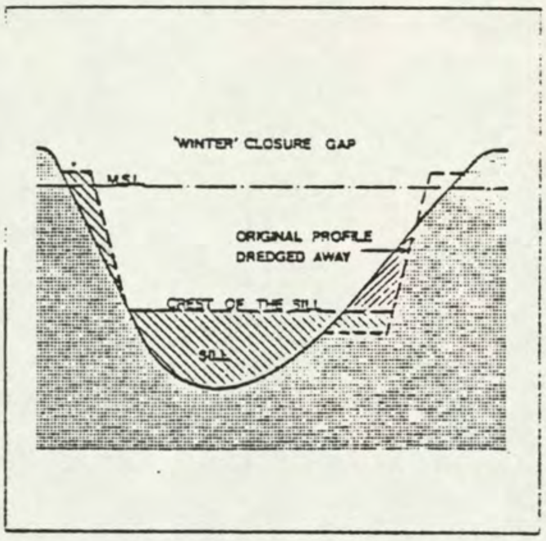
The "Winter" closure gap

The shape of a "winter" closure gap (figure 6.2), to be closed in the following spring or summer by means of caissons, calls for special attention.



(fig 1) sudden ~~closure~~ closure

Fig. 6.1.



Shape of a "winter" closure gap (figure 6.2.)

The height of the sill determines:
- the scouring action;
- the dimensions of the caissons;
- the current velocities during placing the caissons.

The shape of the abutments determines:
- the origin of vortex streets and local scour.

The <u>height of the sill</u> is important because a closure by the caissons method calls for the construction of the sill one or more years proceeding the final closure and the depth and the starting angle of the scour holes increase in time.
A deeply positioned caisson-sill limits the increase of the current velocities and thus avoids deep scourings. Another advantage of a deeply placed sill is that there are more and better opportunities during the turn of the tide for floating and sinking the caissons.
On the other hand, an increasing depth implies that a reasonable levelling of the caisson-sill will become more difficult and the costs of the caisson will rise.
The <u>shape of the abutments</u> have to be designed in such a way that the development of vortex streets is as little as possible.
Downstream of an abutment, with a vertical head on a sill,
a strong string of vortices occur, which cause deep scouring.
A better solution is developed by filling the triangular space between the first caisson and the slope with a specially adapted caisson with a sloping bottom (Volkerak and Brouwersdam closures, see fig. 6.3),
Placing and consolidating this caisson will not take more than a few days, a factor that will greatly reduce scouring.

Fig. 6.3. Oblique abutment caisson "Volkerak" closure.

The interaction between sill and caisson

Caissons used for closures of tidal basins are generally placed on a sill, constructed as a graded filter structure. The upper layer of the sill generally consists of rubble in connection with the stability of the sill for flow over it.
For the design of the sill as a foundation for caissons the following factors are of importance:

- the transferring of the horizontal loads;
- the transferring of the vertical loads;
- the stability of the upper layer of the sill;
- the flatness of the surface of the sill.

Transferring horizontal loads

For caissons that are placed on top of a surface and are not embedded, the horizontal force will have to be transferred to the subsoil by friction of the base over the sill surface.
Due to the fact that the vertical force is low, the surface immediately under the structure is often a critical failure plane.
This is even more so when, in this plane, a poor contact medium has to be accepted such as asphalt, loose sedimentary sand or contact on only a few points of a surface consisting of large stones.
This quite often means that the angle of friction or the H/V value of this layer is low and thus dictates the required effective weight of the structure.
The required weight of the caissons is a very important factor in the total design and costs of an caisson closure. For this reason much attention has to be paid to trying to find a solution with a high friction co-efficient between sill material and the bottom of the caissons.

Transferring vertical loads

The vertical forces that are transmitted to the foundation are the dead weight of the structure (including water displacement, sand ballast, gates, sill, etc) and the vertical components of the wave loads.
It should be mentioned that the rotational moment, due to the waterhead and wave loading, influence the pressure-line of the vertical forces.
One of the important requirements of the sill is to ensure a reasonably even stress distribution under the base of the structure.
Tests indicate that, when à flat stiff surface (e.g. the bottom of a caisson) make contact with an uneven surface of a layer consisting of compacted gravel or stones, very high local stress peaks can occur and hardly any flattening or redistribution takes place. In the floor of the caissons, places can be indicated where high local stress-peaks give no problem at all (mainly under the walls and spans), while local stress peaks at other points can give serious problems (mainly in between the walls).

For the stability of the upper layer it is necessary to examite the situation when the final caissons have to be placed. Then, it can be find out what will be the critical flow for the top layer of the sill, that has to be built as a filter. The head walls and bottoms of the caissons cause trains of eddies that may be a critical factor in the stability of the materials. If the eddies are very strong, they may carry off the stones at much lower average velocities through the closure gap than a normal current with straight flow-pats will do. If there is a real danger of the critical flow being reached, two solutions are possible:

- the sill can be finished off with a heavier material (Volkerakdam);
- the original material may be consolidated by means of an asphalt filling (northern channel Brouwersdam).

Both measures have an effect on the design and completion of the closure.
The coarser and heavier the sill material, the more difficult it is to give the sill a regular, even surface. One possible consequence is the greater risk of seepage after the closure with caissons bearing on an uneven finish of the sill.
Filling of the sill may affect the design of the caissons.
In the design, the horizontal load resulting from waves and difference of head is estimated, while the maximum shear resistance is calculated by using the friction coëfficiënt between sill material and the bottom of the caissons.
The risk is great that the value of the friction coëfficiënt will be reduced as a result of the filling- up process. This means that a heavier caisson would have to be designed.
To determine the value of the friction coëfficiënt prototype tests with a concrete slab on a flat pile-up of normal rubble, and on an asphalt-jointed ragstone pitching were carried out.

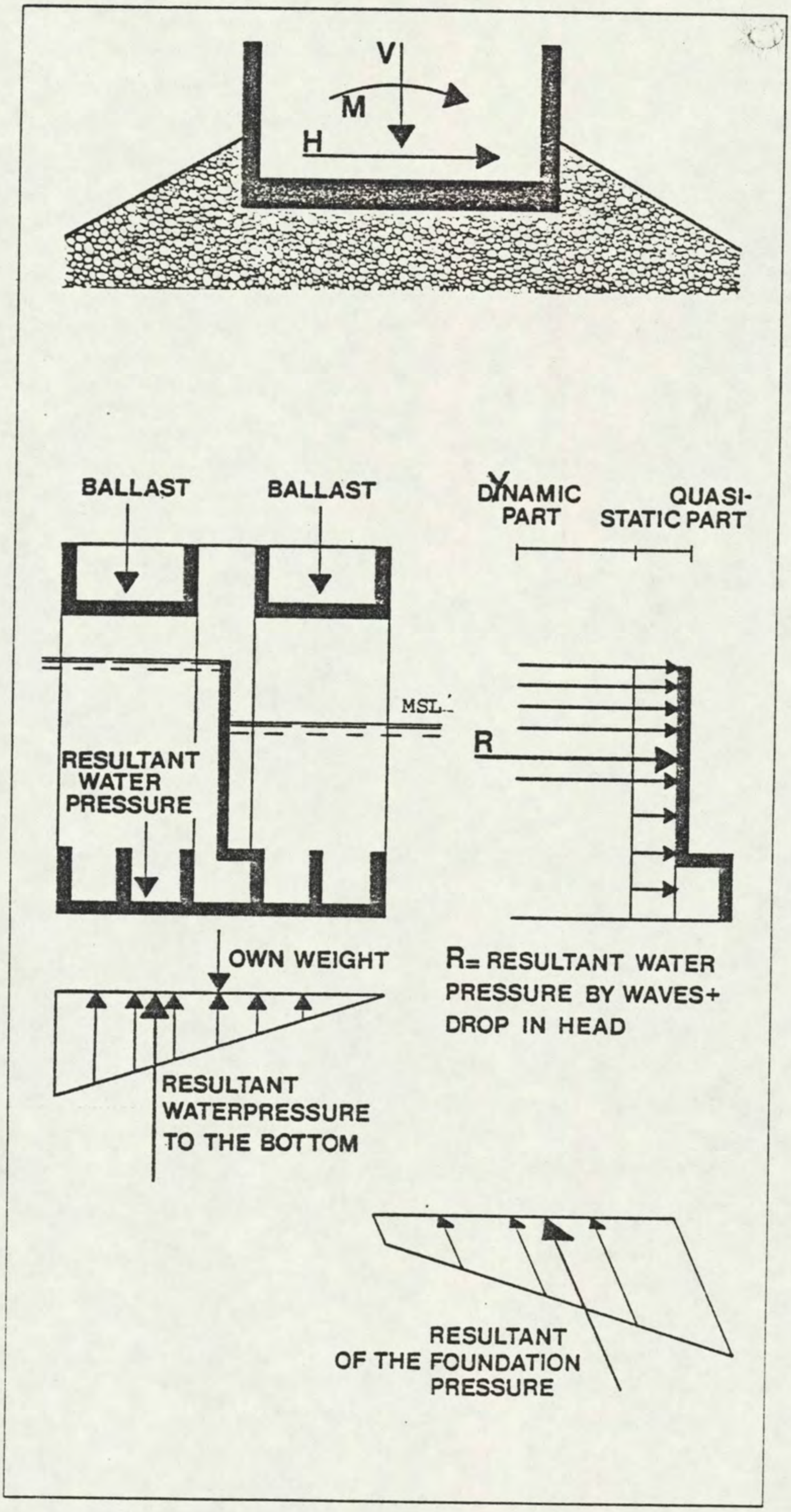

V
M
H
BALLAST
BALLAST
DYNAMIC PART
QUASI-STATIC PART
MSL
RESULTANT WATER PRESSURE
R
OWN WEIGHT
R= RESULTANT WATER PRESSURE BY WAVES+ DROP IN HEAD
RESULTANT WATERPRESSURE TO THE BOTTOM
RESULTANT OF THE FOUNDATION PRESSURE

Fig. 6.4.

The conclusion, from the results of the tests, is that the asphalt filling does indeed cause a fairly serious reduction of the friction coëfficiënt, depending on the quantity of asphalt used in the filling and on the specific pressure under the concrete slab.
The higher the specific pressure and the greater the quantity of asphalt used, the greater the reduction in value of the friction coëfficiënt is.

In the case of flat caisson bottom on a sill of rubble a friction coëfficiënt of 0,5 may be assumed. In the case of a jointed sill, the value may be taken as maximum 0,4.
If the bottom plate of the caissons is ribbed, the value of the friction coëfficiënt increases to 0,6 in the case of a normal rubble sill and to 0,5 in the case of a jointed sill.

The flatness of the surface of the sill

As already mentioned, the sill is not always completely level due to the sizes of the stones in the upper layer. If caissons are placed on the sill, gaps under the caissons will be present with a height at least as great as the diameter of the rubble, depending on the method of execution.
For example when using only stone dumping vessels to construct the sill, the irregularities will be approximately o,5 to 0,7 m (rubble 60 - 300 kg) and after finishing off the surface by means of a bucket dredger, these unevenness will possible be 0,2 tot 0,3 m.

The consequences of these irregularities are:
- the caissons are not fully supported along their entire bases;
- the origin of seepage.

The floor of the caissons for the closures of the Veerse Gat and the Volkerak was completely flat. In these cases, it was uncertain where the caissons rested on the sill. To be certain of the spot where the caissons for the Brouwersdam supports, the caissons which were 68 m long were placed on two seats of 8 x 18 m^2. These seats which had a height of 0,5 m, were situated at 1/4 and 3/4 of the length in order to prevent the caisson resting on the outer ends or in the middle.
The first plans for a permanent caissondam in the mouth of the Eastern Scheldt were to put caissons on a square net of ribs and skirts or on a temporary three-point support and before ballasting to undergrout the rest of the bottom of the caisson.
Another proposed solution was to put each caisson on a three-point support in the permanent situation. These supports had an area of 15 x 15 m^2 and were to be grouted.

High velocities can occur in the gaps under the caissons as a result of the head-loss over the caissons during the closing operations. This can endanger the stability of the rubble on top of the sill and thus the structure as a whole. Therefore, two measures can be taken:

- place narrow ribs under the caissons, which allow them easily to penetrate the sill. The ribs have a function of sealing and increasing the friction. They do not have a bearing function.
- dump stones against the walls of the caissons. If these structures consist of rubble dams, the stability of this dam -especially the down stream one- must be assured (fig. 6.5.).

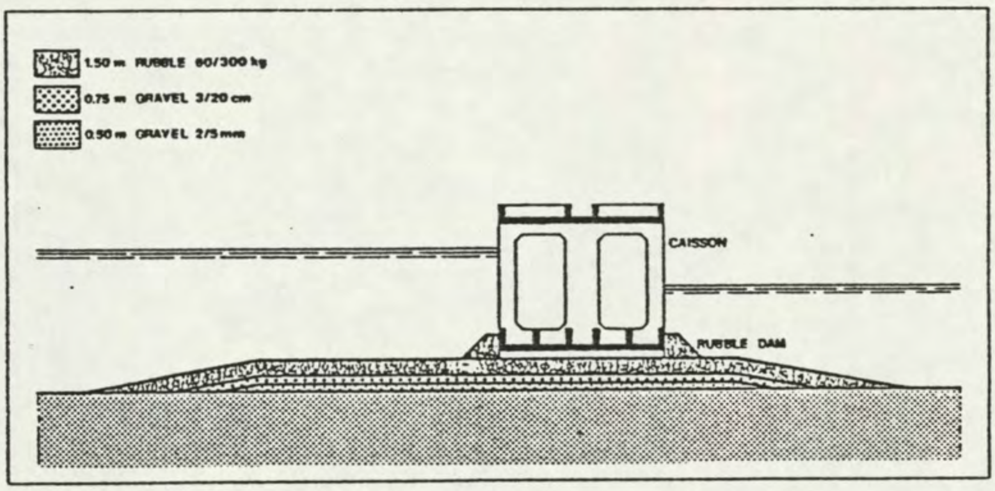


Fig. 6.5.

Distinction should be made between the stability of the rubble dam as a whole and the stability of the individual stones.
Comparing both stabilities, it can be stated that the stability of the downstreams rubble dam as a whole is significant.
This stability is determined by:

$$Sr = (1 - \varepsilon)\, \Delta \cos.\alpha \left(1 - \frac{\tan \alpha}{\tan \varphi}\right)$$

where: ε = porosity

$\Delta = \dfrac{\rho_m - \rho_w}{\rho_w}$ = relatieve density

α = angle of inclination

φ = angle of repose

For the determination of the volume of the rubble dam which must be dumped against the walls of the caissons, it is necessary to know the distribution of the head-loss over the dam(s) and the gap under the caisson.
Model tests have been carried out to obtain insight in this matter. During the tests, the course of the water pressure along the surface of the sill has been measured, so that the distribution of the head-loss could be determined.
It can be concluded that the stability of the rubble dam depends on:
- the porosity;
- the density of the material;
- the head-loss.

The part of the total head-loss over the downstream rubble dam is strongly influenced by the presence of the upstream dam and the gap height.
Influence of grading of the material is manifested in the porosity. No significant influence of the diameter of the rubble could be determined.

If the predicted settlement and deformations of the caissons are too big, a densification of the sill is necessary. Experience with these methods has been gained for the Eastern Scheldt Barrier.

. <u>The design of the caissons.</u>

A caisson has to be constructed in such a way that it is strong enough to be towed from the construction pit towards the closure gap, where it will be sunk and must find a stable place on the sill.
The design considerations concern:
- hydraulic aspects;
- navigation aspects;
- constructional aspects.

<u>Hydraulic aspects</u>

In the case of a sudden closure, it is necessary to maintain as wide as possible a wet cross section, during the placing of the caissons.
The effectiveness of the sluice caissons increases in proportion to their positioning on a deeper lying sill.

1) Design the wet cross section of each caisson as large as possible:
   - large spans between the walls $\longrightarrow$ apply steel diagonals to give the caissons enough torsional strength;
   - thin bottom;
   - place the ballast bunkers, necessary to obtain enough friction between sill and caisson bottom, in the superstructure of the caissons.
2) Design the discharge coëfficiënt μ as high as possible (fig. 6.6.):
   - streamline the diagonals within the spans;
   - apply downstream and upstream rubble dams on the crest of the sill (fig. 6.5.) and give these dams a streamlined shape.

Table Survey of some caisson-closings

	Depth of sill with regard to M.S.L.	Discharge-coëfficiënt sluice caissons
Veerse Gat 1961	- 8 m	not measured
Lauwerszee 1969	- 6 m to - 6,5 m	0,6
Volkerak 1969	- 7 m	0,75
Brouwersdam 1971	-10 m	0,82
Caissons designed for the closing of the Eastern Scheldt	-20 m	1,0

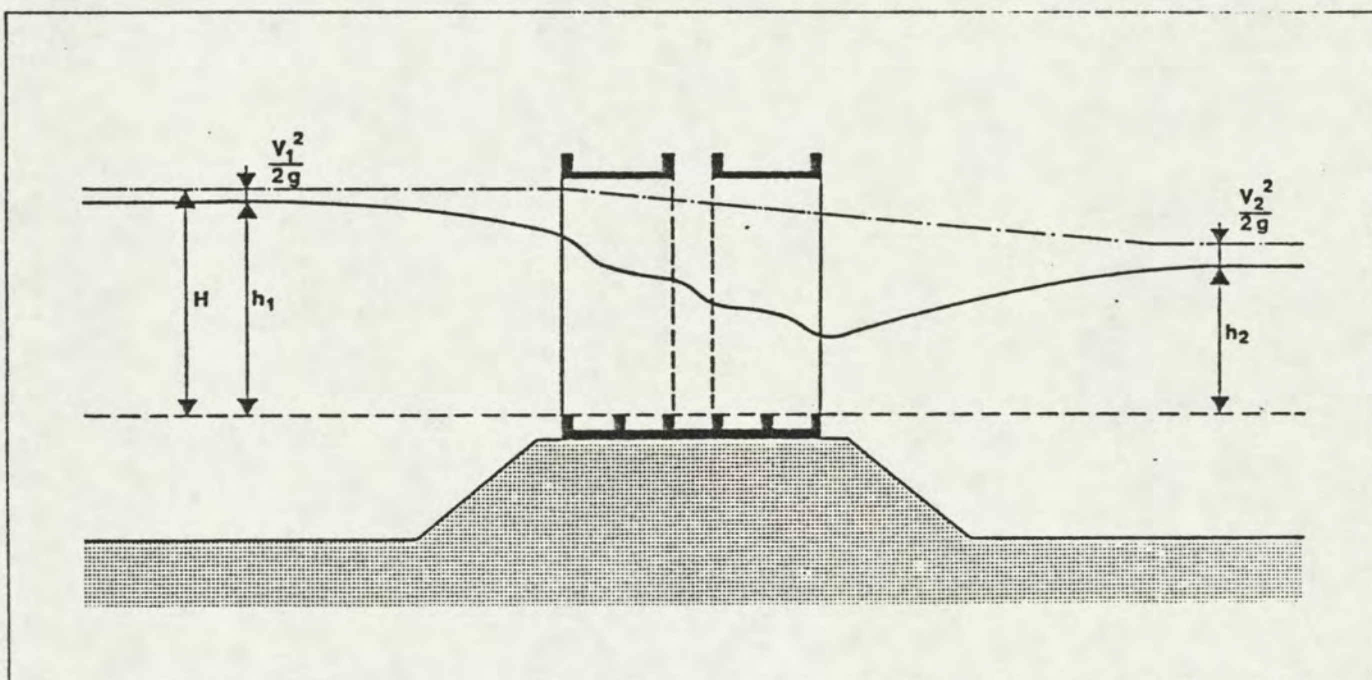


Fig. 6.6.

Discharge coëfficiënt m

$$m = \frac{Q}{6\ hz \quad 2.g.z.}$$

$z = H-hz \quad - \quad hi + \frac{V1^2}{2.g} - hz$

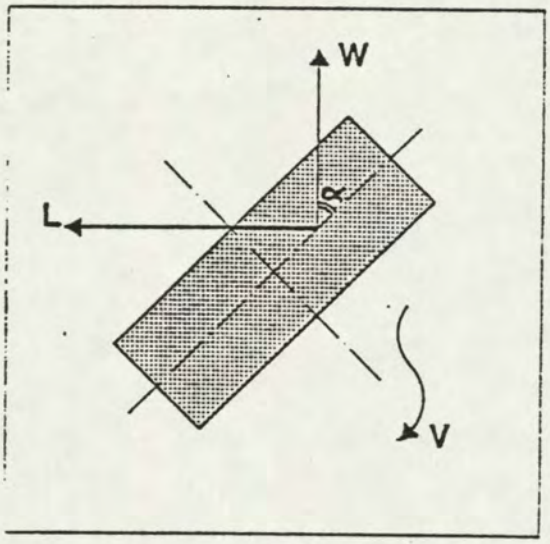
b = length of one caisson
  - thickness of all partitions

Naviagation aspect

From an execution point of view, it is advantageous to limit as much as possible the nummer of caisson-placements at a closing. In addition it is desirable, from nautical considerations, that the relation between width and length of the caisson amounts to 1 : 3 or 1: 4.
Based on these considerations, caissons longer than 50 m are preferred.

The most important navigation conditions affect:
- the draught: depends on the water way and the depth above the sill
- the manoeuvrability: $\frac{L}{B} > 3$, possible to tow two caissons joined behind each other
- flow resistance and immersed volume $\longrightarrow$ number and power of the tugs
- the stability of a floating caisson.

Flow resistance



$$W = C_W \ \tfrac{1}{2} \ \rho_w \ V^2 \ A$$
$$W = C_L \ \tfrac{1}{2} \ \rho_w \ V^2 \ A$$

W = resistance force (newton)
L = lift force (newton)
ρ_w= density of the water (kg/m^3)
V = current velocity, measured about 150 m upstreams of the caisson and 3 m below water surface (m/s)
A = area of the waterplane perpendicular to the flow direction (m^2)
h = water depth (m)

Stability of a floating caisson

It is desirable that the caisson maintain their vertical position during the transport and the sinking. This can best be achieved by building them symmetrically around the longitudinal axis.
The sluice caisson used for closing the Veerse Gat lacked a symmetrical form as, on the one side, gates were suspended and on the other wooden boards.

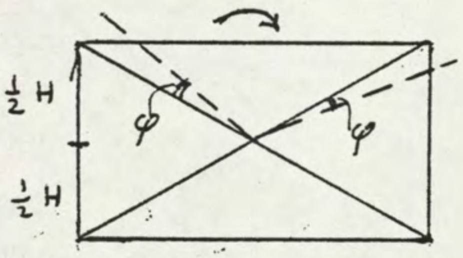
To increase stability during the sinking process, a temporary pendulating board was fixed in the centre of the caisson. With the sluice caisson for the Volkerakdam and the northern channel in the Brouwersdam, wooden boards were used on both sides for the floating. The steel gates for the final closure were placed in the centre. During the towing and sinking these also served as pendulating boards.

A floating unit (caisson) is stable if the centre of buyancy C is above the centre of gravity G.

C = centre of buyancy
G = centre of gravity

$$MC = \frac{I}{V} = \frac{\frac{1}{12} \cdot L.B^3}{V} \qquad \text{if} \qquad \varphi < 10^\circ$$

M = transverse meta centre
I = transverse moment of inertia of the waterplane
L = lenght of the caisson (m)
B = breadth of the caisson (m)
V = immersed volume (m^3)
MG = h_M = height of transverse meta centre above centre of gravity
MG = MC - CZ



If a floating unit is rolling (angle of heel φ is small) the transverse moment is:

$$M = - h_M \cdot \rho \cdot g.V$$
$$M = J \cdot \ddot{\varphi}$$

J = mass inertia moment
j = mass inertia radius
m = ρ.V
J + j 2m
$j = \sqrt{\frac{J}{m}}$

differential equation:

$$J. \ddot{\varphi} + h_M \cdot \rho \cdot g \cdot V \cdot \varphi = 0$$

gives for the <u>own</u> rolling period To

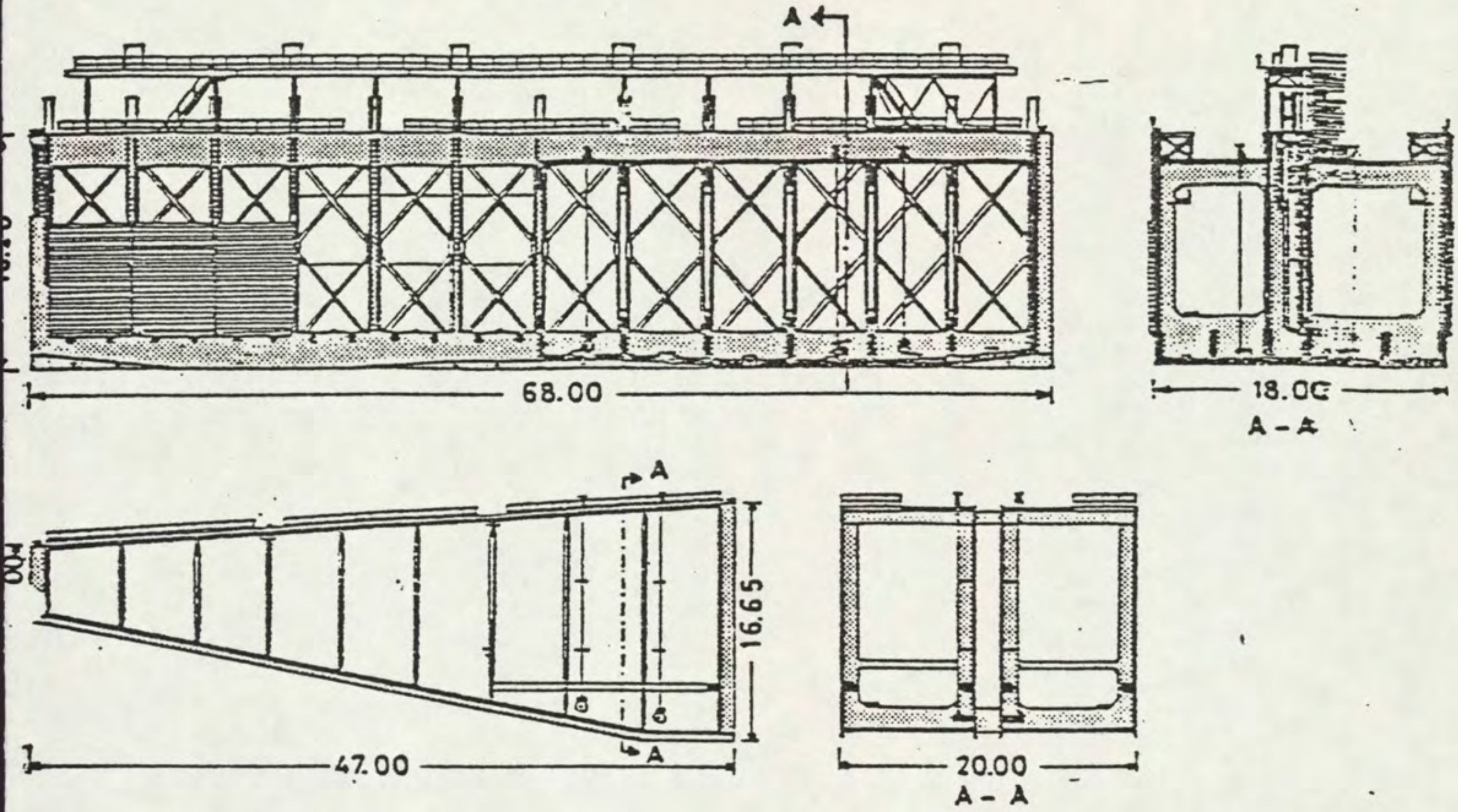
$$To = 2\pi \sqrt{\frac{J}{\rho \cdot g \cdot V \cdot h_m}} = \frac{2 \qquad j}{g.h_m}$$

Constructional aspects

Using sluice caissons has the advantage that they are independent units, each with its own gates.
The caissons must be rigid so that the gates can always be closed, even, if there are unequal settlements.
To pick up a displacement between two adjoining caissons, a space to be filled with rubble is necessary between the head of the caissons.
The caissons allow for deformations as long as the caisson floor would be strong enough to withstand pressure differences.

Due to the fact that a small water displacement is favourable for the navigation conditions, and a high own weight is required for obtaining enough friction between caisson floor and sill surface, ballast bunkers are used. These bunkers are filled with sand or poor concrete immediately after positioning the caisson on the sill.

The caissons have to be constructed in such a way that the supports are situated under the walls as, on these places, high local stress peaks in the floor give no problem.
From the viewpoint of the structural strenght not only the local stress peaks, but also the overall stress distribution can be important, due to the possibility of torsional loading of the structure under its own weight.
The possibility of torsional loading can give great difficulties especially in case of caissons about 100 m ling. A three-point support decreases the torsional loading.
To give the caissons, used for the closing of the Veerse Gat, the Volkerak and the Brouwersdam, enough torsional strenght, steel diagonals were constructed within the spans between the walls (see fig. 6.7.)



Side-view, longitudinal and cross-section of a normal caisson (above) as well as longitudinal and cross-section of one of the two caissons to be placed against the abutments.

Sluice caisson:

immersed volume	7.200	ton
draught	6,1	m
meta centre height	1,2	m
centre of buoyancy	3,17	m above base
centre of gravity	7,10	m above base
volume ballast bunkers	840	m^3/bunker

Fig. 6.7.

Positioning of caissons

In general, a caisson is parked near the closure gap before starting the placing manoeuvre. When the current velocities are sufficiently decreased, the caisson is towed to the sill and moored.
After placing in position, the valves are opened and the caisson is sunk onto the sill.
Preferably, the caissons has to be sunk in position onto the sill just before slack water so that the flow forces at the caisson and the tugs are maintained as long as possible.
L.W. slack is favourable for placing the caissons, because then:
- the required time for sinking the caisson in position onto the sill is less than at H.W. slack.
- the temporary wooden floating boards can be lower.

But, if:
- there is a longer slack water period at H.W.
- the water depth above the sill is too small at L.W. (f.i. an abutment caisson)
- for capacity reasons, there are 2 caissons being placed per day.

then H.W. slack may be favourable!
The required time for placing a caisson depends on the duration of each necessary action. From earlier caisson closures the following placing operation periods were found for a tidal range of approx. 3 m.
For caisson closures, under circumstances with a tidal range of 6 to 8 m, the placing operation has to start, for instance when the current velocity is 2 m/s and has to stop when the velocity is 0,5 m/s. in the opposite direction. In this case, about 1 hour is available to carry out the operation during mean tide. Of course, the placing will be easier during a neap tide. In these cases, 1 hour will be enough to carry out an accurate placing, provided that anchored pontoons equipped with tension winches or fixed anchor points, in addition to tugs are used.

Placing procedure of the caissons

Action Subject	Time before/ after L.W. slack (min.)	Velocity above sill (m/s)
1. Sailing to gap		
2. Parking outside	> - 130	
3. Preparing	~ - 100	
4. Sailing to the sill	~ - 70	
5. "Turn into" manoeuvre	- 55	
6. Make fast	- 30	~ 0,50 (< 0,75)
7. Open valves (sink down)	- 13	≤ 0,30
8. Caisson on the sill	- 5	
9. L.W. Slack	0	0,0
10. Levelling		
11. Remove wooden floating gates	+ 10	
12. Stone dumping besides + ballast	+ 60	
13. Open gates	+ 80	

The duration of the critical phase is determined by the duration of the actions 6, 7 and 8 and amounts to 25 min.
The desired current velocity above the sill is about 0,5 m/s (max. allowed 0,75 m/s) for mooring the caisson.
As a consequence of the placing of a caisson the local velocities increase (with about 20%).

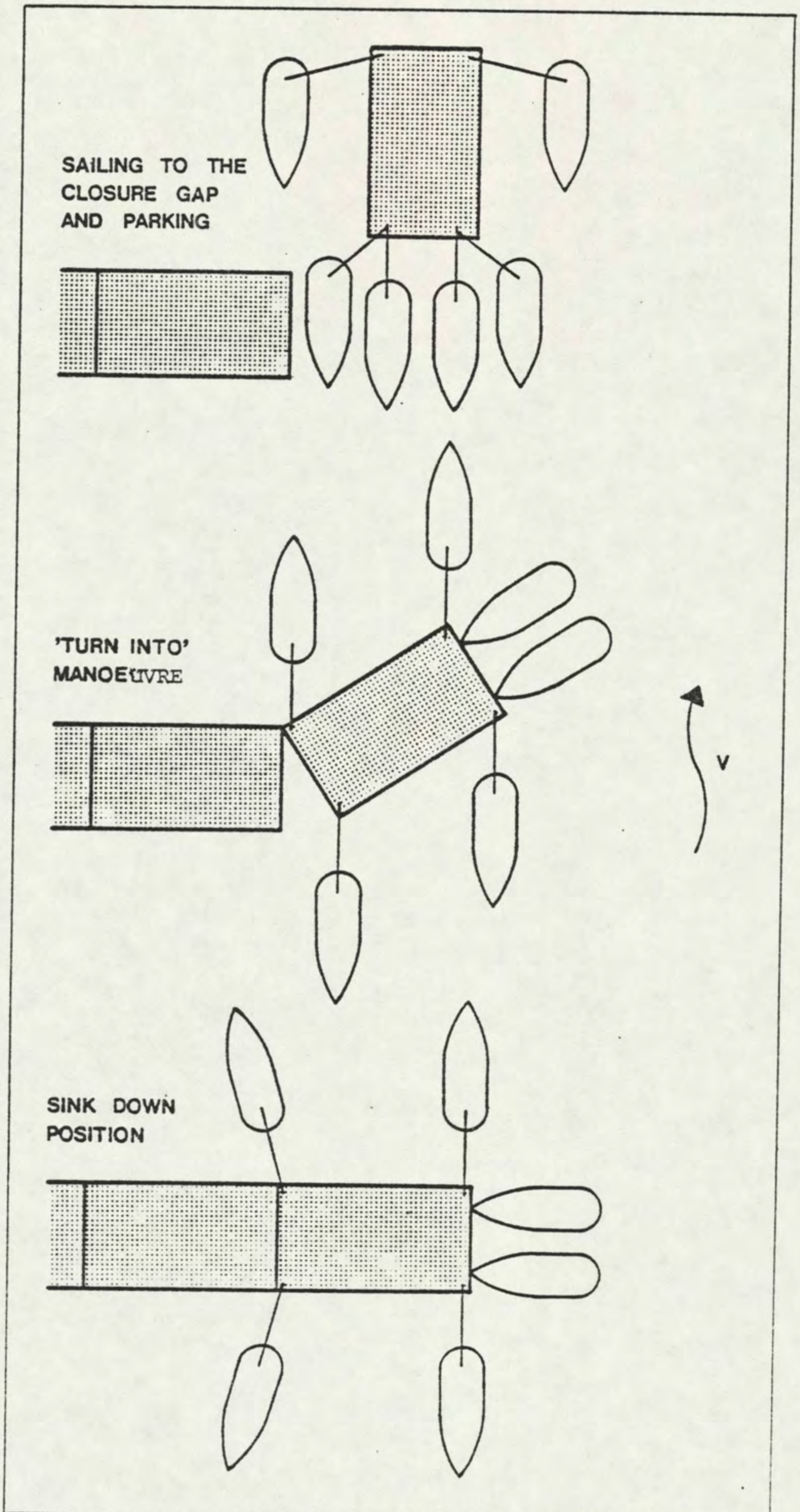


Fig. 6.8. Placing procedure of a sluice

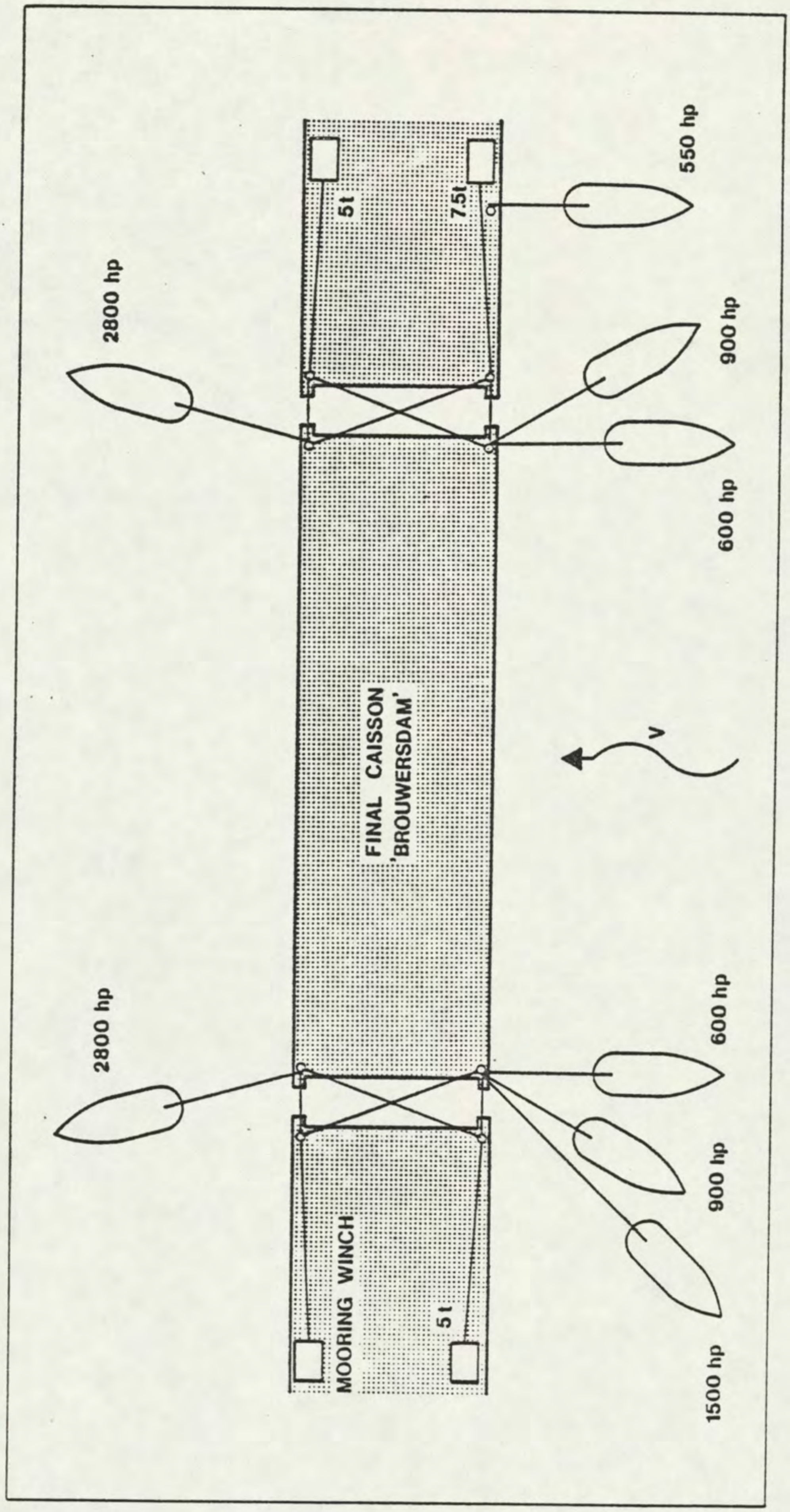
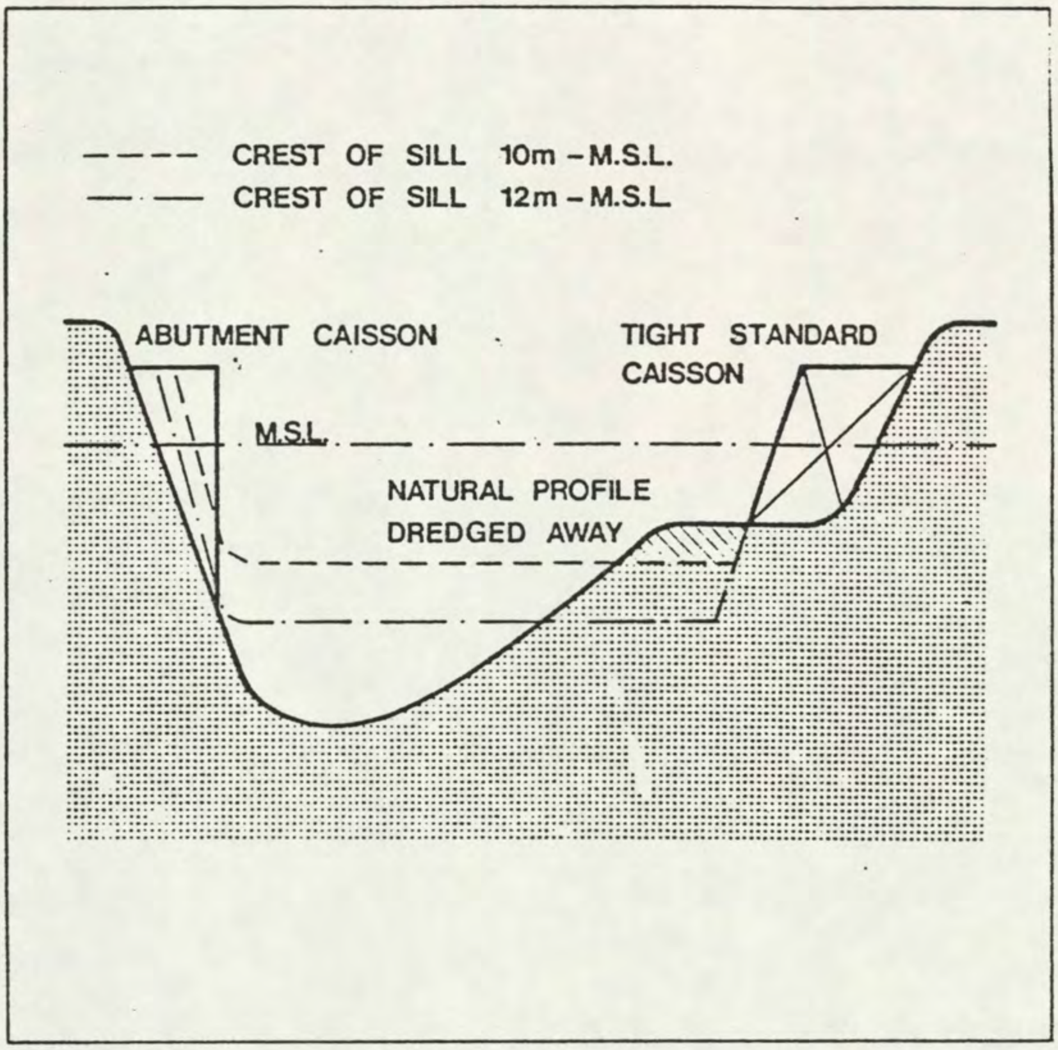

2800 hp
550 hp
5t
7.5t
900 hp
600 hp
FINAL CAISSON
'BROUWERSDAM'
V
2800 hp
600 hp
900 hp
MOORING WINCH
5t
1500 hp

Fig. 6.9.

Example sudden closure of channel Krammer, Philipsdam by means of sluice caissons

9 caissons with a length of 75 m

1 abutment caisson at the north side



If crest of the sill is 10 m - M.S.L. ⟶ breadth of caissons for stability: 16 m.

If crest of the sill is 12 m - M.S.L. ⟶ breadth is 20 m.

Discharge coëfficiënt caissons = 0,85.

Time to build up the sill = 5 months.

Scour depth : sill 10 m - M.S.L. = 6 m

sill 12 m - M.S.L. = 4,5 m

If the sill is higher than 10 m - M.S.L. ⟶ time to place final caisson is too short ⟶ see fig. 6.10.

Duration of placing all the caissons: 2 weeks.

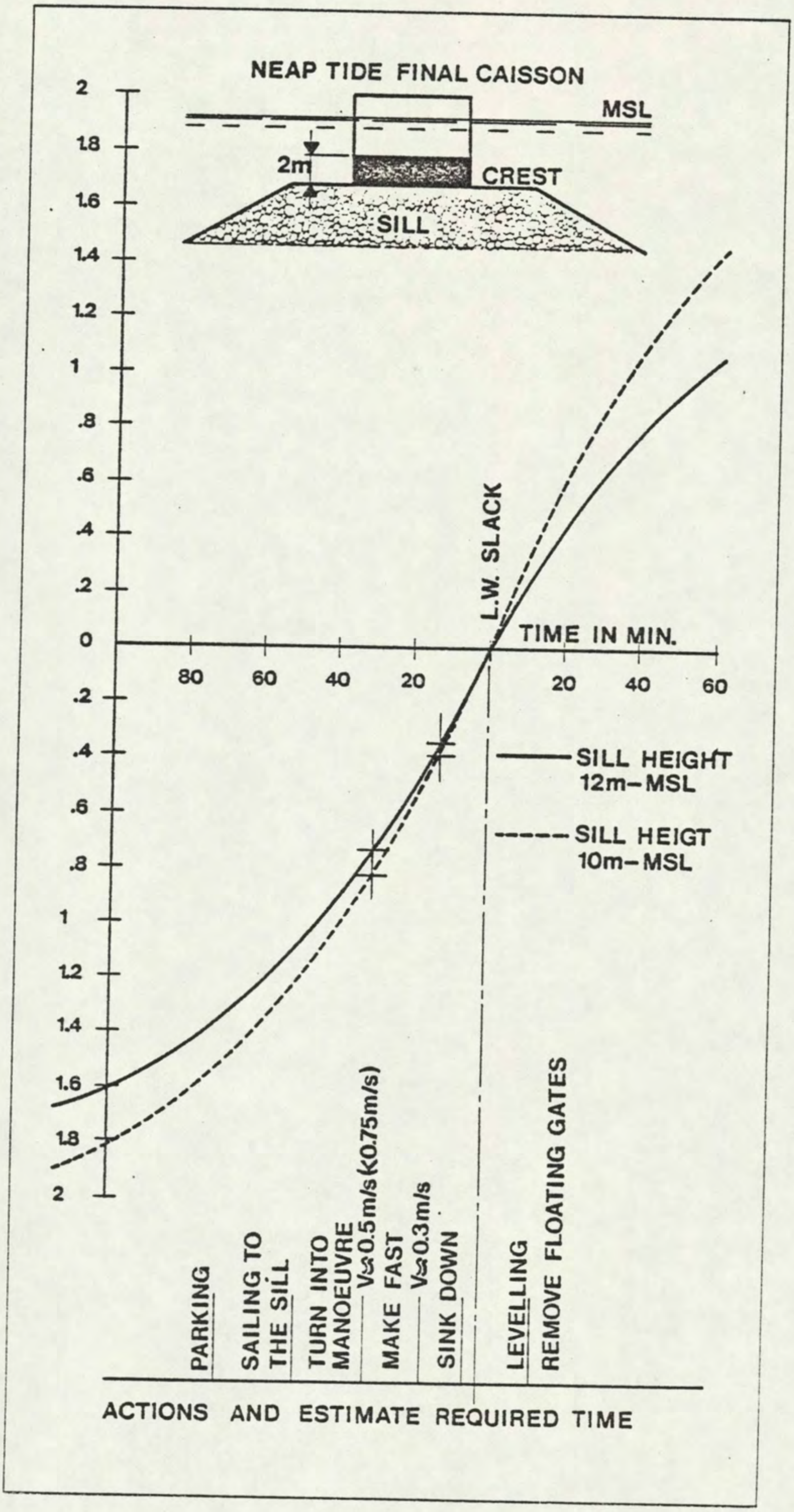


Fig. 6.10. Sluice caisson closure at LW slack channel Krammer $B_{MSL} \approx 675$ m