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(Plans of sites and maps are for illustration and not geographically accurate.)

References

*1 Tractaet van Dyckagie, (in old Dutch)  by Andries Vierlingh.  
reprint of a manuscript written around 1570

*2 The closure of tidal basins,  
-Closing of estuaries, tidal inlets and dike breaches-  
by many authors, editorial co-ordinator W van Aalst.  

*3 Dijken  
-samenstelling, aanleg, onderhoud-  
by T. Huitema.  

*4 Man-made lowlands, (in Dutch: Leefbaar Laagland)  
-History of water management and landreclamation in the Netherlands-  
by many authors, editorial supervision G.P.van de Ven.  

*5 Dredge, drain, reclaim.  
-the art of a nation-  
by Dr. J van Veen.  

*6 Nederland-waterland. (in Dutch)  
-a historical technical survey-  
by Ir. H.A.M.C.Dibbits  
Preface.

Since ancient times man tried to get control over the water moving along in rivers and seas. By trial and error, after many successes and just so many mishaps or even disasters, the engineering technology in this field developed. Practice and science meet in this struggle against (and with) nature. Fortunately, nature controls its processes by rules and laws which can be analyzed and recorded. Thorough knowledge of these processes is the first requirement for successful operating. Therefore, the theory, mainly of hydrodynamics and soil mechanics, lectured in the university as separate subjects, is an indispensable part of the closure technology.

The variation in nature's conditions is immense and the scientific approach to the processes groups these together in several formulae and statistics. Practical coefficients and assumed values, generally to be verified (if possible) for actual cases, are required to make the theory applicable in design engineering practice. Besides, the job has to be executed and the know-how about operational possibilities is a second requisite to obtain a well-considered design.

This book is not meant to be a complete manual for closure design, including all details and available knowledge. It is a guideline covering many (hopefully sufficient) aspects in main lines. Sometimes, in cases where I thought fact or figures are not easily obtainable in the literature, I entered into details. In other cases, even for main aspects, I referred to the generally available theory or descriptions. This book does not present the theory, which is assumed known, but it uses theory in order to show the coherence and application of the various theoretical aspects.

During the execution of a closure work the hydraulic conditions change. Water levels, tides, flow velocities and flow patterns depend on the way the progressive stages of the works influence the change. Therefore, the design and phasing of the closure procedure also influences the possibilities for operating equipment and using closure means. Very many variables influence the decision for selection of the best method of closure. There is never one single answer to the problem of how to close a water course. There are always different possibilities and various answers may be correct from an engineering point of view.

In the text reference is made to actual cases or examples, while results of mathematic calculations illustrate the subject in hand. These paragraphs are given in smaller letter types to ease the following of the main line.

I presented the book in English to make it comprehensible for non-Dutch readers. However, much of the experience is gained in the Netherlands and/or by the Dutch and many cases refer to Dutch circumstances. Terminology sometimes is typically Dutch and sometimes difficult to translate. I tried to find a wording or description which sufficiently clarifies the meaning of the word without trying to introduce a new terminology. May this English not be "double dutch".

The basic technique is old but not outdated. Ancient literature on the subject is rather scarce (ref *1). The art is characterised by experience and craftsmanship. Nobody cared to write about it. Only a few books give details about the state of the art before 1950 (ref *3). After a major flood in the Dutch southern delta in 1953 and the subsequent Deltaplan for reconstruction of the sea defences, extensive reporting took
place. A major effort to denote all experience gained in the last 40 years is the edition of a book, written in English, called: "THE CLOSURE OF TIDAL BASINS" (ref. *2)

Furthermore, several books give details about the history of the Netherlands in relation to the control of water. At first, the accent was given to the successful retrieval of land areas from the sea since the year 1200 (ref *5). Later, it was realised that before that time the same areas had been lost as consequence of human actions as well. It made the Dutch reconsider the achievements and reflect on the impact of human intervention in natural developments. Examples of these aspects are given in ref *4.

The contents of this book can be subdivided into three main parts:

- **chapters 1 and 2**: describe the historic development of the technique and the consequences of the construction of a closure, which have to be considered.
- **chapters 3 and 4**: summarise the theory on hydraulics and geotechnics, as used in the design process. Much attention is paid to the significance for the practical application.
- **chapters 5 to 7**: concentrate on the use of structures and materials in the closure operations, as well as the realisation process and the equipment. An effort is made to present guidelines for achieving a strategy in case a closure design has to be made. Lastly, an example of a closure is detailed. A few options are outlined and motivated and the consequences of the phasing of the process are illustrated. Comparison with a few identical case-histories is made.

The book is primarily written for students at the Technical University Delft. An important part of the technology, apart from the basic theoretical subjects, is the subject-matter presented in the lecture-notes on "Bed, bank and shore protection". In order not to copy too much of this, those aspects are detailed as refresher and not elaborated. Nevertheless, scour hole development and measures to prevent it are an integral part of closure design.

April 1994
F.C.van Roode
List of Symbols

a  waterdepth (above sill)
A  cross sectional flow area
α  correction factor
B  width of gap
Bx  cross sectional storage width
c  wave celerity
cb  distance of Cb above bottom of structure
C  coefficient of Chezy
Cd  drag coefficient
Cb  centre of buoyancy
Cn  soil constant
d  waterdepth
dv  draught of vessel
D  diameter of stone
Δ  relative density
ε  settlement
f  stone stability factor
F  force
g  gravitation
gb  distance of G to bottom of structure
G  centre point of gravity
h  waterlevel
H  energy head
ΔH  head-loss
i  river slope
I  moment of inertia
k  permeability
L  section length in flow direction
m  discharge coefficient
mc  metacentre height
μ  discharge coefficient
p  grain stress
Δp  surcharge
Q  discharge
R  hydraulic radius
ρ  density
σ  normal stress
τ  shear stress
u  flow velocity
uc  critical flow velocity
Wx  wind friction term

1.1. Introduction.

Modern times made much possible that could not be done in the past. However, it is amazing, sometimes even a mystery, what has been achieved in the field of "closing" activities, already centuries ago. "Engineers" and craftsman used their skills gained by playing with nature's laws within the boundaries set by the restrictions of those days. Mechanization and science entered into this technology very late and rather slow. In my opinion not only because of conservatism but because the skills were so very developed that they were difficult to exchange for new.

This paper tries to give full credit to those old skills, as far as recoverable, since they still offer the possibilities of operating in special circumstances like calamities or in situations with limited logistics. Of course, where adequate, modern technology and science has to be incorporated. The old is not outdated, but the new is added.

From the old days (before 1600) only one incomplete copy of a manuscript (3 books out of originally 5) was found in 1920. In 1947 a book was published in which the prevalent state of the art was presented. Apart from the recent developments at that time a detailed description of many closures, realised in the past, is given. Since new developments entered slowly in this technology and by comparing this book with the 350 year older manuscript, a fair idea about the old know-how is obtained.

A closing operation is a struggle with nature. Flowing water on an erodible bed has to be controlled. Every human action to obstruct the flow will immediately be counteracted in some way or another by nature itself. Of course this happens within the laws of nature, of which many (but not all) are known. This knowledge gained in the past by (bad) experience, is supplemented these days by advanced research and experiment. Nevertheless, the changes in conditions during the progress of the closing operation are sometimes difficult to predict.

In the figure, an example of an unpredictable change in topography of a closure is illustrated. During the closure of the River Feni Estuary in Bangla Desh in 1984/85 the longitudinal profile of the alignment enlarged considerably in a month time. This was caused by a meandering secondary gulley and did not change the river's discharge, but a lot more material was needed for the closure.
Every closure design needs a full description of the existing situation in the first place. The hydrology, topography and soil structure of the area and climatological conditions have to be assessed. Then, a calculation is required to establish the change of conditions to be expected after the planned closure is realised. Lastly the intermediate phases of the construction process have to be detailed. Very important is to conclude which stages are critical and determining in the ever changing situation.

Data is but seldom complete and not always reliable. Besides, nature provides unpredictable conditions but sometimes they can be described statistically. Theory is an approximation of practice. Consequently there is always a rather high level of risk involved that things go different than expected. The historic cases clearly show the correctness of this statement. Much attention has to be paid to "what if" aspects of the design without typically over-dimensioning. However, the execution of a closure will always require a lot of improvisation in order to act immediately on nature's reactions.

1.2. **Purpose and Side effects of Closure works.**

Compared with closure works, few engineering works have such an extensive impact on the environment, in all its aspects. The purpose of the closure may be one (or more) of those aspects, the others go along as side effects automatically. A thorough study about these impacts is part of the design process. A feasibility study which does not name and estimate the negative aspects of the closing work is incomplete and invaluable.

Several aspects are of non-technical nature and some cannot be expressed in money, like social and cultural values. Evaluation of such considerations is not part of this book. Nevertheless, the engineer has the task to identify the consequential effects within his ability and to present them in a way that they are understood by decision-makers.

A number of purposes and side effects are listed below for reference.

**Main purpose of closing a watercourse:**
- land reclamation
- shortening the length of sea defence
- creation of fresh water reservoir
- generation of tidal energy
- fixed level harbour dock
- construction dock
- road- or rail connection
- repair of dike breach
- control of upland flow
- fish ponds
- short-cutting river bends

**Various possible side effects (dependent on circumstances):**
- change of tide (amplitude, flows) at the seaward side of the dam
- change in bar- and gulley topography, outside the dam
- disappearance of tides at the inner side of the dam
- change in groundwater level in adjoining areas
- alteration of drainage capacity for adjoining areas
- loss of fish- and vegetation species
- loss of breeding and feeding areas for water birds
- rotting processes during change in vegetation and fauna
- stratification of water quality in stagnant reservoir
- accumulation of sediments in the reservoir
- impact on facilities for shipping
- impact on recreation and leisure pursuits
- change in professional occupation (fishery, navigation)
- social and cultural impacts

1.3. Terminology.

A closure dam will generally be named after the location or the watercourse. To distinguish various types of closure operations several names have been adopted as well. Name adoption has been random rather than systematic although a systematic overview is given in the table below. Some names are typically Dutch and a literal translation may not exist. The name may refer to several aspects. Since every closure combines those distinctions the item considered most relevant determines the name used.

These aspects are:

*typical Dutch names are given in [brackets]*

i  basic method of closure.

1  gradual closure,
   relatively small size but flow resistant material is progressively deposited in small quantities into the flow until complete blockage is attained. This can be either a vertical or a horizontal or a combined closure:
   - horizontal (gradual) closure.
     sideways narrowing the closure gap.
   - vertical (gradual) closure.
     layer by layer upward narrowing the gap.
   - combined vert. and hor. closure.
     a sill is constructed first on which sideways narrowing takes place.

2  sudden closure.
   definite blockage of the flow by pre-installed flap gates or sliding gates, or by placing of a caisson or vessel.

ii  natural conditions.

1  soil conditions.
   never used as reference for a closure name; In broad terms a distinction can be made between:
   - sand, erodible, risk for liquefaction
   - clay, resistant against erosion, risk for slip or squeeze, sensitive for consolidation
   - peat, large consolidation, instable under quick loading, eroding in large lumps
   - rock, weathering and fracturing determine strength and watertightness
2 topography.
- tidal gulley closure.
  [stroomgat-sluiting]
  closure of a deep scoured channel in which high flow-velocities may occur.
- tidal flat closure.
  [maaiveld-sluiting]
  closure across a shallow area, generally drying at low water; characterised by critical flow at certain tide levels.
- reservoir dam (beyond the scope of this book)
  used in mountainous areas; requires temporary deviation of the flow in order to obtain solid foundation in the river's bed down to the bedrock.

3 hydrologic conditions.
- tidal-basin closure.
  characterised by regularly changing flow directions and still water in between; mainly determined by the tidal volumes and the storage capacity of the enclosed basin.
- partial tidal closure.
  a closure in a system of watercourses, such that after closure a waterlevel variation at both sides of the closure dam remains;
- river closure (non-tidal).
  closure determined by upland discharge characteristics and backwater curves.

iii materials used.
1 stacking-up willow mattresses. (opzinken)
  closure realised by successively dropping mattresses, made of willow faggots onto each other, ballasted by clay or cobbles.
2 sand closure.
  closure realised by pumping sand only at a very high rate of production.
3 clay or boulder-clay closure.
  lumps of flow resistant clay, worked up by grabs from floating cranes.
4 stone-dam closure.
  closure realised by dumping rock, boulders or concrete blocks in the gap, either by dump-barges and floating cranes, or by cableway.
5 caisson closure.
  closure with large concrete structures or vessels, self-floating and sunken in the gap, possibly provided with sluice gates.

iv equipment used. (typically for vertical closure)
1 bridge closure.
  closure realised by dumping material from a bridge, pre-installed across the gap.
2 cableway closure.
  dropping materials from a pre-installed cableway.
3 helicopter closure.
  dropping materials by helicopter.

v special circumstances.

1 emergency closure.
  is characterised by improvisation; basic idea is that quick closure even at high risk of failure
  prevents escalation of conditions; mainly used for closing dike breaches; needs
  strengthening afterward.

2 temporary closure.
  used for influencing the conditions elsewhere, for instance by stepwise reducing the
  dimensions of the basin; needs to be sufficiently strong during the required period but easily
  removable afterward.

1.4. Examples of Closure dams and their Locations.

Closure dams have been constructed probably since mankind started agriculture and
needed water for irrigation. Another reason could be political strategy because of road
or navigational connections. Not very much evidence exists of those activities in
ancient times. However, the irrigation projects that once existed in ancient Babylon and
Egypt suggest the presence of such works. As these dams will have been constructed
out of the local available transitory material, no remnants (like pyramids) are found.
They might have been quite extensive however, considering the fact that those people
were able to construct pyramids as well.

Anyway, in the delta area of the
rivers Rhine and Meuse, the dam-
ing of rivers and watercourses
develops in the early Middle Ages.
Because of agricultural expansion,
the moor areas, flooding only during
extreme tides or when rivers are in
spate, are artificially drained. This
makes the soil, mainly peat, settle
and as a result the incidence of
flooding increases. Therefore, the
areas are surrounded by small ear-
then walls and the natural drainage
channels are dammed up. Many
names of cities and villages in Hol-
land are named after these dams
(Rotterdam, Amsterdam). In the
period 1100 to 1300, even the
course of the two main rivers was
drastically changed by damming
activities.

The damming of the river Rhine:

Probably to prevent bank-overflow of the river, choked by sediments, the ruler of Utrecht dammed the
river Rhine at Wijk bij Duurstede around the year 1200. The flow was deviated via the river-branch Lek. Of
course, this dam had its side effects. It excluded the downstream area from further silting up and the outer
delta at the river’s mouth at Katwijk lost its sediment feeder. The coastline locally retreated in the next
centuries by several kilometres and the Roman fortress "Brittburg" disappeared into the North Sea.

The damming of the river Meuse (Maas):

In 1270 the river Meuse was diverted by damming at Maasdam (near the city of Dordrecht) and upstream
near Heusden, where the flow was directed towards the city of Woudrichem. This distorted the delta’s
discharging capacity in extreme conditions and led to a major inundation after the dike breached in 1421
(St. Elizabeth’s flood). Permanent loss of the most developed agricultural area of Holland (the polder Grote
Waard) by erosion of the top soil layer was the result. The region changed into a large tidal freshwater
basin, though unique in its existence.

![Likely course of the river Meuse (Maas) and the polder Grote Waard](image)

After the damming of the River Meuse.

These closure activities were certainly executed at periods of low discharge and
although the water could escape via other river branches, they must have been major
operations.

![Dike breaches in 1421 created a 200 km² tidal lake, gradually silting up.](image)

After the St. Elizabeth’s flood.

Whether or not the results of all these damming activities are to be judged positive or
negative is questionable. For nearly one thousand years all sediments, carried down by
the rivers, evacuated to the sea instead of regularly depositing onto the marshy land. The drainage lowered the watertable and made the peaty soil settle. It changed the landscape, its flora and fauna. Started as a simple water-level control system, it turned out to be a threat to the country. Gradually, large areas of the sinking ground were taken by the sea. The side effects, certainly when considered over very long periods, were tremendous. It left the people of today with a vast area, lower than sealevel, continuously threatened by water, fully dependent on its pumping capability for water evacuation.

The natural restoring process is well demonstrated in this example. The enormous lake created by the 1421-flooding, named Biesbosch, formed a settlement basin and after 550 years this lake was nearly completely silted up again and restored as a marshland.

Between 1500 and 1600 the attack by the sea reached a very critical stage but then new means to get control developed. A major improvement was the use of windmills equipped with a turn-table, able to pump the water up during all wind directions. And so, for hundreds of years the people had to continue the struggle against the sea and the rivers and they became the experts. Their skills were used all over Europe as for instance: along the German Bight, in the Baltic Sea, in the Wash in Britain and in the Gironde Estuary in France. These days very many closures have been realised all over the world and side effects of some recent ones are still awaited. An example is the retreat of the coastline of the Nile delta after the construction of the Assuan dam. Another, is the change in coastal topography in the south of the Netherlands after closing the estuaries within the Deltaworks-scheme.

In this book various examples of closure works will be referred to. For proper understanding they are listed below with the name and/or the location together with the year of closure. The list is not a complete list of closures executed in the past but is given because of its relevancy in this book. Many of the closures are situated along the continental coastline of the North Sea. These locations are indicated in the figure on the next page, numbered 1 up to 25, as in the table below.
Various Closure dams along the North Sea coast

<table>
<thead>
<tr>
<th>nr.</th>
<th>name or location</th>
<th>country or area</th>
<th>year</th>
<th>method or means</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Hindenburgdam</td>
<td>Sylt-Schleswig (Germany)</td>
<td>1925</td>
<td>sheetpile wall</td>
</tr>
<tr>
<td>2</td>
<td>Dagebuell</td>
<td>German Bight (Schleswig)</td>
<td>1633</td>
<td>sunken vessel</td>
</tr>
<tr>
<td>3</td>
<td>Meldorf, various gaps</td>
<td>Schleswig (Germany)</td>
<td>1978</td>
<td>sand closure; sunken barges</td>
</tr>
<tr>
<td>4</td>
<td>Closuredike Lauwerszee</td>
<td>Waddenzee (Neth.)</td>
<td>1969</td>
<td>concrete caissons</td>
</tr>
<tr>
<td>5</td>
<td>Closuredike Zuiderzee</td>
<td>IJsselmeer (Neth.)</td>
<td>1932</td>
<td>boulder clay (crane pontoons)</td>
</tr>
<tr>
<td>6</td>
<td>4 Dikebreaches Walcheren</td>
<td>Walcheren (Neth.)</td>
<td>1945</td>
<td>vessels and caissons</td>
</tr>
<tr>
<td>7</td>
<td>Veerse-Gat dam</td>
<td>Walch.-N.Bevel., Deltaworks</td>
<td>1961</td>
<td>caissons with gates</td>
</tr>
<tr>
<td>8</td>
<td>Stormsurge-barrier</td>
<td>Eastern Scheldt, Deltaworks</td>
<td>1986</td>
<td>steel gates between monoliths</td>
</tr>
<tr>
<td>9</td>
<td>Schelphoeck, var. gaps</td>
<td>Schouwen (Neth.)</td>
<td>1953</td>
<td>caissons and vessels</td>
</tr>
<tr>
<td>10</td>
<td>Brouwers dam, 2 gaps</td>
<td>Schouwen-Goeree, Deltaworks</td>
<td>1972</td>
<td>caissons; blocks (cableway)</td>
</tr>
<tr>
<td>11</td>
<td>Haringvliet-Sluices</td>
<td>Goeree-Voorne, Deltaworks</td>
<td>1971</td>
<td>concrete blocks (cableway)</td>
</tr>
<tr>
<td>12</td>
<td>Prim. dam Brielse Gat</td>
<td>Brielse Maas (Neth.)</td>
<td>1950</td>
<td>caisson</td>
</tr>
<tr>
<td>13</td>
<td>Braakman</td>
<td>Zeeuws-Vlaanderen (Neth.)</td>
<td>1952</td>
<td>sluice caisson</td>
</tr>
<tr>
<td>14</td>
<td>Sloedam</td>
<td>Walcheren-Zd.Beveland (Neth.)</td>
<td>1871</td>
<td>sinking willow mattresses</td>
</tr>
<tr>
<td>15</td>
<td>Ouwerkerk</td>
<td>Duijveland (Neth.)</td>
<td>1953</td>
<td>caissons</td>
</tr>
<tr>
<td>16</td>
<td>Grevelingendam, 2 gaps</td>
<td>Flakkee-Duijveland, Deltaworks</td>
<td>1964</td>
<td>small caissons; quarry stone</td>
</tr>
<tr>
<td>17</td>
<td>Oudenhoorn</td>
<td>Voorne-Putten (Neth.)</td>
<td>1953</td>
<td>caisson with side trap-doors</td>
</tr>
<tr>
<td>18</td>
<td>Kruiningen, var. gaps</td>
<td>Zd.-Beveland (Neth.)</td>
<td>1953</td>
<td>caissons; sandbags</td>
</tr>
<tr>
<td>19</td>
<td>Krammer closure</td>
<td>St.Phiilipsland (Neth.)</td>
<td>1987</td>
<td>sand closure</td>
</tr>
<tr>
<td>20</td>
<td>Bath</td>
<td>Dz.Beveland (Neth.)</td>
<td>1953</td>
<td>ship</td>
</tr>
<tr>
<td>21</td>
<td>Markiezaatskade</td>
<td>Bergen op Zoom (Neth.)</td>
<td>1983</td>
<td>quarry stone, vertically</td>
</tr>
<tr>
<td>22</td>
<td>Volkerak dam</td>
<td>Flakkee-N.Brabant, Deltaworks</td>
<td>1969</td>
<td>caissons with gates</td>
</tr>
<tr>
<td>23</td>
<td>Nieuwerkerk/IJssel</td>
<td>Holl. IJssel (Neth.)</td>
<td>1953</td>
<td>small ship</td>
</tr>
<tr>
<td>24</td>
<td>Ouwerkerk/IJssel</td>
<td>Holl. IJssel (Neth.)</td>
<td>1953</td>
<td>bags filled with sand</td>
</tr>
<tr>
<td>25</td>
<td>Papendrecht</td>
<td>Alblasserwaard (Neth.)</td>
<td>1953</td>
<td>sand bags, quarry stone, clay</td>
</tr>
</tbody>
</table>

In other areas several major closure projects have been realized also, as for instance:

<table>
<thead>
<tr>
<th>nr.</th>
<th>name or location</th>
<th>country or area</th>
<th>year</th>
<th>method or means</th>
</tr>
</thead>
<tbody>
<tr>
<td>26</td>
<td>Seosan</td>
<td>Korea</td>
<td>1987</td>
<td>very large scrap crude carrier</td>
</tr>
<tr>
<td>27</td>
<td>Feni</td>
<td>Bangla Desh</td>
<td>1985</td>
<td>bags filled with clay</td>
</tr>
</tbody>
</table>
2. Development of the Closure Technique.

2.1. Period until 1920.

The historic sources, mentioned in section 1.1. give a fair idea about the old methods used. The damming had to be done with locally available materials, which could be handled by hand and simple equipment. These materials were not stable under conditions of high flow velocities. The essence of the process therefore was to limit the flow velocities during closure progress within the limitation to size and weight of these materials. One way to achieve this was by splitting the basin area up in separate small compartments and consecutively close one part after the other. Experience indicated the maximum area size in relation to tidal rise which could be taken. Furthermore, flow velocities were kept low by using the vertical closing method, as will be clarified in section 3.

The most important material used was branches cut from willow trees. These branches were bound together in two ways. First, branches were bound together as a bundle making a sheaf-shape (called "rijsbos"), with a diameter of about 0.3 meter. Secondly, long strings of 0.1 to 0.2 m diameter and lengths of 50 m or more were composed. These strings (called "wiepen") were used for making a rectangular grating with a grid size of 3 feet (0.9 m).

These strings and sheaves were used for several purposes but the main tool for closure works, made with these components, was a mattress (called "zinkstuk"). Such a mattress consisted of an upper and a lower grating with several layers of sheaves in between. It was made on a strip of beach (called "zate") during the low water period, came afloat during high water and was then launched and sailed to the closure site.
The basic principles of closing a tidal channel were:

1. Protect the bottom of the channel against scour over sufficient length in flow direction. This was done by sinking mattresses by ballasting them with clay-lumps or cobbles. The area to be protected should be much more than the bottom width of the dam’s profile.

2. Raise a sill by sinking a number of thick mattresses one onto the other, called "sinking up" (opzinken). As the mattresses had an interwoven structure with much hollow space, they were stable but rather permeable and very compressible. Every mattress added weight onto the sill and compressed it.

3. The sinking was executed at the moment of still water and consequently could not be done to a higher level than the tide allowed. This was usually slightly above low-water level. At the seaward side, the water level would then fall below sill level for small periods.

4. Next, over the sill a dam had to be constructed, proceeding from both ends towards the centre. This was done, dependent on circumstances, by depositing clay into a sort of clay wall, or by piling up these willow sheaves, knitting and fixing them together in a wall shape, ballasted with clay. (This willow-and-clay wall can be seen as an ancient way of "terre armée" and is called in Dutch: "rijspakwerk").
The closure of the Sloe between the isles of Walcheren and Z-Beveland in the year 1871 is a good example of this procedure. The gap had a width at low water level of 365 m and a maximum waterdepth of 10 m. The tidal range was about 4 m. By sinking mattresses a sill was constructed up to the level of about low water. This sill had side slopes of 1 in 1 and a crest width of 18 m. The next stage was the willow structure "rijspakwerk" on top of the sill. In order to fabricate this wall up to a level of high water, thus 4m height, the sill settled 1.80 m, as consequence of the added weight,. So, in practice a dam of 5.80 m had to be made and that took a full month.

The closure being realised, the actual dam had still to be made. The initial profile was too weak, very compressible still and risk for piping via the willow branches was high. Generally, the initial profile was made part of the definite profile by adding a clay profile against it.

Sometimes, the risk for piping was considered too high during the construction of the closure profile, because of slow progress or of high head loss over the dam. If so, the design was different. The bottom and slope protection, was executed in two parts leaving an unprotected strip in the centre line of the profile. Then two sills, as under 2, were made on both parts leaving a dip in between. This dip was filled by clay only, which, since there are no willow branches running from one side to the other, made an impermeable core in the centre of the dam. This method was much safer but required much more material and took a longer time to construct. And time meant longer exposure to high flow conditions and more risk for getting adverse weather.

The skills required for these operations were, apart from operation managing and logistics, the craftsmanship of how to make a strong willow mattress, how to manoeuvre it to the site under high flow conditions by rowing or sailing, how to position it on the exact location by wires and anchors, and how to sink and ballast it in the limited time of still water. The higher the sill, the more difficult it was. But a higher sill made the last step, the willow and clay crosswall, easier to construct.

At last, if these methods were not successful, it was tried to position a vessel into the final gap and sink that onto the sill. This was not a simple operation, as transport was done by sail or by rowing, and winching by hand was the only driving force. Timely ballasting and prevention of escalation of piping under and around the vessel were very critical.

A historic example is found in the closure of the "Bottschlottertief" near Dagebuell (NW-Germany) in 1633. Clay had to be sailed in from far and it took 5500 labourers to execute the job. The closure was done by sinking a vessel into the gap. It was ballasted and surrounded by clay for the transport of which some 350 carts were used.

A river with mainly permanent flow does not have the advantage of still-water periods. The closing therefore was different, although the materials are identical. A bottom protection had to be sunk, however, not on still water but during flow conditions, of course in a period of low river discharge. Sinking up of a sill is more complex due to the lack of water level- and flow variation. Therefore, the sill could not be made up to a
high level and that made the construction of the willow-and-clay wall impossible. Then another rather complex structure was made called "baardwerk".

In general terms, the technique consisted of the construction of a sort of mattress, straight on the spot, in floating condition proceeding from the shores towards the centre. The grating was not rectangular but diagonal strengthening was provided to withstand the sideways shifting forces by the flow. Every time a part, called beard because of its shape, was ready, it was ballasted, by which it sank, covering its predecessor. It was then sloping down from the water level at its root end, to the sill level at its front end.

2.2. 1920 until 1952.

Gradually mechanisation started influencing the work methods. The steam engine was known already for decades but the equipment was voluminous and heavy, both being troublesome in swift water and on soft ground. However, it could be used as the driving force for winches, for driving sheet-piles and poles, for transfer of materials by cranes and for ship propulsion. Transport by rail, across foreshore and fresh dambodies was easier by locomotive engine, for which a stable railway had to be constructed. So, initially the existing technique was only changed from hard hand labour into engine work. However, better foundations for its transport roads and rails were needed, it was vulnerable to settlement in fresh made ground and transport over water required more water depth.

An illustration of the difficulties encountered gives the closure of the Hindenburgdam, a connection between the Isle of Sylt and the mainland of NW-Germany, made in 1923/27. The area was very shallow and sailing was impossible. The tidal difference was 1.70 m average, but local wind effects much influenced the tides. The selected work method was to build-out a wooden sheet-pile wall. The piling was followed by tipping quarry stone on both sides for support. Transport of the stones was done by rail laid on a bridge piled alongside the sheet-pile wall. Progress was much slower than anticipated and the erosion in front of the works consequently much larger. The piling thus had to be done in high turbulent water in a scour hole that preceded the sheet-pile construction and therefore more stone was needed for stabilisation. On the inshore side, the railway was installed on made ground which regularly subsided and derailments were regular, escalating the problem. Later, the work method was adapted. The preceding scour was prevented by laying a 10 m wide stone protection onto the bottom and the rail foundation was improved. Then the problems were overcome.

Apart from the above mentioned problems, a disadvantage of this type of equipment is that failure of the engine (damage) leads to major break-down of the complete works. The system is less flexible.
Learning how to adapt the methods, using the new equipment, also showed that new possibilities arose as well. The engines could handle heavier units and reach higher production capacities. The advantages of this are:

- **heavier units:**
  - can deal with higher flow velocities,
  - give reduced material losses.

- **higher production capacities:**
  - give a shorter critical phase,
  - give more progress in a still water period.
  - lead to shorter execution time, thus:
    - could do more in a workable period,
    - reduce the risk for incidental bad weather.

Owing to these new techniques larger projects and projects with more critical conditions became possibilities.

For instance, in 1932 a very large closure was realised in the Netherlands when the former Zuydersee was cut off from the sea by the Enclosure Dike. The dam, with a length of 32 km, crossed two main gully systems. During the execution of the works large deposits of boulder-clay, a glacial till, were found. This material appeared to be very stable in the flow and could be handled by large cranes. A complete new set of floating cranes and transport barges was built and the closure was completely made by these large floating units.

Another important change in the closure design was the development of mathematical modelling. Originally a matter of experience and feeling, calculations started to replace the trial and error system. This reduced the risk of failure. For the very large projects this was essential. For the damming of the enormous tidal basin, the Zuydersee (now called IJsselmeer) in 1932, the differential equations for tide-propagation had to be solved. This was achieved by prof. Lorentz. Two questions had to be answered before the job started:

- how will the tide change when the works are ready, as this will affect the closing conditions?
- what will be the design condition for the dike's profile in this new equilibrium state of the sea (water levels and waves)?

A third question, being one of the side-effects, could not be answered:

- how will the sea outside the dike adapt in the long run and change its topography and morphology?

The Enclosure Dike completely changed the tidal conditions on the seaward side. The amplitude of the tide gradually increased to more than two times the former tide with the progress of the closure.

Another challenge was presented when in 1944 for military reasons (world war II), the island of Walcheren was inundated by bombing the surrounding dike in four places. It dislodged the enemy troops and opened the fairway to Antwerp for the army fleet on the one hand but it demolished the sea-defence and opened the low-lying island for tidal penetration on the other hand. Restoring the sea-defence had to be done quickly in order not to loose the island permanently. Again,
the mathematic basis for calculating tide-propagation improved. The four gaps, three on one storage-basin, each with its own tidal amplitude and phase, and the propagation over inundated land with obstacles and ditches, partly drying at low tide, was a very complex system to allow a mathematical approach. And this was needed to establish the most favourable order of progress and also to ascertain risks if in practice a different path would occur. Besides, due to the progressive erosion of gulleys, the hydraulic resistance changed with time.

Immediately after the bombing the gaps in the dike were still relatively small. With the tide flowing in and out twice daily with ranges of 3.5 to 4 m, erosion deepened the gaps and a system of gulleys scoured into the inland area towards the gaps. There was no material nor equipment available (wartime) and the areas were covered with mines. In June 1945 when a start could be made, closure of the gaps was nearly an impossible task. The traditional methods of closure failed because they progressed too slowly, or the equipment and materials could not cope with the circumstances. The four gaps had to be closed simultaneously within a period of four months (before winter storms) and these closures were inter-related.

The only suitable and available means to realise these closures were the caissons of Mulberry Harbour, temporarily used a year before, during the invasion of the Allied Army in Normandy (France). After providing a scour-protection in the gaps, all sorts of large units, like pontoons, caissons, concrete and steel vessels, even large quantities of anti-torpedo-nets were dropped or positioned in the gaps. The job was not ready before the winter and conditions worsened. Several times, initial success failed a few days later due to storm surges and piping. By the end of January 1946 however, the gaps were closed.

Much experience was gained with the handling of caissons and vessels in closure gaps and ideas for the design of purpose-made caissons developed. An improvement would be to create a gap profile in agreement with the caisson shape (and reversed). Besides, the sinking would have to be controlled in a better way by regulating the water inlet by valves and separate chambers.

Different plans to improve the sea defences of the deltaic area of the Netherlands were detailed and several closures were made. In 1950 the river mouth of the Brielse Maas was closed using a purpose made caisson. In 1952 the Braakman, an estuary along the river Western Scheldt, was closed using two caissons, of which one was equipped with sluice-gates. These temporary gates could be opened after positioning of the caisson into the gap in order to reduce the water head in the basin after closure and thus restrict the forces.

2.3. Period after 1952.

A new flood disaster occurred in the southern North Sea on the 1st of February 1953 (according to Dutch tradition this flood should be named: St-Ignatiusvloed). A storm surge together with springtide-high water inundated 2000 km2 of land in the Dutch Delta and created 73 major dike-breaches and very many smaller ones. And again, all
technical experience and equipment and improvisation had to be used on many sites simultaneously, to close these gaps before the next winter season. Not all the gaps had initially the same degree of difficulty or dimension. However, various gaps could not be dealt with immediately because of the disrupted infrastructure and as a result they scoured to tremendous dimensions.

The gap named Schelphoek increased from an initial 40 m width to 525 m after 6 months, while the maximum depth increased from 10 m to over 35 m.

A typical example of successful quick improvisation is the closure of the gap at Ouwerkerk on the IJssel. The storm surge on this spot reached a level of 3.75 m above M.S.L., overtopping the dike. The unprotected inner slope of the dike slid down over a length of about 40 m and the top layer of the dike slid and scoured away. However, the slope protection on the outside remained intact up to the level of +1.70 m, as it rested on a centuries old clay-core. Six hours later, at tidal-low water, still reaching a level of +2.00 m, two small vessels have been positioned onto the outer slope, which broke the force of the waterfall, although piping underneath was severe. Jute-bags filled with sand were carried in by hand and a small embankment was created on top of the dike-remains. At the next high water, +2.80 m, the emergency provision remained intact and could be strengthened.

The many very difficult circumstances led to all sorts of innovative actions, which
resulted in repair within 10 months. The table below illustrates the enormous achievement:

<table>
<thead>
<tr>
<th>date</th>
<th>nr of gaps closed</th>
<th>remaining gaps</th>
<th>inundated area</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 febr.</td>
<td>3</td>
<td>70</td>
<td>2000 km²</td>
</tr>
<tr>
<td>8 febr.</td>
<td>+8 = 11</td>
<td>62</td>
<td>2000</td>
</tr>
<tr>
<td>15 febr.</td>
<td>+6 = 17</td>
<td>56</td>
<td>2000</td>
</tr>
<tr>
<td>1 march.</td>
<td>+20 = 37</td>
<td>36</td>
<td>1400</td>
</tr>
<tr>
<td>1 april.</td>
<td>+17 = 54</td>
<td>19</td>
<td>800</td>
</tr>
<tr>
<td>1 may.</td>
<td>+7 = 61</td>
<td>12</td>
<td>220</td>
</tr>
<tr>
<td>1 june.</td>
<td>+4 = 65</td>
<td>8</td>
<td>150</td>
</tr>
<tr>
<td>1 july.</td>
<td>+3 = 68</td>
<td>5</td>
<td>150</td>
</tr>
<tr>
<td>1 nov.</td>
<td>+4 = 72</td>
<td>1</td>
<td>100</td>
</tr>
<tr>
<td>dec.</td>
<td>+1 = 73</td>
<td>-</td>
<td>getting dry</td>
</tr>
</tbody>
</table>

And again, the experience was used in later developments of the closing technology.

This appears for instance from the following example. The principle of a temporary closure made in 1953 near Kruiningen (Waarde) is copied on a much larger scale, in 1985 to close a major estuary in Bangladesh (Feni River). In the latter case, 1,000,000 bags filled with clay, totalling about 20,000 m³, stored in 12 stockpiles along the alignment, were carried by 12,000 Bangla-Deshi labourers into the 1000 m long gap to shape a dam, within 5 hours.

The disastrous flooding in 1953, with all its negative aspects (1835 people drowned), had an offspring in the decision making for the reconstruction of the sea defence in the Netherlands. Most estuaries would be closed during the next 25 years, in order to shorten the defence length (the Deltaplan). Although many closures were beyond experience, it was considered possible to develop the required methods during that period, working from the small to the large scale projects. This period is therefore characterised by many experiments, a lot of research and the introduction of new materials and technology.

In a later stage it was decided to adapt the plan to the changed views regarding ecologic importance and the largest estuary (Eastern Scheldt) was provided with a storm-surge barrier, which took another 8 years to construct. Since parts of the closure dam were already made and the new design and its execution went parallel, many problems arose in this period. A lot of new ideas had to be generated and tested. The much improved computer- and measuring facilities played an important role. As a result of all these efforts, the present day designer has a lot of rules, formulas, graphs and test-results at his disposal.
3. **Use of Hydraulics.**

3.1. **Upland discharges and Tides.**

Damming a watercourse requires thorough knowledge of the hydraulic behaviour of the total water-regime of which the course is a part. A quick overview of the conditions that may occur, as a result of tides and upland flows, is given in this section. Generally, the impact of the closure on this regime is such that during closure the hydraulic boundary conditions near the closure site change. The change has to be determined before the works start. Calculation of this is the first part of the technical design process. Dependent on circumstances this may vary between a simple calculation and very complex mathematic modelling by computer.

Upland rivers are dammed for different purposes. In case of reservoir construction the river is blocked completely in order to store the upland discharge. Only in case of excess, water is evacuated via a spillway. Damming will have to be done in a low discharge period, but some discharge will always exist. Since in the final stage over the dam (very) large waterlevel differences will occur, the foundation of the dam has to be very stable against piping and sliding. Therefore the flow is usually temporarily diverted to enable removal of the sedimentary subsoil and improving the river’s bed. Analysis of discharge data and the probabilities of their extremes per period are needed for the design of the various construction phases.

Lowland rivers generally show a meandering pattern or they comprise a system of various channels and bars (braided river). Short-cutting a meander or concentrating the flow in one channel requires the closing off of the superfluous channel. During the progress of the closure, the discharge will gradually take its new course. Likewise gradually, the fall in water-head over the closure gap will increase. This reaches its maximum when the full discharge follows the new course and is equal to the fall in head over the length of the new channel. Of course this fall depends on the discharge quantity of the river and the probability of occurrence of this determines the design condition for the closing method. A sudden peak-flow in the upper river (e.g. due to a rainstorm) will raise the water levels and change the head loss and thus the flow velocity. Both, level and flow, affect the condition for the closure. Rating curves will provide a timely warning.

For instance, a channel connecting two points A and B of the main river system (see figure) is sloping down at 2 m fall over 13 km length. The flow velocity will be in the order of 0.80 m/s. A closure is planned halfway, at C. Assume that the water level at A and B do not change. As the closure progresses, three river sections have to be considered. At C, the profile diminishes while the flow velocity increases. The resulting discharge, however diminishes also. Section AC is a river section in which the diminished discharge makes the flow velocity drop while the waterlevel is pushed up by backwater from C. CB however, is a river section in which the level is drawn down. The reduction in
head loss over AC and CB is the fall over the gap at C, responsible for the high flow in C. In the example, a 90% blockage results into a flow velocity in AC of 0.40 m/s, in BC of 0.55 m/s and in the gap of 4.50 m/s. During the final stage, the head over the gap is the full difference in level between A and B.

In deltaic regions, rivers may bifurcate into a complex system of river-branches. Closing one of these branches is comparable to the above situation, with the assumption that the waterlevel on either end of the channel is fully determined by the deltaic regime. The length of such a river section to be closed may be considerable and likewise the fall in head during the final stage of closure may be quite large. For the channel sections, upstream and downstream of the closure, Chezy's formula for open channel flow applies:

\[ u = C\sqrt{R\cdot i} \]

For the gap, the weir formulae, detailed in paragraph 3.2, have to be used.

In coastal areas tide may be the governing factor. Since the tidal conditions vary enormously over the earth, a general description of the tide characteristics follows.

The tidal wave is initiated by the earth's own rotation and the circling around of the moon. Gravitational forces between earth and the celestial bodies result in an uplift of the sea-level at the side of the body and also diametrically opposite. Perpendicularly, a draw-down results.

Relative to the earth, the sun circles around in 24 hours, the moon in 24 hours + 50 minutes and so do the respective uplifts.

Since the only parallel-circle on earth with continuous sea-surface is situated at about 65 degrees South, the initial double-wave circles around across the southern oceans. The circumference of this parallel has a length of about 16,000 km, so the wavelength is about 8000 km. The moon's wave (called M2) thus passes every 12 hrs 25 min., the sun's wave (S2) every 12 hrs.

Although the moon's mass is much smaller than the sun’s, the short distance to the earth makes the moon’s effect the biggest. The two effects coincide when earth, sun and moon are in line, which is during new moon and during full moon (M2+S2 = springtide).

3-2
Due to the 25 minutes difference, the two go out of phase and seven days later the effects are opposite, which is during the first and the last quarter (M2-S2 = neaptide).

The waves on opposite sides of the earth are not equal which means that there is a height-difference in the consecutive daily high and also in the daily low waters. If a waterlevel record is analyzed by harmonic analysis this effect appears as a few diurnal components with a period of about a day. The most important of these are called O1 and K1. Combination of these diurnal components intensify or weaken the resulting daily inequality periodically.

The resulting wave travels along the Southern Ocean from East to West. From there it enters into the Pacific, Indian and Atlantic Oceans going North. As a result of the earth's own rotation, the flow-pattern in a large basin will tend to rotate (Coriolis' acceleration). This is clockwise on the northern and anti-clockwise on the southern hemisphere.

For the tidal wave this may result in a circular propagation along the basin's circumference, while in the centre the tide is negligible (the amphidromic point). This rotation (path of the wave-crest) is the opposite of the effect on the flow, anti-clockwise on the northern and clockwise on the southern hemisphere. The time of travel required to reach any spot on earth determines the time-lag of the tide and consequently that of the spring-neap variation relative to the moon's phase.

The travelling speed can be approximated by the celerity formula:

$$c = \sqrt{gd}$$

which for the oceans is about 200 m/s. Travelling up the Atlantic to the North Sea for instance takes 24 hrs. On the continental shelf the water-depth diminishes to 200 m and in the North Sea further to 25 m, by which the
wave speed reduces to 15 m/s. The tidal wave enters into the North Sea around Scotland, moves south along the British east-coast, east towards Denmark and north to the Norwegian coast. In the North Sea there are two major amphidromic rotations. In the Pacific Ocean, there are six of these rotations, which complicates the picture of the tide-propagation.

The astronomical effects as gravitational forces and rotations are the basis for the tidal wave, but other effects will influence its shape. While travelling, the wave changes as consequence of variation in depth and due to resonance and reflection. Some areas are influenced by tidal waves approaching from various sides, as for instance north of Australia where the waves via the Pacific and via the Indian Ocean meet. Finally, it may be influenced by meteorologic phenomena. A half-yearly constant wind like the monsoon-winds changes the tide with the same frequency as the sun's declination which causes the winter-summer variation.

The resulting tide at any seaside therefore has its own typical characteristics, related to the astronomy, very much adapted during its travelling across the oceans and seas and definitely shaped by the geometry of the area. Generally, the $M_2$ and the $S_2$ are the governing components. Then the tide is called "semi-diurnal". At various locations the diurnal components intensify and the tide is then called a "mixed tide". Very seldom the diurnal effect overrules the semi-diurnal components. In that case the tide is called "diurnal tide". The spring-neap variation, as a result of the semi-diurnal components, does not show in that case. Diurnal components show a similar effect.

To illustrate the differences a mathematical example is given in which four harmonic constants are added up: $(M_2 + S_2 + K_1 + O_1)$. The four graphs differ in the amplitudes of the constants only. Angles and phases are taken identically, while the sum of the four amplitudes is about the same. The resulting graphs show a typical semi-diurnal tide, two mixed tides and a typical diurnal tide.

The semi-diurnal tides are found along the Atlantic coasts generally, the mixed tides along the Indian and large parts of the Pacific Oceans, while the diurnal tides occur more incidentally, e.g. in Vietnam.

At last, the wave enters into estuaries and river-mouths. Shallows, funnel-shapes and upland discharges have their impact on the penetrating wave.
Generally, they result in enlarging the tidal amplitude. The top of the wave (high water) may then propagate faster than the trough (low water) and the wave-front gets steeper. In the harmonic analysis this shows as quarter-daily constants, like $M_4$ and $M_6$. In extremes, the top may overrun the trough and the tide enters as a tidal bore.

The relation between the water level variation and the flow velocities is an important characteristic of the tide. In relatively small basins (length smaller than 0.05 times the tidal wave length) the two variables will be 90 degrees out of phase. At the moment of high water, the basin is full and the inflow stops. And reversed for low water. At the moment of mean level, the flows have their maximum. For a propagating wave this is not true. The still water after flood or ebb may lag behind for some hours. If so, the maximum flood flow occurs during higher water levels on average than the ebb. The mass of water entering the estuary during the flood period, the flood volume, has to flow out during the ebb period with lower levels. Ebb velocities are generally the largest and follow the deep gulleys.

In some shallow river-mouths a typical phenomena occurs. On the average, the flood volume entering during high water, discharges during the low water period. During springtides this tidal volume is larger than during neaps, while the difference in water depth between high and low water is greater. Then, during springtides, the flood may bring more water than the ebb can return. This is particularly true in shallow rivers with high tidal ranges. Water stays behind and the mean level starts rising until spring has got its maximum range. Thereafter, going to neaps, the difference in depth reduces and the system balances out by lowering the mean level. In the harmonic analysis this effect appears as a component with a fortnightly period, called $MS_f$, which goes in line with the spring-neap variation.

It will be clear that the tide characteristics on any spot on earth have to be analyzed very thoroughly before a closure of a tidal basin or in a tidal basin can be designed. Since much of the final appearance of the tide is determined by the geometry of the area near the closure site, the closure itself will have an influence on the shape of the tide. Selection of the boundaries to the area studied has to be carefully done in order to make sure that they are chosen outside the area influenced by the works.

### 3.2. Flow through gaps.

During the progress of closing a watercourse a distinct constriction to the flow develops. This constriction can be in vertical sense by creating a sill, or in horizontal sense by the construction of dam-heads, or both in combination. Chezy's law for flow in open channels does not apply any more. Dependent on the dimensions of the gap and the water depth, different flow patterns may exist. Consequently different formulae apply for calculating the magnitude of the flow and the discharge capacity of the gap.
In case of a horizontal constriction the flow velocity can be approximated by the formula:

\[ u = \sqrt{2gh(H-h)} \]

in which \( g \) is gravity and \( H-h \) stands for the difference in head over the gap. All potential energy is transferred into velocity and friction is ignored. When the water flows through the gap the flow lines contract and behind the gap strong eddies develop. Where the flow is narrowest, the velocity reaches the above value which is its maximum. Generally, this will be downstream of the gap. In the gap the average velocity is smaller by a factor \( \mu \). This \( \mu \) is the ratio between the cross sectional flow areas of the gap (A) and the flow gorge. The discharge capacity of the gap is:

\[ Q = \mu \cdot A \cdot u = \mu \cdot A \cdot \sqrt{2gh(H-h)} \]

If the dam heads have a rounded shape of considerable width, the flow lines follow the dam head contours and energy losses due to friction are rather small. \( \mu \) approaches the value 1. An example of \( \mu = 1 \) is found for sand closures where the shape of the dam head is adapted by the flowing water itself. For closures made by tipping quarry stone a narrow steep dam head exists. Then, in the gap already, the flow separates from the dam head. The profile of the gap does not fully contribute to the effective discharge, so the value of \( \mu \) gets smaller than 1. Besides, a lane of vortices develops in the gap in the shear layer between flowing and stationary water. This results in high energy losses. Instead of a "\( \mu \)", a discharge coefficient "\( m \)" is taken, which takes account of both effects. The value of \( m \) may be as low as 0.8 for these closures.

For the closure operation these aspects mean that the highest flow velocities develop downstream of the gap above the (protected) bottom. The dam is attacked at the point where the flow separates and the shear layer starts developing. The most severe attack on the bottom occurs in the vortex lane along the wake.

A vertical constriction gives a completely different flow pattern. Assuming an infinite length of the gap, the flow pattern can be denoted two-dimensionally in a vertical section. The flow may be sub-critical flow or super-critical flow, dependent on the relative levels of the water and the sill crest.
For sub-critical flow the discharge formula reads:
\[ Q = m \cdot B \cdot h \cdot \sqrt{2 \cdot g \cdot (H - h)} \]

in which \( H \) is the energy level of the upstream water and \( h \) is the water level at the downstream side of the gap, both measured with reference to the crest level of the sill and \( B \) is the gap length. The value \( m \) depends on the shape and the roughness of the sill. In fact, the \((B \cdot h)\) is not the actual flow profile above the sill as the water level there is generally lower than the downstream level. The actual water depth "\( a \)" above the sill is unknown, so the formula has to be used and the error has to be included in the value of "\( m \)", which may therefore reach values higher than 1. The magnitude of the "\( m \)" goes from 1.3 for wide smooth sills with gentle slopes to 0.9 for sharp crested sills. The mean flow velocity above the sill is:

\[ u = \frac{Q}{a} = \frac{m \cdot h}{a} \cdot \sqrt{2 \cdot g \cdot (H - h)} \]

which generally is slightly more than calculated by "\( m \cdot (2 \cdot g \cdot z)^{0.5} \)".

If a closure gap with sub-critical flow is further constricted by progressively heightening the sill, a situation will develop in which the flow becomes critical. This is the case when the vertical distance between sill level and downstream water level is 2/3 of that at the upstream side. The discharge capacity is fully dependent on the upstream energy level in relation to the level of the sill. Then, \( z = \frac{1}{3} \cdot H \)

\[ Q = m \cdot B \cdot a \cdot \sqrt{2 \cdot g \cdot \frac{1}{3} \cdot H} \]

If the downstream water level goes further down, the discharge capacity is not influenced any more and "\( a \)" remains constant viz. 2/3 \( H \). So,

\[ Q = m \cdot B \cdot \sqrt{\frac{2}{3} \cdot H} \cdot \sqrt{2 \cdot g \cdot \frac{1}{3} \cdot H} \quad u = m \cdot \sqrt{2 \cdot g \cdot \frac{1}{3} \cdot H} \]

The value of \( m \) depends on the inflow condition and the sill's roughness, as the flow is independent of the downstream condition. Thus the maximum value of "\( m \)" is 1. In case of sharp crested rough sills it may go down to 0.8 The water falls down along the sill's downstream slope and changes into sub-critical flow via a water-jump. Dependent on conditions and Froude number this may show different flow patterns.
For the closure operations these downstream conditions may be more critical than the conditions on the crest of the sill. Suction forces will attack the downstream slope and endanger stability of the sill. The down pouring jet flow may reach the bottom in case of a closure in relatively shallow water and endanger the bottom stability.

Finally, the combination of vertical and horizontal closure frequently occurs. Then, a situation may exist in which the gap has a vertical constriction by a sill and dam heads on both ends which are progressively build-out. This three-dimensional situation is more complex than the separate ones and difficult to mathematically describe. The general approach is to add the two effects as if they were independently occurring. However, the peak velocity downstream of the gap, usual for horizontal closure, may be compensated by the increase of water depth after the flow has passed the sill.

3.3. Mathematic modelling.

The mathematic modelling of the hydraulic situation in the various stages of the closing process starts with the inventory of the determining boundary conditions and the estimation of the boundaries of the area which will not be affected by the closure. The latter, in tidal basins, is sometimes rather difficult to calculate, but the result is sufficiently accurate when the effect is less than a few centimetres. The resulting error will be less than the influence generated by the natural variation in tidal conditions.

External influences may be:
- Upland discharge(s) coming down the river(s).
- Tidal waves passing along or entering into the basin to be closed.
- Drainage water from surrounding areas.
- Direct rainfall in the basin.
- Wind set-up and draw-down in the area outside the basin.
- Wind set-up and draw-down within the basin.
- The earth’s rotation (Coriolis’ acceleration).

The conditions which determine the hydraulic behaviour within the basin are:
- The network of gulleys and shallows (if applicable).
- The flow profiles (depth/width relation) of the gulleys.
- The storage (level/surface area relation) of the shallows.
- The hydraulic resistance of the gulleys.
- The existence of density (mainly salt/fresh water) currents.
- The occurrence of tidal bores.

The mathematics available to describe the hydraulics are the basic formulae for conservation of mass and of momentum.

\[
\frac{\delta Q}{\delta x} + B \frac{\delta H}{\delta t} = 0
\]

\[
\frac{\delta Q + \delta (a*Q*\mu)}{\delta t} + g*\frac{\delta H}{\delta x} + g*Q*Q - W_x = 0
\]

\[
\frac{\delta Q}{\delta x} = \frac{g}{C^2} \cdot A + R
\]

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These can best be solved by a numerical computation of unsteady flow in a network of watercourses. The extent of the network, the number of storage areas, the schematisation of the flow profiles, the terms in the equations ignored (if any), and the correctness of the guessed hydraulic resistance determine the accuracy of the calculation. Reversed, if the accuracy is not too much affected, the mathematic system (and thus the quantity of desk work) can be simplified. Besides, for a first orientation, and for determination of an order of magnitude, these simplifications are indispensable.

An extreme simplification is to ignore the conservation of momentum in the basin. What remains is the mass balance. For the gap the weir-formulae apply. The waterlevel in the basin is assumed to be horizontally over the full surface area at any time. To represent drying shallows, a variable area for different heights can be taken. The water level changes in time. This change multiplied by the surface area of the water must balance the quantity of water flowing through the gap per the same time laps. The tidal wave propagates at infinite speed within the basin and it does not reflect nor dampen. Besides, the simplification assumes that the water moves towards and from the gap all over the basin without driving forces and without friction.

The deviation is acceptable if the basin is relatively short and not too elongated. And of course friction should be reasonably low, which in practical terms means that the area is well cris-crossed by gulleys.

The quantity of water flowing through the gap per time laps is determined by multiplying the flow velocity and the cross sectional profile of the gap. The first one changes as result of the changing head loss over the gap, the latter as result of the changing water level in the gap. If a vertical closure is considered, a check on the flow condition, non-critical or critical, is necessary for every time step, as different formulae to calculate the flow velocity apply. More gaps than one can be dealt with by a simple summation of the water quantities as long as the condition "no propagating wave" is fulfilled. One gap with variable depths can be schematised by summation of a number of gaps each having different dimensions (length and depth).

In vertical closures various construction stages will be defined, each having a specified sill level. However, during actual construction there will be intermediate stages in which only part of a new layer is present. Besides, a failure may create a local depression in the sill's level. In these depressions higher flow velocities will exist. Therefore, the calculation should always be executed with a gap of variable depth. The plain horizontal sill is a theoretical case and does not represent a determining practical situation.

When closing a dike breach, the storage basin may be an inundated area in which it is very difficult to define a system of gulleys and shallows. Besides, due to erosion of the instable ground the system may quickly change. The Chezy value for this overland flow may be as low as 30. The area will fill up during high water periods but not fully drain during low water. In these calamity situations it will be of the utmost importance to prevent the development of scouring gulleys and to maintain the high resistance of the terrain (See chapter 2.3, the gap called Schelphoek).

As an example, a calculation is made to illustrate the change in the tidal characteristics during the closure of a tidal basin. The calculation is made for two different basins, each having the same water surface area, 50 km$^2$. This is kept constant for all tidal levels, which implies that there are no drying parts in the basins during low water periods.
One basin is relatively short and rectangular, viz. 5 km wide and 10 km long. The bottom elevation is taken constant all over the basin at a level of 6 m below mean sea level (MSL). For this basin the waterlevel will be nearly horizontal at all tidal heights over the full area. Thus a calculation based on conservation of mass only would be appropriate.

The other basin is a long, narrow, funnel shaped estuary, in which the tidal wave propagates. The total length is 41.750 km and the width narrows from 2000 m to 800 m at the end. The bottom level rises from -6 m at the entrance to -2 m at the end (relative to M.S.L). The celerity of the tidal wave is on average about 5 m/s and thus the tide at the end of the basin will lag more than 2 hours behind. The calculation needs to consider the conservation of mass as well as of momentum.

For comparison, both basins are closed by horizontal closure as well as by vertical closure. The results show the influences of the basin's shape and the closure method. The four cases are pictured in four sets of graphs showing curves for water levels just outside and inside the basin near the gap and at the end of the basin, as well as the flow velocity in the closure gap.

The cases thus are characterised by:
- case 1, horizontal closure of short basin
- case 2, horizontal closure of long basin
- case 3, vertical closure of short basin
- case 4, vertical closure of long basin

All four cases have been calculated for stepwise diminishing the gap size in 5 stages (initial gap taken as 100%, 50%, 25%, 10% and 3%). For a vertical closure the gap size is more difficult to define than for a horizontal closure, since for high sills the gap size for ebb differs a lot from the one for flood. Expressed in m² relative to MSL, even a negative size is possible. The sill levels have been selected such that a comparison of the horizontal and vertical characteristics is possible.

For reasons of fair comparison various parameters have been taken identically and constant in the calculations. These are:
- all discharge coefficients are taken as 1.0 for all gap dimensions, in both, horizontal and vertical closures.
- all constructed parts of the closure dam are considered impermeable for water
- the tidal wave at the entrance is a single sine-wave with a range of 3 m, which does not change as consequence of the progressing closure.
- the Chezy-values of all sections in the calculation network are taken 50 for all water depths.
- the calculation is made using the mathematical model "Duflow"

It should be realised that in a practical case the model has to be calibrated by reproducing an actually measured situation. The results of that will give the required data for Chezy-values and dimensions of the sections (depth-width relation to represent an irregular gulley profile).

In the results a few typical characteristics can be observed. For the calculations with the short basin (cases 1 and 3), the water levels at the end of the basin do not show as they are identical with the basin
level near the gap. For the long basin (cases 2 and 4), the waterlevel at the far end (dotted line) is lagging behind, as expected. Besides, at the end of the basin, the high water reaches about the same level as near the gap but low water is a lot higher. Consequently, the mean level in the estuary rises towards the end by about 0.25 m.

For all cases, the water levels in the basin near the gap show a diminishing range for every next stage and the moments of high and low water fall later. For the horizontal closures (cases 1 and 2) identically and for the vertical closures (cases 3 and 4) identically. They differ in the rise in mean level, however. For the horizontal closures in stage 5 the rise is about 0.20 m. This is caused by the fact that during ebb the waterlevel in the gap is slightly lower than during flood. For the vertical closures this is about 1 m, which is caused by the high level of the sill, preventing the basin from discharging.

Very typically for the long basin, the difference in water level curves at the gap and at the end diminishes with the closure progress. In stage 5 there is hardly any difference and the basin apparently acts as if it were a short basin, with nearly horizontal water level all over the basin. This is true for both, horizontal as well as vertical closure. Besides, the rise in mean level is identical to that in the short basin.

The flow velocities for the horizontal closures increase with every stage in a regular pattern. In the first few stages the values for the short basin are slightly higher than for the travelling wave in the long basin, but in the last stage they are identical. There is a completely different flow behaviour in the vertical closures, however. Already in stage 2, during low water at sea, the critical-flow situation occurs. This shows in the straight cut-off line of the curve. Although in later stages the duration of the critical flow is more, the maximum value of the ebb flow is diminishing. (In these calculations the critical-flow does not show in the flood period, which may happen in other cases also.)
Although the flood does not reach critical-flow conditions, the maximum flood value diminishes also. This is caused by the fact that the mean level in the basin rises and thus reduces the head loss during flood. Again, the curves for short and for long basins are nearly identical. For the vertical closures, during low water at sea there is a very large drop of the waterlevel over the sill in stage 4 and 5. As the sill level is m.s.l. and +0.7 in these stages, it can be concluded that the sill does not fall dry. The basin level always remains slightly higher than the sill, which is logical. (In actual cases, sometimes the dam is quite permeable when large quarry stone is used. In that case the basin level may go down further.)

Comparing the flow in the short and long basins for both vertical closures leads to the same conclusion as with the horizontal closures; in the first stages slightly smaller in the long basin, in later stages identically.

Keeping flow velocities low by closing vertically is clearly demonstrated. This can be further visualized in a graph showing the flow maxima against the gap’s profile, as is done for the short basin. (For the long basin the trend is identical.) The results of the five stages calculated, are presented going from about 8000 m² down to nearly closed. For the gap profile the sectional area below MSL is taken, which gives an odd value for the vertical closure in the last stage (negative). For that stage the data has been put in line with a 3% open gap. In the initial stages the velocities during vertical closure are slightly higher than during horizontal closure. However, as vertical closure reaches critical-flow, the first ones diminish while the others still increase.
3.4. Forces on and stability of floating objects.

In various closing methods, floating equipment and submerged or floating closure-means may be used. Some of these have substantial dimensions and when operating in the closure gap they obstruct the flow. As consequence the gap dimension is smaller than anticipated and higher current velocities than expected occur. In calculating the forces, acting on these structures and vessels, this should be incorporated. Various factors determine the force as function of the flow velocity.

-1 the shape of the object,
-2 floating or submerged,
-3 the keel clearance,
-4 the projected obstructed cross profile of the flow,
-5 the dimension in flow direction,
-6 the roughness of the object,
-7 the Reynolds number.

The force can be approximated by the formula:

\[ F = \frac{1}{2} \rho U^2 C_D A \]

in which all items except 4 are covered in the \( C_D \). The estimation of its value may be rather difficult and it has a considerable impact on the resulting force. Besides, the value is not a constant for a vessel, but may increase when the keel clearance decreases.

The \( C_D \)-value can vary between 0.3 for a submerged torpedo-shaped body up to 4 for a large rectangular vessel in shallow water. Keel clearance in closure gaps is often very limited. The \( C_D \)-value is related in that case to the \( \mu \)-value for flow through narrow gaps. A vertical component of the force will also develop and the vessel will be drawn down and eventually may touch the bottom.

The stability of a floating object depends on the centre of gravity on the one hand and the centre of buoyancy on the other. In equilibrium position both centres are in one vertical line. As soon as an outward influence, like wind force on the topside or flow force on the lower side or a pull by a tug wire, brings the object out of its equilibrium, the two centres shift horizontally over different distances, and the forces form a couple. If the centre of buoyancy shifts further than the centre of gravity, the couple is righting, otherwise the vessel is instable and capsizes.

This can be calculated by determining the position of the metacentre \( M \). The distance \( MC \), which is the height of \( M \) above \( C \), the centre of floatation, is \( I/V \). \( I \) being the moment of inertia of the water plane about its central axis and \( V \) being the volume of the submerged part of the vessel. For stability, \( M \) must be above \( G \), the centre of gravity. Generally a distance of \( MG \) of half a meter gives sufficient righting momentum. Adding ballast will increase stability at the cost of more draught of the caisson.

For example, assume a caisson of infinite length, a width of 9 m and a height of 12.5 m. All walls are 0.5 m thick and are made of concrete with density 24 kN/m³. The caisson is made in a dry dock and brought afloat by raising the waterlevel around it. Then (per meter length) the characteristics of the caisson are:

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Stability of a floating caisson.

Since $m_e + c_b$ (3.73 m) is smaller than $g_b$ (4.80 m), the caisson is unstable and will tilt as soon as it comes of the ground. Drawn in a clockwise tilted position, shows the centre C of the submerged part to the left of G. The couple will continue tilting. The meaning of point M is the intersection of the vertical above C with the stability axis considered. Although the caisson capsizes, it will not submerge. At an angle of about 45 degrees a stable situation is reached. The submerged part is triangular and the water line is much larger than the 9 m, which increases the I, thus the $m_e$. The distance MG is positive and quite large, thus stability is very good. M is situated on a new stability axis. This is true as long as the vessel does not get water inside, for instance because of motion or wave action. In the figure, the tilt has been drawn just over its stability point to show the righting couple.

The tilt can be avoided by adding ballast into the caisson. Although solid ballast (sand) will prove to be advantageous, in the example in the next figure water is added. A 2 m layer of water is pumped in and the data change as tabulated below:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>weight of concrete and water (G)</td>
<td>556 kN</td>
</tr>
<tr>
<td>centre of gravity above bottom (g_b)</td>
<td>3.85 m</td>
</tr>
<tr>
<td>mom. of inertia (l)</td>
<td>60.75 m4</td>
</tr>
<tr>
<td>metacentre height (m_e)</td>
<td>1.09 m</td>
</tr>
<tr>
<td>draught of caisson (d_i)</td>
<td>6.18 m</td>
</tr>
<tr>
<td>centre of buoyancy above bottom (c_b)</td>
<td>3.09 m</td>
</tr>
</tbody>
</table>

The $c_b$ increased and the $m_e$ and the $g_b$ decreased. As a result, the $m_e + c_b$ became 0.33 m larger than $g_b$ and stability seems to be reached. However, if an external influence gives the caisson a little tilt, (see middle-figure below) then the liquid cargo starts moving and the centre of gravity shifts so much that the resulting couple is enlarging the tilt. In the calculation a correction for the I is needed. The water area inside the vessel has to be subtracted. This could have been avoided by subdividing the caisson with bulkheads into compartments. Using solid ballast also avoids the dynamic effects (waves) of the cargo.

The sinking in the closure gap, is usually done by further ballasting with water. Therefore, stability has to be calculated for all stages of that operation. Although the sinking is intended to ground the vessel, a list will at least result in a position out of place.

The above calculations are valid for situations where the outer pressure around the
Stability of a water-ballasted caisson

vessel is hydrostatic (Archimedes). This may not be true for a vessel in a high current flow, as for instance in a closure gap with too little keel clearance. A flow net around the vessel is needed then to give the outer pressure distribution and enable the determination of the position of the centre of buoyancy.
4. **Use of Geotechnics.**

4.1. **Geotechnical data**

Damming river branches and tidal basins generally takes place in alluvial deposits of sand, gravel, silt and clay, stratified in various compositions and of variable characteristics. Bedrock will underlay these layers at various depths. Sometimes local deposits as boulderclay or cobbles may be encountered. In several deltaic regions layers of peat may also be present. These soil layers form the foundation bed for the dam structure and need to withstand groundwater flow and differential water pressures. During dam construction they are the subbase for the vehicles and equipment driving across the site, while submerged they will be exposed to eroding currents. Therefore, a thorough knowledge of the soil types present, their characteristics and the stratification is a prerequisite for the design and execution of a closure operation.

Geotechnical data can be obtained by sample-borings and penetrometer-tests executed in the field and analyzed in laboratories. The number of borings generally is limited in relation to the area considered and an overall examination, together with historical and geological information, is required to obtain a reasonable picture of the subbottom. Specially in deltaic areas and tidal inlets, former gulleys filled up with different types of soil may give sudden changes in the subbottom profiles of soil layers. Very local deviations of the general picture as well as obstacles (fossil trees, large boulders) very seldom appear in the results of a field survey. The most important parameters of the subbottom are stratification, soil type and phreatic levels. Laboratory test on samples of every of layer of clayey soil will give the values for cohesion, angle of internal friction, Atterberg limits and water content. Granular soil types are characterized by the grading of the grain sizes, the sharpness (roundness) of the grains and the pore volume (relative density).

Cohesion and friction-angle are the most important parameters for soft soils. The field tests, done by Standard Penetration Tests (SPT), give the number of blows to hammer a pin down into the bottom, or done by the Dutch Cone, gives the force to push a cone down. These values are sometimes transferred via relation-tables into values for cohesion and friction-angle. Far more accurate is triaxial testing on soil samples. These tests can be done drained or undrained and consolidated or unconsolidated, which leads to different values for the same soil. Which of these is the most appropriate depends on the purpose.

A triaxial test, done undrained and unconsolidated (UU), corresponds to a field situation in which the soil is surcharged without having the possibility to compress. Pore water can not escape. This is true for saturated soil of very low permeability which is quickly charged. If so, the resulting friction-angle is zero. There is cohesion only. Deformation takes place without volume change. Line UU in the $\tau-\sigma$-diagram.
If the test is done consolidated, the sample is charged first by lateral support (water stress) and open drainage. Compression leads to increase of strength. Then the drain is closed for undrained testing (CU) and surcharge is applied to determine the moment of failure. Next, the drain is re-opened, the water stress is increased to a higher level and further consolidation is waited for (for low permeable soils this procedure takes a very long time and the method is adapted, but the result is the same) Then the drain is closed again and testing by another surcharge gives a new failure level. Without the intermediate drainage the same would happen as in the UU situation: friction-angle zero, but on a higher cohesion level (the sample is stronger due to the initial consolidation). The meaning of these test-results (CU) for practice is that the same soil, taken from a deeper level, which is adapted to the existing higher water stress on that level, is initially stronger. However, if charged above that strength, it also has no friction angle if water cannot escape. The line CU is a succession of points, each true only for a sample depth corresponding to the water pressure in the bottom. Usually, data for cohesion and friction-angle given in soil-test reports are these consolidated, undrained tested values: c and \( \varphi \).

If all testing is done with open drainage (CD), water can escape and thus over-stresses in the pore water will disappear if loading goes slower than the permeability allows. The stresses in the grains remain. This happens in sandy soil mainly and cohesion will be close to zero. The values for cohesion and friction-angle give the ultimate strength of the soil under long lasting surcharge. Generally, these data are presented in test reports by an index: \( c' \) and \( \varphi' \). In the construction phases of closure dams the surcharge is applied quickly and soft soil layers have little or no time to adapt themselves because of the low permeability. Therefore, for dam construction in deltaic areas the values \( c \) and \( \varphi \), for undrained and consolidated tests have to be used generally.

For granular material (sand), the relative density and the permeability are the most important parameters. The grain structure itself is strong enough to withstand considerable surcharges. Problems may arise if some grains want to rearrange in a slightly more dense packing and the pore water can not escape. The latter is the case if the permeability is low. This applies for sand with a high content of fines. Generally, the 10% finest part of the sieve curve determines the permeability.

Apart from the subsoil, the materials used for the dam construction form part of the total soil mass to be considered. Sand fill, rock masses and clay cores are surcharge on the one hand and subject to instability on the other hand. Stability criteria have to be determined not only for the final design but also for the various construction stages. As described above, surcharging compressible soils with low permeability will result in an increase of the water stress, which fade away slowly and thus the rate of loading may be a determining factor. Therefore, the construction planning is important too.

Several mathematic models are available for calculating the soilmechanic behaviour. Of course, the accuracy of the results and predictions depends largely on the schematization of the soil layers in the model, uniformity of the soil parameters, the correct input of the execution phases and timing, as well as the validity of the basic assumptions implemented in the model used (such as sliding planes being circular). A good soil investigation should be quite extensive. Obviously permeability and pore volume are very important parameters, but these are the most difficult to determine because generally the sampling instrument will have disturbed the sample. A soil investigation program, being actual field and laboratory work, is a rather expensive part of
the design process, which is office work. Savings on the number of borings and the extent of the program may however lead to very costly mishaps in the construction phase of the project.

4.2. Geotechnical stability

The most important property of the soil is its bearing capacity, which frequently is a determining condition for the design and a limiting factor for the operations. The soil mechanics problems, related to bearing capacity, are:
- sliding,
- squeeze,
- liquefaction.

Sliding:
In dam construction, this is the instability of an earthen, or quarry stone embankment along the side slope or at the construction front. The driving force of the slip is the surcharge on the subsoil by the own weight of the embankment. The steepness of the slope is an important parameter. In closure dam construction two aspects have to be considered in particular. One is the water level variation at the dam site, the other is the erosion of the soil in front of the dam.

Water level variations influence the amount of surcharge. The dam made of quarry run will have a specific weight above water of about 17 kN/m³, however, while submerged only 10 kN/m³. As the lower part of the dam may be submerged during high water, the weight gradually increases during falling water by loss of buoyancy. Such a dam, built out by tipping with dump trucks gets very steep slopes naturally, (up to 1 vertically in 1.3 horizontally). Built-out without problems during the high water period, the dam may suddenly fail during low water.

A dam, made of sand, constructed by hydraulic filling, results in far more gentle slopes. Below the water line this will be in the order of 1 in 5 to 1 in 15 (dependent on the grain size of the sand), and even much flatter above water. The fill provides fully saturated material, which has a weight of about 20 kN/m³, and 10 kN/m³ for the submerged part.

The possible erosive action alongside and in front of the embankment as a result of the high flow velocities of the currents around the dam head are more difficult to predict.
Erosion pits may develop rather quickly and take away part of the soil, assumed to provide the counterweight, necessary to keep the dam stable. These erosion pits seldom appear on the design drawings. Besides, their shape, size and depth are hard to predict. However, a designer has to include these conditions in his considerations for a safe operational procedure. Preventive measures, as for instance dumping a protective layer of quarry run ahead of the progressing works, will avoid the erosion.

For the calculation of the risk for slip the method of Bishop is generally used. In this mathematic model the subsoil and the surcharge layers are schematized in vertical slices and the forces keeping the individual slices in place are detailed by attaching values for cohesion and angle of internal friction to the different types of subsoil and the dam construction layers. The instability is assumed to take place in a circular plane intersecting the horizontal layers. Each layer contributes to the resistance by a different value dependent on the length of the circle segment in the layer and the friction force it can yield. When sliding occurs, the vertical slices within the circle turn all together until a new equilibrium is reached. Basic assumptions for the calculation are the correctness of the soilmechanic values of the soil layers and the dam, the uniformness of the composition of every layer and the circular shape of the plane that first reaches over-stressing. The method determines the circular plane which has the lowest ratio between the shear resistance along the full plane and the driving force by the total weight. For the value 1 sliding should occur.

In practice, sliding should be prevented or purposely used. For prevention, the safety against sliding has to include all circumstances during the construction of the initial closure dam profile. In a later stage of construction the dam profile will be enlarged by further heightening and widening and finishing of the slopes. In many cases, with the knowledge that the soil will gain strength after some time, the construction of this final profile may be scheduled such that stability is not critical. The calculations for initial and final profile require different cohesion values.

Sliding is used purposely if a soft layer of limited thickness is situated on top of a well-bearing subsoil. The design choice is to remove the soft layer by dredging (and backfill the created trench by good sand) or to construct on top of the soft soil and press the embankment down into it by its own weight, which involves sliding. The first choice is the safest and applies for critical parts and operations of the closure dam construction anyway. Nevertheless, the other method is used sometimes. The risk is that the soft soil is only partly pressed away and that in later stages deformation continues. For such a procedure the safety factor should be much lower than 1 as the embankment has to fail during construction in favourable conditions also. Controlled failure is far more difficult than ascertained stability.

The subsoil encountered during damming activities (period 1970 to 1976) in the rear of the tidal basin called Eastern Scheldt, consisted of large areas of peat, criss-crossed by farmer stream gulleys of the River Scheldt filled up with clay and sand. Part of the Markiezaatsdam was hydraulically filled with sand up to specified profile and several slips leading to heaps of peat along the side of the dam occurred. Another part was build out by tipping stone with dump-trucks. This dam dropped almost instantaneously down into the soft material. It was then decided to exchange the subsoil at the final gap site. Although closure was designed as vertical closure, consolidation time between layers would have been too short. In a later design for the nearby Oesterdam in comparable conditions, hydraulic filling was prescribed again but this time specifying a profile to be made with gentle slopes and a limited height in first stage. The filling required some extra equipment and rehandling of sand, but no failures occurred.
Squeeze:
It is clear that in a situation that very weak subsoil is sur-charged by very stable dam material, instability may occur within the subsoil only. Instead of sliding along a plane, deformation of the soil layer occurs comparable with an ice-cream squeezing out between two wafers. Over-stressing will than start at one location and locally lead to deformation without changing volume. The stress transfers to the surrounding area in various directions by which the deformation expands. The dam body on top sinks into the subsoil and the same volume of weakened soil escapes on the edge of the dam. This type of instability is called squeeze. As this failure starts by over-stressing in one point and then progressively expands via deformation, the mathematic approach is different from the slip along a plane by basically undeformed bodies. For the calculation of instability by squeeze another mathematic model, (e.g. Plaxis) has to be used.

The figures show that slip and squeeze are completely different. By observation in the field this is more difficult to establish. The dam profile after instability is seldom as ideal as sketched and in both cases a heap of subsoil material rises up in front (or at the side) of the dam. Taking samples at the toe of the slope will show the facts. In practice the difference between slip and squeeze has few implications. A choice has to be made in the design between soil improvement or purposely induced subsidence.

Liquefaction:
This may occur in loosely packed sands. The strength of a sand mass is determined by the transfer of the forces from grain to grain. In saturated sand, the voids between the grains are filled with water. This water is hydrostatic and does not bear the grains. If, for some reason, a shear force leads to movement of the grains, they will try to obtain a more dense packing, which is possible only when some of the water is driven out of the pores. At that moment, the water is taking part of the load from the grains. If the permeability of the sand is small, for instance because of a high content of fines, the water can not escape and the grains loose their stabilising contact forces. A fluid mass of water and sand grains, with a density of 1800 kN/m3 results (quicksand).
From the above, it is clear that liquidizing may occur in loosely packed sand of poor permeability and only after an initial event triggers a disturbance of the grain structure. In nature, such deposits of sand occur in areas where the conditions for settling of the sand particles was ideal, no turbulence, no waves and not to much current flow. Made bodies of sand, placed under water, for instance by hydraulic filling, always have a loosely packed grain structure. Then, the quantity of fines determines the sensitiveness for liquefaction. The initiating event may be of various nature. If the grains are in a stress situation, for instance because of a developing scour hole, a little vibration or shock may start the flow. Different from slip-circle and squeeze, liquefaction will result in a flow slide with very gentle slopes at the toe.

In practice many stability failures occurred in closure operations. Generally they resulted from variations in soil characteristics, changes in the works program, limited soils information and unforeseen conditions. Apparently, soil conditions on the sites where closures are executed, give little margin. Of course, it is important to know what should be done and not done in such a case. After a failure, the soil is disturbed, the soil structure distorted, over-stressed water will still be present and the new situation will have a very vulnerable equilibrium state. Shifted material will be in the wrong place, for instance at the toe of the slope, and the top layer of the dam will be too low. However, every corrective action by taking away the wrong toe material or extending the dam by recharging the top will lead to continuation of the failure. The only possible measure to improve the situation is to let the soil mass consolidate and let the stresses in the water diminish. This takes time and can hardly be speeded up. If it is possible to install them, artificial drainage by vertical drains may be helpful. Otherwise, it is better to bypass the area and thereby provide time for the water pressure to reduce to normal. Next, preferably working from the toe side uphill and very gently, layer by layer the profile of the dam can be restored.

4.3. Settlement

Closure dams are constructed with various materials. Sand is used for dam bodies which are built out into the flow as long as the current velocities are modest. Furthermore sand is used to enlarge (widen and/or heighten) the dam’s profile after an initial profile has been made by using other materials. Lumps of clay and boulderclay are suitable for closures with medium current flow velocities because of their resistance against erosion. Otherwise, quarry stone or concrete blocks are used. Apart from the clay, these materials will show little settlement of the created profile. The clay however, may give a considerable settlement and, when used for the closure’s initial profile, generally will be strengthened afterwards by a sand profile alongside. The main problem with settlement therefore is the subsoil.

The magnitude of the settlement can be estimated by using the formula of Terzaghi:

\[ \varepsilon = \frac{1}{C} \ln \frac{\Delta p + p}{p} \]

in which \( p \) is the grain stress, \( \Delta p \) the surcharge and \( C \) is a constant, dependent on the soil properties. For peat \( C \) may be as low as 3 to 10, for clay 10 to 20 is used. Compressible soils consisting of loam, clay or peat may show considerable settlement after some time. The development in time can be calculated on the basis of the soil's
permeability. Water has to be driven out of the soil and escape to the nearest draining soil layer. As in many cases the permeability is very low and the distance for the water to travel may be large (thick layer), the settlement may take several years to complete. The problem of settlement therefore is mainly a matter of the determination of the final design profile after closure than part of the closure.

In practice, the expected settlement is added to the design height of the dam to be constructed. Consequently, the surcharge onto the subsoil increases initially with the extra height of dam material and may put the profile at risk for failure. The over-dimension is not necessary at first instance and may, in a critical situation, be postponed until the subsoil has gained strength by some consolidation. Then after a period of time the dam's top layer has to be completely reconstructed. Otherwise, the settlement-sensitive soil layer can be dredged away and be replaced by better soil (sand).

An example of such a case was the closure of two small tidal creeks near Odafado in Nigeria as part of a canalization project. The subsoil consisted of a thick layer of rather soft silty clay and considerable settlement was expected. The extra added height of dam to compensate for the settlement enlarged the load thus the settlement and in the end the design engineer decided to select the alternative and have the soft soil removed and exchanged by sand. Unfortunately, this increased the total construction cost disproportionately. At the suggestion of the contractor and after consultation with the local authorities it was decided to execute the closure process without soil exchange, by stepwise building the final dam. For the initial closure profile the lowest possible dam height was selected (for instance by making a very gentle outer slope to reduce on wave run-up). Besides, the closure was based on the old Dutch methods using trees and branches ballasted with soil, which together was the least heavy dam material that could be used. Of course regular maintenance to cope with the settlement was required afterward.

4.4. Groundwater

Damming up watercourses will influence the groundwater pressures in the vicinity of the dam and in the dam itself. These changes may be short term variations, e.g. in relation with the daily tide levels, or long term adaptions, such as lining up with the rise in mean level of a basin. Differential pressures lead to flow of the groundwater. As long as the Re-number is smaller than about 5, the flow velocity according to Darcy's law is:

\[ u = k \cdot i \]

in which \( k \) denotes the permeability of the soil and \( i \) the pressure head-loss per unit length. The flow velocity \( u \) has a complex relation to the actual velocity of the water in the pores of the soil. The quantity of water passing through the soil is given by the \( u \) multiplied with the full cross sectional area \( A \) of the soil, as if there were no soil particles.

\[ Q = k \cdot d \cdot b \cdot \frac{\Delta H}{L} = u \cdot d \cdot b = u \cdot A \]

In granular soil like sand for instance, the pore volume will be in the order of 40% only, which means that, if the soil is considered as a solid mass provided with 40% volume of straight narrow tubes, the real flow velocity would be 2.5 times as much as \( u \). The actual flow pattern is very complex as the water has to force itself through the grain structure by acceleration and slackening. These actual flow velocities act on the grains and may transport loose fines which do not form part of the structure. Differential water levels consequently may lead to:
instability of the total soil mass if the force by the pressure head is more than the weight of the soil layer (macro stability).
- instability of individual grains at the surface of the soil layer by the out-flowing water (slope stability under seepage conditions)
- instability of grains within the soil layer (micro stability), leading to transport of the finest soil particles.

In damming activities sometimes a choice can be made between a rather permeable initial closure dam or a profile provided with an impermeable (which in practice generally means with very low permeability) core or structure. The selection will depend on various circumstances. The main consideration is that in a permeable profile the phreatic pressure goes down gradually but the flow in the full profile has to remain within limits to ascertain micro stability. A watertight core leads to a high pressure difference over the core which endangers macro stability. Besides, locally along the bounds of the core or structure, the high pressure difference may lead to micro instability.

In the design process the various combinations of water levels and soil parameters for the sub soil and the structure details have to be considered. Permeability coefficients have to be tested or assumed. Determining conditions in the various construction phases have to be concluded. Next, ground water flow nets can be drawn and analyzed. In three dimensional situations (aside a caisson or at the end of a sheet pile wall), the flow net is difficult to draw and great care should be given to these details. A problem may arise in case infill of fines gradually plugs the soil. If so, the permeability changes and the pressures start building up. The reverse, erosion of fines, may lead to higher flow velocities which worsen the erosion and finally lead to a scour pipe in which seepage changes into piping.

Generally, the most critical stage is the closure of the final gap. This is a situation in which a bottom protection against scour has been laid, on which in a short period of time an initial dam or structure is made. Differential water levels attack the freshly made profile. Particularly with caissons, the stability of the foundation bed and of the side connections is important.

In several closure designs made for the Delta schema in The Netherlands, an asphaltic mat was used for protecting the bottom. Although permeable mats composed of graded stone with an asphalt binder can be made, these mats were practically impermeable. The phreatic level in the soil underneath the mat, between the sea and the basin gradually diminished according to a long stretched flow net. Since the protected length (L) is large, i is small and so is u (Darcy). After construction of the dam profile in the centre of the protected area, the water level difference increases and thus u also increases. As shown in the figure the phreatic level underneath the mat, just downstream of the dam results in an lift force. The relatively thin mat, meant to prevent uplift of impermeable bed protection

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scour, suddenly gets another task: counteract the upward pressure by its own weight. If lifting would occur, the mat might tear and the water would break out. Then, the seepage length \( L \) would be about half the original and (in a two-dimensional case) \( u \) would double. In practice there would be one location which gives way first and the flow would converge to this point. The flow would start a piping process and after that the whole dam profile would collapse. Consequently, the mat had to be much thicker than in case a permeable mat had been designed.

Another example of piping actually occurred during the closure of the River Feni in Bangla Desh. During preparatory operations, stockpiles of jute bags filled with clay had been stored along the initial closure alignment. These stockpiles had been build on geotextile sheets strengthened with a bamboo grating. During the closure operation the bags would be carried from the stockpiles into the longitudinal initial dam profile, sufficient to block the flow. This profile would immediately be strengthened afterwards by a heavy clay profile behind. The procedure prescribed the removal of the bamboo gratings but time did not allow the complete removal and the clay profile covered several remnants of these bamboo strings. Then, during high water, the ground water found its way underneath the clay along these bamboo and several wells gave artesian water. The problem was solved by dumping some extra clay on top of the wells. This appeared to be sufficient to block the flow and prevent escalation of the piping.

If piping occurs, several measures can be taken to prevent escalation. The best measure is to block the inflow at the upstream side. However, the outflow point is known but where the water enters is generally unknown. The next measure is to ballast the well as in the above example. But sometimes, water may break out every time again. Then, the well can be covered by a permeable sheet sufficiently ballasted with cobbles. Due to the permeability the upward pressure will remain low and further ballasting is possible. The flow will carry soil particles which get stuck in the sheet and gradually the flow blocks while the pressure increases. Ballast and soil on top will ultimately stop the flow. In shallow water and above, a method is to construct a ring-shaped wall around the well of sufficient height. Within this ring the water level will rise until the high level at the pipe’s entrance is reached. Then the flow stops and any sort of material can be dropped onto the well and block it.

### 4.5. Moving on impassable sites

Bearing capacity is not the only determining factor which makes a site passable. This is clear for everyone who tried to drive with a normal car on a sandy beach. On the dry beach a four wheel driven car with very low gear is needed. In non-granular soil the difference shows for instance in heavy consolidated laterite clay, which is a good base for driving when dry but absolutely impassable after some rain. Since damming activities generally take place in deltaic areas in which sand, sandy clay, clay, silt and peat frequently occur, moving across the site with all sorts of equipment and vehicles under various weather conditions may be a problem. Besides, driving on top of a quarry stone dam will be a problem if the stone is coarser than about 0.5 m in diameter.

A distinction has to be made between the heavy operational equipment like dump trucks, cranes, hydraulic excavators, bulldozers and the exploration-equipment used for soil-investigation, positioning finding and measuring. This last group generally consists of light-weight equipment, which is used frequently however, in original terrain conditions. Very special vehicles have been developed for these specific circumstances, although usually they are not readily available and quite expensive sometimes. Hovercraft or amphirol are examples of these vehicles. The hovercraft supports itself by an air-bell confined within rubber skirts, maintained underneath the vehicle by continuous pumping. It can be used on land, on mud flats and on water. The amphirol is supported by two horizontal cylinders provided with opposite winded archimedean screws. Motion
is obtained by rotating the cylinders. Steering is realised by the horizontal orientation of the cylinders and the rotation direction. The amphiroil moves well on soft grounds, mud flats and on water. These vehicles are very useful for small equipment and special assignments.

For the heavy equipment the right system of movement has to be selected. If moving speed is important or if driving on site and on public roads has to be combined, the only solution is to use pneumatic tires. Variables with tires are the dimension of the tire, the pressure inside, the number of tires and how many are driven and can be steered.

A wheel has to transfer the forces onto the ground and its motion depends on the reaction forces of the ground. A non-driven wheel transfers a vertical load and a lateral force, while a driven wheel has the load and a rotational momentum. Usually the pressure in a truck’s tire is quite high (300 to 500 kN/m²) and the tire will hardly deform. The reaction force of the road surface or the ground is also high. The maximum lateral force is the friction coefficient times the support force. Deep profiles on the outside of the tire may give it sufficient grip in loose ground and then the shear force in the soil determines the friction coefficient. If the friction is exceeded, the wheel slips. This is the case in the above mentioned example of wet laterite clay.

If the soil cannot stand the point load of the wheel, the tire will sink into the ground until the load is spread over sufficient bearing area. Consequently, when turning, the wheel has to move up against a slope, by which the support area shifts to the front side of the wheel. A much larger momentum is required to achieve driving and quite soon the friction reaches its maximum. Then the wheel starts turning without lateral movement and it digs itself further down into the soil. This is the case in the dry-beach sand example. A momentum exerted on all wheels (four-wheel drive) and very slow turning will improve the situation.

With a low air pressure in the tire, it is able to deform which gives it a larger support area without subsiding into the soil. Therefore, vehicles with special low-pressure tires (e.g. wheel-loaders) can move much easier but their motion is very springy. For heavy transport that is generally not allowed and then increasing the number of tires is the only solution. For exceptional heavy transport a large number of axles, each with a set of wheels is used. In that case all the wheels are provided with integrated steering capacity.
The next step is to take crawlers instead of wheels (e.g. crawler cranes and bulldozers). They spread the load over a very wide area. Support during driving is not very determining but the shifting of centre of the load and sufficient lateral force during pushing or pulling are important. For different purposes, wide or narrow crawler-tracks can be used. Driving on large quarry stones, for instance, requires narrow tracks. The high point loads exerted by the spanned stones have to be kept close to the centre-line of the track in order not to damage it.

Another solution is to prepare a number of road-ways across the area along which surfacing is made to allow vehicles to move along. Those roads are of a temporary nature and should be removable and relatively inexpensive. Two systems are frequently used. One is to pave with large, steel-framed wooden slabs or steel planking. Draglines and cranes can position these themselves and then drive on them. The other system is to provide a road-base direct on the existing soft ground. A geotextile sheet is unrolled and ballasted by a layer of sand or gravel. As soon as the wheels of a truck move on to the ballast layer, the road-base underneath is pressed down into the soil. Due to the sag-bend in the sheet part of the vertical load is transferred sideways into tension in the textile. This horizontal force is taken by friction in the soil for which sufficient length and ballast has to be available. If the trail of the wheel into the ballast bed is too deep, direct sheer of the tire on the sheet will occur and the sheet will tear. Therefore the ballast layer needs to be quite thick and trucks have to prevent deep ruts by regularly shifting tracks. The geotextile has three functions. It separates the subsoil and the ballast material, it transfers the surcharge to the subsoil and spreads the load via tension and elongation to the sides. Besides, due to the separation between ballast and subgrade, the removal of the temporary road is more simple and the ballast material can be re-used easily.

A typical event occurs with bulldozers driving on a hydraulic fill. The freshly settled sand still has a large pore volume and is saturated with water. The vibrations of the dozer and its weight (although spread over a large area), will fluidize the top layer of the sand. Generally, this remains within such limits that the bulldozer can continue driving while the sand resettles in a more dense grain-structure. Bulldozing leads to densification. However, sometimes the liquefaction covers a too large area and the bulldozer may sink down into the fluidized sand. A fill of fine silty sand (smaller than about 120 micron) is very sensitive for this and hardly passable. Then a long time is needed to await evaporation of the pore water before driving is possible.
5. Using Structures as Components of the Closure.

5.1. Closure by hydraulic filling with sand only

In a number of cases, a tidal basin can be closed by pumping sand only. The principle is simple. As long as more sand is pumped into the closure gap than the flow erodes away, the gap narrows. Due to the development of dredgers with very high capacities (5000 to 8000 m³ sand per hour), this has become realistic. Bulldozers are needed to spread the sand-water mixture over the fill and shape the desired profile of the dam. Besides, they prevent the erosion of gulleys on the fill slope and densify the deposited sand. This equipment, used to control the fill process has improved likewise.

The main question thus is how much capacity is needed. A very high capacity can be attained by using many delivery lines from various dredgers. However, to keep the fill under control, every delivery line needs a certain width for these operations. More capacity means a much wider fill-profile, by which progress is not improved. So, there is a practical limit to the supply. Of course, the gap can be approached from two sides.

A sand closure is a horizontal closure, with a progressively narrowing gap, in which the flow increases until the very last gap can be blocked. The flow in the gap has a sand transporting capacity which is related to the flow velocity to the power of the order of four. This means that when the flow increases from 2 m/s to 2.5 m/s the transport of sand multiplies by a factor 2.5 and when it goes to 3 m/s by a factor 5.

The normal process of building a sand fill dam is as follows. A sand water mixture is pumped by the dredger via a delivery line to the fill, sufficiently high above H.W. (point A1 in the figure). The mixture runs down the slope to the water line (A2) and into the water, while the sand settles. The sand creates a slope above water (A1-A2) which is much flatter than the submerged slope (A2-A3). In the ebb period, going from H.W. to L.W., the progress is as shown in the figure from A1-A2-A3 to B1-B2-B3. Progress seems small, as the delivery point hardly shifts (A1-B1). All the sand goes into the toe circle. During flood however, the line B1-B2-B3 shifts to C1-C2-C3. The nett progress per tidal cycle is A1-C1. Scour will erode the original bottom in front of the dam and enlarge the profile to be made. Besides, part of the supplied sand will be taken by the flow also and carried outside the alignment area. The further the dam extends, the higher the flow in front of it and the smaller the nett progress per tidal cycle.
When hardly any progress is made, the final blocking has to be enforced. This is a special operation, which starts at the moment of H.W., shown in the figure by the line K1-K2-K3-K4-K5 for two fills approaching the gap from both sides. Progress during the ebb may seem negative even, as the line K1-K2 goes backward but the gap size at L.W. is virtually small (L1-L2-L3). Then, just before slack water, the profile has to be blocked. This requires the temporary interruption of hydraulic transport with recourse to earth moving plant thus obviating the erosive action of the hydraulic transport water. Bulldozers and cranes will have to shift as much sand as possible into this L.W.-gap to shape a tiny ridge into the triangular profile (L1-L2-L3). This sand is taken from the above water slope (K2-L1). The ridge being ready, the flow is blocked and all the sand supplied by resumed pumping will settle in the profile. The ridge, protruding above L.W. has to be heightened and widened to stay ahead of the rising outer water. The volume required increases tremendously with the rising level, as the ridge length also increases. To create a stable profile over the full length (K2-K5), able to stand the head difference of the full tidal range, in a couple of hours, requires a very skilled fill procedure and a sufficient sand delivery capacity as well.

In total, the principal questions are: how large a gap can be closed in the last tidal cycle and what will be the sand transporting capacity of the flow in that gap? The last gap’s operation takes place in one tidal cycle, so for a semi-diurnal tide in about twelve hours. The twelve hours before that is the last tidal cycle in the normal fill procedure, in which some narrowing progress is still to be made. Consequently, the possibility to close is fully determined by the operations during the last day. The capacity required thus is determined by:
- the normal process to attain the size of gap that can be closed in one tidal cycle,
- the ebb-phase to shape the tiny ridge at the L.W.-slack period,
- the flood-phase to build a stable profile before the next H.W.

With that capacity laid down, the progress in the days (weeks) before can be calculated by phase wise determining the nett progress per cycle. Summing-up gives a reasonable approximation of the total time needed for the construction of the dam.

In practice several sand closures have been realised. It appeared that the maximum flow velocity which could be accommodated was in the order of 2 to 2.5 m/s, dependent on the grading of the sand. These velocities occur for a head difference of about 0.30 m, according to the flow formulae in gaps (section 3.2), for rounded (sand-)dam heads. The gap size for which this boundary condition exists can be calculated using a mathematic model of the closure procedure.

For instance in the mathematic example "case 1" of section 3.3, the flow reaches about 6 m/s in the final stages (4 and 5), when the head difference is slightly more than half of the tidal range (1.5 m). The 2.5 m/s is reached already at stage 2, which is for a gap dimension of 4000 m². This gap size is far too much for the final day’s operation. Nevertheless, the sand-closure of the "Krammer" (in the rear of the Eastern Scheldt basin) in 1987, closed a basin of about 55 km² in which a tidal range of 2.70 m existed. However, by that time, the storm surge barrier in the entrance of the estuary was operational and the tidal range was artificially reduced to 0.60 m during the closure. Thus, flow velocities were kept under 2.5 m/s to enable the closure. This shows that if the tidal range is smaller than 0.6 m, even very large basins can be closed by pumping sand.

Basins with larger tidal ranges can only be closed by sand pumping if the basin’s storage area is much smaller. An example of that is the sand closure of the Wohrder Loch in northern Germany near Meldorf in 1978. The tidal range during the neap tide on closure day was 3.20 m. The storage area was 10 km², the grading of the sand about 350 micron, the total installed dredge capacity 8000 m³ per hour and 14 bulldozers and 8 hydraulic excavating machines were busy at the fills. The length of the gap at the water
line during H.W. (K2-K5) in that case was 120 m. The flood phase capacity, to strengthen the ridge, appeared to be the determining factor.

A few conditions determine the possibility to close with sand only:
- the tidal range or the storage area of the basin has to be sufficiently small,
- large quantities of good quality sand must be available nearby,
- high-capacity dredgers have to be used,
- a well-organized fill-procedure by cranes, back-hoes and bulldozers is required.

As long as the original bottom in the gap has a resistance against erosion comparable to the sand used for closing, a scour protection is not relevant. Scour is acceptable unless it endangers stability of structures in the proximity. A considerable volume of sand is carried by the flow outside the desired profile. This reduces the progress but is part of the method. The lost sand is not considered a loss. Actually, instead of providing an expensive bottom protection, scour is compensated by using an extra quantity of sand. The many machines operating at the fill require a good passable subsoil. Sand closures with very fine or silty fine sand are hardly or not possible.

5.2. Scour prevention by mattresses or filter layers

Every closure of a watercourse leads to a situation in which the flow accelerates, circles around dam heads or crosses over materials with different hydraulic roughness. Each of these results in increase of the capacity to erode. In the sand closure process the scour is accepted since its magnitude is limited because of the restrictionsto the flow velocities which offer the possibility to apply the method. For all other methods and dependent on the resistance of the bottom material against erosive action, scour holes may develop, which can endanger the stability of the closure dam. This has to be prevented by the placing of bottom protection means at all relevant locations. These do not prevent the scour completely but shift its bearing towards less vulnerable locations and may reduce the scour depth. Scour prevention therefore is part most closure processes and generally one of the first actions in practice.

Generally speaking the scour resistance of the bottom material is difficult to predict. Rock and stiff clay will be very resistant, soft clay is rather resistant, peat may stand the attack quite long and then suddenly break out in large lumps. The behaviour of sand has been investigated intensively and several formulae have been derived to predict the scour hole development. Since in practice a sandy subsoil is but seldom homogeneous, the actual scour may still deviate from the predicted values.

In short, scour holes can be expected at places where:

i. the flow velocity increases in course of time

ii. the flow distribution over the vertical changes

iii. the flow is not saturated with sediment

iv. the turbulence intensity increases

These aspects occur in closure processes for instance:
- when diminishing the gaps profile (item i)
- at the end of a stone protection as consequence of change in roughness (item ii)
- due to reduction of the discharge quantity in the approach gulley (item iii),
- around dam heads, structures and obstacles (item iv)

Of course combinations of these four often occur.
In one-directional flow, the scouring process creates a hole which is characterised by its steep slope at the upstream side, its depth and its gentle downstream slope. In tidal areas, where the flow changes direction in every tidal cycle, the shape of the hole will be different. The reversing flow smooths the hole out slightly by which the slopes become more gentle. The development of the hole goes quickly in the beginning but gradually slows down. By creation of the hole, the bottom topography adapts itself to the flow's eroding capacity and in the end an equilibrium state is reached. The depth of the scour hole develops in an exponential relation with time. In many cases the equilibrium state is reached so quickly that the intermediate stages are of no importance. However, if a number of caissons are placed one after the other in a couple of weeks time, the flow pattern changes stepwise in short periods and so does the scouring capacity. The scour hole then develops as a summation of intermediate successive stages.

The development of scour holes in itself is not dangerous. Only in cases where they come too close to either the closure dam or adjoining dams or structures, will they endanger stability of these structures. Then, uncontrolled scour should be prevented. A scour protection by a bottom mattress or a filter layer will be required. Since the costs of these protections in closure works generally are considerable, minimizing the dimension is important. However, the installation has to be done in advance of the determining situation (construction phasing) and a too short protection may give a large risk. The longer the protected area, the further away the hole develops and since on that spot the attack will be less, for instance because of spreading of the flow or diminishing of the turbulence intensity, the equilibrium depth of the hole will be less. Both aspects, further away and lesser depth, improve the stability consideration of the endangered structure. Usually, as a first approximation, a protected length of about 10 times the water depth is considered safe. For detailed engineering in case large costs are involved, the optimization requires physical model investigation in a hydraulic laboratory.

For the stability considerations the dam in the closure gap and the joining bottom protection act as a total structure. Consequently groundwater flows and potential head differences will build up over this protection. Therefore the protection has to:
- be flexible to follow changes in bottom topography
- be well connected to the bottom, leaving no room for piping,
- be sufficiently heavy to prevent flapping in the flow,
- be extra ballasted at its end to prevent turning up when the tide turns,
- be impermeable for the soil material underneath.
- be stable in all flow conditions in all relevant construction phases.
- be permeable for water to prevent high pressures underneath,
  (sometimes the requirement for an impermeable part is combined with the scour protection, in spite of the pressure increase; see figure on page 4-8).
Installing the protection is done by various methods. Usually, the area to be protected is divided into rectangular plots. Each plot is covered by one mat. Overlaps of 2 meter in the width and 5 meter in length allow for inaccuracies of placement. The method which is generally used is to bring the mat to the plot in floating condition, position and stretch it between anchored pontoons and then lower it to the bottom by ballasting. Forces by the flow on the mat are considerable due to the large surface area of the mat and oblique flow is a hindrance. Therefore, the maximum practicable dimensions of these plots are taken about 30 meters wide and 80 to 120 meters long (in flow direction).

Another method is to wind the mat onto a cylinder, and unroll the cylinder either at the water surface or close to the bottom. For this method the grating is omitted, while some ballast has to be tightly fixed to the geotextile in order to keep it down on the bottom straight after unrolling. Generally, in all methods, the protection of the bottom is laid in an early stage of the closing works as flow velocities during lay-operation are still limited. Besides, flow attack on overlaps and erosion along the mat during the lay operation will also be less and there is less risk for an upturned rim or a piping channel underneath the mat.

A granular filter, consisting of various layers of material of increasing coarseness, from sand via gravel to quarry stone, also serves the purpose. The advantages of this protection are:
- they can be laid much easier and quicker than the geotextiles,
- they are self correcting for small damages (for instance for a dropped anchor), which makes them less vulnerable,
- there are no structural joints,
- they are relatively simple to remove (dredging),
- towards the end of the area to be protected, they can gradually be faded out.

The disadvantages are:
- there is no structural coherence, they disintegrate on steep slopes,
- a major damage escalates, as it exposes more fine material underneath,
- the construction height is large as each layer needs to be quite thick to allow for inaccuracies during the placing.

A granular filter may be a good solution if scour develops a larger hole than expected. Then, well in time, the steep gradient of the hole can be stabilized by quickly dropping a cover of gravel or quarried stone.

Sometimes a design option is considered in which the protection has to be watertight. In practice this is very difficult to realise. The laying of large membranes in flowing water is a difficult operation and to obtain watertight joints is nearly impossible.
5.3. Quarry stone dams, dumped or tipped

Quarry stone is frequently used for closure operations. Various methods apply and the method, the equipment and the conditions are interrelated. A major distinction is the difference between a horizontal and a vertical closure which results in completely different flow- and water level parameters. Different construction methods or equipment may result in dam profiles with different slopes and thus influence stability calculations. The grading of the stone depends on the flow- and stability criteria.

During a horizontal closure the dam heads are build out by tipping stone, for instance with dump trucks. Then, the dam head slopes are steep natural slopes of 35 to 45 degrees. The flow will attack the dam at the dam head, where the flow turns around the spearhead, contracts and loses contact, starting a line of eddies. The attack is most severe at the upstream side in the gap where the eddies start. The dam heads can proceed with a dump capacity of 250 to 300 m$^3$ per hour per dam head. The final gap, which is the gap during the last tidal cycle, is much smaller than with sand closures. A tidal basin closed with quarry stone will therefore have a larger head difference over the final gap than with sand closure (if possible) for the same basin.

For stone stability calculations, the formula of Shields is used. According to the formula, for flow over a stone bed, there is a relation between the critical flow velocity $U_k$, and the submerged density $(\Delta)$ and diameter of the stone $(D)$. A distinction has to be made for the stones in front of the dam head and on the slope of dam head. In front of the head the stone bed may be part of the bottom protection, which gets the determining flow condition at the moment the damhead approaches. To account for the three dimensional flow conditions Shields formula has to be amplified by a factor $K$:

$$\overline{U_k} = \frac{1}{K} \cdot C \cdot \sqrt{\frac{\psi}{g}} \cdot \sqrt{\Delta gD}$$

For the stability of the stones on the front slope of the dam head, another correction factor is used. The formula then is:

$$\overline{U_k} = \log(3 \cdot \frac{h}{D}) \cdot \sqrt{\Delta gD}$$

An advantage of the gradual progress of the dam head is that the line of eddies also shifts and so does the flow attack on the bottom. Time for scour downstream of the protected bottom, is short as long as dam head construction proceeds without interruption. While progressing with the closure, the potential head over the dam increases and seepage flow will increase. Down the side slope of the dam at the side of low water level slope stability may then be critical. If this is a determining condition, timely reshaping of the initially made dam profile has to be done.

When closing vertically, the flow pattern will be completely different. Building up layer after layer the flow will reach a stage in which critical flow starts. Stability of the profile depends on its shape which depends on the way the dam is built. There are two different ways, dumping in a line or dumping in wide layers. Dumping in a line along the
dam axis, for instance from a temporary bridge or a cableway, creates a steep triangular profile, gradually increasing in height. The stones on the top are the least stable and may tumble down the slope locally. Since the number of stones in the top is small, the level of the top will be very irregular. This is a negative aspect as long as the top of the dam is submerged. This is particularly true in case of critical flow, since peak flow velocities are higher in these depressions in the dam. Later, when the dam has reached a more elevated level, the downstream slope will be the most vulnerable for instability. Seepage flow together with the overflow of water over the top can take the stones out of the steep profile. This method requires a high initial investment in the construction of a temporary bridge or cableway but has the advantage of being able to reach any place along the alignment at any time to proceed with construction or to repair the profile.

A different profile can be obtained by dropping the stones over a broad band instead of in a line. Then, a trapezium-shaped sill can be heightened layer by layer. This has many advantages. First, in contrast with the line-dump, every successive layer contains less material for the same rise. Creating the top thus takes little time, shortening the critical period. Second, the slopes of the dam sides can be created in a flatter profile, dependent on the width of every individual layer. Thus stability of the profile will be much higher. Third, a few dislodged stones will not create a depression in the sill over the full cross-section.

The operational side of the latter method, is more difficult, however. It requires specialized equipment, regular surveying and good positioning equipment. At first it may be possible to drop stones using dump barges or split barges. The barges can be used head-on in the flow, as the width of the band will fit the length of the hopper. As the sill gets higher, the keel clearance of the vessels reduces and finally the process cannot proceed any more. The last couple of meters has to be done in a different manner. Cranes, conveyors or the like have to be used to reach outboard and that limits the capacity very much. Besides, the operation with vessels is increasingly difficult as result of the worsening flow conditions.

For stability calculations of the stones on the sill, the formula of Shields is also used. However, the relation has to be adapted for the typical phases, mentioned above, of a vertically built stone dam under construction. The triangular or trapezoid profile and the broad sill are not in accordance with Shields test conditions. Therefore the formula can be adapted into:
in which \( a \) is the water depth above the sill and the factor \( f \) goes from 1 for the very first horizontal layer via 0.75 for a broad sill to 0.25 for a very sharp crested sill.

In a vertical closure the most critical situation occurs if, in a rather advanced phase, a local instability leads to a depression of some extent in the sill's crest. The vertical closure method is used generally to limit the increase in flow velocity, usually in order to use smaller size stones. In a major depression the flow concentrates and reaches much higher values than accounted for. Thus, the stone underneath in the depression is not stable either and the dip will soon deepen considerably. The vertical closure is changed into a horizontal gap. Most likely, the bottom protection will not be designed for those conditions and the failure will become a major disaster.

Due to the difficulty to construct the toplayers of the dam, generally, the last phase of closure will be a horizontally built top on the vertically built trapezium. Then, three-dimensional flow patterns develop. This has to be considered in the stability calculations and some extra safety in the dimensions should be taken.

5.4. Caissons, closed or provided with sluice gates

Caissons, in closure design, are large, artificially made structures or vessels used to block a final closure gap in one effort or in a few major steps. In emergency cases existing ships, pontoons or the like have been used, sometimes after adaption to fit the gap dimensions. In normal circumstances caissons are specifically designed for the purpose. Generally they are made of concrete, have a box shape and are self floating. Three typically different systems can be distinguished:

- The final gap is closed in a single operation by placing one or a few caissons simultaneously.
- Several identical units are made, which together fit into the gap. They are placed one after the other in period of several days.
- Several units are used of which a number (or all) are provided with sluice gates. Every unit is placed with its gates closed, but after stabilising of the caisson, the gates are opened. As soon as all caissons are rigidly in place, all the gates are closed at a suitable moment.

Which system is used depends on circumstances and conditions. They are progressively more expensive.

The caisson is intended to block (part of) the gap and thus will be positioned transverse in the flow. Since dimensions are generally considerable, even small flow velocities will result in high forces for manipulating and positioning of the caisson. Therefore placing will always be done during slack water when the tide turns. In practice the moment with flow velocity zero does not exist. Generally the tide starts turning near the bottom first and later at the surface and usually not over the full gap length at the very same moment. Slack water therefore is the period (time window) in which velocities are smaller than about 0.5 m/s in either direction. In a tidal cycle there are two of these periods, during high-water when flood changes into ebb and during low-water from ebb into flood. A number of considerations determines which of the two is selected:
duration of that time window, which is not the same for the two slack periods.
available keel clearance in the approach route of the caisson; sailing during high
water may be preferred.
draft and stability of the floating caisson, the ballasting operation and the
sinking depth.
the desired waterlevel in the basin after closure.
the way of placing and the equipment used.

The last item relates to the fact that caissons are preferably positioned by pushing (or
pulling) them against the current flow direction. The advantage is that if something
goes wrong, the caisson is pushed back by the flow in the free space, while in the
opposite case the caisson may float into the gap and get damaged or cause damage.
The procedure thus starts by bringing the caisson into the gap against the diminishing
flow well before the moment of still water. The most commonly used way of bringing
the caisson into the gap is to position it, in advance, head-on on one side of the gap, to
connect a corner to a fixed point on the shore (as a hinge) and to swing it around that
point like a door into the gap. Then, by ballasting, the caisson is lowered and put down
just before (or at) slack. Further ballasting will stabilize its position while the flow
direction, for flow underneath and around the caisson, turns.

For an operation at high water slack, when the tide turns from flood into ebb, the
positioning needs to be done opposite the flood flow, thus from the basin side towards
the open water. Therefore, the caisson has to be sailed into the basin via the gap
during a preceding high water period (assuming the fabrication dock to be outside the
basin). If the (last or the only) caisson is pushed into the gap by tugs, they become
trapped in the basin.

After the caisson has been put onto the foundation bed, four aspects are important:
- the load of the caisson onto the bed should be well-spread, which defines
tolerance of the bed level and the structural strength of the caisson.
- flow underneath the caisson will soon reach high values but piping (with scour
of bed material) should be prevented.
- the gap size needs to be longer than the length of the caisson to allow for
tolerances and diagonal length during the swing motion; however, outflanking of
the flow along the sides has to be blocked immediately.
- the caisson will soon be subjected to a high potential head, which will try to
either shift or overturn the caisson; generally, a linear decline of the potential
head underneath the caisson is assumed. However, if permeability of the bed is
lowest at the downstream side, the upward pressure is higher than average.
Besides, seepage flow in the bed material concentrates along the lower edge.

An example of an unusual concept of closure in which these aspects can be demonstrated clearly, is the
closure in 1978 of the Mieie, a main gulley in the tidal flat area near Meldorf in the north-west of
Germany. The originally planned closure method failed and left a closure gap with limited possibilities to
enforce a closure before the next winter. The gap was 320 m wide and the bottom elevation was 3.60 m
below MSL. The tide had a range of 3.5 to 4 meter. It was decided to try a closure by caissons, adapting
five identical sand transport barges. A new bottom protection had to replace the distorted one. A sill had
to be created as foundation for the barges. The limited water depth did not allow a high sill, not even large
stones. Therefore flow velocities had to be kept low and the five barges could not be placed one after the
other. The problem was how to put one composite caisson, consisting of five rather fragile steel barges
onto a 320 m long sill without risk for breaking, nor for piping underneath, nor for piping at the ends. The
solution found was as follows:
The barges were provided with heavy steel H-profiles underneath, along the two sides, to improve longitudinal strength, to improve penetration into the bed material and reduce free-spanning along uneven bed levels. For stability calculations it was assumed that the bed underneath the downstream H-profile would be the most impermeable and determine the upward water pressure. The barges were assembled into two composite caissons, one consisting of two and one of three barges. The connection between the barges was made by flexible material which allowed every barge to settle independently (within reasonable limits). Much attention is given to smoothen the sill to avoid high spots which would pierce the barge bottoms. The two caissons were positioned near the gap at the two shore ends, where they were connected to a hinge (pole) by steel wire. Both caissons were swung into the gap simultaneously at low water slack. For ease of positioning, to prevent the "doors" to swing too far, steel tubes had been piled in the alignment of the gap. Tugs on the seaward side had to gently push the caissons against these tubes. Being in line, the caissons showed a wide slit where they met. By pushing from the shore-ends and releasing the wire hinges, the slit was closed, while the space was divided over the two shore connections. Ballasting was done by pumping water and sand into the barges. Stones were dumped in the shore end slits and via a floating pipeline sand was pumped at high capacity along the full length of the caissons to prevent piping. As soon as the rising water allowed dump vessels to sail, stone was dumped alongside the barges also. (After a sand dike was provided at the basin side of the caissons, the barges were emptied, refloated, refitted and taken back in normal operation again.)

Generally several caissons will be placed one after the other in a period of several days. Every caisson blocks part of the gap's profile and the next caisson will be more difficult to position. Flow velocities increase, time window diminishes and the turbulent eddies in front of the caisson will be more severe. Although the program will try to work from spring tides to neaps, the last caisson to be placed will be determining for the design of caisson dimension and foundation bed material. The advantage of this method is that the operational phase in which flow velocities are very high is relatively short. This means that in areas with limited workable period, for instance because of weather or river discharges, the progress is within schedule. Besides, the duration of exposure to high flows with large eroding capacity is small.

For large closures the last caisson may need unrealistic dimensions. Then, the use of caissons provided with sluice gates is a good option. In that case, every caisson blocks a small part of the gap profile only viz. side walls, diaphragm walls, bottom structure and ballast hold. The gates will provide an opening of 80 to 85% of the submerged section of the caisson, which is to be multiplied by a discharge coefficient in the hydraulic calculations. Again the determining conditions are those during placing of the last caisson. This flow condition determines the dimension of the total opening provided by the gates. Multiplication by 1.3 gives the total gap size to be blocked by caissons with gates.

5-10

6.1. Decisive circumstances

There is not one single prescription suitable for all closures. There are too many variables and boundary conditions. The most unequivocal case is a well defined tidal basin with a single closure gap of uniform dimensions. In practice, frequently, the situation is more complex. Sometimes, special conditions may be so determining for a case that they restrict or offer possibilities. Five typical examples of such criteria are detailed below.

-a- the basin’s area can be easily subdivided into separate compartments.

Basically, this is a matter of costs. Subdividing the area diminishes the storage capacity of the individual areas. Every closure therefore can be a lot easier, probably with locally available materials and the total cost for these small-size closures may be less than the cost for one closure of the total area. However, additional costs may be needed for the construction of the embankments, separating the compartments.

Because of the later use of the area, those embankments may have to be removed. Sometimes, re-use of materials is a possibility, but part of the materials is certainly lost. Dependent on the lay-out of the area, subdivision can be designed in two ways. An elongated basin with a single channel can be taken in successive sections, while a complex channel pattern may ask for adjacent sections, as shown in the figure.

-b- the basin is penetrated by tide via two separate entrances.

Closure of the basin means closure of both entrances, with the option to close either one first and the other one next or to close both simultaneously in any combination of method, materials and phasing. All actions in one entrance will doubtlessly affect the conditions in the other entrance and the balance between the two may be quite sensitive. In case of a major unbalance, the tide conditions in the whole basin will also be affected and lead to changes in flow and subsequent erosion at several locations.

In such a case the mathematic hydraulic model may be complex and difficult to calibrate. The problem is that somewhere in the basin the tidal waves will meet. Since these waves have a different history, their shape, phase or amplitude are not exactly the same. Nevertheless, generally the tidal meeting (in Dutch called “wantij”) is characterized by low flow velocities and an unusual relation between water level and flow. The difficulty is to estimate the correct Chezy-value for the gulley system in this meeting area. For the existing situation, a wide range of values, used in the
mathematical model, may give acceptable results and thus calibration gives no clue. However, as soon as the tide changes due to the progressing closure works and the meeting area dislocates, the unchecked value may be very important. Calculations with various assumed values will at least show the possible impact on the conditions.

For simultaneous closure, the impact of every combination of construction phases on the tide penetration has to be determined. As for a single closure, this is done by calculating with weir formulae in which several schematic simplifications and practical coefficients are used. The resulting deviation has little impact for a single closure but for a dual system the balance may soon become instable. Therefore, closure plans must allow for these deviations.

Nevertheless, also in case of a very well thought-out plan of concurrent closure methods, a set-back in the execution of one, affects the other. Besides, a major failure in one closure may lead to a complete disaster as the other one has to be dismantled to maintain the basin's balance.

The easiest way to overcome the problem is to make a (temporary) closure dam across the meeting point, which separates the two tidal systems and divides the basin into two compartments. Then, the two primary closures of the basin area are fully independent. The mathematics are more simple and reliable. The closure design for each one is independent of the other one and so is the execution. Constructing the separating dam is a partial closure and frequently an obvious solution. However, in some cases it may not be allowed, for instance because it blocks a navigation route.

Another method is to close the two entrances purposely one after the other. The order of activities then is:
- fix the bottom topography of entrance "A" by protecting bottom and shores against future scour.
- close the other entrance "B" by any closure method and accept the change in tide and conditions in the basin as well as in entrance "A".
- next, close the entrance "A".

The advantage is that the closures are independent in design method and execution. In respect of the uncertainty about the response of the basin on the imbalance after the first closure, this has to be covered by assuming that deep gulleys scour across the meeting area and by taking the whole basin area as storage for the tide calculations for the second closure. Compared with the closures for separated basins, the closure "B" may be easier because "A" is still an open entrance. However, "A" with the full basin behind the gap will be much more comprehensive.

It might have been possible to stabilize the meeting area and prevent the erosion of deep gulleys. Then, the flow velocities will increase but the topography remains intact. However, the cost involved in such erosion prevention will generally be more than providing a temporary partial closure dam to fully separate the systems.
-c- the closure profile consists of two (or more) main gulleys and shallows.

In between the main gulleys there will be an area of tidal flats. These more or less separate the gulleys during the low water periods. During rising and falling tide they are storage area on the one hand and balance between the gulleys on the other hand. Although not considered a tidal meeting, there is a lot in common with that. For this case too, the first problem for the designer is the mathematic model. Tide penetration is calculated by adopting a gulley network and tidal flats are assumed storage area only. However, imbalance creates flow, results in erosion and the development of a gulley across the shallow. How quickly will that go, how deep will it be and what will be the Chezy roughness? Separating the systems by dividing the basin is not possible as both gulleys run into the same main storage area. Therefore, after construction of the dam-section across the shallow, the only possibilities remaining are:
- close the two gulleys simultaneously in a very balanced way.
- close one gulley first, accept or prevent erosion across the tidal flat, and close the other gap accounting for a fully adapted situation.

In the latter case, most likely the dam section across the tidal flat will be built before the closure of the first gulley. Since the main gulleys are relatively close, the erosion of the short cut across the tidal flat will most likely develop along this dam section. Thus, the toe of this dam has to be heavily protected. Besides, flow conditions in the remaining gap will be very adversely affected. A better solution may be to create a short-cut by dredging at an appropriate location to guide to tide towards the last gap.

-d- the gap to be closed is not in an equilibrium state.

This situation occurs in case of a calamitous breach of a dam or dike. It may also happen when a construction phase goes wrong and creates unexpected conditions at the gap’s location. In these cases, time is a very important factor. Every day, natural processes will try to achieve the equilibrium state and change the existing situation.

A first consideration is to analyze how quickly definite measures can be taken. Over that period the situation will adapt and the magnitude of the change has to be guessed in order to plan the right measures. If this change is undesirable, temporary measures can be considered to halt or retard the deterioration. Such temporary measures can be:
- stabilize the attacked bottom of the gap by dropping coarse material. Stabilizing the gap’s sides is easier but may enforce a deeper scour. Generally, scouring deeper is worse than scouring wider.
- try to avoid erosion of gulleys in the storage area, for instance by protecting critical spots with mats or quarry stone. A less resistant gulley network will result in an easier penetration of the tide and enlarge the tidal volume.
In the meantime data on the existing conditions can be measured and gathered, while a definite closure strategy can be drafted.

Usually, the existing situation has to be fixed and secured before any construction phase can start. In some cases a direct counter-attack is justified. This is in calamity cases where life is endangered. The risk in such a case is that if the action fails, generally, the situation is much worse than it was before the action. If an emergency closure can not be obtained in a couple of days, certainty is more important than speed.

An example of a successful emergency closure was the blocking of the dike breach near "Nieuwerkerk a/d IJssel" (Holland) during the major storm flood in February 1953. A relatively small breach cut the dike which secured a densely populated, vast area of Holland, north of Rotterdam. A couple of hours later that same night, a small vessel was taken and put onto the remainder of the outer slope of the dike, without any erosion protection nor any re-profiling of the gap to fit the vessel's shape. Piping under the vessel and around stem and stern could easily have scoured another gap. Then, the vessel would break and be pushed away leaving a very large gap. However, the piping was blocked by using tarpaulins, ballasted with bags filled with sand. The closure was a success and this central area of Holland remained dry.

- various alignments with different longitudinal profiles can be selected.

This is the case for instance, in a river branch with variable bottom topography. In a river bend there may be a deep triangular channel while in the crossings between bends a shallow box-profile may be available. (Likewise the alignments 1 or 2 in the upper half of the figure on page 6-1, but this time as alternative locations.) Which of the two alignments is preferred?

Another example gives the situation which occurs after a dike, situated in a shallow area breaches. The breach will erode a deep scour hole very close to the original dike's alignment. Due to spreading of the flow the surrounding shallow area will remain intact for a certain period of time, although erosion will gradually create gulleys. The option is to restore the original dike or to go around the scour hole, either along the river(sea)-side or via the inside. Various considerations determine which option is the most attractive.

An important parameter is the amount of material needed to block the gap. The flow is determined by the nett cross-sectional profile of the gap in \( \text{m}^2 \) while the gap has to be blocked by \( \text{m}^3 \) of material. For instance, a dam with slopes \( \text{1 in 1} \) with height "s" along a gap length "l", used to block a profile "l x s", has a volume of "l x s^2". An identical dam, with half the height along twice the length, blocks the same profile but consists of half the volume only. On the other hand, the bottom protection (if needed) is twice as wide but may be more than half as long (in flow direction).
Another parameter is the equipment, the materials and the closure method used. A shallow gap may be difficult to approach by large operating vessels. Caissons are used in deep gulleys preferably. For a vertical closure, however, a long gap is advantageous because of the resulting current velocities. This will be demonstrated in the examples of chapters 7.2 and 7.3.

The latter remark makes clear that for many dike breaches in the past, closing around the scour hole was preferred; the old method being vertically "sinking mattresses". In order not to loose area, where possible, the alignment at the river side was taken, by which the scour hole became situated within the enclosed polder. Nowadays, in the Netherlands, these former scour holes still can be seen in the landscape as small circular ponds just at the inner toe of the dike where the alignment of the dike winds around it in a semi-circle. Such a pond is called in Dutch a "wiël".

6.2. Main outlines and considerations

Closure design starts with a thorough analysis of the existing situation and an investigation into the effects of the closure on the environment. Secondly, the main outlines of the closure process are approximated. Which possibilities for the closure procedure are realistic alternatives. What are the advantages and disadvantages in respect of the intentions for later use of the enclosed water area? Is it one final gap or several gaps, simultaneously or consecutively? Is it possible to split-up the basin into compartments? Are there any restrictions in the use of materials or equipment, for instance because of non-availability, bad working conditions (wave climate, water depth, impassable terrain) or high mobilisation costs?

From this analysis generally quite a number of options still remains. Each of these will have to be further detailed for making a final selection. Dependent on the complexity of the area a mathematical model is required to study the hydraulics in various closure phases. Then, closure phases can be transferred into construction phases with selected materials. Dimensions and quantities can be determined and together with estimates of production capacities a time schedule can be drafted. This has to be done for all the options to enable comparison. At last, selection can be done on the basis of several criteria as for instance the budget-price or the construction time.

A very good example of the difficulty of selection is the closure of the Miele at Meldorf in northern Germany. The design made by the design engineer prescribed a closure by simultaneously lowering numerous sliding gates mounted along a long temporary bridge placed across the gap. A joint venture of experienced contractors, apart from estimating the prescribed design, tried to come up with alternatives which would give them advantage over the competition. Each partner in the group had its own preference based on his equipment and experience. As a result three alternatives were detailed and estimated:
- a different type of bridge, using the same sliding gates.
- a closure by dumping quarry stone.
- a closure by using adapted barges as caissons.

The estimation of the cost-prices showed no significant difference.

Another alternative, for which no experience existed, was also detailed:
- a closure by water inflation of a large diameter nylon tube anchored to the bottom.

This appeared to be cheaper but needed an extra sum to allow for unforeseen aspects.

Therefore the four alternatives were presented to the client for the same price, leaving the choice for him. He selected the inflatable tube, which was further detailed and executed. However, though thought to have an adequate safety in strength, the tube failed after inflation when the water level difference was about 1.80 m.

After the failure, a closure had to be enforced in a short period of time (before the winter season). The other three alternatives were reconsidered. The decision was influenced by:
- ramming the bridge would take too long,
- supply of quarry stone could not cope with the required production capacity.
- the barges as caissons were possible, provided a solution was found to place them all in one tide.

The latter was selected and realised as described section 5.4.

The number of boundary conditions and aspects which may have to be considered in an actual case of design or execution of a closure is quite extensive. Without limitation, the list below gives a summary of items that might be relevant:

A. Hydrologic data

a. river discharge characteristics,
   - statistics on discharge quantities,
   - relation between waterlevel and discharge (rating curve),
   - celerity of peak discharge wave,
   - catchment areas and run off coefficients,
   - location of measuring (gauging) stations.

b. tidal characteristics,
   - the main components of the local tide,
   - the variation between spring and neap range,
   - daily inequality of subsequent waves,
   - shallow water components,
   - time lag of the tidal wave in relation to the moon's phase,

c. seasonal changes,
   - typical seasons (monsoons, melt water, wind, frost),
   - statistics on the occurrence and magnitude of these,
   - influence of these on items a. or b.

d. meteorologic effects,
   - effect of short duration (wind, precipitation, fog, frost, evaporation),
   - statistics on the occurrence and magnitude and duration of these,
   - cyclones, hurricanes or typhoons,

e. waves,
   - sea state and swell,
   - daily conditions for workability,
   - extremes for design and survival conditions,

f. topography,
   - depth contours,
   - flow pattern across shallows,
   - storage areas in relation to tidal level,

g. special circumstances,
   - occurrence of tsunami's
   - occurrence and magnitude of earthquakes,

B. Geotechnic data

a. soil characteristics,
   - soil layers,
   - clay; Atterberg limits, water content, cohesion
   - peat; density, content of sand,
   - sand; grading, pore volume, silt content,
   - penetration values; Dutch cone, SPT,

b. phreatic behaviour,
- phreatic level and variations,
- overstressed (artesian) water in soil layers,
- permeability,
- potential head differences.

c. derived properties:
- settlements
- bearing capacity
- resistance against scour

C. Availability of materials and equipment

a. most frequently used materials,
   - (coarse) sand, gravel
   - clay or boulder clay,
   - quarry stone, (various density and grading),
   - aggregates for concreting,
   - geotextiles and willow or bamboo,
   - sheet piles and the like.

b. possible equipment,
   - dry earth movers (dump trucks, dozers, cranes, loaders)
   - various types of dredge equipment,
   - stone-dump and crane vessels,
   - various equipment for
     - position finding
     - sounding water depth
     - towing, hoisting, ramming and the like.
   - facilities to build caissons

Furthermore, several other considerations may play a role in the choice for a specific design option. Such as:

- to adopt a system which can be executed by a large number of inexperienced people (because of operating in developing countries).
- to minimize construction time to avoid a high financing costs.
- to build in separate phases because of availability of funds.
7. **Examples, Alternatives and Cases.**

7.1. **Closing an Estuary, creating Final Gaps in the Tidal Channels.**

In the foregoing chapters various details of examples and cases have been given in relation to the subject discussed. However, a plan showing the construction phases of the closure of an estuary entrance, comprising several channels and tidal flats was not detailed yet. In this chapter, a few examples of a hypothetical closure will demonstrate various possibilities. A number of alternatives will be outlined and the relation with some historic cases will be discussed. Data on flow velocities and discharges is taken from mathematical calculations, which are not detailed. However, data relevant for the motivation is presented.

The example assumes a tidal estuary which has to be closed along a fixed alignment. The longitudinal profile of the total closure consists of (see figure: phase 0):

- a foreshore, 250 m wide, 0.5 m lower than mean sea level,
- a secondary gulley of 200 m width and an average depth of 4 m below mean sea level (MSL),
- a tidal flat 300 m wide, with a level of about MSL,
- the main gulley 250 m wide, with an average depth of 6.5 m below MSL and the largest depth along the bank.

The longitudinal profile of the closure gap thus is 4000 m$^2$ at high and 1800 m$^2$ at low water. The tide is a semi-diurnal sine wave with a range of 3 m. In all calculations the tidal range is taken constant; neap-spring variation is ignored. The storage area of the basin is 20 km$^2$ at high and 5 km$^2$ at low water.

Three main options will be studied:

- **a**- dam sections across the shallows first, next closing the gulleys. (in this section)
- **b**- dealing with the gulleys first and closing the shallows last. (in section 7.2)
- **c**- all simultaneously. (in section 7.3)

Each option may have several alternatives.

The option "shallows first", is detailed below.

Dam sections across the shallows will create two gaps. Then, many possibilities exist, but a few alternatives are unattractive. The secondary gap for instance is very shallow for using caissons. And for a vertical closure, the total length of the (two) sills, 450 meters only, is rather short (this will be clarified when detailing the options b and c). A mathematic model is needed to get the required data for a well-considered decision.
Horizontal closure by tipping quarry stone in both gaps is a very good possibility, but for the purpose of this example, a combination of placing caissons and tipping quarry stone will be detailed.

In the next three figures some construction phases are presented. The program reads:

Two gaps are created by damming the shallows. Bottom protection is provided for in the remaining gaps. Then, both closure gaps are slightly diminished in sectional profile by creating sills. For a caisson closure abutments are also made. These are concrete structures or sheetpile walls which shape the vertical sides of the closure gap. Next, caissons are positioned in the main gap and finally the secondary gap is closed by tipping quarry stone.

Expressed in phases:

<table>
<thead>
<tr>
<th>phase and action</th>
<th>foreshore</th>
<th>sec. gulley</th>
<th>tidal flat</th>
<th>main gulley</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 original state</td>
<td>250m; -0.5</td>
<td>200m; -4</td>
<td>300m; MSL</td>
<td>250m; -6.5</td>
</tr>
<tr>
<td>1 bott.prot. + shallows</td>
<td>dammed</td>
<td>200m; -3.5</td>
<td>dammed</td>
<td>250m; -6</td>
</tr>
<tr>
<td>2 partial sills in both</td>
<td>dammed</td>
<td>200m; -3</td>
<td>dammed</td>
<td>250m; -4.5</td>
</tr>
<tr>
<td>3 final sill, abutments</td>
<td>dammed</td>
<td>200m; -2.5</td>
<td>dammed</td>
<td>190m; -4.5</td>
</tr>
<tr>
<td>4 first caisson in place</td>
<td>dammed</td>
<td>200m; -2.5</td>
<td>dammed</td>
<td>128m; -4.5</td>
</tr>
<tr>
<td>5 sec. caisson in place</td>
<td>dammed</td>
<td>200m; -2.5</td>
<td>dammed</td>
<td>66m; -4.5</td>
</tr>
<tr>
<td>6 third caisson in place</td>
<td>dammed</td>
<td>200m; -2.5</td>
<td>dammed</td>
<td>closed</td>
</tr>
<tr>
<td>7 narrowing on sec. sill</td>
<td>dammed</td>
<td>100m; -2.5</td>
<td>dammed</td>
<td>closed</td>
</tr>
<tr>
<td>8 further narrowing</td>
<td>dammed</td>
<td>50m; -2.5</td>
<td>dammed</td>
<td>closed</td>
</tr>
<tr>
<td>9 very last gap</td>
<td>dammed</td>
<td>10m; -2.5</td>
<td>dammed</td>
<td>closed</td>
</tr>
</tbody>
</table>

As the two gaps are blocked virtually one after the other, there may be quite some imbalance in the tidal system between the gaps. This may require extra measures. To prevent this, the tipping into the secondary gap has to run together with the placing of the caissons. The risk then is that the flow conditions during the sinking of the last caisson are too high. In fact, this is already questionable in the above phasing.

Checking on these conditions gives the data in the next table. At the moment that two caissons are placed (phase 5) the maximum discharge via the secondary gap has doubled and reaches about the same magnitude as the main gap had originally. To the contrary, the main gap halved its discharge. The secondary channel will have to accommodate the doubled quantities, for which its profile will be quite small. A scouring of a gulley across the shallow from the main to the secondary channel therefore is the likely consequence of the imbalance.
(The values in the table have been calculated for the same tide variation at sea; possibilities of using spring/neap variation is discarded. Maximum flow does not necessarily coincide with maximum discharge, neither do the maxima of the two gaps always coincide)

<table>
<thead>
<tr>
<th>phase situation</th>
<th>secondary gap</th>
<th>main gap</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>during ebb</td>
<td>during flood</td>
</tr>
<tr>
<td></td>
<td>Umax</td>
<td>Qmax</td>
</tr>
<tr>
<td>0 orig.</td>
<td>1.09</td>
<td>915</td>
</tr>
<tr>
<td>1 bp + dams</td>
<td>1.33</td>
<td>1010</td>
</tr>
<tr>
<td>2 sills</td>
<td>1.67</td>
<td>1065</td>
</tr>
<tr>
<td>3 abutm</td>
<td>2.12</td>
<td>1090</td>
</tr>
<tr>
<td>4 1 placed</td>
<td>2.71</td>
<td>1305</td>
</tr>
<tr>
<td>5 2 placed</td>
<td>3.57</td>
<td>1550</td>
</tr>
</tbody>
</table>

(phase 2 is pictured in the figure)

The positioning of the last caisson takes place in the situation of phase 5 during HW-slack, as at LW there is insufficient water depth. At the end of the flood period the flow diminishes as follows:

- 30 min. before slack: \( u = 1.50 \) m/s,
- 20 min. before slack: \( u = 1.20 \) m/s,
- 10 min. before slack: \( u = 0.70 \) m/s.

For a safe sinking operation, these values are far too high. Consequently, the program has to be adapted. Instead of using ordinary caissons, they can be equipped with sluice gates. The program then reads:

<table>
<thead>
<tr>
<th>phase and action</th>
<th>foreshore</th>
<th>sec. gulley</th>
<th>tidal flat</th>
<th>main gulley</th>
<th>sluice gate</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 first placed, opened</td>
<td>dammed</td>
<td>200m; -2.5</td>
<td>dammed</td>
<td>128m; -4.5</td>
<td>56m; -3.5</td>
</tr>
<tr>
<td>5 sec. placed, opened</td>
<td>dammed</td>
<td>200m; -2.5</td>
<td>dammed</td>
<td>66m; -4.5</td>
<td>112; -3.5</td>
</tr>
<tr>
<td>6 third caisson placed</td>
<td>dammed</td>
<td>200m; -2.5</td>
<td>dammed</td>
<td>0m;</td>
<td>112; -3.5</td>
</tr>
<tr>
<td>7 narrowing on sill</td>
<td>dammed</td>
<td>100m; -2.5</td>
<td>dammed</td>
<td>0m;</td>
<td>112; -3.5</td>
</tr>
<tr>
<td>8 further narrowing</td>
<td>dammed</td>
<td>50m; -2.5</td>
<td>dammed</td>
<td>0m;</td>
<td>112; -3.5</td>
</tr>
<tr>
<td>9 very last gap in sec.</td>
<td>dammed</td>
<td>10m; -2.5</td>
<td>dammed</td>
<td>0m;</td>
<td>112; -3.5</td>
</tr>
<tr>
<td>10 close sluice gates</td>
<td>dammed</td>
<td>dammed</td>
<td>dammed</td>
<td>0m;</td>
<td>closed</td>
</tr>
</tbody>
</table>

(phase 4 is pictured in the figure)

This time, the positioning of the last caisson takes place in phase 5 with the sluice gates of the two other caissons opened (assumed to have 56m effective width each and a floor thickness of 1m). Then, at the end of the flood period the flow conditions appear to be acceptable:
30 min. before slack: $u = 0.75 \text{ m/s}$,
20 min. before slack: $u = 0.50 \text{ m/s}$,
10 min. before slack; $u = 0.20 \text{ m/s}$.

At first, the balance between the gaps will be better than in the former schema, as the gates provide a flow profile while tipping starts in the other gap. Although the secondary gap is closed before the gates close, the main channel does not exceed its original discharge. The balance can still be improved by closing a number of gates simultaneously with the tipping. However, that worsens the conditions for the tipping.

The flow conditions are:

<table>
<thead>
<tr>
<th>phase situation</th>
<th>secondary gap</th>
<th>main gap **</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>during ebb</td>
<td>during flood</td>
</tr>
<tr>
<td>(m/s,m$^3$/s)</td>
<td>Umax</td>
<td>Qmax</td>
</tr>
<tr>
<td>5 1+2, open</td>
<td>2.60</td>
<td>1280</td>
</tr>
<tr>
<td>6 3 placed</td>
<td>3.35</td>
<td>1480</td>
</tr>
<tr>
<td>7 100m gap</td>
<td>3.87 *</td>
<td>830</td>
</tr>
<tr>
<td>8 50m gap</td>
<td>3.78 *</td>
<td>410</td>
</tr>
<tr>
<td>9 10m gap</td>
<td>3.62 *</td>
<td>80</td>
</tr>
</tbody>
</table>

* means critical flow.
** via the sluice gates

Critical flow occurs during the ebb, also in the sluices. Per tidal cycle it lasts for nearly 2 hours in phase 8 and for 2.5 hours in phase 9. Probably, this can be prevented by providing the third caisson with sluice gates as well (which was not required for the sinking operation).

If so, the conditions for the tipping of the stone are better, but the imbalance between the gaps increases. Whether it is worth the extra expense, depends on the savings on stone tipping and bottom protection.

The flow and discharge conditions for three sluice caissons are:

<table>
<thead>
<tr>
<th>phase situation</th>
<th>secondary gap</th>
<th>main gap **</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>during ebb</td>
<td>during flood</td>
</tr>
<tr>
<td>(m/s,m$^3$/s)</td>
<td>Umax</td>
<td>Qmax</td>
</tr>
<tr>
<td>7 100 m gap</td>
<td>3.35 *</td>
<td>720</td>
</tr>
<tr>
<td>8 50m gap</td>
<td>3.55 *</td>
<td>385</td>
</tr>
<tr>
<td>9 10m gap</td>
<td>3.49 *</td>
<td>80</td>
</tr>
</tbody>
</table>
Critical flow in the sluices now occurs during the very last operation and lasts for half an hour only. Maximum flow velocities in the secondary gap reduce by about 10%.

A historic case of the above system of closing a tidal basin is the construction in 1965 up to 1972 of the "Brouwersdam", damming the "Brouwershavensche Gat" by which the "Lake Grevelingen" was created. Dimensions of the channels and the basin were much larger. The total dam alignment had a length of 6 km. In that case the minor gulley was closed by sinking 12 sluice-caissons, each 68 m long, on a sill levelled at 10 m below MSL. The main gulley was closed by gradual closure with concrete cubes of 50 kN each. The profile of this gap was about 13000 m². Contrary to the example, the gradual closure was a vertical closure, dropping the cubes by means of a pre-installed cableway. As in the example, a limiting factor for the progress of the gradual closure was the flow condition during positioning and sinking of the last caisson.

7.2. Blocking the Main Channel first.

In this section the same estuary as above is closed by reducing the profiles of the gulleys first. Then the main gulley will be blocked completely. Next, the secondary gulley will be further reduced and finally the total rest profile will be blocked. This option is worth consideration if it leads to a cheaper closure operation. The obvious disadvantage, when the channels are blocked first, is that the bottom across the shallows has to be protected against scour also. Cost savings on the other items need to compensate for this expensive extra. Such savings may result from a possibly reduced dimension of the protection in the gulley and from cheaper caisson design. A major saving would result by using caissons without sluice gates, while another saving could be obtained by using two caissons only.

A determining factor for the decision to omit sluice gates is the positioning of the last caisson. The flow conditions will be best when the flow profile is as large as possible at that moment. This is the case when there is no sill in the secondary gap. Assuming the same dimensions of the caissons, the determining total flow profile is the original profile, diminished by bottom protection over the full length, by the abutments on both sides of the main gulley, by the foundation sill and by the caisson(s) already placed. The HW-slack period then is characterized by:

- 30 min. before slack: \( u = 0.80 \text{ m/s} \),
- 20 min. before slack: \( u = 0.60 \text{ m/s} \),
- 10 min. before slack: \( u = 0.25 \text{ m/s} \).

Positioning will not be a problem. There is however, a substantial imbalance between the two gulleys. The maximum flood via the secondary gulley is 1975 m³/s, which is more than twice the original. A sill in this gulley, up to the level -3m, brings the discharge down to 1860 m³/s. The effect is small and flow velocities in the main gulley increase up to:

- 30 min. before slack: \( u = 0.95 \text{ m/s} \),
- 20 min. before slack: \( u = 0.65 \text{ m/s} \),
- 10 min. before slack: \( u = 0.35 \text{ m/s} \).

The sinking is possible, but further raising the sill is not acceptable. The construction of the sill up to -3m in the secondary gulley can best be done simultaneously with the foundation sill in the main gulley. After that, caissons can be placed at short intervals to limit the duration of the imbalance.
Saving on the number of caissons depends on the flow conditions for creating the smaller gap for the (two) caissons, as the narrowing of the gulley has to be done by pumping sand or the like. The flow profile available is the original profile diminished by bottom protection and by the island for the abutment on the shallows. The longitudinal profile then consists of 250 m foreshore, 200 m secondary gulley and 250 m shallows, all provided with bottom protection, an island section on the shallows of 50 m length and adjoining island in the main gulley along 75 m, leaving a gap length for two abutments of 25 m each, two caissons of 60 m each and 5 m extra (see figure). In the secondary gap, bottom protection will be present and some of the sill construction may exist. The calculations show that the maximum flow velocities in the gap are 1.70 m/s during ebb and 1.60 m/s during flood, which is no problem for the construction of the island.

The conclusion is that blocking the gulleys first can be done by closing the main gap with two simple caissons, while a restricted sill is present in the secondary gulley. The remaining flow profile consist of two 250 m long shallow sections and a partly blocked gap in between.

The construction phasing up to this moment thus reads:

<table>
<thead>
<tr>
<th>phase and action</th>
<th>foreshore</th>
<th>sec. gulley</th>
<th>tidal flat</th>
<th>island</th>
<th>main gulley</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 original state</td>
<td>250m; -0.5</td>
<td>200m; -4</td>
<td>300m; MSL</td>
<td>none</td>
<td>250m; -6.5</td>
</tr>
<tr>
<td>1 bott.prot + island</td>
<td>250m; MSL</td>
<td>200m; -3.5</td>
<td>250m; +0.5</td>
<td>125m; 175m; -6</td>
<td></td>
</tr>
<tr>
<td>2 sills in both</td>
<td>250m; MSL</td>
<td>200m; -3</td>
<td>250m; +0.5</td>
<td>125m; 175m; -4.5</td>
<td></td>
</tr>
<tr>
<td>3 sill, abutments</td>
<td>250m; MSL</td>
<td>200m; -3</td>
<td>250m; +0.5</td>
<td>150m; 125m; -4.5</td>
<td></td>
</tr>
<tr>
<td>4 first caisson in pl</td>
<td>250m; MSL</td>
<td>200m; -3</td>
<td>250m; +0.5</td>
<td>150m; 65m; -4.5</td>
<td></td>
</tr>
<tr>
<td>5 sec. caisson in pl</td>
<td>250m; MSL</td>
<td>200m; -3</td>
<td>250m; +0.5</td>
<td>closed</td>
<td></td>
</tr>
</tbody>
</table>

Flow velocities and discharges are as follows:

<table>
<thead>
<tr>
<th>phase</th>
<th>situation</th>
<th>secondary gap **</th>
<th>main gap</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>during ebb</td>
<td>during flood</td>
<td>during ebb</td>
</tr>
<tr>
<td>(m/s, m³/s)</td>
<td>Umax</td>
<td>Qmax</td>
<td>Umax</td>
</tr>
<tr>
<td>0 orig.</td>
<td>1.09</td>
<td>915</td>
<td>1.07</td>
</tr>
<tr>
<td>1 botpr + isle</td>
<td>1.61</td>
<td>1155</td>
<td>1.52</td>
</tr>
<tr>
<td>2 sills</td>
<td>2.01</td>
<td>1175</td>
<td>1.85</td>
</tr>
<tr>
<td>3 abutm</td>
<td>2.42</td>
<td>1355</td>
<td>2.19</td>
</tr>
<tr>
<td>4 after 1st</td>
<td>3.06</td>
<td>1585</td>
<td>2.68</td>
</tr>
<tr>
<td>5 after 2nd</td>
<td>3.98 *</td>
<td>1860</td>
<td>3.37</td>
</tr>
</tbody>
</table>

** the central 200 m section only (the shallows falling dry during low tide).
The next step could be a horizontal closure by tipping quarry stone, from either one side or from both sides. It creates high flow velocities in the secondary gap. The situation is comparable with the former option, except for the sluice gates. The flow velocities will rise in this case to about 4.5 m/s. Therefore, it is more appropriate to try to reduce the profile of the secondary gap, maintaining the flow across the shallows. Dumping quarry stone by dump-vessels will be impossible because of draught restriction. However, vertical closure is possible by means of a temporary bridge (to be installed in the previous period) or a cableway (ditto). The length is considerable (700 m) but 500 m of the bridge crosses shallow water and so, the foundation cost is limited.

Another method, with a difficult operational procedure, is to recognise the fact that during LW the dam section across the shallows falls dry for several hours. The first step is to bring the sill level up in two layers to above LW, so from -3 to -2 and then to -1. The water level in the basin will not follow the sea level and the relation between the levels of the sea, the basin and the sill determine the operational possibilities. The equipment to be used is a shallow-draught crane vessel and dump trucks with hydraulic cranes, approaching via the drying dam sections. The operational period per tidal cycle, the work-window, is small and the production is low, but the equipment is available on the market and investment in bridge or cableway can be avoided. Gradually, layer by layer, the sill will be raised. For every layer, the determining moment exists when the last 10 m has to be made. That missing part of the layer is a dip in the sill level which is subjected to higher (critical) flow.

The crane vessel can operate when anchored near the gap during the periods that flow velocities are smaller than 2 m/s. For the layer from -3m to -2m (phase 5 to 6) this is 2 hours during HW and 1 hour during LW on average. For the next layer (phase 6 to 7), up to -1m, this is 1.5 hours at HW and three quarters of an hour during LW. After that, dump trucks may start to assist during the LW-period, as the water level at the seaward side will fall lower than the sill level. At the start of the next layer up to MSL (phase 7 to 8), the basin level falls below that level during about 1 hour (b-b in the figure). When finishing the layer, the water levels
are less than 0.5 meter above the sill’s level for two hours. Though risky, trucks can operate in water depth of less than 0.5 meter.

The construction phasing thus continues for the 700 m section vertically:

<table>
<thead>
<tr>
<th>phase and action</th>
<th>foreshore</th>
<th>sec. gulley</th>
<th>tidal flat</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 first layer</td>
<td>250m; MSL</td>
<td>97m; -2</td>
<td>97m; -2</td>
</tr>
<tr>
<td>7 first layer</td>
<td>250m; MSL</td>
<td>97m; -1</td>
<td>97m; -1</td>
</tr>
<tr>
<td>8 level foreshore</td>
<td>250m; MSL</td>
<td>97m; MSL</td>
<td>6m; -1</td>
</tr>
<tr>
<td>9 level tidal flat</td>
<td>222m; +0.5</td>
<td>6m; MSL</td>
<td>222m; +0.5</td>
</tr>
<tr>
<td>10 level +1</td>
<td>347m; +1</td>
<td>6m; +0.5</td>
<td>347m; +1</td>
</tr>
<tr>
<td>11 final layer</td>
<td>dammed</td>
<td>6m; +1</td>
<td>dammed</td>
</tr>
</tbody>
</table>

The flow velocities for the various levels of the sill are:

<table>
<thead>
<tr>
<th>Umax. in m/s:</th>
<th>deepest part</th>
<th>deepest but one</th>
<th>deepest but two</th>
<th>deepest but three</th>
</tr>
</thead>
<tbody>
<tr>
<td>phase situation</td>
<td>ebb</td>
<td>flood</td>
<td>ebb</td>
<td>flood</td>
</tr>
<tr>
<td>5 after 2nd</td>
<td>3.98</td>
<td>3.37</td>
<td>2.34</td>
<td>2.85</td>
</tr>
<tr>
<td>6 up to -2</td>
<td>4.22</td>
<td>3.43</td>
<td>3.81</td>
<td>3.84</td>
</tr>
<tr>
<td>7 up to -1</td>
<td>3.82</td>
<td>3.38</td>
<td>3.28</td>
<td>3.68</td>
</tr>
<tr>
<td>8 up to MSL</td>
<td>3.27</td>
<td>2.92</td>
<td>2.50</td>
<td>2.91</td>
</tr>
<tr>
<td>9 up to 0.5</td>
<td>2.32</td>
<td>2.67</td>
<td>1.98</td>
<td>2.32</td>
</tr>
<tr>
<td>10 up to +1</td>
<td>1.86</td>
<td>2.18</td>
<td>1.05</td>
<td>1.55</td>
</tr>
<tr>
<td>11 up to HW</td>
<td>0.88</td>
<td>1.55</td>
<td>high water free</td>
<td></td>
</tr>
</tbody>
</table>

* means limited by critical flow condition.

In the table the deepest part represents the dip in the sill mentioned above. Although in all sections of the sill critical flow limits the flow velocity, in the dip this is only true for the ebb. Besides, ebb is determining for sill levels up to about MSL, above that flood flows are higher.

Considering the above results, it appears that the maximum flow velocity in the secondary gap during the raising of the sill is not very much less than in case a horizontal closure had been designed (4.22 m/s instead of about 4.50 m/s). This is due to the fact that the limiting critical flow condition for the -2m sill level under these circumstances is about the same as the normal flow condition for a narrow gap with a 3 m tidal range. The determining condition occurs for the dip in the low sill in the 200 m gap. In that stage of the process, the 500 m shallow sections are too elevated to be useful. This example proves by its exception that the general rule that vertical closure leads to smaller flow velocities than horizontal closure is (not always) true. Neither the difficult operational procedure and small production capacity, nor the investment for a bridge or cableway, can compete (money wise) with the dump trucks operating from two sides for horizontal closure.

A rather critical point in the closure phasing is the situation near the island after the caissons have been placed. Then, the main gulley is blocked and the secondary gulley more than doubles its discharge. An easy way for the water to pass through the gap is to follow the main gulley, to circle across the shallows around the island and to return into the main gulley. Scouring a short-cut like that is a typical example of the consequence of the imbalance. It would be a disaster and has to be prevented.

A historic case of a closure in which firstly gulleys were blocked by caissons and then
the shallows were closed, is the closure of the Schelphoek, one of the major dike breaches in the south west of the Netherlands (1953). The situation of the breaches and the closure alignment are pictured in section 2.3 showing the development of erosion gulleys. The picture giving the situation after 20 weeks shows the two gaps that had been shaped, typically suited for caisson placing, while the long overland stretches had been protected by mattresses. After the caissons had been placed the overland sections followed by horizontal closure. A large number of shallow concrete units was placed every tide, in such a short sequence that the overland flow could not scour a short-cut.

The vertical closure procedure, using dump trucks, driving on the sill and using hydraulic cranes to level the cobbles, was executed during the closure of the Markiezaatsskade (1983), one of the secondary dams required in the Deltaworks. The dam closed a shallow tidal basin of about 20 km$^2$ with a tidal range of about 4 meters. The final gap was 800 m long and had a basic sill level of 2 m below MSL. That time the vertical closure was an advantage as the full 800 m had the same low sill level. The quarry stone dam was constructed in layers of 1 or 0.5 m thickness.

7.3. Closure over the Full Dam Length.

The option to close over the full length of the alignment is logically a vertical closure or a combined closure. The principle is to block the gulleys partly by sills first, until one level exists over the full length. Then, either vertically or horizontally, the 1000 m long gap above that sill is closed. The difference with the option to close (one of) the gulleys first is that there are no caissons, that the imbalance will be smaller and that flow conditions are more favourable due to the longer weir. The first two phases of construction are:
- a bottom protection along the full alignment,
- sills to be dumped up to the level of -2.5m (vessels' draught permitting).

Then, horizontal or vertical closure has to be selected. With a horizontal closure one very final gap is created. While proceeding to that gap the flow conditions on the lowest part of the sill will be determining. As that sill level is -2.5m, the situation of the former options is created again. The same final stage is reached than at the cost of a bottom protection over the full length. The bottom protection in the shallow areas therefore is superfluous, so the full length option is meaningful only in case vertical closure is taken. The next phases therefore are:
- a next layer bringing the level of the sills up to -1m,
- further layers of 0.5 m thickness up to HW.

The procedure is identically to the last phases of the former option. The difference is
the weir length which is 1000 m instead of 700 m.

The phasing of the closure thus reads:

<table>
<thead>
<tr>
<th>phase and action</th>
<th>foreshore</th>
<th>sec. gulley</th>
<th>tidal flat</th>
<th>main gulley</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 original state</td>
<td>250m: -0.5</td>
<td>200m: -4</td>
<td>300m: MSL</td>
<td>250m: -6.5</td>
</tr>
<tr>
<td>1 bot.prot + sill (-3.5)</td>
<td>250m: MSL</td>
<td>200m: -3.5</td>
<td>300m: +0.5</td>
<td>250m: -3.5</td>
</tr>
<tr>
<td>2 sills dumped (-3)</td>
<td>250m: MSL</td>
<td>200m: -3</td>
<td>300m: +0.5</td>
<td>250m: -3</td>
</tr>
<tr>
<td>3 sills dumped (-2.5)</td>
<td>250m: MSL</td>
<td>200m: -2.5</td>
<td>300m: +0.5</td>
<td>250m: -2.5</td>
</tr>
<tr>
<td>4 sill by trucks (-1)</td>
<td>250m: MSL</td>
<td>200m: -1</td>
<td>300m: +0.5</td>
<td>245m: -1 5m: -2.5</td>
</tr>
<tr>
<td>5 up to MSL</td>
<td>445m: MSL</td>
<td>5m: -1</td>
<td>300m: +0.5</td>
<td>250m: MSL</td>
</tr>
<tr>
<td>6 up to +0.5</td>
<td>445m: +0.5</td>
<td>5m: MSL</td>
<td>300m: +0.5</td>
<td>250m: +0.5</td>
</tr>
<tr>
<td>7 up to +1</td>
<td>445m: +1</td>
<td>5m: +0.5</td>
<td>300m: +1</td>
<td>250m: +1</td>
</tr>
<tr>
<td>8 up to HW</td>
<td>445m: +1.5</td>
<td>5m: +1</td>
<td>300m: +1.5</td>
<td>250m: +1.5</td>
</tr>
</tbody>
</table>

The influence on the flow conditions appears from the lists below (to be compared with the table in the former option for the 700 m long weir):

<table>
<thead>
<tr>
<th>flow and discharge</th>
<th>secondary gap</th>
<th>main gap</th>
</tr>
</thead>
<tbody>
<tr>
<td>phase situation</td>
<td>during ebb</td>
<td>during flood</td>
</tr>
<tr>
<td></td>
<td>Umax</td>
<td>Qmax</td>
</tr>
<tr>
<td>(m/s, m³/s)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0 orig.</td>
<td>1.09</td>
<td>915</td>
</tr>
<tr>
<td>1 botpr + sill</td>
<td>1.78</td>
<td>1230</td>
</tr>
<tr>
<td>2 sills -3</td>
<td>2.06</td>
<td>1180</td>
</tr>
<tr>
<td>3 sills -2.5</td>
<td>2.48</td>
<td>1110</td>
</tr>
<tr>
<td>4 sill -1</td>
<td>2.99*</td>
<td>710</td>
</tr>
</tbody>
</table>

The discharge quantities in the remaining gap diminish with the progressing construction of the sill. The determining flow conditions are those in the narrow dip of every layer. The flow velocities in the various parts of the 1000 m gap are:

<table>
<thead>
<tr>
<th>phase situation</th>
<th>ebb</th>
<th>flood</th>
<th>ebb</th>
<th>flood</th>
<th>ebb</th>
<th>flood</th>
<th>ebb</th>
<th>flood</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 sill -1</td>
<td>3.62*</td>
<td>3.09</td>
<td>2.99*</td>
<td>3.49*</td>
<td>2.24*</td>
<td>2.74*</td>
<td>1.68*</td>
<td>2.27*</td>
</tr>
<tr>
<td>5 up to MSL</td>
<td>2.97*</td>
<td>2.87</td>
<td>2.33*</td>
<td>3.05*</td>
<td>1.98*</td>
<td>2.32*</td>
<td>not applicable</td>
<td></td>
</tr>
<tr>
<td>6 up to 0.5</td>
<td>2.32*</td>
<td>2.64*</td>
<td>1.95*</td>
<td>2.41*</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>7 up to +1</td>
<td>1.90*</td>
<td>2.05</td>
<td>1.11*</td>
<td>1.55*</td>
<td>not applicable</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8 up to HW</td>
<td>0.88*</td>
<td>1.55*</td>
<td>high water free</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td></td>
</tr>
</tbody>
</table>

Comparing the phases 4 to 8 of this option, with phases 7 to 11 of the former option, it appears that the maximum flow velocity is considerably less (3.62 m/s and 4.22 m/s) as consequence of the longer gap length. The phase with the determining flow conditions is phase 4. Except for the last dip in the sill, which exists as consequence of the construction of the layer up to the level of -1 m, all flow conditions reach the critical flow stage. In the dip the flood flow is still sub-
critical. The magnitude is slightly less than the critical flow above the -1 m elevated sill, which is due to the low discharge-coefficient used for this narrow gap.

Even more important is the fact that the time window for the equipment to operate on the sill is much longer than in the former option. This is due to the lower low-water level in the basin for the comparable level of the sill. On the other hand, an operational difficulty of the present option is that the layer construction has to advance over 500 m from each side instead of 350 m.

An alternative to avoid this problem of 500 m driving distance, is to prepare an approach-road towards to centre of the sill via an artificial island on the tidal flat. The construction can then advance from four sides along 250 m each at the cost of a major transport to the island and of installing transhipment facilities. The total sill length reduces with the width of the road connection only. If the island is situated in the alignment, the advantage of the long sill length decreases. At certain dimensions the method even changes into the first option with the final gaps in the channels, using vertical closure instead of caissons.

An example of a closure by constructing sills in the gulleys and providing one level over the full length of the alignment is the damming of the River Feni in Bangla Desh in 1984/85 (see also page 1-1). The tides were very variable, due to shallow water effects. Spring ranges doubled the neap ranges while low water levels were always about the same (see section 3.1, influence of MSf). During spring tides the tidal wave entered the estuary as a tidal bore. Therefore, conditions during neaps were a lot more favourable than during springs and the last layers of the vertical closure had to be forced in a major effort over the neap tide period. During that closure day a neap-tide-safe profile had to be constructed on top of the sill, which had to be heightened up to a spring-tide-safe profile within a week’s time.

The tide on closure day rose from -0.5 m to +2.65 m (range 3.15 m) while the start level of the sill was +0.70 m. On closure day an embankment was constructed up to the level +3 m by piling up jute bags filled with clay. In total 1,000,000 bags were positioned by 12,000 Bangla Deshi people, all by hand, in five hours time. In order to minimize the hauling distance the bags had been stored in 12 stockpiles along the alignment of the dam, which reduced the total gap’s length to about 1000 meter. The enlargement of the profile to the spring tide safe profile was done by trucks, tipping clay.