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Fighting Against the Current

Restoring dike breaches and closing tidal channels by simple means - from past to present

van Duivendijk, J.

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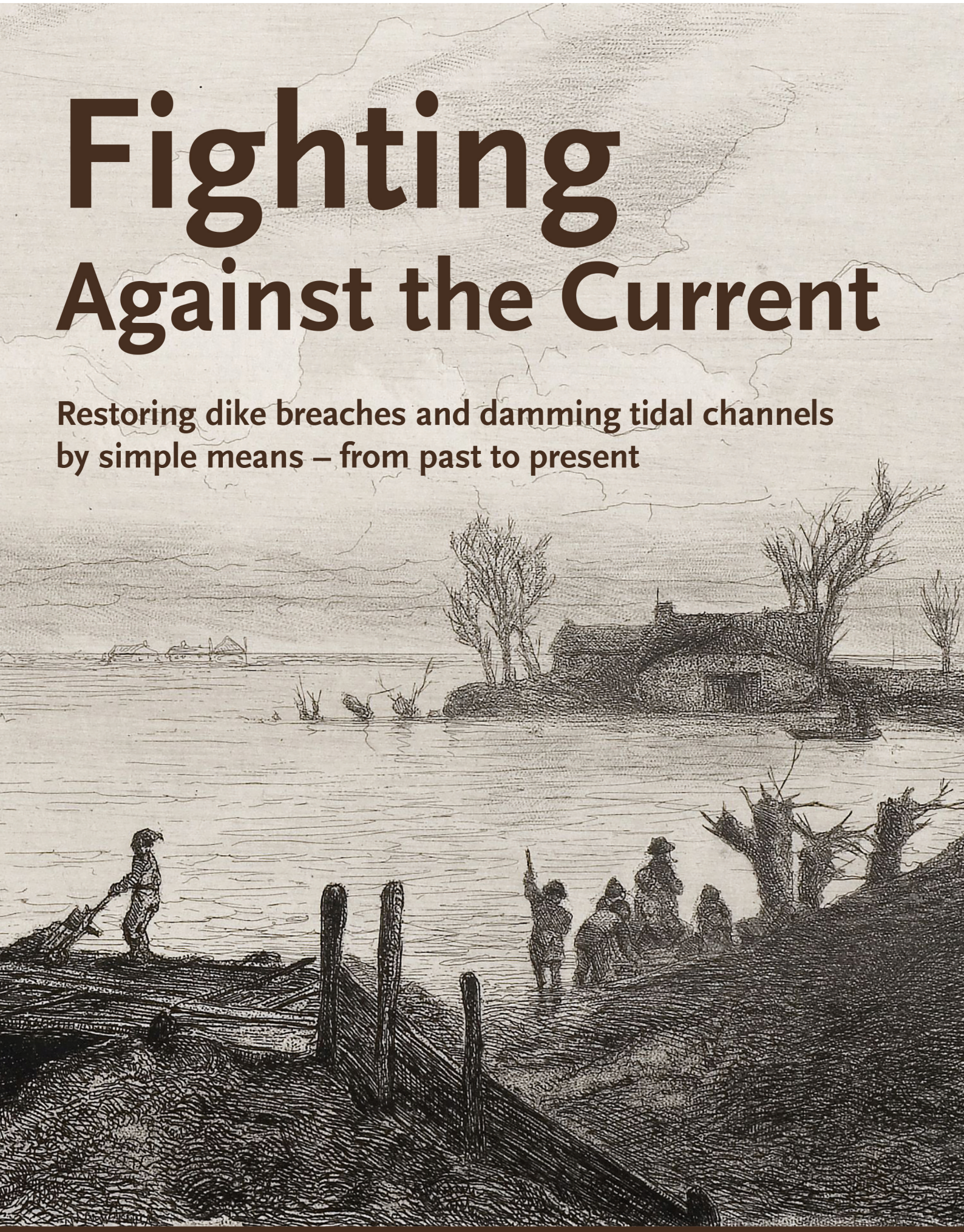
Fighting Against the Current

Restoring dike breaches and damming tidal channels by simple means – from past to present



J. van Duivendijk

Fighting Against the Current

Restoring dike breaches and closing tidal channels by simple means – from past to present

Dissertation

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prof.dr.ir. T.H.J.J. van der Hagen
chair of the Board for Doctorates
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Monday 17 June 2024 at
12.30

by

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Cover picture: Cofferdam built for the breach at the Spees in 1855 (southern bank Nederrijn near Opheusden); courtesy: Atlas van Stolk, Rotterdam, etching made by Herman and Mari ten Kate
Back picture: see curriculum vitae

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PROPOSITIONS

pertaining to the dissertation

Fighting Against the Current

Restoring dike breaches and closing tidal channels by simple means – from past to present

by

J. van Duivendijk,

Delft, 17 June 2024

1. Techniques developed in earlier centuries for closing tidal channels by simple means can still be applied in the present day. (this dissertation)
2. The so-called fascine pole or sink-fascine (a sausage-shaped structure made of brushwood, with a core of clay clods or stone) is the predecessor of the traditional floating fascine mattress and can be considered as hydraulic engineering heritage. (this dissertation)
3. There is a tendency among hydraulic engineers to give little or no publicity to mistakes made, resulting in those mistakes being repeated by others. (this dissertation)
4. Jumping to conclusions when scrutinizing the scarce data about hydraulic engineering projects constructed during past centuries, is dangerous.
5. Intelligent young men who started as dike workers in the past have done more for development of hydraulic engineering than academics trained at a university.
6. The frequent use of computer models to do engineering calculations does not necessarily lead to a greater understanding of the subject matter.
7. It is surprising that in a country where hydraulic engineering and water management are of such importance, it has not been deemed necessary for many years to appoint a specialist minister to the relevant ministry.
8. The name change from ‘technological college’ to Technological University of Delft would not have been necessary as far as the quality and reputation of civil engineers is concerned.
9. The very informal way in which the Graduate School approaches its older PhD students is in stark contrast to the way in which those PhD students have to address their much younger opponents on the doctoral committee.
10. Those who listen learn a lot, those who think know more.

These propositions are regarded as opposable and defendable, and have been approved as such by the promotors Prof. dr. ir. S.N. Jonkman and dr.ir. B. Hofland and the co-promotor dr. Ing. M.Z.Voorendt

STELLINGEN

behorende bij het proefschrift

Fighting Against the Current

Restoring dike breaches and closing tidal channels by simple means – from past to present

van

J. van Duivendijk,

Delft, 17 juni 2024

1. De technieken in vroeger eeuwen ontwikkeld voor het sluiten van getijgeulen met eenvoudige middelen kunnen in de tegenwoordige tijd nog steeds worden toegepast. (dit proefschrift)
2. De zogenaamde dodeman of zink-fascine (een worstvormige constructie van rijshout, met een kern van schorklei of steen) is de voorloper van het traditionele rijshouten zinkstuk en kan beschouwd worden als waterbouwkundig erfgoed. (dit proefschrift)
3. Er is een neiging bij waterbouwers om weinig of geen ruchtbaarheid te geven aan gemaakte fouten, waardoor die fouten opnieuw door anderen worden gemaakt. (dit proefschrift)
4. Het overhaast trekken van conclusies op basis van als schaarse informatie over waterbouwkundige projecten, gebouwd in het verleden, is gevaarlijk.
5. Intelligente jonge mannen die in het verleden als dijkwerker begonnen hebben meer voor ontwikkeling van de waterbouwkunde betekend dan academici die waren opgeleid aan een universiteit.
6. Het veelvuldig gebruik van computermodellen voor het doen van technische berekeningen leidt niet noodzakelijkerwijs tot meer inzicht in de materie.
7. Het is wonderlijk dat in een land waar het waterstaatkundig beheer van zo groot belang is, het al vele jaren niet nodig wordt geacht om een vakminister op het betrokken ministerie te benoemen.
8. De naamsverandering van Technische Hogeschool in Technische Universiteit Delft was voor wat betreft de kwaliteit en reputatie van civiel ingenieurs niet nodig geweest.
9. De zeer informele wijze waarop de Graduate School zijn oudere promovendi benadert staat in schril contrast met de wijze waarop die promovendi hun veel jongere opponenten in de promotiecommissie moeten aanspreken.
10. Wie luistert leert veel, wie denkt weet meer.

Deze stellingen worden opponeerbaar en verdedigbaar geacht en zijn als zodanig goedgekeurd door de promotoren prof. dr. ir. S.N. Jonkman en dr.ir. B. Hofland en de copromotor dr. ing. M.Z. Voorendt.

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Abbreviations

AP	*Amsterdams Peil*; datum in parts of the Netherlands from 1683 until 1818; from 1818 until 1875 it was the datum for the whole country
BWDB	Bangladesh Water Development Board
GHW	gemiddeld hoog water
GLW	gemiddeld laag water
HWS	high water spring
HWN	high water neap
KZGW, KZGdW	Koninklijk Zeeuws Genootschap der Wetenschappen
LB	left bank
LWS	low water spring
LWN	low water neap
MHT	mean high tide
MLT	mean low tide
MSL	mean sea level
NA	National Archives (*Nationaal Archief*)
NAP	*Normaal Amsterdams Peil*; datum in the Netherlands since the year 1875; depending on the location more or less equal to MSL
NLD	Ghana survey datum; at Keta, MSL = NLD +0.40 m (Ch.11, Sub-section 11.2)
NTD	Neap tide dam
OGL	original ground level
PWD	Public Works Datum in Bangladesh; at the Nalian closure site (Ch. 15) MSL = PWD + 0.46 m
RB	right bank
R.L.	Reference Level = PWD in Bangladesh, see CH. 14
RWS	Rijkswaterstaat
SOB	Survey of Bangladesh (in use as datum during the eighties of the 20th century)
TBD	*Technisch Bureau der Domeinen* (Technical Department for state-owned lands)
WSTD	Winter springtide dam
ZA	Archives of the province of Zeeland (*Zeeuws Archief)*

List of symbols

The following List summarizes the main symbols which have been applied in this dissertation

Roman symbols

b_t	width of closure gap at bed level	(m)
b_w	width of closure gap on waterline	(m)
D	typical diameter of block	(m)
D_n	nominal block diameter, or equivalent cube size, $D_n = (M/\rho_{app})^{1/3}$	(m)
D_{n50}	median block diameter, or equivalent cube size, $D_{n50} = (M_{50}/\rho_{app})^{1/3}$	(m)
D_s	diameter of equivalent sphere	(m)
Fs	shape Factor	(-)
g	gravitational acceleration	(m/s^2)
H	water level upstream of a dam or sill, relative to dam crest	(m)
h	water depth; water depth at structure toe	(m)
h_1	water depth upstream of closure gap or dam, relative to bed level	(m)
h_2	water depth in closure gap, relative to bed level	(m)
h_3	water depth downstream of closure gap or dam, relative to bed level	(m)
k	permeability coefficient according to Darcy	(m/s)
M	mass of a unit (for instance a rock)	(kg)
M_{50}	mass of particle for which 50% of the granular material is lighter	(kg)
Q	discharge	(m^3/s)
u, v, w	local velocities, usually defined in x, y ,z directions	(m/s)
u_b	near-bed velocity	(m/s)

Greek symbols

Δ	relative buoyant density of material, i.e. for rock ρ_r/ρ_w - 1	(-)
μ	discharge coefficient	(-)
ρ_r	mass density of rock	(kg/m^3)
ρ_w	mass density of water	(kg/m^3)
Ψ	non-dimensional shear stress parameter or Shields number	(-)

Summary

Over the last centuries people have developed several ways to close dike breaches and to dam tidal channels. These closure methods were needed to repair damages after floods and to reclaim land. This dissertation aims to provide a systematic overview and analysis of closure methods that have been developed over the last centuries.

At the start of the studies a number of research questions was formulated which focussed on the kind of closure methods used, in the past and at present, to repair breached dikes and to close tidal channels by simple means. By simple means one must think about the application by (mainly) manual labour of local materials and small-scale tools and equipment.

This research was in the first place aimed at Dutch closure methods as practised in the Netherlands, Germany (Schleswig Holstein) and Ghana/Bangladesh (20th century). But it was also considered useful to check the closure methods used in other countries than the ones just mentioned.

Typical topics addressed have been hydraulic features, materials and equipment applied, the importance of manual labour in these closures and their output.

Related topics addressed in this dissertation are the development in time of closure methods and the reasons why sometimes closures failed.

The research has been performed on the basis of research in archives, and is also based on the personal experience of the doctoral candidate with closures in Bangladesh, Ghana and Netherlands.

A closure effected by simple means took place for the last time in the Netherlands in 1954 and the study of relevant literature was expected to indicate details about closures during past centuries. But the search of works aimed at dike building for reclamation and breach repair (both requiring such closures) constructed during past centuries, in most cases, did not provide the wealth of information as hoped for. Though flood embankments and sea defences as part of reclamation works have been constructed from the 11th/12th century onward, limited technical information about their construction as well as possible subsequent repairs after breaching, could only be traced for six projects carried out during a period of more than 400 years (1263 – 1675).

More information could be traced about the methods and constructions used in general but not for specific projects. It is only during the 18th and 19th century that a few experts in hydraulic engineering started to write about closure methods being used.

The information consulted in the National Archives and various regional archives in the Netherlands about reclamation projects, breach repairs and closure of tidal channels for damming purposes, in the 19th and 20th century, provided a good insight in the magnitude and method of tidal closures which had to be performed as part of these projects.

For data about closures in Germany (Schleswig Holstein) and elsewhere (i.e. China, East-Pakistan, England and Japan) technical literature (books and articles) were studied, published in those countries as well as in the Netherlands and the Unites States.

Finally, the 20th century tidal closures performed under Dutch guidance in Ghana and Bangladesh could be analysed thanks to the published articles and reports written by the Author and/or his former colleagues.

A total of 75 closure projects have been identified and documented in this dissertation.

The closure methods used during the period 1200 – 1700, nearly always started by dumping a horizontal sill of clay clods up to the level of Low tide. After this initial phase, the gap was narrowed by dumping clay clods, and the actual closure was, subsequently, effected by lowering constructions (called 'sows') made up of sink-fascines (fascines imprisoning a gravel core or clay clods) and clay clods. Also, such sink-fascines could be rolled into the gap by manual labour or placed in a vertical position when the gap was too deep and current velocities too high to construct a sill. Alternatively, a cofferdam could be constructed across the flow gap. The cofferdam was then filled by manual labour

dumping clay clods or stones inside the cofferdam. In case of low tidal ranges, a row of sheet piles or piles, closely placed to each other, was driven across the gap. Sometimes, ships were ballasted and sunk on top of the sill.
At the end of the 17th century floating fascine mattresses were introduced in closure works. They were ballasted and immersed in the breach or tidal channel and functioned as bed protection of the easily erodible soil. Moreover, the aforementioned sill could now be constructed by stacking fascine mattresses on top of each other, again up to the level of Low tide. Subsequently, the closure was performed gradually in a horizontal direction by constructing a bund in sections, comprising fascines, clay clods, stones and wattle work (such a bund section was called a pakwerk).
From the end of the 19th century up to the middle of the 20th century jetties were constructed across the gap to be closed. A narrow gauge railway track on the jetty enabled the dumping of clay clods or stones in the gap from side tip wagons. Until the Second World war the side tip wagons were loaded by hand and in most cases pushed by manual labour.
The aforementioned closure methods were applied in the Netherlands for closing tidal creeks in reclamation schemes, for the closure of flow gaps in dike breaches and for damming tidal channels when dams had to be built to connect islands with each other or to the mainland.
The closure by simple means of tidal creeks for reclamation turned out to be relatively easy. The same was more or less true for damming tidal channels as these dams were built on or near the tidal divide. But closure of dike breaches could turn out to be problematic when tidal prisms of flooded areas were large and related current velocities in the closure gaps high. During past centuries in many cases simple means were not adequate to close such breaches and, consequently, the flooded polders could not be reclaimed and had to be given back to the sea.
Special mention must be made of the low overflow dams, built to further accretion at both sides of these dams, for future reclamation. In all four cases recorded, these dams failed repeatedly and, finally, were abandoned or a dam having a non-overflow crest, was constructed instead.
In the period 1969 – 1985 Dutch closure techniques based on the principles described above were introduced for tidal closures in Ghana and Bangladesh. This required the adaptation of the said closure techniques to locally available construction materials, equipment and skills. Also here, a significant input of manual labour was part of the closure operation.
Closure techniques applied in China, East-Pakistan (which, later became independent as Bangladesh), England and Japan were also studied. It was surprizing to learn that the sink-fascine (having in each country another name, other dimensions and composition) had a major role in closure projects in China and East-Pakistan and, to a lesser extent, in Japan. Moreover, the application of ropes and hawsers (as in the Dutch sink-fascine, the 'sow' and the floating fascine mattress) turned out to be of vital importance. In China and Japan the closures were required to repair dike breaches along non-tidal rivers. In East-Pakistan and England the closures studied were in the tidal zone.
Finally, a tidal closure performed in Bangladesh (at the Nalian River) by manual labour and simple means in the year 2020, has been studied. Materials, comprising synthetic fibres, resulting in the use of polypropylene in geotextile membranes, in nylon ropes and in sand-filled geo-bags and synthetic bags, have meanwhile replaced vegetal materials like brushwood (*rijshout*), jute, palm leaves, millet stalks and reed. The application of these 'new' materials in a tidal closure was a success. An advantage of using sand-filled geo-bags and synthetic bags as bed protection is no doubt that less skilled labour is required than in the Dutch closure techniques introduced in the same country in the period 1978 – 1985: dumping sandbags or clay bags is easier than construction and immersion of a floating mattress constructed of bamboo, palm leaves and reed.

The case histories presented in this dissertation reflect the art of river and tidal closures by simple means over a period of nearly 760 years (1263 – 2020) in six countries. In most cases the area of the tidal basin which could be closed by simple means was less than 2 km^2 while the tidal volumes were

less than 2 million m^3. Production rates in tidal closures varied between 0.26 and 0.62 m^3 of fill placed per manhour.
But most of the simple means described can be considered as history. There are, however, countries where cheap manual labour is abundantly available and tidal closures could still be constructed by simple means. Such closures by simple means have been described for the year 1969 (Ghana) and the period 1979 – 2020 (Bangladesh).
If one compares the closures of 1979 – 1985 with the Nalian closure of 2020, it is noted that three aspects have stayed: (a) the construction of a horizontal sill prior to any narrowing of the tidal river/creek, (b) the protection (screening) of the dumping area against removal of closure materials by the current, and (c) carrying and dumping by manual labour of fill material contained in bags.

The insights from this dissertation will therefore also be useful for future closures constructed by using simple means.

Samenvatting

In de afgelopen eeuwen heeft men verschillende manieren ontwikkeld om dijkdoorbraken te dichten en getijdengeulen af te dammen. Deze afsluitmethoden waren nodig om schade na overstromingen te herstellen en land in te polderen. Dit proefschrift beoogt een systematisch overzicht en analyse te geven van afsluitmethoden die in de afgelopen eeuwen zijn ontwikkeld.
Bij het begin van het onderzoek werd een aantal onderzoeksvragen geformuleerd die zich richtten op het soort afsluitmethoden die in het verleden en nu worden gebruikt om doorgebroken dijken te repareren en getijgeulen af te sluiten met eenvoudige middelen. Bij eenvoudige middelen moet men denken aan de toepassing door (voornamelijk) handarbeiders van lokale materialen, handgereedschap en kleinschalige werktuigen.
Dit onderzoek was in de eerste plaats gericht op Nederlandse afsluitingsmethoden zoals toegepast in Nederland, Duitsland (Sleeswijk-Holstein) en Ghana/Bangladesh (20e eeuw). Maar het werd ook nuttig geacht om de afsluitingsmethoden na te gaan die in andere dan de zojuist genoemde landen werden toegepast.
Typische onderwerpen die aan bod kwamen waren waterloopkundige aspecten, toegepaste materialen en werktuigen, het belang van handarbeiders bij deze sluitingen en hun output.
Verwante onderwerpen in dit proefschrift zijn de ontwikkeling in de tijd van afsluitingsmethoden en onderzoek naar de redenen waarom sluitingen soms mislukten.
Het onderzoek is uitgevoerd op basis van onderzoek in archieven, en is ook gebaseerd op de persoonlijke ervaring van de promovendus met afsluitingen in Bangladesh, Ghana en Nederland.

Een sluiting met eenvoudige middelen vond voor het laatst plaats in Nederland in 1954 en verwacht werd dat de studie van relevante literatuur details zou geven over sluitingen in de afgelopen eeuwen. Maar het zoeken naar werken gericht op dijkaanleg voor inpoldering en herstel van dijkdoorbraken (die beide dergelijke afsluitingen vereisen) die in de afgelopen eeuwen zijn uitgevoerd, leverde in de meeste gevallen niet de gehoopte schat aan informatie op. Hoewel rivierdijken en zeeweringen als onderdeel van inpolderingswerken vanaf de 11e/12e eeuw zijn aangelegd, kon voor slechts zes projecten die in een periode van meer dan 400 jaar (1263 - 1675) werden uitgevoerd, beperkte technische informatie over de bouw ervan en mogelijke latere reparaties na doorbraken worden achterhaald.
Er kon meer informatie worden opgespoord over de methoden en constructies die in het algemeen (dus niet voor specifieke projecten) werden gebruikt. Het is pas in de 18e en 19e eeuw dat enkele deskundigen op dit gebied van de waterbouwkunde de gebruikte afsluitmethoden op schrift gingen stellen.
De geraadpleegde informatie in het Nationaal Archief en verschillende regionale archieven in Nederland over inpolderingsprojecten, doorbraakherstel en het sluiten van getijdengeulen, voor afdammingsdoeleinden in de 19de en 20ste eeuw, gaf een goed inzicht in de omvang en de methode van getijdenafsluitingen die als onderdeel van deze projecten moesten worden uitgevoerd.
Voor gegevens over afsluitingen in Duitsland (Sleeswijk-Holstein) en elders (d.w.z. China, Oost-Pakistan, Engeland en Japan) werd de technische literatuur (boeken en artikelen) bestudeerd die in die landen en in Nederland en de Verenigde Staten was gepubliceerd.
Ten slotte konden de 20e-eeuwse getijdensluitingen die onder Nederlandse leiding in Ghana en Bangladesh zijn uitgevoerd, worden geanalyseerd dankzij de gepubliceerde artikelen en rapporten van de auteur en/of zijn voormalige collega's.
In totaal zijn 75 afsluitingsprojecten geïdentificeerd en gedocumenteerd in deze dissertatie.

De afsluitmethoden die in de periode 1200 - 1700 werden gebruikt, begonnen bijna altijd met het storten van een horizontale drempel van kleikluiten tot het niveau van laagwater. Na deze eerste fase werd de doorstroomopening verkleind door het storten van kleikluiten en de feitelijke afsluiting werd vervolgens uitgevoerd door het laten zinken van constructies ('zeugen' genoemd) bestaande uit zinkfascines (bossen rijshout die een grindkern of kleikluiten insluiten) en kleikluiten. Dergelijke zinkfascines konden ook met de hand in het sluitgat worden gerold of in verticale positie worden geplaatst als het gat te diep was en de stroomsnelheden te hoog om een drempel te bouwen. Als alternatief kon er een kofferdam door het sluitgat worden gebouwd, die vervolgens werd gevuld door met de hand kleikluiten of stenen in de kofferdam te dumpen. Bij getijden met kleine amplitude werd een rij damplanken of palen, dicht bij elkaar geplaatst, door het gat geheid. Soms werden schepen geballast en bovenop de drempel afgezonken.
Aan het einde van de 17e eeuw werden zinkstukken geïntroduceerd in afsluitingswerken. Ze werden geballast en gezonken in het stroomgat of de getijgeul en functioneerden als bodembescherming van de gemakkelijk erodeerbare ondergrond. Bovendien kon de bovengenoemde drempel nu worden opgebouwd door zinkstukken op elkaar te zinken, opnieuw tot het niveau van laagwater. Vervolgens werd de afsluiting geleidelijk in horizontale richting uitgebouwd door de aanleg van een afsluitkade in secties, bestaande uit bossen rijshout, kleikluiten, stenen en betuining (zo'n sectie in een afsluitkade werd pakwerk genoemd).
Vanaf het einde van de 19e eeuw tot het midden van de 20e eeuw werden er steigers gebouwd over het te dichten sluitgat. Een smalspoor op de steiger maakte het mogelijk om kleikluiten of stenen in het sluitgat te storten met behulp van kipkarren.
De bovengenoemde afsluitmethoden werden in Nederland toegepast voor het afsluiten van getijdenkreken bij inpolderingen, voor het dichten van stroomgaten bij dijkdoorbraken en voor het afdammen van getijdengeulen als er dammen moesten worden gebouwd om eilanden met elkaar of met het vasteland te verbinden.
Het met eenvoudige middelen afsluiten van getijdenkreken voor inpoldering bleek relatief eenvoudig te zijn. Hetzelfde gold min of meer voor het afdammen van getijdengeulen omdat deze dammen op of nabij het wantij werden gebouwd. Maar het dichten van dijkdoorbraken kon problematisch blijken te zijn als de komberging van overstroomde gebieden groot was en de daarmee samenhangende stroomsnelheden in de sluitgaten hoog. In de afgelopen eeuwen waren eenvoudige middelen in veel gevallen niet toereikend om dergelijke doorbraken te dichten, met als gevolg dat de overstroomde polders niet konden worden drooggelegd en moesten worden teruggegeven aan de zee.
Speciale vermelding verdienen de lage overloopdammen, die werden gebouwd om de aanslibbing aan weerszijden van die dammen voor toekomstige inpolderingen te bevorderen. In alle vier de gevallen faalden deze dammen herhaaldelijk en werden uiteindelijk opgegeven of er werd een niet-overstroombare dam gebouwd.
In de periode 1969 - 1985 werden Nederlandse afsluitingstechnieken, gebaseerd op de hierboven beschreven principes, geïntroduceerd voor getijsluitingen in Ghana en Bangladesh. Dit vereiste aanpassing van de genoemde sluitingstechnieken aan lokaal beschikbare bouwmaterialen, materieel en vaardigheden. Ook hier maakte een aanzienlijke inzet van handarbeid deel uit van de sluitings-operatie.
Zoals eerder vermeld, werden ook afsluittechnieken bestudeerd die werden toegepast in China, Oost-Pakistan (dat later onafhankelijk werd als Bangladesh), Engeland en Japan. Het was verrassend om te leren dat de zinkfascine (die in elk land een andere naam, andere afmetingen en samenstelling heeft) een grote rol speelde bij afsluitingsprojecten in China en Oost-Pakistan en, in mindere mate, in Japan. Bovendien bleek de toepassing van touw en trossen (zoals in de Nederlandse zinkfascine, de 'zeug' en het traditionele zinkstuk) van vitaal belang. In China en Japan waren de afsluitingen nodig om

dijkdoorbraken langs bovenrivieren te herstellen. In Oost-Pakistan en Engeland lagen de onderzochte afsluitingen in het getijdengebied.
Tot slot is een getijdensluiting bestudeerd die in het jaar 2020 in Bangladesh (bij de rivier de Nalian) werd uitgevoerd door handarbeiders met eenvoudige middelen. Materialen bestaande uit synthetische vezels, resulterend in het gebruik van polypropyleen in geotextiel-membranen, in nylon touw en in met zand gevulde geo-bags en synthetische zakken, hebben inmiddels de plaats ingenomen van plantaardige materialen zoals rijshout, jute, palmbladeren, gierststengels en riet. De toepassing van deze 'nieuwe' materialen in een getijdensluiting was een succes. Een voordeel bij het gebruik van met zand gevulde geo-bags en synthetische zakken als bodemverdediging is ongetwijfeld dat er minder geschoolde arbeid nodig is dan bij de Nederlandse afsluitingstechnieken die in de periode 1978 - 1985 in hetzelfde land werden geïntroduceerd: het dumpen van zandzakken of kleizakken is eenvoudiger dan het samenstellen en afzinken van een zinkstuk bestaande uit bamboe, palmbladeren en riet.

De casestudies in deze dissertatie geven de kunst weer van afsluitingen bij rivieren en in de getijzone met eenvoudige middelen over een periode van bijna 760 jaar (1263 - 2020) in zes landen. In de meeste gevallen bedroeg het oppervlak van het gebied onder getijinvloed, dat moest worden afgesloten, niet meer dan 2 km^2 terwijl het getijvolume niet groter was dan 2 miljoen m^3. Productie cijfers bij getijafsluitingen varieerden van 0,26 tot 0,62 m^3 grondverzet per manuur.
Maar de meeste van de beschreven eenvoudige middelen kunnen worden beschouwd als geschiedenis. Er zijn echter landen waar goedkope handarbeid in overvloed beschikbaar is en waar getijafsluitingen nog steeds met eenvoudige middelen kunnen worden gerealiseerd. Dergelijke eenvoudige afsluitingen zijn beschreven voor de jaren 1969 (Ghana) en de periode 1979 - 2020 (Bangladesh).
Als men de afsluitingen van 1979 - 1985 vergelijkt met de Nalian afsluiting van 2020, dan valt op dat drie aspecten zijn gebleven: (a) de aanleg van een horizontale drempel voorafgaand aan een eventuele vernauwing van de getijdenrivier/kreek, (b) de bescherming (afscherming) van het stortgebied tegen het wegstromen van afsluitmateriaal, en (c) het dragen en storten door mankracht van afsluitmateriaal in zakken.

De inzichten uit dit proefschrift zullen daarom ook bruikbaar zijn voor toekomstige afsluitingen die met eenvoudige middelen tot stand worden gebracht.

CHAPTER 1 Introduction

1.1 Background

Over the last centuries people have developed several ways to close dike breaches and to dam tidal channels. These closure methods were needed to repair damages after floods and to reclaim land. This dissertation aims to provide a systematic overview and analysis of closure methods that have been developed over the last centuries.

The idea to write a dissertation came up in the autumn of 2018.

A decision on the subject to be studied and researched was not difficult. During the author's professional life he has, inter alia, been involved (planning, design, supervision during construction) in the closure of tidal channels in developing countries.

In 1976, in Bangladesh a number of tidal channels still had to be closed as part of a coastal embankment project and the Netherlands' Government was prepared to provide technical assistance as a local closure method was only applicable to small tidal creeks.

At that time a plea was made to stop the ruthless introduction of western standards and techniques in developing countries and to apply appropriate technology instead[1].

Following that plea about appropriate technology, the author proposed to the Ministry of Development Cooperation in the Netherlands to introduce in Bangladesh old Dutch techniques for closing tidal channels. The closure method would be adapted to local circumstances and much local labour would be involved as the said techniques are very labour intensive. Prior to this proposal in 1976 for activities in Bangladesh, the author had already successfully closed (1969) a tidal inlet at the coast of Ghana in this manner (Van Duivendijk, 1970). Ultimately, between 1979 and 1985 four tidal closures were constructed in Bangladesh. In this respect reference is made to a number of reports written in the period 1976 – 1982 (BWDB, 1976 – 1982) and to Van Duivendijk & Te Slaa (1987).

1.2 Aim of this dissertation

The author came to the conclusion that a suitable subject for his dissertation could be:

'Restoring dike breaches and closing tidal channels by simple means – from past to present'

Even when often used in that way in international discussions on technologies, the expression 'simple means' is not used in this dissertation in a normative sense. The term 'simple' is reserved for the type of closures studied that have the following common characteristics:

- They were carried out by (mainly) manual labour;
- local materials are used such as locally available soil and vegetal materials; these vegetal materials are for instance used in fascine constructions;
- small-scale equipment is used like barrows and wheelbarrows, while also aprons and head baskets are used to transport clay and sand. Moreover, horse-drawn carts are utilized. The application of motorized equipment for these closures is an exception.

As a result, the studied closures are very labour intensive and require much planning and coordination. For example, in many closures sandbags and bags filled with clay are carried by man power to the dumping site. As such, these closures are actually complex to arrange. Nevertheless, the term 'simple' is kept being used in this dissertation for pragmatic reasons.

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[1] Two definitions of appropriate technologies are quoted:

- 'Tools and machines that are suited to labor-intensive production and fit LDC small farms, small businesses and small incomes' (AID, 1976, 7). LDC stands for 'less developed countries'.
- 'Appropriate technology is the set of technologies which makes optimum use of available resources in a given environment' (Morawetz, September 1974).

In the two part reference numbering, as in (AID 1976, 7), the first number refers to the year of publication and the second to the page in that publication to which reference is made.

The aim of this dissertation can be described as follows:

- to recall and describe (from a technical perspective) the repair of dike breaches and the damming of tidal channels as done in the past and at present by using simple means;
- to make an inventory of the documented closure[2] methods in use during the past centuries up to the present time;
- to compare and, in so far as collected data permit, analyse these closure methods (development in time, differences between methods and countries);
- to determine more in particular the hydraulic engineering aspects, logistics, labour input, time frame and (if data permit) financial features of closure works.

1.3 The research questions

In order to achieve the aim of this dissertation the following **research questions** have been formulated:

1. Why did dikes breach so frequently during past centuries?
2. What kind of closure methods were used and what are the characteristics and differences between the various methods?
3. What materials, tools, equipment and temporary works were applied for these closures and what was the role of labourers?
4. Is there a certain relationship between the closure method(s) in use and the natural resources/social conditions in that particular region or country?
5. How successful were the closure methods used in the Netherlands and in other countries during past centuries? And if they failed, what was (were) the reason(s) for failure?
6. Has there been any technical development in the closure methods in the course of time?
7. Is it possible to compare physical output by labourers during closures in different centuries and different countries?
8. Is it possible to define, on the basis of closures studied, certain factors stimulating the success of manual closures, also at present?

1.4 Definition and delimitation of closures to be studied

(a) Only those closures will be studied which were carried out by the application of (mainly) simple means as defined in Section 1.2.

(b) The focus in this study will be on closures which took place in the past in the Netherlands and, to a lesser extent, in some foreign countries: East-Pakistan/Bangladesh, China, England, Germany (Schleswig Holstein), Ghana and Japan.

(c) Closures must have been carried out. But they can have failed.

(d) Closures must have taken place in tidal or river flow. In fact the closure of flow gaps (*stroomgaten)* is discussed. However, the definition of a flow gap as defined in Rijkswaterstaat (1961, 159) is not completely followed. In this dissertation the bed level of the breach or tidal channel must have an elevation low enough to allow for a continuous flow. In other words the breach or tidal channel does not run dry during the tidal cycle.

(e) During this study the emphasis will be on the technical aspects of closures.
Such technical aspects comprise hydraulic engineering, logistics, labour input and construction planning and, more specifically, their mutual relationship.
Other aspects like economics, ecology and local culture will only be mentioned if relevant within the context of a particular case study.

[2] From now on the word closure is used in most cases for the repair of dike breaches as well as for the damming of tidal channels.

(f) Tidal closures which, for the greater part, have been built by utilizing large mechanical equipment (like suction dredgers and floating cranes) and sluice caissons, will not be discussed. Such closures took place in the Netherlands in order:

- to build the Zuiderzee Enclosure dam (*Afsluitdijk)*,
- to dam the tidal river Brielse Maas and tidal inlet Braakman,
- to repair the dikes breached on the island of Walcheren during the Second World war,
- to repair most of the dike breaches caused by the Storm surge flood of 1953,
- to build the estuary dams required as part of the Delta Project.

1.5 Research approach

1.5.1 General

The research approach has been first of all a study of literature on the topic.
Following information found in the literature, relevant documents were traced in a number of archives in the Netherlands. The main sources in this respect have been the National Archives in The Hague and the archives of the provinces of Friesland (Tresoar) and Zeeland.
Last but not least the special reading rooms must be mentioned of the libraries of Delft University (Trésor) and Radboud University, Nijmegen.

1.5.2 Literature studied

As breaches in dikes in most cases are caused by river floods or storm surges, it was decided to scrutinize Gottschalk's '*Stormvloeden en rivieroverstromingen in Nederland* ' (1971-1977) on references to documents in which case histories on river floods and storm surge floods are presented. Gottschalk gives numerous references up to the year 1700 and, moreover, states always whether the document is thought to be reliable or not.
Gottschalk only refers to printed documents. It was assumed that in the latter also unpublished data would be referenced.
As Gottschalk only describes the storm surge floods and river floodings up to the year 1700, a large number of texts books and articles published during the last three centuries have been checked as well on possible case histories[3], relevant to the subject of this dissertation.
In all, more than 200 books and some 100 articles have been reviewed on the subject to be elaborated upon.
This included the following documentation[4]:

- Books on hydraulic engineering topics including repair of dike breaches and closure of tidal channels.
- Books about water management, more in particular about the creation of polders.
- Articles about repairs of dike breaches and closure of tidal channels in (hydraulic) engineering journals and in magazines on historical geography.
- Dissertations related to these topics of dike breaches and tidal closures (in most cases these dissertations had been written by historical geographers).
- Reports about the topics within the scope of this dissertation.
- Monographs, on specific topics and items relevant to the dissertation[5].

[3] For instance Huitema (1947, 117 -123) discusses at length the problematic construction of the Hindenburgdamm to the island Sylt in 1926-1927.
[4] Though the lockdown periods following the outbreak of the pandemic caused by the virus Covid-19, has hampered the number of visits to libraries considerably, the digitization of many old and rare books by Google and others has turned out to be a blessing in disguise.
[5] For instance the monograph on the wheelbarrow by Stutterheim (2017)

Moreover, it was checked whether well-known diking experts from the past have left a 'testament'. Mentioned in this regard are Andries Vierlingh, who wrote his 'Tractaet van Dyckagie' around 1570[6], Jan Matelé (1731) and Adriaan Bommenee (1750). But most experts in the field of closure works only started to put their technical designs, considerations and experience in writing from the 19th century onward.
Though most books/articles discussed projects on the aforementioned topics in the Netherlands, also books/articles on similar activities in other European countries have been consulted. In this respect it is noted that activities by Dutchmen on drainage took place during the past centuries in countries like Germany, Poland, Russia, Italy, France, Great Britain etc. (vide the overview by Van Veen, 1962, 47 et seq.). But, apparently, these drainage activities did not require the closure of flow gaps in breached dikes or the closure of tidal channels as part of reclamation schemes (polders). In any case these specific subjects are not mentioned anywhere.
Also in Great Britain flow gaps in tidal areas have been closed but this was not done by the Dutch.
The author's own publications on indigenous closure methods in use in East-Pakistan (now Bangladesh) have been reviewed as well. The particulars of this method have been studied in 1976 – 1978 (BWDB, first, second and third interim report). Also reports on closures constructed in Ghana and Bangladesh during the period 1969 – 1985, as mentioned already in Section 1 .1 of this Chapter, have been studied.
Publications by various engineers, having worked in overseas countries, enabled the author to scrutinize closures made in countries like China, England, Germany (Schleswig Holstein) and Japan.
Finally, thanks to the assistance by the World Bank and Bangladesh Water Development Board, a recent closure in Bangladesh (Nalian closure, 2020), carried out by using simple means, could be analysed.

1.5.3 Documentation traced in the Archives

Originally, it was intended to look at inventories of archives of waterboards in order to find case histories on repair of dike breaches and/or construction of reclamation schemes. But this idea was abandoned after discussions with historical geographers who declared that such case histories are completely missing in the archives studied by them.
Consequently, the possible presence of non – transcribed manuscripts on closure works in old archives of water boards and the like has in general been ignored. It would cost many man-years to scrutinize all these archives. In most cases only a list of contents has been made by others of these archives. Already a study of all such, possibly relevant, lists of content would be a major operation. For instance, the list of contents of the archives of the authorities governing the island of Walcheren (later called the Polder Walcheren) during the period 1511 – 1870 (De Waard, 1914) comprises 932 pages! And that is only one of the hundreds of waterboards which have existed in the past.
Nevertheless, on the basis of a number of comprehensive publications[7] about the history of specific waterboards, islands or regions, references could be traced to documentation about the repair of dike breaches or the closure of tidal channels (e.g. as part of reclamation schemes) in the archives mentioned in Sub-section 1.5.1.

[6] Andries Vierlingh's manuscript was written around 1570, it has been transcribed and annotated by De Hullu & Verhoeven, who published the transcript with annotations in 1920.
[7] In this respect are mentioned in alphabetical order: Baars (1973 & 1973a), Dekker (1971), De Kraker (1997), De Vleesschauwer (2013), De Vries (1876), Dibbits (1950), Gottschalk (1983), Priester (1915), Rienks & Walther (1954), Schilstra (1987), Sjoerds (1767), Van Dam (1992) & (1998), Van de Ven (ed.) (2004), Van Veen (1962).

1.6 Data traced and knowledge gaps

1.6.1 Overview of the information found

As outlined above, the study of literature and research in the archives has been quite comprehensive. But the length of the period during which the closures may have taken place (roughly eight centuries) as well as the impossibility to look at all archives filed at regional archives and waterboards, render it impossible to guarantee that all relevant closure projects have been traced. Below an effort has been made to summarize the available information in global terms.
Coming back to the area which is now called the Netherlands[8]: From the literature studied it became clear that most dikes along rivers, around islands in estuaries and on the coast have been built during the 11th and 12th century. It means that the dike breaches of interest to the Author will have taken place between, roughly, 1200 (when dike building was more or less complete[9]) and 1953[10].
That period can, on the basis of available information, be divided into three sub-periods:
1200 – ± 1570
± 1570 – 1833
1833 – 1953
The years ± 1570 and 1833 require some explanation.
Andries Vierlingh wrote his *Tractaet van Dyckagie* (published in 1920) approximately in 1570 while the first comprehensive book on coastal engineering in the Netherlands was published by Abraham Caland in 1833.
Now, these three sub-periods will be looked into.

<u>Sub-period 1200 – 1570</u>

During this sub-period numerous dike breaches occur. Gottschalk (1971 – 1977) determines the events 'storm surge' and 'flooding by rivers' mainly on the basis of reports etc. regarding damage to and/or breaches of dikes. It therefore stands to reason to study her masterpiece[11], searching for dike breaches and related information. The storm surges and river floods are discussed by Gottschalk in chronological order. A major merit of Gottschalk's work is her critical approach. She reads everything there is about the subject and when she considers, by comparison or for other reasons, the author of a report or a story to be an unreliable source , she will say so. On the basis of her (reliable) references, a list of books, reports, charters, accounts, dissertations, etc. was made to be looked at. This search has not been very successful. A few reports, books and dissertations contain information about the subject of this dissertation but the voluminous printed collections of charters and accounts are only about conditions and rulings, decisions, financing and (sometimes) quantities and cost concerning the repair of dike breaches.
In Part II of this dissertation, because of lack of suitable documentation, only two closures as part of land reclamation schemes during this sub-period will be discussed and one repair of a breached dike.

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[8] This expression is used because in former centuries the area which is now called Zeeuws-Vlaanderen was part of Flanders (Belgium).

[9] Obviously, the word 'complete' refers to the dikes which were needed at that time to protect agricultural lowlands, towns and people against flooding during high river stages or during storm surges. After the 12th century dike building continued as part of new reclamation schemes and necessary re-alignment of dikes because of meandering river channels or shifting tidal channels.

[10] Many dike breaches due to major storm surge flood, please refer to:
https://en.wikipedia.org/wiki/North_Sea_flood_of_1953 (accessed on 21 March 2024).

[11] Three volumes, a total of 1 950 pages, which cover the period 516 – 1700.

Still, a few conclusions could be drawn:

- the repair of dike breaches along non-tidal rivers is in general of no interest for the dissertation as the repair is nearly always carried out by means of a perimeter bund around the scour hole (*wiel*) after retreat of the flood waters[12];
- the frequent breaching of dikes during this period is not so much caused by the magnitude of the storm surge or river flood but by, inter alia, bad management, bad quality of the work and lack of maintenance; this aspect will be elaborated upon in Chapter 2, Section 2.7;
- so far, the information on dike breaches in the North of the Netherlands (Groningen and Friesland) is scarce; if there were indeed less dike breaches than elsewhere it might be due to smaller tidal amplitudes, moderate wave action behind the Frisian Islands (*waddeneilanden*) in the shallow Waddenzee and/or, on average that is, a higher ground level than in the south-western part of the Netherlands. In this respect the article of Zhu et al (2020) is of interest. In this paper the authors report about their research and modelling regarding the impact of salt marshes (*schorren, kwelders*), in front of sea defences, on wave run-up and size of dike breaches. The presence of salt marshes, and more specifically their width, attenuates the waves (and consequently the wave run-up) and limit the size (depth and length) of dike breaches. This situation exists on part of the Waddenzee coast and could be the reason why there were less dike breaches and less flooding in the past than, for instance, in the province of Zeeland.

Apart from the repair of breached dikes, tidal channels had to be closed during this period as part of reclamation schemes.

Sub-period 1570 - 1833

The two outstanding documents of this sub-period about coastal engineering have been written by Andries Vierlingh (*Tractaet van Dyckagie*, in manuscript, ± 1570) and Abraham Caland (*Handleiding… Dijksbouw en Zeeweringskunde*, printed book, 1833) respectively.

During the period of 260 years between these two dates many people had the responsibility to repair dike breaches and to construct dikes, surrounding new polders. That could be dike reeves (*dijkgraven*), sinking masters (*zinkbazen*), self-made engineers or land surveyors.

Some of them have put down their technical considerations and experience in writing. In this respect are mentioned in chronological order: John Perry (1721),Willem Loré (± 1730)[13], Jan Matelé (1731), Pieter van Bleiswyk (1745), Adriaan Bommenee (± 1750)[14], Bastiaen Nebbens (1771), Cornelis Velsen (1768) and Andries Schraver (1807).

This list is by no means complete and it is observed that the 17th century is completely missing!

Nevertheless, by combining the information given by Vierlingh and Caland with those given by the aforementioned gentlemen one gets a good idea how dike breaches were repaired and tidal channels closed. It has for instance been possible to comprehend the '*liggende en staande waesen'*, mentioned by Vierlingh (1920, 439).

Regrettably, the origin of the well-known floating fascine mattress could not be established but a clear reference to its existence in 1675 (and suggested as something new) was found in Hunnius (± 1720, page nn).

Sub-period 1833 – 1953

During this sub-period many books on hydraulic engineering matters have been published. However, the number of case histories on tidal closures and repair of breached dikes is limited.

[12] Exceptions are the closure of the Lekdijk Bovendams in 1747 and the closures of breaches in river dikes in China and Japan.

[13] Ref. De Crane (1835) and Eekhoff (1835)

[14] Ref. Gerhardt (Ed) (1988).

The repair of breached dikes after the 1953 storm surge flood has been well documented in articles published in technical magazines and in the *Verslag over de stormvloed 1953* (Rijkswaterstaat, 1961). One would expect similar documentation about earlier storm surges. But in the eight *Verslagen*[15] (reports, which are all digitally accessible), about earlier storm surges, nothing is reported about the closure of the various flow gaps as part of dike repair.
Various articles have, however, been found about the closure of tidal channels for other reasons, such as reclamation or the construction of dams between islands or from an island to the mainland, in the 19th as well as in the 20th century.
Obviously, only those tidal closures are of interest in which simple means, play a significant role.

1.6.2 Knowledge gaps

Prior to Vierlingh's *Tractaet van Dijckagie,* written ± 1570, no technical information has been laid down regarding closures. And after Vierlingh another 160 years passed before Matelé put down some thoughts about closures in writing.
More in general one can state that closures (made prior to the 19th century) are described in global terms but details are missing:

- descriptions of closures have not been made from an engineering perspective but from a financial, managerial or administrative point of view;
- it is not known how successful certain closure methods were in specific circumstances;
- the stability of certain closure materials in high currents has nowhere been quantified.

1.7 Structure of the dissertation

The dissertation has been divided in four Parts.

In **Part I** the chapters 2 to 5 (incl.) provide the information required to understand the contents of the various case histories and the analysis thereof.
First of all the environment in which closures take place is described (Chapter 2). These are in most cases the dikes which have a certain build-up and may breach.
Subsequently, the simple means used to repair these breaches or to close tidal channels are determined. In this respect one can distinguish labour, construction materials, tools and equipment, temporary works (Chapter 3).
Because of the important role of fascines and fascine constructions in closures a special chapter is dedicated to these items (Chapter 4).
Finally, the various closure methods, applied within the context of simple means are discussed (Chapter 5).

Case histories of closures are presented in **Parts II** and **III** of this dissertation. When certain vital information was missing, the author has tried to make reasonable assumptions in order to 'reconstruct' the closure as it may have taken place.

In **Part II** closures are discussed which took place during the period 1263 – 1985 in the Netherlands but also in other countries (Germany, Ghana, Bangladesh), provided, the input from Dutch side has been high (Chapters 6 – 11) .

In **Part III** closures are discussed as constructed outside the Netherlands without any (or small) input from Dutch side: China, East-Pakistan/Bangladesh, England, Germany, Japan (Chapters 12 – 18) .

[15] It concerns *Verslagen* (reports) about the storm surges of 1877, 1881,1883, 1889, 1894, 1906, 1911 and 1916.

In **Part IV**, the outcome of the analyses carried out for the various case histories of closures in Parts II and III will be compared with each other. Also some simple rules will be drawn up regarding closures by simple means (Chapter 19).
Finally, a summary of the answers to the Research Questions, formulated in Section 1.3 of this chapter, is presented in Chapter 20.

PART I Breaching of dikes and dams and methods and means used for closures

CHAPTER 2 Dikes and dams constructed in the period 1200 – 1900

2.1 Introduction

This dissertation is not about dikes as such but for a good understanding of the topic to be discussed (i.e. closure of dike breaches and the damming of tidal channels) one should have a certain knowledge about the dikes built during past centuries in the area which is now called the Netherlands.
In the sub-sections which follow the various aspects which play a role in dike design in (mainly) the Netherlands during the period 1200 – 1900 will be discussed.
From archaeological investigations it is known that dikes have already been built during Roman times, say around the year zero. But it is questionable whether these dikes were built to protect the land against flooding (Borger 2018, 7-8). Probably, they were built in order to provide a flood-free link between two points. In that period also wooden culverts with wooden flap valves, placed in small dams, were constructed (De Ridder, 1999, 10-17). But the network of dikes, as a means to prevent flooding caused by storm surges and by river floods, only started around the year 1000 and was complete between 1200 and 1300[1].
Consequently, the year 1200 is an arbitrary starting point for the period to be studied.
The year 1900 was chosen by the Author as the end of the 'primitive' dike and dam building period as, at that time, more and more mechanical equipment, like the suction dredger, was introduced to build dikes and dams (Verhoeven, 1924, 191) while the application of stony materials in slope protection and on the crest of the dike (*muraltmuurtjes*) gave dikes and dams much more resistance against wave and current attack. Still, also the year 1900 is an arbitrary date.
The repair of dike breaches by applying simple means as well as the closure of tidal channels in a similar manner continues in the Netherlands up to the year 1954 and in Bangladesh until the year 2020.

2.2 Definitions

When, in the following, the word dike is used, an artificial earth structure is meant having a more or less trapezoidal shape.
A dike is a water retaining structure, built up of soil, which has an elevation and strength able to retain the water under extreme circumstances.
When a dike borders the sea or an estuary, also the word sea defence (*zeewering*) is correct while along rivers, one speaks about flood embankments and also about river dikes.
A low dike (up to 1 – 2 m high) is called a bund (*kade*) but one speaks also about a summer dike (*zomerkade*).
Not used here is another technical meaning of dike: large drain.
In the USA one speaks about a levee (after the French word: levée).
The word dam is used instead of dike when the earth structure, after construction, will have water bodies at both sides. In this regard the name *Afsluitdijk*, for the dam separating the Zuiderzee from the Waddenzee, is actually wrong.
The shore (*voorland*) is defined as the area in front of the dike which runs dry during low tide and is called mud flat (*slik*). But it is also the area above high tide in front of the dike and then it is called marsh land or salt marsh (*schor, kwelder*).
The bank (*oeverlijn*) in tidal waters is the line of the low water level at the edge of the shore or, in rivers, the edge of the floodplain (*uiterwaard*).
The area in front of the bank in tidal waters is called the foreshore (*vooroever*).

[1] As explained in Chapter 1, Sub-section 1.6.1, 'complete' in this context means complete given the existing environment at that time of agricultural lands and towns. New reclamation schemes and re-alignment of dikes continuously necessitated the construction of new dikes after the said completion.

In rivers one has groynes (*kribben*), also called jetties[2]. Their function is to enable one deep channel to exist for navigation outside the flood season. Groynes are low and armoured over their full length because during floods they will be overtopped (like a weir). They are built from the bank into the river.
Spurs[3] (*hoofden*) look sometimes like groynes but have another function: to keep currents and the river- or tidal channel at a safe distance from the dike and/or to reduce wave attack. They are usually higher than groynes and start at the dike rather than the bank.
According to CIRIA, CUR & CETMEF (2007, 985) a hard point is a miniature spur, a localized revetment protruding into the river, tidal inlet or estuary. According to this definition the old Dutch construction called *bol* (also called triangle, Van Breen, 1920, 112) can be translated as hard point.

2.3 A few remarks about dikes built in the Netherlands

Below, dikes are discussed along the coast, in estuaries or tidal inlets. When dikes along rivers are discussed this will be explicitly stated.
The early dikes are low, 1-2 metres high. They are easily overtopped and breached. But the land is still high in relation to the average sea level and tidal differences rather modest. If a shifting tidal channel comes near the existing dike and threatens to destroy it , a new dike, initially acting as secondary defence (*inlaagdijk*) is built at some distance behind the old dike. Such secondary defences are already being built in the 13th century (Raad van de Waterstaat (1862, 6-7). *Inlagen (insete*), which are the areas between primary and secondary sea defence, are mentioned in a Pronouncement of Count Floris V about the dike of the island of Schouwen on 13 February 1290 (Van Mieris I, 1753, 501).
Through the centuries much land is lost due to these shifting tidal channels and also because many storm surges and flow slides result in dike breaches which are too large to be repaired.
Moreover, subsidence of the reclaimed land occurs as, either, the drained peat layer subsides and oxidizes, or is removed to use it for salt production or for heating and cooking. This in turn requires higher dikes while, because of the increase in flooded area, the tidal prism becomes larger, tidal channels deepen and new tidal inlets are formed like the Zuiderzee early in the 13th century (Van de Ven, 2004, 40-41) and the Braakman in 1375/1376 (Gottschalk, 1983, Vol. I, 162-163). Also the tidal inlet Lauwerszee and the Eems estuary Dollard are enlarged by the sea (Van de Ven, 2004, 104-105).
However, new land is reclaimed from the sea as well. Sediments transported to the estuaries by the rivers Eems, Rhine, Meuse and Scheldt are deposited on the shoals and foreshores. When the mud-flats[4], thus formed, acquire a level which is more or less that of average high tide and are for the greater part covered by marsh grasses '*schorgrasse*n' (like in the 20th century Ricegrass, Spartina Townsendii, see Verhoeven, 1938, B 123), they are considered to be 'ripe' for reclamation (Verhoeven 1924, 189). Obviously, the area to be reclaimed must also be large enough to warrant the investment.
Reclamation (also called polderization) implies in this case[5] the building of dikes surrounding the area and the damming of tidal channels entering the area. But it includes also construction of roads and a drainage network inside the new polder as well as sluices to discharge rainwater and seepage water.

[2] But a jetty is also a berth (*steiger*).
[3] Called wing dams in USA.
[4] Also called tidal lands, salt marshes.
[5] This reservation is made because the word reclamation is also used when desert lands are made suitable for cultivation or habitation.

2.4 General considerations for cross-sectional design

Below, an effort is made to formulate the considerations used in the past for the cross-sectional design of dikes.

2.4.1. Cost of construction

For individuals and organizations (towns, monasteries) , investing in reclamation schemes, the first consideration is cost of construction. In practice, this implies that dike fill, regardless of quality, is borrowed as near as possible to the dike axis. Also the cross-section is 'skinny' to save on fill material: slopes are steep and the crest of the dike has a minimum width (sometimes less than one metre). Last but not least, the dikes are low, they are built for the benefit of agriculture. The safety of the farmers living in the reclaimed polder is not considered to be important (Ferguson, 1991, 45).

2.4.2 Height

2.4.2.1 Up to the middle of the 20th century

Vierlingh (1920, 111) states 'the greatest benefit comes from the height of a dike'. But then it is already 1570. The dikes of the 13th century are low because the level of the land is still 'high', i.e. peat layers have not yet been drained or removed and tidal amplitudes are relatively small.
In the course of the centuries it turns out that repeatedly dikes have to be heightened and one tries to arrive at a logical set of aspects which have to be taken into account when deciding on the height of a dike.
Huitema (1947, 22) lists eight aspects which determine, in his opinion, the dike height; but some aspects are missing and the aspects listed are not all applicable between 1200 and 1900 or they cannot be measured/ quantified anyhow during that period.
Up to 1900, the following aspects appear to be relevant[6] :

1 The highest storm surge on record in the area where a dike is going to be built, as part of a reclamation scheme, or has to be repaired / heightened after a storm surge.
Caland (1833, 22-23) points out that the highest storm surge on record in the province of Zeeland is that of 14-15 January 1808. But the reader will note the qualification 'on record'. Only in a few cases it has been possible to locally establish the height of a storm surge prior to 1800. On the basis of (only) four records of the said storm surge, Caland gives as average height for the storm surge of 1808: 2.25 m above mean high tide (*GHW*). But after 1808, as can, for instance, be seen in Fig. 2.1, higher storm surges have been recorded.

2 The expected subsidence of the subsoil under the dike body and the settlement of the (non-compacted) dike body proper after its construction.
But nobody during these past centuries knows how high these values will be. Caland (1833, 47) recommends to reckon with a figure of 10 % of the constructed height of the embankment.

3 Waves (and also wave run-up but the latter will be discussed in the next sub-section).

[6] Sometimes one encounters unusual considerations for determination of the dike height. Van Vliet (1961, 56-62) reports that the northern dike of the river Lek (known as the Lekdijk bovendams) seldom overtopped because the policy of the waterboard was to keep its crest elevation higher than that of the southern Lekdijk. So, if a high flood occurred, it would overtop and breach the southern Lekdijk. In 1872 the height of the southern Lekdijk had been increased and the situation had been reversed. Moreover, flood levels were increasing. After years of argument about who was going to pay for the required heightening, the new, much higher, Lekdijk bovendams was finally realized in 1880 – 1881. Highest elevation was at Amerongen (1,5 m above highest flood level on record) but more downstream the elevation was only 1,0 m above highest flood level on record in view of the lower elevation of the Lekdijk at the opposite bank!

Fig. 2.1 Flood-stones in Willemstad, marking the highest water levels reached by a number of storm surges; source: Van Veen (1962, 25)

The height of the storm surge as given above has been measured in a situation without currents and without waves. So the question is: how high are those waves, associated with storm surges. During nearly a millennium, nobody is able to calculate (or even observe accurately) wave height, but one is aware of a number of factors which determine waves.
First of all the direction of the wind in relation to the direction of the dike. Then the location of the dike, i.e. is its location to the lee-side or the weather-side of an island. A third factor is the water depth (during a storm surge) on the shore and foreshore. These three factors all have a bearing on the degree of wave attack on the dike. Whether, during the said millennium, dike builders in the Netherlands are also aware of the importance of other factors like wind speed, duration and fetch[7] for the development of wave height, its period and length, is not clear. But in any case, around the middle of the 19th century, Thomas Stevenson (1818 – 1887) develops already a simple formula for wave height which is solely based on the fetch. (Stevenson, Townson & Wilson, 1981, 907-909).
On the basis of the aforementioned considerations, Caland (1833, 26-27) recommends for the province of Zeeland to take waves into account by adding 50 cm to the height of dikes which are not (much) exposed to waves and 2.50 m in case of heavy wave attack. Both values include freeboard. This results in elevations of the dike crest varying between 3 m and 5.25 m above mean high tide (*GHW*). But, already in 1837 a storm surge reaches 2.80 m above mean high tide, excluding waves

[7] The free distance to any point over which the wind can travel in raising waves, i.e. the distance from the nearest coast in the direction of the wind (Scott, 1991, 163).

(Storm Buysing, 1854, 596). So it is not surprising that Caland in a later publication (Caland, 1862, 111-115) mentions a possible extreme water level due to waves in front of the dike of 6.00 m above mean high tide.
Storm Buysing (1854, 597) recalls that the dike on the island of Schouwen-Duiveland near Scharendijke, during the years 1842 – 1843, has been raised to a crest elevation of 6.50 m above mean high tide. This dike is fully exposed to waves coming from the North Sea.
Wave action in breach development is nowadays mainly of importance in terms of overtopping. Since the middle of the 19th century revetments of dikes are such that waves during storm surges do not cause breaches by damaging these revetments but because they overtop the crest of the dike. For wave attack on the seaward slope of the dike see Sub-section 2.4.3 in this chapter.
The overtopping by waves, in combination with overflow because of water levels being higher than the dike crest, weakens the dike body as water enters cracks in the dike crest (see for instance Fig. 9.40 in Ch. 9) and also erodes the (mostly) steep landward slope. This results in a saturated dike slope which subsequently slips because of water pressure from inside.

2.4.2.2 Extreme water level criterion for dike design

Until the second World War the highest water level on record and observed in history during a storm surge is the point of departure for dike design. Wemelsfelder (1939, B 31-35) rejects this approach and demonstrates that storm surge water levels, observed during a period of 50 years, fit nicely in an exceedance curve. He points out that 'highest water level on record' is a misleading criterion for dike design.
Valken (1953, B 163-167) presents an 'estimate' of the elevation of the dike which separates the tidal inlet Braakman from the Westerschelde. This dike (in fact a dam) is complete in 1952 and Valken's article has been written prior to the Storm surge flood of 1953. Valken bases his calculations on an exceedance curve for high water levels observed during a period of 40 years at Terneuzen.
In accordance with the recommendations in a preliminary report of the 'Stormvloed commissie 1939', he takes a water level, which has a probability of exceedance once every 300 years, as criterion. That implies a water level of NAP + 4.50 m. But he also notes that on 1 February 1953 the water level at Terneuzen reaches NAP + 4.96 m!
Valken also points out that the designer of the Braakman dike, Technisch Bureau der Domeinen (TBD) bases its calculations for the crest elevation of the dike on the highest water level on record at Terneuzen (NAP + 4,27 m in 1906).
Crest levels as calculated by Valken (not everywhere the same due to differences in horizontal alignment of dike sections and expected subsidence) are NAP + 7.2 to 8.0 m. The aforementioned designer TBD comes to NAP + 6.75 to 7.85 m. If the allowances for expected subsidence are subtracted from these four figures, one gets NAP + 6.6 to 6.9 m and NAP + 6,75 m respectively. These elevations can be compared with the one given in Sub-section 2.4.7.2 for the nearby dike of the Nieuw-Neuzen polder in 1950: NAP + 6.30 m.
Apparently, the storm surge flood of 1953 was required to have the approach of Wemelsfelder generally adopted[8]. Because, also the dam in the Zuider-Sloe, completed in 1950, has an elevation based on the water level reached in the immediate neighbourhood during the storm surge of 1906 (Verhoeven, 1951, 588).

[8] In any case, Ir. J. .van Veen (Rijkswaterstaat, 1940) already in 1940 calculated the desired dike height in Rotterdam (as part of the reconstruction plans after the 1940 bombings) using the probability of exceedance of water levels.

2.4.3 Seaward slope

During the past centuries the angle of the seaward slope is determined bearing in mind the presence of ice floes and the attack by waves on the vulnerable slope protection of that time: grass, cramp matting (*krammat*), fascines (*rijshout*). The more gentle the slope, the less the effect of ice floes and waves on the slope protection will be.

Accordingly, during the 18th and 19th century a number of hydraulic engineers and others are in favour of applying very gentle slopes. In this respect the publications of Van Bleiswyk (1745), Velsen (1768), Van Peyma (1827) and Loré (De Crane, 1835) are mentioned. Depending on local circumstances, the authors recommend seaward slopes between 1:6 and 1:12 (Fig. 2.2).

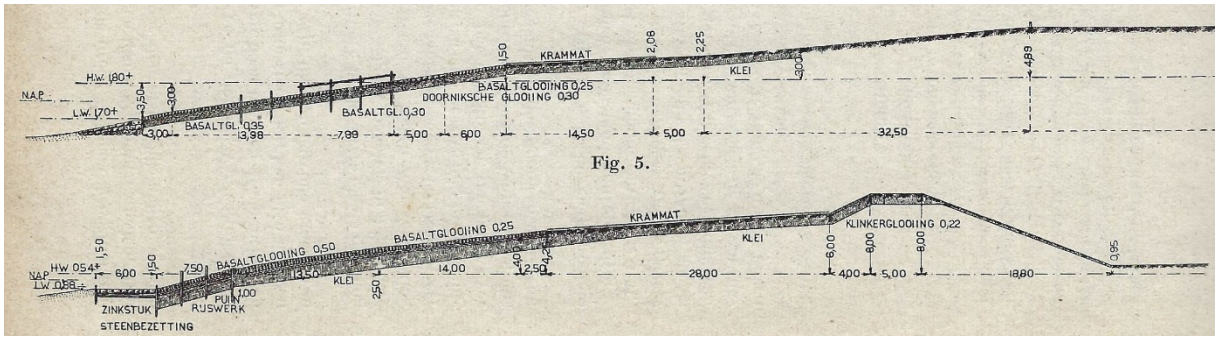

Fig. 2.2 Cross-sections of two sea defences having very gentle slopes: at Westkapelle (top) and Petten (below); source: Colijn (1921, 18)

Wave run-up is a more complicated issue than wave height as it not only depends on wave characteristics, the angle of wave approach and the angle of the slope but also on the roughness and porosity of slope protection.

As recent as 1947, Huitema (1947, 14) states: "it is impossible to calculate the wave run up". But this observation is not correct: Lorentz (Staatscommisie, 1926, 256) presents already a graph for the determination of the wave run-up.

Caland (1833, 68 et seq.) recommends to give the seaward slope a camber *(tonrondte*) such that the slope lower down becomes steeper (but not steeper than 1:3) and the slope near the dike crest more gentle. One can say that by doing that, the slope angle has been adapted locally to the magnitude of wave attack at that level (Fig. 2.3). However, this is the opposite of the practice to construct a *zeebek* (literal translation: sea beak). The *zeebek* is mentioned in Baars (1973, 50 et seq., 56) and is shown in Ch. 7, Fig. 7.9. It was in any case applied to some of the dikes surrounding the polders Klein and Groot Zuid-Beijerland, described by Baars. These dikes were constructed between 1615 and 1653. Baars notes that Vierlingh also mentions the *zeebeck* (Vierlingh, p. 316) but observes rightly that in Vierlingh the word means something different.

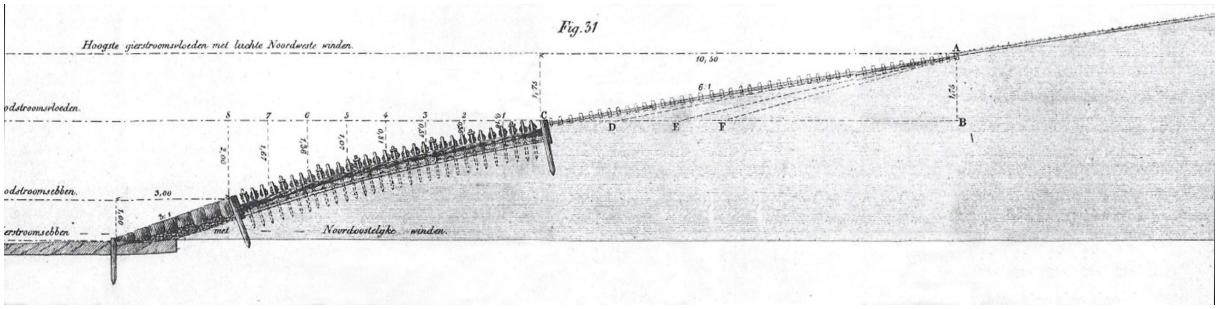

Fig. 2.3 Camber in the seaward slope as proposed by Caland (1833, Plate IV, Fig. 33)

By applying the gentle slopes as mentioned above the wave run-up is much reduced and not so relevant anymore for the dike height. But in the second half of the 19th century the application of stone pitching on seaward slopes of dikes leads again to steeper slopes. Steeper slopes require less expanse to be covered by stones than gentle slopes and the stone pitching is far less vulnerable to wave attack than the old types of slope protection. So the wave run-up comes back as a factor to be taken into account in dike design.

2.4.4 Storm surge berm

In order to economize on the cost of slope protection and earth works , Caland (1862, 111-115) introduces in 1838 in the province of Zeeland[9] a so-called storm surge berm in the seaward slope of sea dikes without foreshore. The width of the berm depends on the wave attack to be expected locally and is 5 to 15 m. The berm has a very gentle slope (1:20). Also its elevation is a function of the wave attack: 1 to 2 m above mean high tide (*GHW*). Slope protection is only required on the slope below the storm surge berm. Apart from savings on earth works and on slope protection (which, in the middle of the 19th century has to be renewed at many dikes in Zeeland) Caland argues that, because the dike crest is now at a greater distance from the toe of the dike, a possible flow slide will not result in a dike breach.

It is noted that Caland does not mention the reduction of wave run-up as a consequence of the existence of the berm. He must have observed it but then decided not to mention it to water boards which would find it a good reason to reject, reduce or postpone dike heightening.

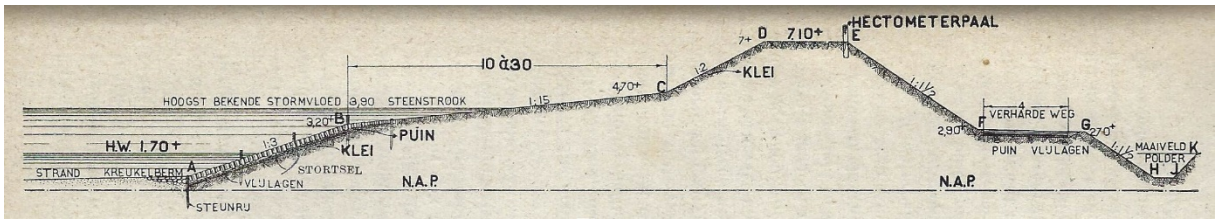

Fig. 2.4 General cross-section of a dike having a storm surge berm; note that the length and slope of the berm differs from what has been proposed by Caland; source: Colijn (1921,18)

2.4.5 Width of crest

If one intends to allow traffic (i.e. pedestrians, carts drawn by horses) on the dike, then a minimum width of 4 – 5 m is required. But not all dike managers are in favour as, in a time without brick paving, let alone an asphaltic or concrete pavement, the 'road' on the dike crest is easily deteriorating which may lead to potholes and rut, caused by the passage of wheels. This is subsequently followed by infiltration in the dike body of rainwater or water from wave run-up. Also wind erosion during dry spells and, last but not least, local lowering of the dike crest by rain may happen.

Finally, during storm surges, it is better to have a maintenance road on a high berm at the land side slope available. Consequently, not many dikes bordering the sea, an estuary or a tidal inlet will have a wide crest and a road on top. Mostly, the crest width is 2 to 4 metres, sometimes 1 to 2 m or even less.

[9] Caland is a government official (Rijkswaterstaat) and normally would not have the power to insist on the implementation of his new dike design in polders being managed by independent waterboards. But many polders in Zeeland (Caland mentions a number of 36) are necessitous (*calamiteus)* and have to accept governmental control in order to receive subsidies.

River dikes, however, frequently have a road on top. Van Vliet (1961, 61) notes that the 'Lekdijk bovendams' being reconstructed in 1880, gets a crest width of 7 metres (but this might also have been done in view of future heightenings).

2.4.6 Land side slope

For obvious reasons, as noted above, the seaward slope, during past centuries, has a gentle slope. But this does not apply to the land side slope. One tries to keep that slope as steep as possible, i.e. 1:1½, 1: 1¾, sometimes even 1:1 but also 1:2.
More gentle slopes are seldom found (Rijkswaterstaat,1961, 161).
The above applies to sea dikes. The situation is different for river dikes. Here slopes are at 1: 1½ to 1 : 2½. Because high water levels may have a long duration, in a number of cases high berms are applied at the land side to prevent seepage water to flow out of the dike body.
The angle of the land side slope should, beside geotechnical and fluid mechanics' reasons, be determined bearing in mind the need for a good grass cover and cattle moving up and down the slope.

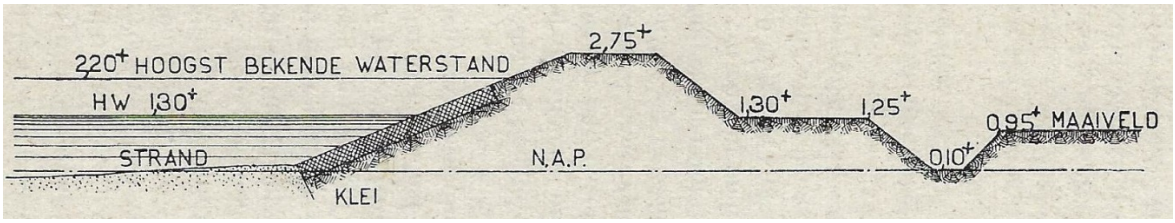

Fig. 2.5 Typical cross-section of a sea dike situated at the leeward side of an island; source: Colijn (1921, 18)

2.4.7 Case histories

In order to give the reader an idea of the magnitude of dike heightening's and elevations during the past and more recently, some data have been collected which are presented below.

2.4.7.1 Dike along the Dollard in the province of Groningen

In 1717, a storm surge breached and damaged many dikes in the province of Groningen. Under the leadership and at the expense of the provincial government the dikes were heightened and the slopes made more gentle. Figure 2.6 shows clearly the difference between old and new cross-sections for dikes along the Eems estuary (Dollard). In this particular case (fig. 14) the dike was heightened from 10 to 12 feet (2.9 to 3.5 m) while its base was widened from 45 to 86/87 feet (13 to 25 m).

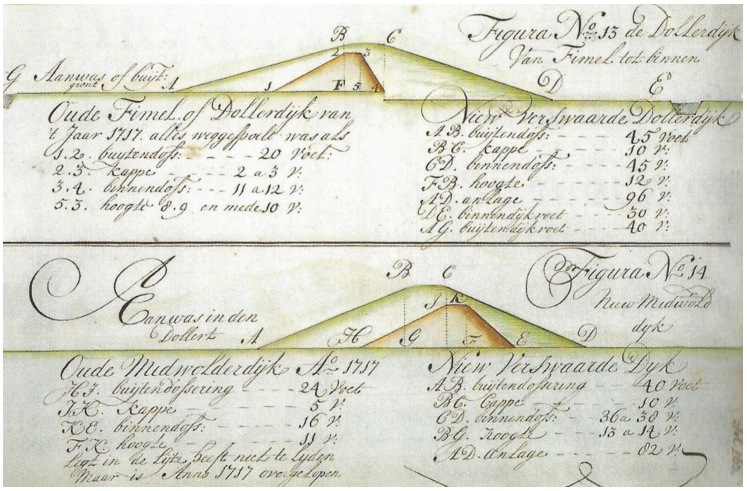

Fig. 2.6
After a storm surge in 1717 the dikes in the province of Groningen had to be heightened and to be given a wider base; cross-sections of the dikes (old and new) along the Dollard are shown; source: Van Seeratt (1721, 55)

2.4.7.2 Dike of the Nieuw- Neuzen polder in Zeeuws-Vlaanderen

From 1559 onward this dike along the Westerschelde has an elevation of 3.30 to 4.10 m above mean high tide (*GHW*).

In 1872 these values are 3.6 to 3.8 m above mean high tide but the latter has increased meanwhile by 0.45 to 0.73 m. In 1862/1870 mean high tide is at NAP + 1.83 m. So the elevation of the dike crest in 1872 is approximately at NAP + 5.43 to 5.63 m. Around 1950 the said elevation is approximately NAP + 6.30 m.

After the storm surge flood of 1953 the elevation at the west side becomes NAP + 9.75 m, which elevation goes down to NAP + 8.0 m at the east side. These last values are based on a probability of exceedance of the water level by a storm surge once every 4000 years.

The data presented in this Sub-section have been derived from De Kraker (1997, 233 and 236) and De Vleesschauwer (2013, 176 and 190).

2.5 The fill in the dike body

As Huitema (1947, vii) points out:

> "dikes and bunds are constructed using earth fill, preferably the earth fill locally available; and the characteristics of soil types used as dike fill not only vary considerably but one type may already show a great variation in characteristics while the moisture content and the method of placing used, again may enhance important differences."

Huitema published his book about dikes in 1947 and refers mainly to the standards and methods in use for the placing of earth fill in dikes during the first half of the 20th century. One can assume that, before that period, the situation might have been the same but, most probably, worse. That situation is aggravated by the fact that one could not measure the characteristics of the various soil types, let alone specify quantitively the desired value of certain soil mechanical parameters like grainsize, grading, density, moisture content, permeability or degree of compaction. In this respect it is recalled that the study of soil mechanics only starts in the Netherlands after the train disaster near Weesp in 1918 (Bokhoven 1990, 19).

Depending on what is locally available in the borrow pits on the foreshore the fill to be placed in the dike body can be peat, clay, silt, sand or a mixture of these materials (Fig. 2.7).

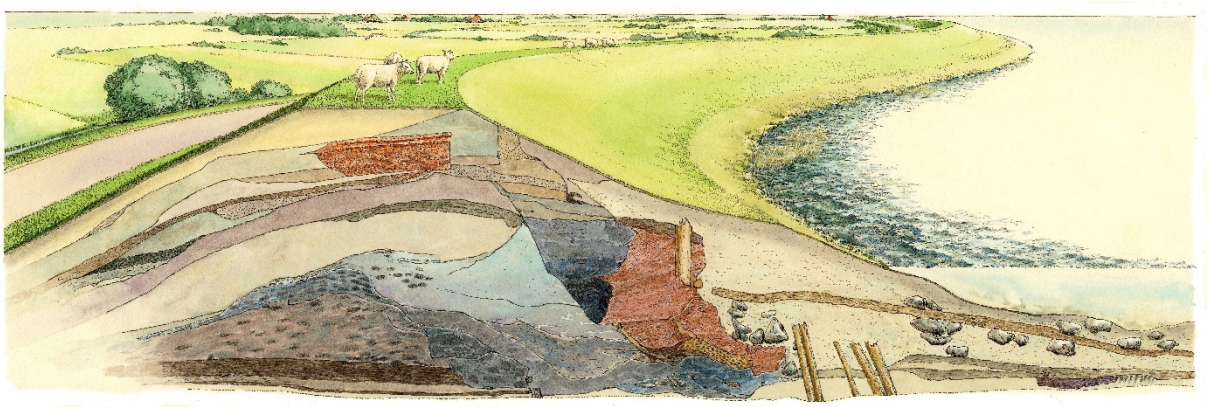

Fig. 2.7 Artist impression of a cross-section of a dike near Enkhuizen; in the course of time a great number of different materials have been incorporated in the cross-section, like sand, clay, peat, shells, sea-weed, a willow mattress, bricks, reed and twigs; source: Bob Brobbel

The so-called clastic '*klastische*' sediments can be ranked on the basis of grainsize (Table 2.1). Clay will always, to a greater or lesser extent, be mixed with sand. One speaks about clay when the soil

contains 25 % of clay grains (lute) or more. The greater the sand or silt content the lesser the cohesiveness of the clay will be. And it is its cohesiveness which determines its resistance against scour. Clay has a low permeability. Fat clay has a low sand and silt content. It is called '*spier*' in the province of Zeeland. It has a blue-grey colour.

Table 2.1 Ranking of clastic materials on the basis of grainsize; Source: Deltares (2008, 8)

Gravel	2	to	63	mm
Sand	0.063	to	2	mm
Silt	0.0002	to	0.063	mm
Clay	< 0.0002			mm

Loam is a soil type which consists predominantly of silt, while also sand and clay are part of it. Löss found in the province of Limburg is loam and was deposited by wind.
Boulder-clay (*keileem*) is an unclassified mixture of boulders, gravel, sand, clay and loam. It has a high resistance against scour. It has a low permeability.
Peat consists of humidified organic material. It can have a very loose or a stiff (after compression that is) structure. In the latter case it will be, like clay, quite scour resistant, De Kraker (1997 ,237); see also Aan de Brugh (2020, W11).
But Van Bleiswyk (1745, 9) notes "The Frisian peat-earth is still more porous, lighter and less cohesive than the peat in Holland, accordingly, the former is very unsuitable for the building of dikes".
Contrary to what is generally believed, during past centuries many dikes are already built up of sand covered by a layer of clay. See in this respect Vierlingh (p. 269-270), Anemaat (1739, 26-28), Van Bleiswyk (1745, 18, 25); Van Leeuwen (1930, B.99) and also Baars (1973, 43), who stipulates that sand in the 16th century was permitted in the dike body, provided a protective layer of 1.2 m of clay was applied.

2.6 How erosion of the dike body is prevented

2.6.1 General

Erosion of the dike body may happen due to the following forces: rain, wind, currents caused by tidal or river flow, ice floes, wave attack, carts, horses, pedestrians, grazing cows or goats.
An important role in the protection of the dike body is a good grass cover. For many dikes, which are not exposed to wave attack and (salt) water levels varying as a function of the tide, a well maintained grass cover suffices as protection to the eroding forces just mentioned.
But a good grass cover cannot be maintained in the tidal zone and it will not prevent damage to the dike body in case of heavy wave attack, strong currents undercutting the toe of the dike or ice floes ripping the grass cover.
Other materials, in addition to the dike fill just described, have been introduced to obtain the required protection against erosion. In this respect are mentioned: fat clay clods (*kloetelingen*), fat clay sods, straw, reed, fascines, sea-weed, piles, timber, bricks, boulders, stones.
These materials are applied whether or not in combination with each other and this leads to certain constructions in which each material has its own specific function.
Below, the various constructions used in the past to protect the dike body against erosion will be shortly described. More details about each construction mentioned can be found in Appendix A.
A distinction will be made between:

- direct protection of a sea side slope of a dike by a protective layer on the slope;
- protection of the slope by means of a 'wall';
- protection of the dike by keeping the currents away from the dike body;
- protection of the foreshore;
- protection against scour of the toe of the dike.

2.6.2 Protective layer on the slope

There is direct link between the effect of wave attack and the gentleness of the sea side slope of a dike; the more gentle a slope, the lower the effect of wave attack. So, the first dikes exposed to wave attack had very gentle slopes and only a grass cover. In the tidal zone the dike fill was protected by cramp matting and, later on, by a combination of cramp matting and fascines. If the wave attack was heavy, stones were fixed on top of the fascines. The disadvantage of this type of protection is the need to renew it regularly. Also the naval shipworm destroyed the fascine protection. Finally, in the 19th century, stone pitching got applied everywhere as the most efficient protection against wave attack and currents along the slope. Further details can be found in Sections A.2 – A.7 of Appendix A.

2.6.3 Protection by a 'wall'

As suitable stone (i.e. having an angular shape) was not everywhere easy to get and/or expensive, the slope was in certain regions replaced by a, nearly vertical, 'wall'. This 'wall' consisted mainly of sea-weed or reed (both were locally 'harvested') or comprised just one row of wooden (sheet) piles. Also a combination of a row of piles at the toe of the dike combined with a light protection on the slope (brick revetment) was applied in one region. Further details can be found in Sections A.8 – A.11 of Appendix A.

2.6.4 Protection to keep the currents away from the dike body

In order to prevent scour of the toe of the dike by the current and damage to the protected slope by ice floes, spur-shaped constructions were applied on the shore or foreshore. These spur-shaped constructions in most cases looked like cofferdams, comprising two or three rows of piles, the space in between having been filled with either stones, or fascines ballasted by stones or fascines only. Apart from their shape, undesirable from a hydraulic point of view, the spurs required much maintenance which was a costly affair.

At some places, along rivers or in estuaries, a kind of hard points (small groynes) having, in horizontal projection, a triangular shape, have been applied. Apparently, their usefulness has not generally been agreed upon.

Further details can be found in Sections A.12 – A.15 of Appendix A.

2.6.5 Protection of the foreshore

Protection of the foreshore has been promoted by Caland who dedicated 138 pages to this topic in his book which' title can be translated as 'Manual about dike building and the art of sea defence' (Caland, 1833, 142 – 279). This protection has only a remote relation to the subject of this dissertation. For more details reference is therefore made to Appendix A, Section A.16.

2.6.6 Protection against scour of the toe of the dike

The toe of the dike as well as the area immediately in front of it are vulnerable to scour by breaking waves. Also the transition from protected slope to non-protected (fore)shore requires certain measures. Support of the toe of the dike and protection of the (fore)shore against scour can be provided by placing a fascine mattress (called *kraagstuk)*, ballasted by stone. For further details, see Appendix A, Section A.17.

2.7 Why dikes frequently breached during the past centuries

2.7.1 Introduction

During the storm surge flood of 1953 in the Netherlands many dikes were breached or seriously damaged. Rijkswaterstaat (1961, 158) notes that:

> "the overwhelming majority of the damages (to the dikes) was found on the slopes at the landward side and at the crest. Really spectacular damages to a seaside slope at places

where landward slope and crest were not damaged or only a little bit, were seldom found. Most of the dikes in the disaster area showed a dike having a very much damaged landward slope while the seaside slope in most cases stayed undamaged."

The reason for the said damages to the landward slope (at many places followed by breaching) was nearly always water, and/or wave run-up, overtopping the crest due to extreme water levels,.
Only in a few cases other reasons for damages or breaching could be established, like foreign objects (partly) located in the dike body (culvert sluices, bunkers, pipe line crossings, houses) or a badly backfilled trench (p. 161).
Obviously, in the past there were also many storm surges and river floods which caused breaches of dikes and, subsequently flooding. It is however not known, whether these storm surges or floods were of the same magnitude as the 1953 storm surge. The *Rapport Deltacommissie* (1960, Vol. 1, 27-28), suggests that even the storm surges causing serious disasters (like those in 1421 and 1570) cannot have reached much higher water levels than in 1953.
In the 20th century there were only a few storm surges (4) and river floods (2) which breached dikes, followed by flooding. For storm surges in 1906, 1911, 1916 and 1953, see Rijkswaterstaat (1961, 13); for river floods in 1920 and 1926, see Burgers (2014, 308)
But in the 15th , 16th and 17th centuries each and every one of the nearly 100 storm surges, listed by Gottschalk[10] for (parts of) the area which is now known as The Netherlands, resulted in a small or large number of dike breaches.
Also river floods caused many dike breaches but the information collected by Gottschalk is not as consistent as for storm surges. First of all one speaks already about a flood in case of bank overspill and flood water stored in and flowing over the flood plain (*uiterwaard*) without breaching. Secondly, breaching can also be caused by a high water level in the river and flood plain due to ice jamming without a flood.
Because of the enormous number of dike breaches in former centuries in the Netherlands along its rivers, in the south-western delta of Rhine and Meuse and along the coast, an effort has been made in the following sections to answer the question *Why did so many dikes breach* ?

2.7.2 Why do dikes breach in general

There are six basic causes for dikes breaching:

- quality of the initial product (i.e. the newly completed dike section) is not to standard;
- unusual external loading of the dike higher than the design resistance;
- there is no periodic or regular maintenance; no timely repair of damages;
- no regular inspection and survey of the dike which may lead to a long-term deterioration and to hidden faults;
- a change in the original natural conditions;
- wilful breaching in time of war.

Obviously, a link may exist between, for instance, lack of inspection, no maintenance and insufficient strength. Each of these basic reasons for dikes breaching will be elaborated upon in Sub-sections 2.7.3 to 2.7.8 inclusive.

2.7.3 Low quality of the completed dike section

Most of the dike breaches are due to one of the following reasons:

1. The crest of the dike is too low from the start. The entrepreneurs, chartered to undertake a new reclamation scheme (*inpoldering*), do not want to spend much money. They are first and for all interested in a high agricultural yield and they are less concerned about the safety of the farmers in the new polder. See in this respect Ferguson (1991, 45).

[10] (Vol II, 1975 and Vol. III, 1977, Diagrams IX to XIV incl.)

Gottschalk (1983, II, 139) suggests for a particular case that dikes were kept low in order that they could be overtopped during storm surges without getting breached.

2. The dike profile is 'skinny'. The slopes are too steep and the crest is too narrow. It implies that wave run-up is high and overtopping of the dike by waves easily occurs. Reference is made in this respect to Schraver (1807, 92), De Bruin (1953, juli, 16) and Niemeijer (1975, 78).
3. The dike has been constructed by using light peaty material, Rienks & Walther (1954, 25). When the water level is high, the dike is pushed sideways and parts start to float away. It happened for instance in 1675 near Hoorn where, according to Van Bleiswyk (1745, 9) during a storm surge a dike was pushed from its base over a length of 5 700 m.
4. According to Fockema Andreae (1934, 77) up to the middle of the 16th century all structures in the dikes are made of wood. It happens regularly that, during high water levels, such a wooden sluice is being pushed out of the dike and is floating away with the current through the just created opening in the dike; see Van Dam (1998, 151), De Vries (1876, 108), Such a wooden sluice is pre-fabricated and subsequently shipped to the site (De Kraker et al, 1993, 643).
5. During construction of the dike it happens regularly that unsuitable materials are incorporated by the contractors in the dike body. Obviously, this is strictly forbidden but nevertheless it happens! Vierlingh (p. 132) specifies, inter alia, in a tender document, that it is forbidden to put empty barrels in the dike. In 1494, prior to start of construction of the Landdijk (Zeeuws-Vlaanderen), all straw and tree trunks laying in the area after the flooding were collected and burnt; this in order to prevent contractors from incorporating these items in the dike body (De Kraker et al, 1993, 189). In another article in the same publication (p. 204) a tender document is mentioned in which it is specified that it is forbidden to bury empty barrels, straw or dead horses in the dike body.
6. Fill brought into the dike profile during construction is insufficiently compacted or not at all, resulting in settlement of the dike crest. The contractors building a dike do not like compaction because it means more work. There are also no adequate compaction[11] methods available at that time and, moreover, no method to check the degree of compaction. In 1605 the mud flats of Oud-Prinsenland (north-east of Steenbergen) are reclaimed (Baars,1982, 32) It is specified in the contracts that the dike has to be built up in layers of 100 to 135 cm. The author of the article notes that this requirement, though still not adequate (layers should be much thinner) is new; before, there was no compaction apart from horses and carts crossing the fill already placed. Verhoeven (in Vierlingh , 1920, XXXV) notes that the thickness of the layers of earth to be placed is missing in the specifications given by Vierlingh.
7. The new dike or dike section in case of breach repair has to be connected to already existing dikes. Often, this is done badly and during the next storm surge the dike breaches again, at the same spot as before. The same applies to the connection between an old dike crest and the heightening made on top of it (a so-called '*muts*'). For instance, De Vleesschauwer (2013, 15-16) relates that the storm surge of 1570 causes a large breach near Hulst at the same spot as the breach of 1532.
8. A seaside slope protection is not strong enough to resist heavy wave attack and the dike collapses[12]. The slope protection will also be seriously damaged by ice-floes flowing along the slope. In this respect, one should realize that up to the end of the 18th century the slope protection consists in most places of non – durable materials (fascines, straw, wooden structures) which (a) have to renewed frequently , (b) do not have the strength to endure heavy wave attack,

[11] Other than by repeated human traffic, walking over the new construction.

[12] This was definitely not the case during the storm surge in 1953. In this respect, Rijkswaterstaat (1961, 158) notes the following: "Really spectacular damage to the seaward slope , when the landward slope and crest of the dike had no damage or only small damage, was quite rare. …Most dikes in the disaster area showed a landward slope which was seriously damaged while the seaward slope most of time had no damage whatsoever."

and (c) are destroyed by ice-floes and the naval shipworm. One of the actions in that situation is to apply very gentle slopes.
Schraver (1807, 91) mentions a slope angle of 5° (1 in 11.4) for the sea side slope of the dike at Westkapelle. He refers in this respect to calculations made by Van Bleiswyk (1745) and also to Velsen. Finally, Loré (Eekhof, 1835, 46 and 61) designs dike profiles having slopes of 1 in 10.[13]

2.7.4 Unusual external loading of the dike for which no counter-measures do exist

Dikes should only breach under extreme conditions during which design boundary conditions are exceeded. Storm surges may just exceed the design height of the dike (which was usually based on experience and not linked to a return period).
As already stipulated in Sub-section 2.4.2.1, the overtopping by waves, in combination with overflow because of water levels being higher than the dike crest, weakens the dike body as water enters cracks in the dike crest and also erodes the (mostly) steep landward slope. This results in a saturated dike slope which subsequently slips because of water pressure from inside.
But there is also the case of ice jamming, which, in the past happened frequently. The ice jamming occurs during winter or early spring when the river is still frozen and floods created by snow-melt already arrive from upstream. Moreover, the irregular profile of the rivers , which are at some locations very shallow, and human causes (obstructions built in the river bed) are a great help in the forming of ice jams. The ice jamming only stops after the rivers, for navigation purposes, have been normalized during the 19th century and powerful icebreakers have been developed. (Burgers, 2014, 200-205).

2.7.5 No periodic or regular maintenance; no timely repair of damages

If maintenance is lacking and repairs are not carried out, this can be due to (one of) the following reasons:

- there is no money to carry out the maintenance,
- no good governance,
- disagreement about who is going to pay,
- a warring situation exist in the region which prevents remedial action.

In practice all four reasons occurred many times, alone or in combination, and the situation is aggravated by the fact that, apart from damage, by river floods and storm surges, which has to be repaired, these dikes require periodic and regular maintenance anyhow. Nowadays, it suffices to make regular inspections and most probably no action is required. The dike can lay there for many years and maintenance is limited to periodically mowing the grass. But during the past centuries that was different. Obviously, all four reasons mentioned are of an institutional character.
The people living in the polder (mostly peasants) were either supposed to pay for certain periodic maintenance or to carry it out themselves. Usually, they were too poor to pay and also too unskilled to do the work themselves as well as reluctant to do it.
The man who was supposed to organize and supervise the work, the dike reeve, had been appointed because he had a good relationship with the regional ruler, not because he knew anything about hydraulic engineering, let alone about dikes.
Then there were the cities and/or the waterboards who had endless arguments about the share in the cost each party had to pay.
Still, these were exceptional situations, and in particular also due to the very fact that all conditions listed were present at or before the event. That was an unhappy coincidence, and that is how disasters come about.

[13] See in this respect also Sub-section 2.4.3 and Fig. 2.2.

Two typical examples are given of such disasters:

- The All-Saints storm surge of 1 November 1570 had disastrous consequences for the whole of the Netherlands. Also in Friesland the damages were great, may be even greater than anywhere else. It has been described in detail by Gottschalk (1975, 681-687). There was so much disagreement between the various parties who had to pay their share in the repairs that the repair works only started after 3½ years, in March 1574. And in fact this was only because the representative of king Philips II, Caspar de Robles, after the parties had argued for three years, cleared the situation by making firm decisions and by heavily punishing those who did not follow his orders.
- During the storm surge of 5 November 1530 all polders around the town of Reimerswaal were flooded. Subsequently, the central Government in Brussels took a number of decisions. Everybody who had left was ordered to return and nobody was allowed to leave the area. All people had to work at the dikes to be repaired and one was also supposed to work on saint's days. Measures were taken to stop increases in prices and flat-deck barges could not any longer be used to transport salt and peat but should transport clay sods instead. There were a large number of breaches and it had been possible to have these closed in a planned sequence but some stupid decisions and measures were taken. First of all the local ruler, the lord of Lodijke, decided that the breach near his castle should not be closed as he hoped that the tide should scour a nice harbour for him. As a consequence, In a few days the breach, who could easily have been closed immediately after the storm surge, developed into a large gap, 18 metres deep and 200 metres wide. Secondly, the other breaches were closed except one (*Kreeke*). But this one was at a very low spot and because of the closure of the other breaches also this breach developed into a nearly irreparable gap (Vierlingh, p. 331, 332, 335).
  Still, various works around the town of Reimerswaal, aiming at repair of the breaches and reclamation, proceeded, but in 1531 part of the landholders was not any longer able to pay for their share in the cost. As a result the completion of the repair works was delayed while sixteen villages were still flooded. In March 1532 the work was resumed and should have been completed by May 1533. But then came the storm surge of 2 November 1532 and all new work was lost. Only the town of Reimerswaal survived for another 100 years. But the six storm surges between 1552 and 1570, followed by warring, plundering and conflagration, meant the end of that town (De Bruin, 1973, 6-9, 13).

2.7.6 No regular inspections or surveys

A systematic inspection of a dike will show what is wrong. For instance, the grass cover has locally disappeared because of grazing and, subsequently, the slope has eroded during heavy rain or by wind. Surveys would also show that the dike crest is lower than before and this is due to settlement of the badly compacted dike body and/or peaty layers in the subsoil which are being compressed by the weight of the dike body (Colijn, 1921, 123; Van Vliet, 1961, 43-44). This can be a slow process going on for years and, without regular inspection and surveys, nobody will notice the slow deterioration.

2.7.7 Change in natural conditions

At the time the dike is being constructed the physical environment will be acceptable for the dike being built at that particular location: the level of land inside the (future) dike and that of the shore or flood plain is known as well as the water levels and wave run-up to be expected during storm surges and/or river floods.
But that situation may change in time:

1. A deep tidal channel changes its course and now runs near the dike. As a consequence a flow slide takes place in the foundation of the dike which in turn causes slips in the seaside slope of the dike, thus seriously decreasing its functionality. In the past numerous flow slides did happen in

the province of Zeeland. Gottschalk (1975, 824-825 and 1977, 427-428) reports about some 115 known flow slides in the period 1365 – 1698.
Rijkswaterstaat, Deltadienst (1958 – 1988, no. 37, 348) estimates the number of flow slides since 1800 at 1 000. Already in 1290 mention is made of secondary sea defences *(inlaagdijken*) to counteract the effect of flow slides (Raad van de Waterstaat, 1862, 6-7). Before 1725 one tried already to prevent flow slides by protecting the foreshore at vulnerable locations with fascine mattresses (Boddaert et al, 1753, 262-263). But, in view of the description given, one can conclude that it concerned the suspended fascine mattresses described in Chapter 4, Section 4.3. Also fascine spurs (*rijshoofden*) were applied to keep the deep tidal channel away from the dike. These fascine spurs were expensive in construction and maintenance and in early spring, with ice-floes in the tidal current, they were easily damaged (De Kraker, 1997, 94 and 173).
The *Raad van de Waterstaat* (1862) in its report *Verslag Oeververdediging in Zeeland*, pages 82-83, after discussing all kind of systems protecting the foreshore against flow slides, recommends a non-interrupted thick layer of rock along the foreshore supported at the sea side by fascine mattresses, ballasted with rock. In the 20th century, after geotechnical studies and bearing in mind the high cost of fascine mattresses, it is decided to prevent future flow slides by applying a thick layer of coarse gravel on the foreshore placed at a slope of I in 3 or more gentle after having filled scour holes under that layer by dumping fine gravel or coarse sand (Rijkswaterstaat, Deltadienst, 1958 – 1988, no. 43, 137-143).

2. The river has a significant sediment load. As the river has not been regulated the river bed will gradually become higher because of sediment deposition. Nobody is aware of it and during river floods the water levels are much higher than before and the dike is overtopped (Krayenhoff, 1821, 1).
3. There is originally a thick layer of peat under the clay layer at the surface of the shore. Then, in order to obtain salt for consumption the peat is dug out and burnt. Consequently, the level of the shore is now much lower and waves much higher than before and, as explained above, the protection of the dike against wave attack is low; Ferguson (1991, 42); Lendering (2005, 32).
4. In the course of the past centuries during storm surges the sea has intruded into the mainland. The Zuiderzee is then formed as well as the tidal inlets in the south western part of the Netherlands. Because of storm surges lands are flooded and the area under the influence of the tidal regime increases temporarily or permanently. This in turn leads to deeper and wider tidal channels and to increased tidal amplitudes[14]. It implies that high water levels increase, also during storm surges, and once again, it results in more dikes overtopping and breaching. As an example the tidal ranges of the average tide are given for various years or periods for three towns at the banks of the Westerschelde and Antwerp (Table 2.2).

Table 2.2: Mean tidal ranges along the Westerschelde

Year or period	Flushing	Terneuzen	Bath	Antwerp
1000				1.00 m
1500				2.85 m
16th century		3.43 m		
1854	3.62 m	3.95 m	4.25 m	
1990				4.90 m
2001-2010	3.79 m	4.14 m	4.88 m	

Sources: De Kraker (1997, 233), Storm Buysing (1854, 377-378); Coen (2008, 70), Tienjarig Overzicht Zeescheldebekken (2016, 18)

5. Ice jamming as caused by human intervention has already been mentioned in Sub-section 2.7.4. But ice jamming is also related to lower temperatures during the Little Ice Age which started in

[14] But one should bear in mind that tidal amplitudes may also increase because of human intervention like dredging for navigation as has been done in the Westerschelde (Table 2.2).

1400 – 1500 and lasted about 400 years. It resulted in the Netherlands in a maximum lowering of the temperature during winter by the end of the 16th century and could lead to some very cold winters.

6. Finally, there is a sea level rise as well as a lowering of the land area. Up to the 21st century the sea level rise has been small (according to different sources[15] some 104 cm to 200 cm in 10 centuries) but the lands reclaimed from the sea have become much lower in level because of peat winning for domestic purposes (for heating, cooking and to obtain salt), peat oxidation and, last but not least, subsidence of the peat layers in the subsoil because of drainage. It implies that, depending on the location, the ground level has gone down by 2 to 4 metres (Van de Ven, Ed. 2004, 18; Schilstra, 1987, 26; Kool-Blokland, 2003, 17; etc.). These low land levels as well as the (moderate) sea level rise render the repair of dike breaches much more difficult and costly than before.

2.7.8 Wilful breaching in time of war

The last time wilful breaching of dikes happens in The Netherlands is during the Second World War: the Germans breach the dikes of the polder Wieringermeer and the Betuwe while the Allied Forces bomb the dikes of Walcheren. But it also happened many times in the past and inundation[16] is then even a well-established way[17] in war time of defending parts of the country or a city. Still, it is not always as controlled as one would wish. For instance, during the 80-years war in, what is now called, Zeeuws-Vlaanderen, various dikes are wilfully breached and De Vleesschauwer (2013, 16) writes:

> "The consequences of these war inundations for the landscape were already irreparable after a few months. The impact on the landscape was greater than what the storm surges had been able to achieve."

Another outcome of warring-actions is the delay in remedial measures (i.e. repair of the breaches) and a lack of regular maintenance of the dikes, which' situation may last many years.

[15] A graph in Schilstra (1987, 26) gives 80 cm for the period 1000 – 1850; In 'Sea level rise Dutch coast' a figure of 23 cm is given for the period 1890 – 2014. If these data are combined one finds 104 cm for the period 1000 – 2000. In Van de Ven (Ed.) (2004, 316) a graph is shown which shows also 104 cm for the same period. According to Ir. J. van Veen (Rijkswaterstaat, 1940) the sea level rise is 20 cm in a 100 years period..

[16] There is a distinct difference between flooding and inundation. Flooding is normally an uncontrolled operation, the flooding can be deep or shallow. Inundation is a military operation aiming at a thin layer of water on the land thus preventing vehicular traffic (lorries, tanks) and navigation because the water is not deep enough.

[17] There was a '*Hollandse waterlinie*' as well as an '*IJssellinie*' and the '*Stelling van Amsterdam'*.

CHAPTER 3 Description of simple means

3.1 General

Within the context of this dissertation the expression simple means implies that most of the physical work was done by manual labour. Machines[1] and tools played no role or a limited one. The latter was either not (yet) available or too expensive[2].
In order to close the flow gap in a breached dike or to dam the last gap in a tidal channel, one will need;

- labour,
- construction materials,
- tools and equipment to be cheap and simple,
- temporary works[3].

These four topics are discussed below bearing in mind the focus in this dissertation on closures carried out in whole or in part by using simple means. The last closures which satisfy this definition are, in the Netherlands, the damming of the tidal inlet Dijkwater (island Schouwen-Duiveland) in 1954 and, internationally, in Bangladesh, the closure of the Nalian creek in 2020.
One could in fact read simple means as 'to be carried out at low cost' because closure operations in former centuries always have to be carried out requiring a low investment. Such approach may result in great risks being taken, in many cases leading to failure of the closure operation during or immediately after the event. This is also pointed out repeatedly by Vierlingh (1920, see for instance 347 et seq.).
Low cost implies in fact also:

- cheap labour,
- construction materials to come preferably from the neighbourhood of the closure site;
- tools and equipment to be cheap and simple;
- temporary works to comprise cheap materials, easy to construct and easy to remove after closure.

In the following sub-sections each of these four topics will be discussed in more detail.

3.2 Labour

During the past centuries the energy required to effect a closure, to build a dike, etc. is provided for the greater part by people carrying out the work manually. To avoid misunderstandings: manual comes from the Latin *, manunualis* (by hand). It definitely does not mean 'labour by man'.
During or immediately after storm surges also women work at the dike (see Sub-section 3.3.1) and, as shown in 1969 in Ghana (Ch. 11, Section 11.2), they may have an important role in the background of a closure operation.
Above, it is stated that the energy is provided for the greater part by manual labour. But, to a lesser extent, energy is also provided by horses drawing carts, by the wind and the (tidal) current moving ships loaded with closure materials, or by the current helping in transporting a floating fascine mattress.

[1] In this respect one must think first of all about dredgers, floating cranes, cable ways, large pile driving equipment, controlled stone dumping vessels. But a pile rig operated by a pile-driving crew is considered as simple means.
[2] In Parts II and III of this dissertation one will note that a relative small part of a project may still require the input of mechanical equipment.
[3] See for definition Chapter 1, Sub-section 1.4.2.

The physical output[4] of a labourer through the centuries during closure works will be analysed for a number of closures in the case histories of Parts II and III and summarized in Chapter 19. Here, it is only recalled that always many labourers are involved which is illustrated by three examples:

- for the closure of the '*Grote Gat'* (the large gap) in the Spaarndammerdijk near 't Hart in the summer of 1510[5] during the peak of the work 900 labourers are employed (Van Dam, 1992, 455);
- In December 1575, at Scharwoude[6] 4 800 labourers strengthen a secondary, 14 km long dike in a period of two weeks (Thierry, 1930, A 53);
- In 1985 the remaining openings (adding up to 775 m) in the sill of the Feni closure bund[7] are closed in 7.5 hours by 10 000 labourers (Van Duivendijk & Te Slaa, 1987, 24).

In the past, in an emergency situation, i.e. during a storm surge or a high river flood, when a dike breach is imminent, all inhabitants of a polder were supposed to come to the dike and help[8]. Nowadays, a group of volunteers (*het Dijkleger*) have agreed in advance to inspect the dikes during threatening situations and to go to the spot where the dike might be in danger of breaching.

3.3 Construction materials

3.3.1 Materials available in nature

For materials, suitable for closure works and available in nature, one must think on the one hand about granular/ stony materials like clay, sand, boulders and rocks and on the other about (parts of) plants like brushwood, wooden sticks, poles and stakes, reed, straw, bamboo, palm leaves, millet stalks.
Below, a few characteristics[9] are given about each item and their typical use in closures described.

3.3.1.1 Clay

Clay is one of the clastic sediments. These sediments can be ranked on the basis of grainsize.
Clay will always, to a greater or lesser extent, be mixed with sand. One speaks about clay when the soil contains 25 % of clay grains (lute) or more. The greater the sand or silt content the lesser the cohesiveness of the clay will be. And it is its cohesiveness which determines its resistance against scour. Clay has a low permeability k[10] ($k = 10^{-10}$ to 10^{-12} m/s). Fat clay has a low sand and silt content. It is called '*spier*' in the province of Zeeland. It has a blue-grey colour.
If clay is used for closure works it is, either packed in bags or comes as a cube-shaped (15-20 cm sides) clod[11], at one side grown over with grass and dug from a 'ripe' saltmarsh.
These clods are either directly used as a closure material (dumped, in a vertical closure), or to ballast a fascine mattress, or as a scour resistant layer in a closure gap (Matelé, 1731, negenste Cap.).

3.3.1.2 Sand

In most cases sand, available near the closure site, is too fine for use in closure works as the current will carry it away. Very fine sand will also leak from jute bags[12]. In Ghana (Keta closure bund, see Ch. 11, Section 11.2) the sand grains in the sand ridge along the lagoon have a diameter of 1-2 mm. They could therefore be characterized as very coarse and had a certain resistance against scour.

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[4] For instance the number of bags carried during one day.
[5] Ref. Ch. 9, Section 9.2.
[6] Ref. Ch. 9, Section 9.4.
[7] Ref. Ch. 11, Section 11.4.
[8] See for instance De Bruin (1952).
[9] It is realized that a small part of the text below has already been presented in Ch. 2, Section 2.5 but it is repeated here in order to provide the full picture.
[10] k = permeability coefficient according to Darcy.
[11] A clay sod has 20 or 30 cm sides and a thickness of 4-5 cm (Huitema, 1947, 102).
[12] For that reason jute bags in Bangladesh were filled with clay instead of sand.

Accordingly, the sand could be used to narrow the closure gap. The permeability k for clean sands is 10^{-4} to 10^{-6} m/s.

3.3.1.3 Boulders

Boulders (*rolstenen)* is the general term for pieces of rock having obtained a more or less spherical shape by abrasion during transport in a mountain river or through rotation in the current under or just downstream of a glacier. Because of their shape boulders are not suitable for use in stone pitching but can be used very well as fill in a cofferdam or to ballast a fascine mattress. It is thought (by Van Duivendijk) that so-called *cattebollen* (or *cattebecken*, *kattebollen*) are in fact small boulders (diameter 10 cm or less). See also Vierlingh (1920, 427). The maximum size of boulders used in these closures will never be more than 25 to 30 cm in diameter as these boulders have to be transported manually.

Boulders are more or less spherical with diameter D_s = 25- 30 cm and the density ρ_r = 2 600 kg/m^3. Then is, according to the formula below, the block mass M = 21 to 37 kg.

$$M = \left(\frac{D_s}{1.24}\right)^3 \text{ x } \rho_r$$

If one uses rocks, as indicated in Table 3.1, which are not spherical in shape, the relationship between the mass M of a rock and its dimensions is more complicated (see below).

3.3.1.4 Rocks

There are three groups of rock which comprise 20 rock types (CIRIA, CUR &CETMEF; 2007, 86 et seq.). But it is beyond the scope of this dissertation to go into much detail. During past centuries in the area which is now the Netherlands rock is only used in closure works if there is no other option because it is expensive. This rock comes from quarries in the neighbourhood of Vilvoorde and near Doornik (both in Belgium) and from Norway. The rock from Vilvoorde is a calcareous sandstone while the rock from Doornik is classified as a layered limestone. Rock from Norway can be granite, gneiss, porphyry or Quartzite. Rock from Belgium is used predominantly in the province of Zeeland and in the South of Holland, while the rock from Norway is used in the North of Holland, Groningen and Friesland.

Caland (1833, 392) is probably the first one in the Netherlands who specifies some rules regarding the range in sizes and weight which should be adhered to in slope protection works for the two types of rock mentioned (Table 3.1). For rock to be used in closure operations the most important parameter is in fact the weight (mass) of the individual stone.

Though the rock from Vilvoorde is very irregular one can calculate from these figures by approximation that rock density ρ_r = 1 800 – 2 000 kg/m^3 for rock from Vilvoorde while ρ_r = 2 500 – 2 600 kg/m^3 for rock from Doornik. Nowadays (CIRIA, CUR & CETMEF; 2007, 110), one would say that the rock from Doornik has (more or less) a standard light grading of 40 to 200 kg and the rock from Vilvoorde is a 10 to 60 kg standard light grading. The stone from Norway has a density ρ_r = 2 600 – 2 900 kg/m^3 and is a 60-300 kg standard light grading.

These figures are worthwhile to remember when closures are analysed in Parts II and III of this dissertation.

Table 3.1 Range of acceptable dimensions and weights of sedimentary rock according to Caland (1833, 392) and of igneous rock from Norway according to Storm Buysing (1854, 537)

source of rock	type of rock	height (cm)	length (cm)	width (cm)	rock mass (kg)
Doornik (B)	limestone	20 – 40	40 – 50	40	80 – 200/210
Vilvoorde (B)	sandstone	24 – 40	30 – 40	10 – 20	13 - 60
Norway	granite, gneiss, porphyry, quartzite	30 – 50			75 - 300

If D refers to the square opening sieve size, the dimension of the equivalent cube D_n (m) is used as the nominal diameter in design, while:

$$D_n = 0.84\ D$$

The factor 0.84 is an average based on a great number of measurements (CIRIA, CUR & CETMEF, 2007, 107)[13] .
The median sieve size D_{50} , the median nominal diameter D_{n50} and the median mass M_{50} are related and determine the shape Factor F_s of a rock:

$$F_s = M_{50} / (\rho_r D_{50}^3) = (0.84)^3 = 0.60$$

F_s varies from 0.34 to 0.72 but a value of $F_s = 0.60$ is considered best practice.

3.3.1.5 Fascines, stakes, sticks, withes and binders

Watson (1875, 158 et seq.) states:

> "fascines, called in Dutch '*Ryshout*', are derived from copses of willows, osiers, etc. , on the low lands and islands in the estuaries of the Rhine and the Meuse. Willow, with a small admixture of ash and alder, is grown as a business, the crop being cut every third or fourth year. …… There are several sorts of *ryshout: Hollandsche, Brabantsche, Geldersche, Limburgsche, Schouwsche,* and *Gaasterlandsche*, so called from the different provinces where the wood is grown. Fascines of oak and hazel are procured from the higher districts, but the wood generally used is willow.
> *'Hollandsche ryshout'* is of this latter wood, very full of small branches and twigs, the main sticks growing tolerably straight and parallel. The largest are used for cask-hoops; the smaller are cut and tied in bundles."

It is noted that fascine is not the correct translation of '*rijshout*'; fascine is, like faggot, a bundle of brushwood (*rijshout)* of a certain type, the bundle having specified dimensions.
According to Caland (1833, 389) a bundle of '*Hollands Rijshout'* has a length of 3.50 m and a maximum circumference of 50 cm. For '*Brabants rijshout'* these dimensions are 2.00 m and 40 cm, respectively.
There are many more parts of the brushwood which have a function in fascine works. Caland, and also Watson, mention, inter alia, '*staken, tuinlatten en gaarden'* (stakes having lengths of 1.20, 2.20 and 3.00 m, respectively). Also are mentioned '*wiepband, knijpband en kruisband*' which Watson translates "as osiers to be used as withes and binders".
What is written above is only meant to be indicative. The dimensions quoted are those of the 19th century and the standards may have changed in time. There are also accepted tolerances in these dimensions and, moreover, the dimensions differ for each kind of *rijshout* .
The materials described above are used to construct the following 'composite' fascine constructions[14]:

- rubble or clay-filled fascine sausages (*doodemans)*
- suspended fascine mattresses ('*zeugen'*, translated as 'sows')
- floating fascine mattresses *(zinkstukken)*
- *pakwerken* (no proper English translation found)
- *baardwerken* (,, ,, ,, ,, ,,)

As these fascine constructions in the past have played a major role in closures in the Netherlands and nearby parts of Belgium, Chapter 4 is solely dedicated to these constructions.
For completeness sake it is mentioned that fascines are also used as 'fill' in cofferdams.

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[13] But see also http://resolver.tudelft.nl/uuid:f3204674-3a72-4515-92f5-92a8f91d8a15 .
[14] Here only the fascine constructions used in closure works are discussed but there are many more such constructions functioning as, or being part of, groynes, spurs, slope protection, quay walls, road foundations.

3.3.1.6 Straw

The application of straw in cramp matting for slope protection on dikes has already been mentioned in Appendix A (Section A.4) to Chapter 2.

Straw is also used on a limited scale in closure works:

- Vierlingh (1920, 325) recommends to apply cramp matting as an emergency protection to stop further scour of the base (*zate*) in a dike breach. Obviously, that would be a breach which runs dry during low tide. Such cramp matting only has to function during the short period that the labour, materials and equipment for the closure of that breach are being mobilized.
- Straw is sometimes used as a lining in a cofferdam type of bund to prevent soil (clay, sand) particles to be washed out by waves and currents.
- Straw is pressed in straw packs. These are placed as a core in a closure bund (Fig. 3.1) to economize on the use of sand bags (closure of the eastern breach in the sea defence of the Everingenpolder in 1953, Rijkswaterstaat (1961, 574 et seq.).

Because of the limited period the straw application has to function in a closure operation no requirements regarding quality of the straw stalks exist. Vierlingh (p. 331) recalls that for a breach near Middelburg (in 1530) the straw originating from the beds of deceased plague patients could aptly be used in the closure works!

3.3.1.7 Reed

Generally, in the Netherlands for reed a distinction is made between *groenriet* or *bladriet* and *droogriet* or *dekriet* [15] .

Groenriet or *bladriet* is harvested in summer. It is used including the leaves. It provides a denser cover than *droogriet* or *dekriet*. The bundles have a length of 2.50 m. It is used for light slope protection and in cramp matting. If in closures the bed of the tidal channel consists of fine sand, the middle layer in the fascine mattress is replaced by a layer of reed. The reed layer will prevent 'leakage' of the fine sand through the fascine mattress.

Droogriet or *dekriet* is harvested in Winter. In bundles 1/3 of the stalks must have a length of 2.25 m and 2/3 a length of 1.75 m.

The reed in the Netherlands has a limited tensile strength and is quite fragile (brittle). It implies that in a floating fascine mattress never all three layers of fascines can be replaced by reed. This comment is made because in Bangladesh it proved feasible to construct a floating protection mattress solely consisting of three reed layers and bamboo strings (*wiepen)* for under and upper grating.

3.3.1.8 Bamboo, palm leaves and millet stalks

The materials bamboo, palm leaves and millet stalks are not used for closure works in Western Europe. Bamboo and palm leaves have been used for closure works in Bangladesh. Millet stalks have been much used in closures in China. These materials will be discussed in Chapters 11 and 13 of this dissertation.

[15] It is not known whether the English language also makes this distinction; in any case *dekriet* is used for thatch (roofing) but thatch can also be straw or palm leaves.

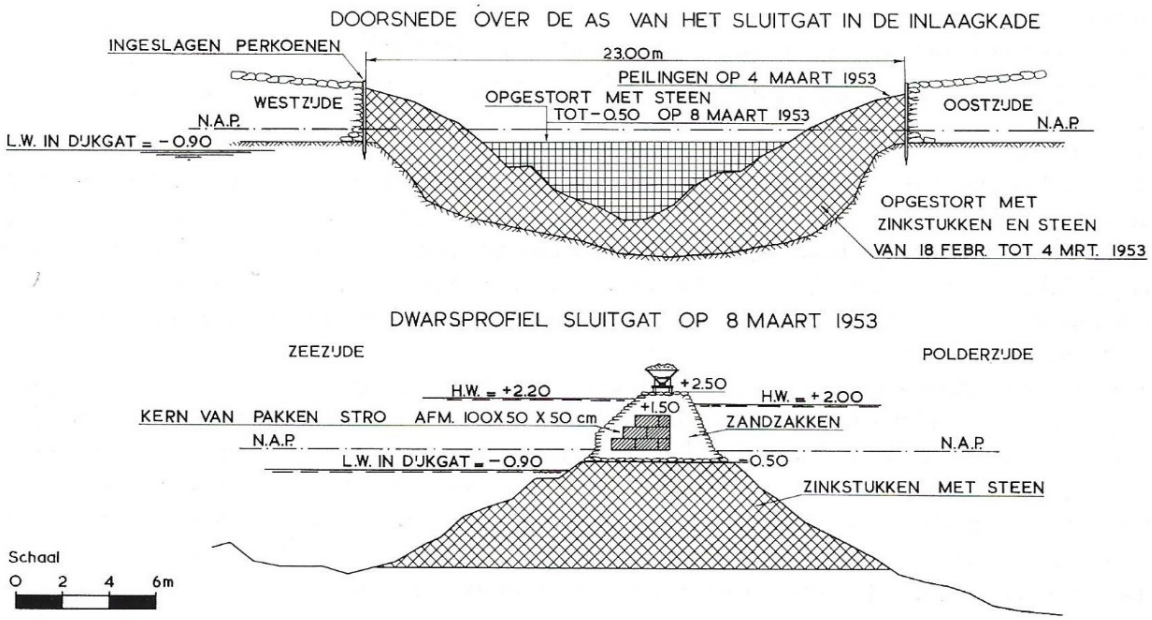


Fig. 3.1 Closure of the eastern flow gap in the Everingerpolder in March 1953; note the core of pressed straw packs in the closure bund of sandbags; source: (Rijkswaterstaat, 1961, 575)

3.3.2 Materials obtained by relatively simple processing or manufacturing

3.3.2.1 Bags

The bags come in various types and sizes. During the closure works after the storm surge flood of 1953 it was first thought that the so-called La Plata bags were ideal for the job but the fine *Scheldezand* leaked out of these bags. The most suitable jute bag turned out to be the so-called 'twill' bag made from second hand Kurachees or Cuba bags. The dimensions decided upon were 50 x 70 and 50 x 80 cm. Obviously, these dimensions are based on what weight people can carry over a certain distance. During 1953 in all 17 500 000 bags have been transported to the various closure sites (Rijkswaterstaat, 1961, 364 et seq.).

In various sources it is stated that bags to be carried have to be small and only to be filled to 2/3 of their volume. Deltares[16], in this respect, mentions bags[17] having a volume of 15 dm^3, filled by 10 dm^3 of sand which represents a mass of 15 kg dry sand or 20 kg wet sand. This results in 100 bags in 1 m^3. A similar standard sandbag is suggested by the US Army Corp of Engineers[18]: size 14" x 26" (35.5 cm by 66 cm); mass of sand in bag 18 kg, number of bags in one m^3: 110[19].

Further details shall be discussed for the individual closures in Parts II and III of this dissertation.

3.3.2.2 Rope and hawsers

Rope and hawsers are made of hemp, jute, flax or Manila hemp. Ropes and hawsers made of Manila hemp are stronger than those made up of the other fibres mentioned. But hemp and flax (growing in the Netherlands) and jute (growing in countries like India , Bangladesh and China) was apparently more easily available than Manila hemp (coming from the fibres of the leaves of the Abaca tree

[16] https://v-web002.deltares.nl/sterktenoodmaatregelen/index.php/Hoe_om_te_gaan_met_zandzakken (accessed on 20 Nov. 2023).

[17] Its dimensions (cm) are approximately 30 by 60 by 8.

[18] https://usace.contentdm.oclc.org/digital/collection/p16021coll11/id/0/ (accessed on 20 Nov. 2023).

[19] Actually, in the examples given this number varies between 109 and 112.

growing in the Philippines). In Bangladesh it is quite common to manufacture jute rope and hawsers, to the length and thickness as required, on the closure site (Fig. 3.2).

Fig. 3.2 Rope making from jute at Madargong Closure site, Bangladesh; source: BDWB (1978)

3.3.2.3 Hurdles

Hurdles (*dijkhorden)* are made of wattle work around 13 thin stakes (Fig. 3.3). According to Van Breen (1920, 73) they have a rectangular shape: length 2.30 m, height 0.70 m. They are for instance used to build a provisional cofferdam on the dike when overtopping is imminent. They are stored in sheds, built at regular distances along the dikes, containing materials for use in case of an emergency.

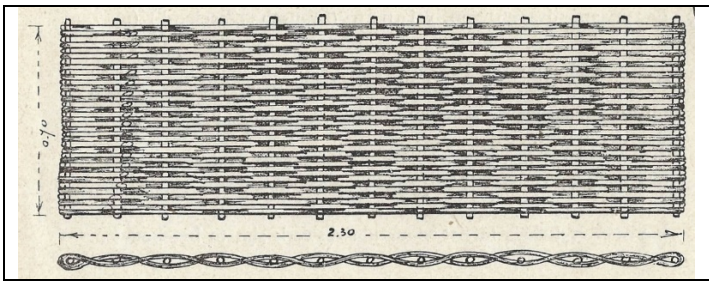


Fig. 3.3
Hurdle for use in dike maintenance and closure works; source: Van Breen (1920, 58, Plate 6)

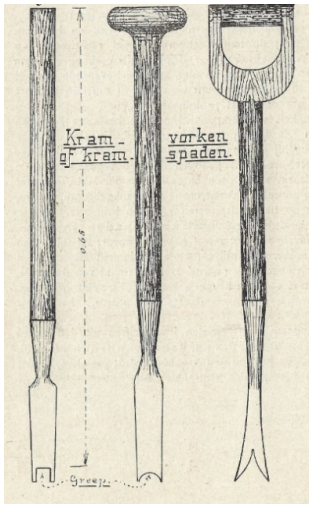
-3.4 Tools and equipment

3.4.1 Tools: shovels, forks, spades, sacks, baskets, barrows, aprons, head pans

In the past, having a certain tool at your disposal more or less determines the work you are doing. For the closure of the 'Grote Gat' near 't Hart (1510- 1511) Van Dam (1992, 453 et seq.) mentions the payment to 'pairs of workmen ', one man has a shovel, the second a fork. She suggests that the one having a fork is probably the *'ofzetter'* who spreads the dike fill, makes the layers of dike fill level and lays the sods.

Other workmen have no tool and carry in pairs a barrow, made available by the authority organizing the repair of the dike (the waterboard), for transport of dike fill from the ships to the closure gap.

During past centuries barrows are quite common for transport of dike fill. It is one of the items stored in the sheds built for dike maintenance and emergencies along the dikes.
Vierlingh (1920, 340) notes that it is more efficient to carry sacks filled with earth or sods to the closure gap than to carry baskets (*draagkorven, hotten, botten*) filled with earth.
Vierlingh (p. 330) also mentions large baskets (*groote packmanden*) filled with earth and sods which are used to stop the flow through drains and ditches at the land side of a dike breach. Apparently, these baskets were not emptied but used as heavy closure elements.
Spades are very important in dike construction. They are used for digging dike fill but also for cutting sods (to cover slopes) and clay clods for ballasting fascine mattresses or defending *slikkerdijken* (see Appendix A to Ch. 2, Section A.2). Spades come in various sizes and shapes (Fig. 3.4). A special type of spade is used for cramp matting. Also this spade comes in various shapes (Fig. 3.5).
For the sake of completeness one should also mention the tools used by women, who, when breaching of the dike is imminent, or has occurred, use their aprons (Vierlingh, 1920, 309) to carry earth.
The Arne dike (near Middelburg) breaches after Christmas 1517 and 71 yards of canvas (*93¾ el canevas)* are then bought by the authorities for making aprons (*scortecleeren).* See Kesteloo (1888, 358).
During the closure of The '*Grote Gat'* (large flow gap) in the Spaarndammerdijk in ± 1510 (Van Dam, 1998, 257) aprons are used to carry earth to the flow gap where it is dumped.[20]
In many countries soil (and other loads) were (and are) still being transported using head pans, see for example Fig 3.6 which shows dike building in Bangladesh.



Left:
Fig. 3.4
Different types and sizes of spades

Right:
Fig. 3.5
Cramp spades; source:
Van Breen, (1920, 55, Plate 5)

[20] The transcripted text reads in Dutch: *"Men sal gereet maken een getal van schortcleen om aerde in te draeghen, van daer men se op sal werpen ende die leggen ende in 't gat te werpen…"*

Fig. 3.6 Transport of dike fill using head pans in Bangladesh; the 'pillars' in the centre and at the right are kept for measurement of quantities excavated

3.4.2 Equipment: bicycles, wheelbarrows, shovel shaped sleds (*molborden*), carts, side tip wagons, boats, piledriving installations

3.4.2.1 Bicycles

Though it is not common in the Netherlands it should be mentioned that in Asian Countries like Viet Nam bicycles are utilized to transport loads like clay clods. These bicycles have been adapted to the work to be done, they do not have pedals and one can only walk with it.

3.4.2.2 Wheelbarrows

In 'De Geschiedenis van Zeeland' (Brusse & Henderikx, 2012, Vol 1, 202) it is stated that the wheelbarrow is being used for transport of dike fill from approx. 1300 onward.
Other sources suggest later dates. Stutterheim (2017), after having made a historical review, concludes that, until the 15th century, the wheelbarrow is a rare thing in Western Europe.
In 1610 Jan Claesz introduces the wheelbarrow for dike construction in Sleeswijk-Holstein. It gave him his surname: Rollwagen. (De Roever, 1944, 205).
Stutterheim also states that a wheelbarrow is six times as expensive as a barrow[21]. It could be a reason why wheelbarrows in the past are not as frequently utilized as barrows but one should also realize that pushing a loaded wheelbarrow up a steep (dike) slope is not very easy.
Another reason limiting the use of wheelbarrows is the need to have a system of paths made up by laying planks. Planks were expensive because wood was relatively scarce, in particular in the coastal areas. Planks at sites of dike repair start to appear after the Middle Ages in the northern Netherlands. The invention of the wind-driven sawing mills in the 17th century must have been important to drive the price down.
The expensiveness of a wheelbarrow is illustrated by the fact that the contractors constructing a dike section normally do not have wheelbarrows. But it can also be a matter of scarcity. The authorities organizing the work then purchase these wheelbarrows in one lump and make them available to the

[21] The Oxford Dictionary gives as one of the meanings of barrow: rectangular frame with short shafts used by two or more men for carrying loads on, stretcher, bier;

contractors. This is for instance the case for the reclamation of the *Wijde Wormer* in 1624 when 600 wheelbarrows are required. These cannot be obtained in Amsterdam. In Dordrecht, however, 470 nos. are available but these wheelbarrows turn out to be of inferior quality. Ultimately 200 nos. of reasonable quality can be bought from a timber-merchant in Barsingerhorn (Van Zwet, 2009, 94).

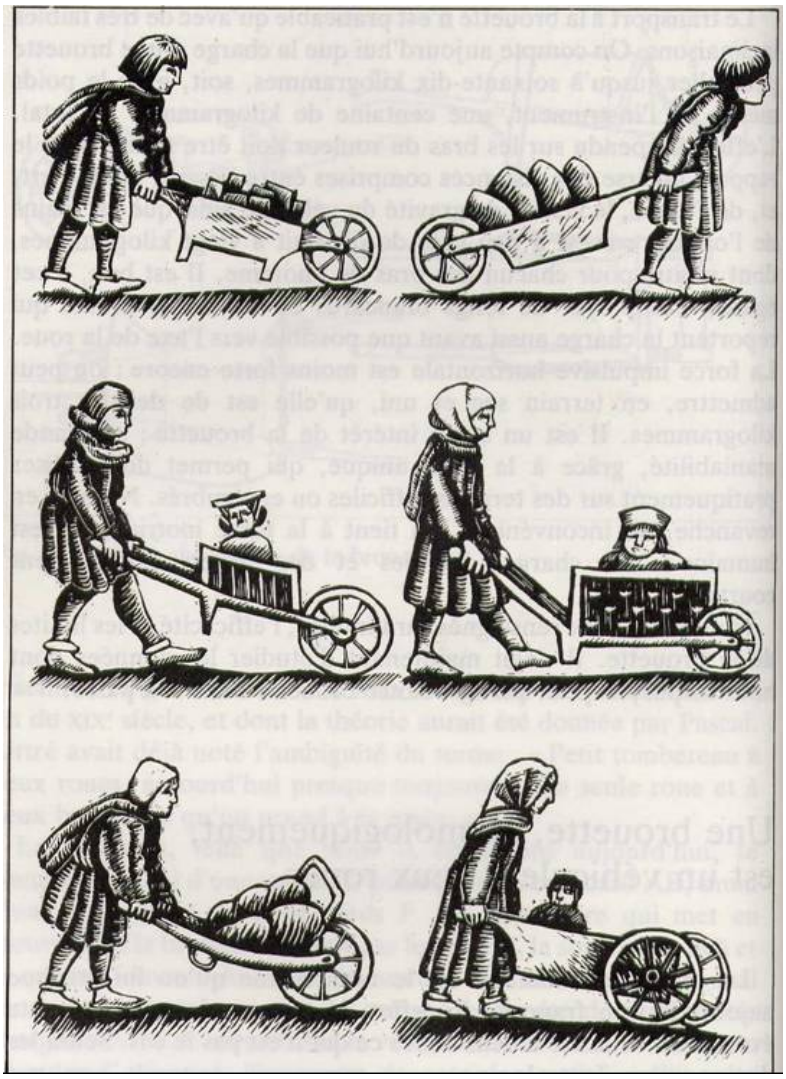


Fig.3.7 A review of different types of wheelbarrows in the 13th century; source: Gille, (1983)

Fig. 3.8
The use of wheelbarrows at the Noorderdijk near Andijk in 1916; source: Schilstra (1987, 122)

3.4.2.3 Molborden (shovel shaped sleds)

In 1553, Vierlingh (1920, 292 et seq.) is asked to give advice about the reclamation of the Zijpe in North- Holland.

As part of the future dike, a temporary bund, a so-called *voor-verskade* (Fig.3.9) is being constructed for which sand fill is used. This sand fill is dug and transported using '*molborden*'. That is a kind of large shovel (s*chop*) drawn by two horses. Vierlingh recommends a more speedy completion of this bund and within a few days 1000 horses are working at the site.

The *molbord* has a handle which allows the driver to scoop the sand on the *molbor*d (boarded at three sides and measuring 9 by 12 dm). The horses then pull the *molbord* as a kind of sled across the sand to the bund where it is emptied. Obviously, *molborden* can only be used when the soil is very sandy. They cannot be used on the fat clay of the tidal flats.

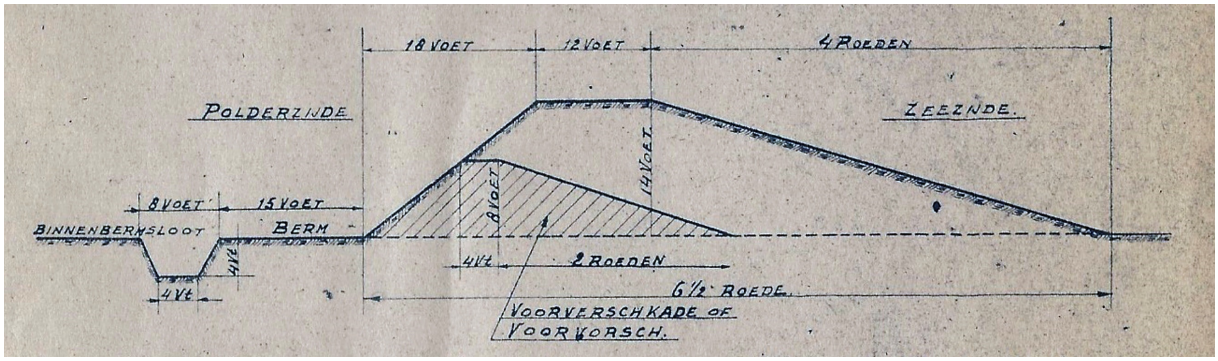


Fig. 3.9 Profile of dike of the Zijpepolder (± 1660); source: De Jong (1937, 376)

3.4.2.4 Carts drawn by horses

Provided the subsoil is not too soggy (*drassig*), carts drawn by horses can be used to transport dike fill. Horses and carts also bring about a certain degree of compaction of the fill.

At the time of a storm surge which may lead to a dike breach all farmers in the polder (as well as other inhabitants) are summoned to come to the dike taking with them such items as shovels, spades, straw, brushwood, planks, hurdles, wheelbarrows and sails as well as their carts and horses (if available). See for instance Gottschalk (1977, Vol. III, 230) and Sjoerds (1767, Vol 2, Part 1, 462).

In 1682 the dike at Westkapelle is seriously damaged and the Government of the province of Zeeland, at the request of the authorities of the island of Walcheren, orders the farmers (though it is harvest time) to come daily to the dike with 150 carts and horses. If the farmers don't come, they are fined and, moreover, carts and horses are hired at their cost (Gottschalk, 1977, Vol. III, 310).

3.4.2.5 Side tip wagons[22]

Side tip wagons (Fig 3.10) are in use on construction sites since 1870 (Niemeijer, 1975, 14). One wagon can contain 2 m^3 of material (clay, sand, gravel, etc.) and are drawn by horses or locomotives. During periods of much unemployment the wagons are pushed by labourers. The trains run along narrow gauge rail tracks. In 1871 the tidal channel Sloe is dammed and side tip wagons drawn by a locomotive transport the dike fill for the final profile of the Sloe Dam (Neyt, 1872, 223).

[22] One can argue that a steel side tip wagon on a narrow gauge railway track and pulled by a locomotive cannot be considered as simple means. But in the closures described in Part II of this dissertation such side tip wagons in a number of cases were loaded by hand, pushed along the railway track by manual labour and tipped manually.

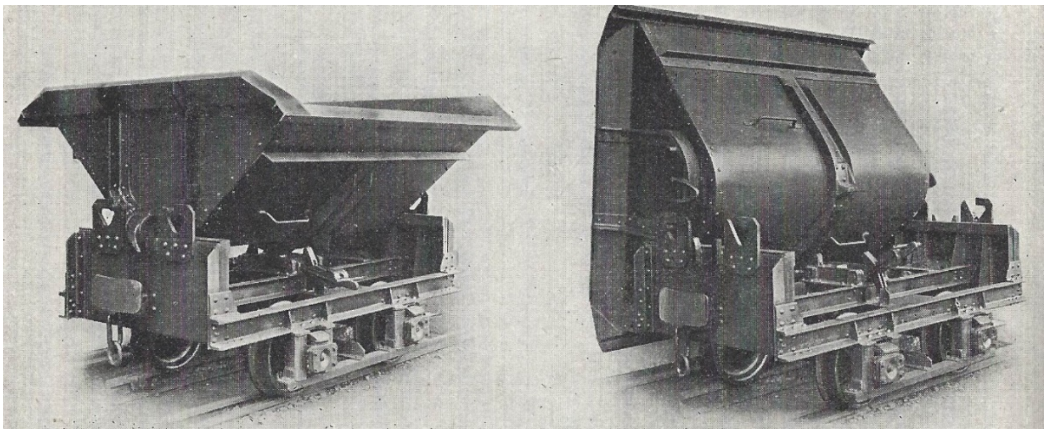


Fig. 3.10 Side tip wagon (2 m^3) for narrow gauge rail tracks on building sites; source: Visser (1946, 248)

3.4.2.6 Boats

When a dike breach is not accessible for carts because of the flooding caused by the breach or there is no dike fill available in the neighbourhood of the breach, boats will have to transport the fill from elsewhere. Again, the magnitude of such operation can be enormous. For instance, Van Dam (1992, 457) speaks about a fleet of more than 100 ships of all types and sizes (fishing boats, rowing boats, etc.) transporting the dike fill required for the repairs at the Spaarndammerdijk in 1510.[23]

3.4.2.7 Piledriving installations

Piledriving might be necessary as part of a closure operation. One may need to drive (sheet) piles in order to construct a cofferdam or one requires a jetty as temporary works to enable certain closure operations to proceed (see below in Section 3.5). Obviously, the pile hammer is operating thanks to manual labour.

The peaty sub soil in the towns and cities in the province of Holland requires a piled foundation for buildings. Accordingly, at an early stage, the art of pile driving is well developed. In this respect it is not surprising that one finds (sheet) piledriving for closures predominantly in Holland (including West-Friesland) rather than in Zeeland where piled foundations usually are not required.

Piledriving on the water requires a broad base while boats are small and not very wide. The solution is the catamaran consisting in this case of two boats. The similarities between the floating piledriving installations in Holland (Fig.3.11 ,18th century) and Bangladesh (Fig.3.12, 1982) are remarkable.

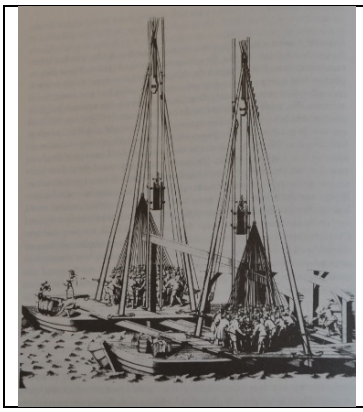


Fig. 3.11 ←

Piledriving in Holland 18th century; source: Van der Horst & Polley (1736)

Fig. 3.12 ←

Piledriving at Amtali closure, Bangladesh; source: BWDB (1982)

[23] In December 1570, after the All-Saints'- storm surge, the waterboard Polder Schouwen obtains the privilege for the requisition of ships and boats required for transport of sods. Mentioned are in this respect: *"…plat becken, cromstevens, drijrnmeleers, hooukers, Vlaemsche Seuyen, kaeckscbuyten, veynschuyten, vletscbuyten ende alle andere, mogende laden oft voeren van twee schacht zooden tot acht schachten incluys,…."* (Fokker, 1909a, 77).

3.5 Temporary Works

The only item of any significance used for the type of closure works as temporary works here discussed is the jetty across the closure gap. Obviously, there are other temporary works on closure sites, like tool sheds, temporary living accommodation for the labourers, offices for the work supervisor, surveyors and paymaster, stalls for the caterers (*zoetelaarsters)*, but discussion about these temporary works is beyond the scope of this dissertation.

A jetty across the closure gap may serve the following purposes:

- to act (in so far as already constructed) as a support of the machine driving the piles of the jetty. Figure 3.13 shows a piledriving dragline[24] on a jetty being constructed,
- to carry a narrow gauge railway[25] for transport and dumping of closure materials (clay, sandbags, rocks) by a train comprising a locomotive and side tip wagons (Fig. 3.10),
- to carry a machine driving a row of sheet piles for a wall or piles for a cofferdam,
- to act as support for the said row of sheet piles or for a frame work for sliding gates (see for instance Chapter 17, Section 17.4),
- to function as a walkway for labourers carrying sandbags or bags filled with clay to be dumped in the closure gap (Fig. 3.14),
- to form part of a cofferdam to be filled with clay, straw packs, brushwood, sandbagging (Fig. 3.14).

Fig. 3.13 Piles being driven for a jetty across the closure gap in the dam closing off the tidal inlet Dijkwater from the Grevelingen (23 July 1954); source: photo De Feyter

Fig. 3.14
A jetty across the closure gap has been transformed by means of bamboo stakes (see left) into a cofferdam; labourers dump bags filled with clay; Amtali closure, Bangladesh; source: BWDB (1982)

[24] Obviously, a dragline is not considered to be a simple means, but not all items in a closure by simple means have to fall within the definition.
[25] Same comment as above.

CHAPTER 4 'Composite' materials and constructions based on the use of fascines

4.1 Introduction

In Chapter 3, Sub-section 3.3.1.5, fascines and other basic closure materials of vegital origin[26], like stakes, sticks, withes and binders, have been shortly discussed.
The 'composite' fascine constructions mentioned in that particular Sub-section, i.e.:

- stone or clay-filled fascine sausages (*doodemans)*
- suspended fascine mattresses ('*zeugen'*, translated as 'sows')
- floating fascine mattresses *(zinkstukken)*
- *pakwerken* (no proper English translation found)
- *baardwerken* (,, ,, ,, ,, ,,)

are discussed below in Sections 4.2 to 4.6 (incl.)

4.2 Stone or clay-filled fascine sausages

4.2.1 Literature review

The stone or clay-filled fascine sausages (hereinafter also called sink-fascine), discussed in this section, have been developed in the course of the centuries in many different countries. Accordingly, many different names exist for a construction which will comprise materials and have dimensions solely determined by local conditions and a particular application. Table 4.1 presents an overview.

Table 4.1 Names and particulars of stone or clay-filled sausages in various countries

name	country	'skin'	'fill'	size[27]	references
doodeman, dooman, waese	Netherlands	fascines	clay, rubble, bricks, rocks	10 x 0.5	Gerhardt (1988, par. 242); Nebbens (1771, 51); Vierlingh (1920)
Senkfaschine, Sink-walze, Sinkwelle	Germany	fascines	coarse gravel, small rocks	(8-100) x (0.8-1.0)	Kreuter (1910, 174 – 180) Von Wiebeking (1811, 601)
fascine pole	England	no details available			Kurz (2004)
saucisson	France	fascines	no details available		Bossut & Viallet (1764, Planche III)
salciccia	Italy	fascines	no details available		Box 4.1 in this dissertation
mata	Bangladesh	palm leaves	straw, earth	(15-35) x 0.9	BWDB (1976)
'rolled kids'	China	millet stalks	earth, gravel	20 x (1-2)	CNCD (2005, 85), Appendix J
'willow-stone sausage'		fascine strings	stones	(10-18) x 1.0	Kung (1936, 19)
Yakago ('stone basket')	Japan	bamboo netting	boulders	10 x (0.7-1.2)	De Rijke (1900, 8)

Below, some information collected about these fascine sausages is presented.
During a meeting on 29 April 1862 at the Institution of Civil Engineers in London, U.K., three papers were presented regarding sea dykes and reclamation works[28].
After the presentation of the papers by the authors, the papers were commented upon by those present at the meeting. Mr. Vignoles[29] was one of the speakers. His statement is presented here in full (Box 4.1).Obviously, not everything in Mr. Vignoles' story necessarily has to be correct. He also is

[26] These naturel fibres, if kept permanently under water (i.e. below LWS), will not rot or deteriorate. In case they are above LWS (for instance on a slope) they have to be renewed periodically. Still, the naval ship worm may do serious damage but one tries to limit the damage by embedding the fibres by stones or clay.
[27] Length (m) x diameter (m).
[28] The authors of the papers were: Paton (1862), Oldham (18620 and Muller (1862).
[29] Mr C.B. Vignoles (1793 – 1875) was an influential British railway engineer who, during a long professional life, designed many railways and railway bridges in the U.K as well as in other countries (among these: Germany, Ireland, Spain, Brazil, Ukraine). During his stay in Germany he became acquainted with the Hamburg City engineer Heinrich Hübbe. The latter probably told him about the fascine works in Schleswig Holstein. When Vignoles had problems with scour around bridge piers in Kiev, Hübbe advised to employ Dutch '*rijswerkers*' in order to have them construct a fascine bed protection round the bridge piers.

rather vague about the transition of a *salsiccia* into a *zinckstuk* . But it is worthwhile to read the underlined sentences.

Subsequently, the following was found in the literature:

Gerard Mulert makes a report of an inspection field trip after the storm surge of 1530 and notes that in Zierikzee people are complaining that it is forbidden in Vlaanderen to export 'sinckhoute' which are required for dike repairs; see Fruin (1908, 163 et seq.). Most probably fascines are meant which can be used to make sink-fascines.

Cornelis Meijer, a Dutch hydraulic engineer, working in Italy, published a book in 1696 having as title 'Nuovi Ritrovamenti'. This was in fact a new extended version of a book having the same title published in 1689. Meijer, describes, inter alia, the repair works to the breakwaters which he undertook at the port of Pesaro after the storm of September 1695. He did not replace the cofferdam-like spurs but placed fascines on the (shallow) foreshore outside the harbour basin and the destroyed spurs. Subsequently, rapid siltation followed. Meijer does not explicitly mention the use of sink-fascines but he placed these fascines under water and states that no mattresses were used. The original text, a translation in Dutch and his sketches are presented in Appendix E to this chapter.

Adriaan Bommenee, a self-taught engineer, wrote a 'Testament', completed in 1750, and he describes (Gerhardt, 1988, par. 242), what he calls a '*dooman*':

> "When an eroding foreshore is too steep to place fascine mattresses, it is possible to protect the slope with '*doomans*'. That are bundles of brushwood (*rijshout*) placed turn and turn about and loaded inside with stone……"

BOX 4.1 Mr Vignoles' statement at the Institution of Civil Engineers, London on 29 April 1862[30]

Vignoles had practised the system described in the last Paper. It had been correctly stated by the Author, that the principal difficulty in reclaiming land did not consist in making the embankment, but in carrying it across the creeks and gullies, or deeper parts, where it was liable to sink, and to be breached. Until within the last seventy years, the mode of closing breaches, and of making the dams across these creeks, or gullies, adopted in most countries, was to proceed from both sides, by driving one, or more rows of close piling from each side towards the deepest part, until the aperture in the middle became so narrow, that the closing became gradually, more and more difficult to effect, and at last, from the great depth of water, was almost impossible. About a century ago, in consequence of the difficulties and failures which arose from its employment, it was found necessary to have recourse to some other method. The history of this subject as recorded in the archives of Holstein, Hamburg, and the Low Countries, was extremely interesting. Until 1789 the Engineers, who directed the works, considered it an innovation, to attempt to close a breach, otherwise than by piling across it. The more intelligent of the country people, however, contended that the best method was to use bundles of fagots, or fascines filled with as much gravel or broken brick as could be bound up in them.[31] After this controversy had existed for some time, some of the leading Engineers had the common sense to adopt the suggestion of the country people, and eventually, they so far improved upon it as to produce, what might be considered as the perfection of the new system, namely, that of protecting the bottom of the gap, with an elastic covering of similar materials, but formed into what was now called a 'mattress.' Instead of narrowing the opening by working from the sides, they made it shallower, by laying the mattresses across the entire extent of the gap. They thus filled it up horizontally. In proportion as the bottom was raised, the velocity of the water was decreased, until a certain level was attained at which the velocity became so small as to enable them to close the breach without difficulty. Subsequently this system was almost universally adopted in Holstein and in the north of Europe. The employment of fascine work, or of bundles of bushes, mixed with gravel or broken brick, bound up tightly, and heavy enough to sink in a stream, and to remain in the position in which they were deposited, must have been common in Holland and the Low Countries, at a period of at least three centuries, before its revival under the preceding circumstances. These materials had probably been used in the first instance from want of stone, and had then been found to answer the purpose better. It was recorded that the Italian Engineers had used these materials about that period, and that they gave the fascines the name of 'salsiccias' which was subsequently changed in the north of Europe to the present name of ' zinkstucken.' In the fifteenth century the Spanish Engineers also adopted the employment of these fascines for military and other works, the Italian name being retained under the French expression ' saucisson.' The system appeared, however, to have been forgotten in the Spanish Peninsula, as the old method of piling had been advocated there very recently. It was singular, that the art of employing fascines for closing breaches should have been neglected, until it was forgotten by the technical men in the Low Countries. No doubt its revival was due to some traditional idea on the subject, which had been kept up among the working class of the country people.

[30] From: Minutes of the Proceedings of the Institution of Civil Engineers, Vol. 21, 1862, pages 478-479.

[31] Underlining added by Van Duivendijk.

A similar description is given by Nebbens in his prize-winning submission on the origin, prevention and repair of flow slides (Nebbens, 1771, 51). He calls such a fascine a '*doodeman*':

> "…which '*doodemans*' are made of fascines, ; the whole is bound round or spirally with osiers, and loaded inside with bricks, or better with rocks from Vilvoorde or other quarries, in such manner that, along a uneven surface, they will roll to the lowest spots, which they will fill up and so make an even level on which then clods or bricks are dumped."

Fokker (1908) gives numerous quotations from documents in the archives of the water board Polder Schouwen: in the quotations from the 18th century the use of *doomans/doodmans/dooijemans* is mentioned nine times. In nearly all cases they are used in stabilizing a slope after a flow slide.
The next quotation in this Section is from Van Breen (1920, 235-236):

> "In foreign countries thick fascine strings (*wiepen*), called sausages, are being made for sea defences and river training works. They are filled with gravel, rock or rubble and instead of osiers wire is used for binding. Because of their heavy weight they cannot be transported and they are made in situ. Layers of these sausages are fixed to the beach or bank by means of stakes. But it is not a very sustainable protection against wave attack."
>
> "Short thick sausages, also called sink- fascines, can be very well used to fill up not too deep holes in a slope below the water level. They are sunk with their length perpendicular to the slope. These fascines have to be transported and therefore should not be too long or heavy."

Subsequently, Van Breen refers to the *doodemans* , promoted by Nebbens in his 1771 publication, mentioned above.
Last, but not least, reference is made to the 'Tractaet van Dyckagie' by Andries Vierlingh (1920, 439) as annotated and published by De Hullu & Verhoeven. In the Tractaet repeatedly mention is made of standing and lying w*aesen.* It is stated in the Glossary, made by De Hullu & Verhoeven, that the word *waese* most probably originates from the German word *Wase* which means *Bündel, Faschine,* while *Wasen* can be '*ein Bündel Reiser'* (Schiller und Lübben, Mittelniederdeutsches Wörterbuch).
De Hullu & Verhoeven, admittedly, cannot explain what type of constructions are meant by the expressions standing *waesen* and lying *waesen* . But, the reader, having read the statement given by Vignoles as well as the quotations from Gerhardt, Nebbens and Van Breen may assume that in fact standing and lying sink-fascines (*doodemans*) are meant. The next step is then to find out where and when these constructions were actually used. For lying *waesen* (*doodemans)* this will be done in Sub-section 4.2.2.
Vierlingh (1920, 327) explains that, in case it is impossible to construct a bund around the perimeter of the scour hole (which is formed by the heavy current in a dike breach) and, moreover, application of lying *waesen* in the heavy current is also out of the question, standing *waesen* may be applied. But that is not easy and, to prevent the current from moving the *waesen,* immediately after launching, sods should be dumped against the *waesen* at the inland and sea side. Vierlingh suggests that only a thickness of sods of a few feet is required because the standing *waesen*, if made well and loaded with stones, will not be lifted up because of their weight.
In fact Vierlingh describes the start of, what is called, a 'horizontal closure'. This closure method will be discussed later in more detail (Chapter 5).
The standing *waese* is also used to prevent a dike breach from getting wider. After breaching, the dike heads are protected against erosion by the current by placing standing *waesen*. It is very well demonstrated in Figure 4.1. One observes that apparently, some kind of fabric (canvas? jute bags?) has been used to prevent the stones, sods or rubble from falling out of the waese.
In a French book (Bossut & Viallet, 1764, 29) fascine works are, inter alia, discussed (Des Revête-ments en fascinage, Ch. II, Sect. III,). One of the applications mentioned in passing (p. 30) is the

protection of a steep slope using saucissons. The sketch in the book (Planche III, figure 32) is clear and copied here for easy reference as Figure 4.2.
Vignoles translates the rubble or clay-filled fascine sausage into French as saucisson. Larousse (1952, 925) gives, as one of the meanings of saucisson '*fascinage fait de branchages liés en forme de boudin*'. An English-French Technical Dictionary (Kurz, 2004) notes: 'Fascine pole (*saucisson*): a fascine imprisoning a gravel core'.
In Part III of this dissertation it will be seen that constructions, very similar to the *doodeman*, have been used until recently in Asian countries (Bangladesh, China and Japan).

Figure 4.1 Repair of a dike breach at Vianen , river Lek, in 1624; the perimeter bund (*vingerling*) is under construction in the background; the dike head at the left has been protected, using standing *waesen*, against erosion by flowing water, immediately after the breaching; source: Atlas van Stolk, Rotterdam

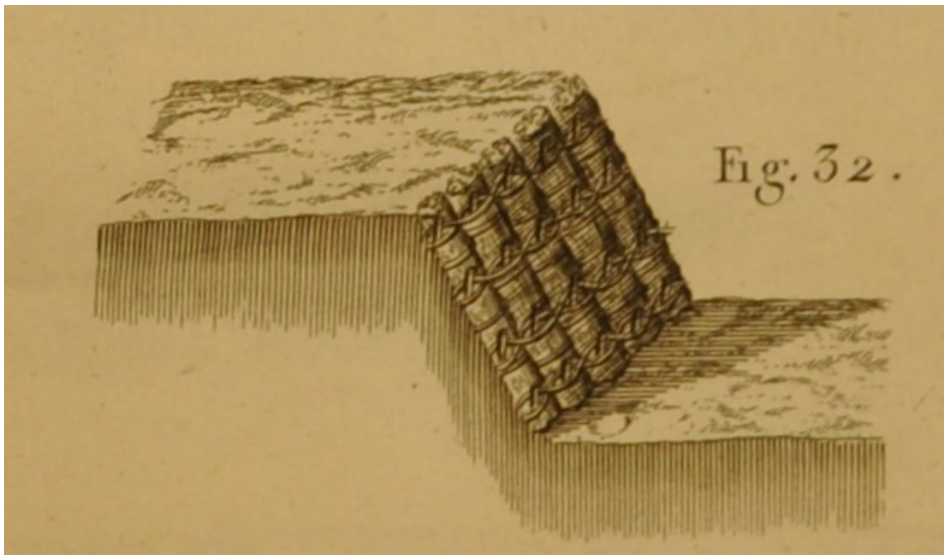


Fig.4.2 Protection of a steep slope applying saucissons; source: Bossut & Viallet (1764, Planche III)

4.2.2 Design of a fascine sausage (*doodeman*)

From the literature review above it follows that a fascine sausage (from now on called **sink-fascine**) either is made in situ or is constructed at a certain (dry) location and then transported and/or rolled down a slope into its final position below the water level. In the first case the sink-fascine will be heavy and difficult to handle by manual labour. In the second its dimensions must be such that handling is possible by a reasonable number (say 30 to 50 depending on its length) of labourers. This pre-fabricated sink-fascine must satisfy the following requirements:

- it must have a more or less cylindrical shape,
- its overall dry density must be more than 1 000 kg/m^3, i.e. so much higher that it will resist movement by the prevailing current and does not start to float,
- the sink-fascine must be flexible but also of a solid construction to prevent 'leakage' of the 'fill' from the core or disintegration of the bundled fascines.

In Box 4.2 a detailed calculation of a sink-fascine having a diameter of 53 cm is presented.

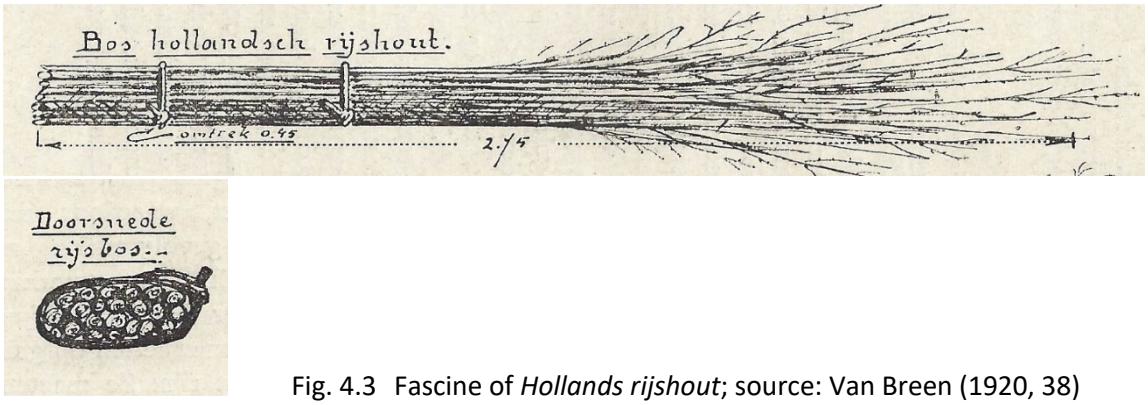


Fig. 4.3 Fascine of *Hollands rijshout*; source: Van Breen (1920, 38)

Below, in Table 4.2, the values of the relevant parameters for the design are presented for different values R of the core.

Table 4.2 Particulars of a sink-fascine for various values of the core radius R; see calculations in Box 4.2

R of core (m)	downward force $F_D / \pi g$	uplift force $F_U / \pi g$	F_D / F_U	diam. sink-fascine (cm)	weight sink-fascine (N/m^1)
0.12	6.25	6.13	1.02	53	643
0.15	9.76	7.11	1.37	60	1 005
0.20	17.36	8.74	2.0	70	1 786
0.25	27.12	10.37	2.62	80	2 791

From the figures in this Table it would appear that a diameter of 60 cm is a maximum for a sink-fascine which is to be transported manually.

There is a distinct difference between standing and lying *waesen* (as outlined by Vierlingh).

The standing *waese* is a *sink-fascine,* which has to be pre-fabricated, imprisoning a gravel core at the thick end. It then has to be transported manually to the spot where it has to be placed in a more or less vertical position. As this is all done by hand its dimensions (and weight) have to be modest. A diameter of 50 cm by a length of 4.1 m would appear to be the maximum that can be achieved.

Vierlingh is rather vague about lying waesen. It is not always clear whether he means a sink-fascine or a much larger construction, as described in Section 4.3: the suspended fascine mattress. In a number of cases (for instance on p. 154/155 of the '*Tractaet*') he speaks about '*waesen met welinge af te werpen*', and this would imply the suspended fascine mattress. But also application of the sink-fascine sec as bed protection is suggested.

Box 4.2 Design of a sink-fascine

Van Breen (1920, 235- 236) describes a *zink fascine* (sink-fascine) as a short thick sausage and notes that it is called *dodeman* by Nebbens (1771, 45 et seq.). But both authors do not provide dimensions. The sink-fascine is also mentioned in Maschek (1863, 73-74). Apparently, in the 19th century it used to be part of military defences. Mentioned are a *waterfascine (*long 2.5 to 3 m, diam. 32 cm) while a shorter type (long 2 m, diam. 32 to 48 cm) is called *waterfascine* as well as *zinkfascine*. But nothing is said about its application or a gravel core.

Below, for all practical purposes a sink-fascine is being designed on the basis of *Hollandsch rijshout* and a core of bricks/stones/gravel.

A fascine of this type of brushwood (*rijshout)* has a length of 2.75 m and a circumference of 45 cm (Fig. 4.3), which implies a diameter of 14.3 cm. It should be possible to make a sink-fascine having a length of 4.10 m by placing the fascines alternately (the thicker end and then the top).

When R is the radius of the brick core; its volume is πR^2 m^3 per linear m. Bricks in the Middle Ages will have had a low density, say ρ_{br}= 1 450 kg/m^3, while for salt water ρ_w= 1 016 kg/m^3.

Ignoring for the time being the voids, one comes to a downward force F_D under water of:

$$\pi g \times (1\,450 - 1016) \times R^2 \text{ N/m}$$

One layer of fascines around this core would have a volume of:

$$\pi \{(R + 0.143)^2 - R^2\} = \pi (0.287R + 0.0196) \text{ } m^3 \text{ per linear m.}$$

According to Caland (1833, 214 et seq.) the volume of fascine material in 1 m^3 is 0.2925 m^3 while the density of the fascine material ρ_f = 628 kg/m^3. Accordingly, the uplift force F_U of the fascine 'cylinder' around the core of bricks is:

$$\pi g (0.287R + 0.0196) \times (1016 - 628) \times 0.2925 \text{ N/m}$$

The downward force F_D has to be larger than the uplift force F_U, i.e. :

$$F_D / F_U > 1 \longrightarrow (\pi g \times (1\,450 - 1016) \times R^2 / \pi g (0.287R + 0.0196) \times (1016 - 628) \times 0.2925 > 1$$

or

$$434 R^2 > (0.287R + 0.0196) \times 113.49$$

Now determine the value of R from:

$$R^2 - 0.0750 R + 0.0051 = 0 \longrightarrow R_1 = 0.1182 \text{ m}, R_2 = -0.0432 \text{ m}$$

So the minimum radius would be R = 12 cm while the diameter of the sink-fascine would be 53 cm. Its weight above water[32] would be: $\pi g \times 1450 \times (0.12)^2$ N/m = 643 N/m.

But: there are a number of uncertainties like the density of the bricks and the percentage of voids in the brick mass. It is therefore recommended to check the value of F_D / F_U for other values of R. This has been done in Table 4.2.

4.3 The suspended fascine mattress

In his Tractaet van Dyckagie, Vierlingh describes several times how lying *waesen* (sink-fascines) are used as a kind of bed protection and as a fascine dam in so-called vertical closures. He also describes the closure of the final gap by, what could be called, a suspended fascine mattress. This name is chosen in order to distinguish it from the floating fascine mattress (described in Section 4.4).

Below, this suspended fascine mattress, comprising fascines, fascine strings (*wiepen*), ropes, stakes, wattle work and clay clods, is discussed.

[32] Ignoring the weight of the fascine material.

The descriptions given by Vierlingh (see, for instance, page 160 in the Tractaet) are not always clear but an effort has been made to combine the information given on various pages:

- At the location where the suspended mattress is required a number of long *weliën* is spanned parallel to each other between the bank and a ship or between two ships. Probably, weliën are cables made of brushwood (like *wiepen*), but it might also be ropes or hawsers[33] and this stands to reason because the tensile force on the weliën will be considerable and *wiepen* wil not endure a high tensile force (see in this respect Van Breen, 1920, 59-60). Also Boddaert (1753, 262-263) speaks about ropes (*touwen)*.
- After spanning the *weliën* , *waesen* (fascines) are rolled, pushed and shoved onto the *weliën* using long stakes and poles.
- A few *waesen* are laid parallel to the *weliën* for *'rijswerkers'* [34] to stand on it as they will connect the lying *waesen* by wattle work (*tuinen*), stakes and ropes making the whole into a suspended fascine mattress; they will also place additional *waesen* (a second layer so to say) and as many clay clods in between the layers as necessary to lower the mattress by sagging to the water level.
- Finally the suspended mattress is immersed by easing off all *weliën* simultaneously.

No doubt the last action is a tricky operation. Though Vierlingh does not state it explicitly, the sinking operation will be carried out at the turn of the (low) tide.
Apparently, no sketches or drawings exist of the work as described above but photographs of a similar construction made in China are presented in Part III of this Dissertation: Chapter 13, Figures 13.12 to 13.14 (incl.) refer.
Smallegange (1696, 197) describes the suspended fascine mattress as follows:

> "The sea works are being made of fascines, laid on a number of long cables and then filled with good, tough, and stiff sods; on top of which comes another layer of fascines: then, finally, the cables put together and well closed, the whole construction is sunk. As such the art is to have a good relationship between weight and fascines. It is better to have stone inside these 'sink-works' than sods , but many polders consider this too costly." [35]

The best description was found so far in Schoorl (1973, 204 et seq.) The author describes the bad state of the coastal defence at Huisduinen. In 1664 a committee is sent to the island of Walcheren to visit the dike at Westkapelle and to learn how that dike is protected against attack by the North Sea. During their visit the committee members attend the sinking of a large *rijswerk,* locally called 'sow' *(zeug)* at a depth of 13 fathoms (= approximately 24 m).

A commissioner of the Waterboard Zuid Watering of Walcheren, Gerrit van Laecken, is then invited to advise about the works to be carried out at the coast near Huisduinen. Subsequently, it is decided to protect near Huisduinen the spurs and the seaward slope of the dike sections in between against scour by placing a total of 181 'sows' (Fig. 4.4). In the report of the committee (dated 27 March 1664) the working method applied to construct and sink the 'sows' is described

[33] In the Glossary to the Tractaet on page 440 it is mentioned that the word *wele* appears in G. van der Schueren "Theutonista of Duijtschlender' and means rope, cable.
[34] *Rijswerkers* are workmen skilled in the application of all kind of fascine works.
[35] The original Dutch text has been presented in Appendix B to this dissertation. According to De Hullu & Verhoeven (in Vierlingh, 1920, 439) this quotation can also be found in "Staatkundig Zeeland", written by Anthonis de Jonge van Bruijnisse (1607 – 1669). Moreover, also Bosch (1969, 52) quotes it in his book.

in detail (p. 207, footnote 23). For a good understanding of the construction and placing of the suspended fascine mattress this description[36] is translated and presented here in full:

> "Two ships, firmly anchored at the location of the mattress to be made and placed, are connected to the shore by hawsers; they are connected to each other by beams. One ship is loaded with fascines *(rijs)* , the other with stones, Between both ships an opening remains of 15 to 16 ft. (± 4.5 m). At equal distances 6 hawsers (16 to 18 cm thick)[37] are stretched between the two ships and firmly fixed to the bollards. The hawsers get a sheer (*zeeg*) of 7 to 8 ft (± 2 m). In the same manner 24-25 fascine strings (*wiepen)* are stretched between the ships. These fascine strings have been made using bitter willow (*bittere wilg*).
> Subsequently, a large labour force places fascines lengthwise over the hawsers and fascine strings to a thickness of 2 to 3 bundles. This mat of fascines is then made into one unit by four rows of wattle work using 1.25 m long stakes. In the next step stones are dumped on the mat until the top of the mat has sunk down to the water level. This is followed by the placing of two layers of fascines lengthwise and transverse. The three layers are connected to each other using stakes and wattle work. This results in a 'sow', thick ± 2 m, having a width of ± 3.4 m and a length of ± 20 m. Finally, the 'sow' is well girded using willow strings and at the call 'everywhere' all hawsers are eased off, in such manner that the suspended fascine mattress sinks uniformly to the seabed."

This type of protection against scour near Huisduinen is still in use in 1730 (p. 291, 294).

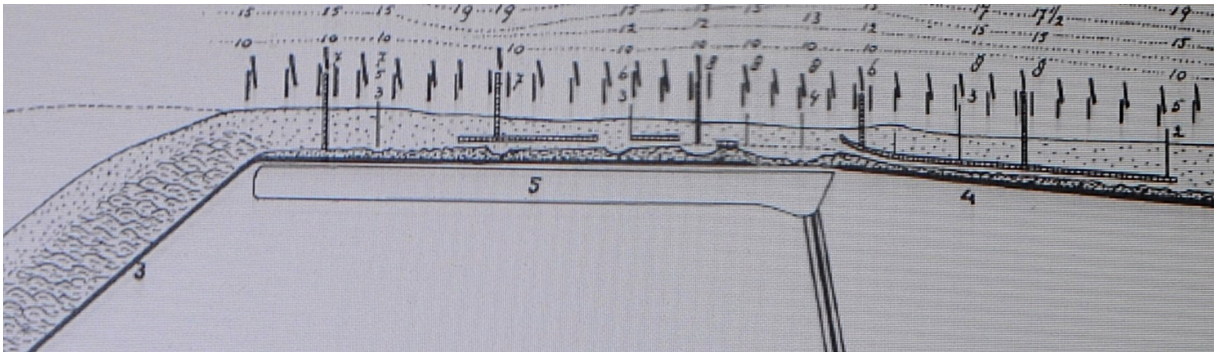


Fig. 4.4 Plan made by Van Laecken to place 'sows' along the coast at Huisduinen (after a plan view made by Dirck Abbestee; source: Schoorl (1973, 204)

On the basis of the description, presented above, four sketches have been made by Van Duivendijk of the construction of a sow (Figures 4.5 – 4.8 incl.).
When the lying *waese* is made as described in Schoorl (1973, 207, footnote 23) a much larger *waese,* called 'sow', is obtained than the sink-fascine, discussed in Section 4.2.
Still, it implies that a final flow gap to be closed by such 'sows', i.e. by lying *waesen*, as suggested by Vierlingh (1920, 158, 160, 349) must have had a limited width (7 m at the most) as only two *waesen* can be made in situ and sunk at the same time (one from each side).
Also Nebbens (1771, 52 – 56) describes the suspended fascine mattress and stipulates that for repair of the damage caused by a flow slide at a sea dike you can apply the floating fascine mattress as well as the suspended fascine mattress.

[36] The description made by Schoorl has been derived from a much longer text found in the Archives of the province of Noord-Holland, Haarlem. The said description has been transcribed by Van Duivendijk and presented in Appendix C to this dissertation.
[37] The text reads 6 to 7 thumbs thick; probably the circumference is meant.

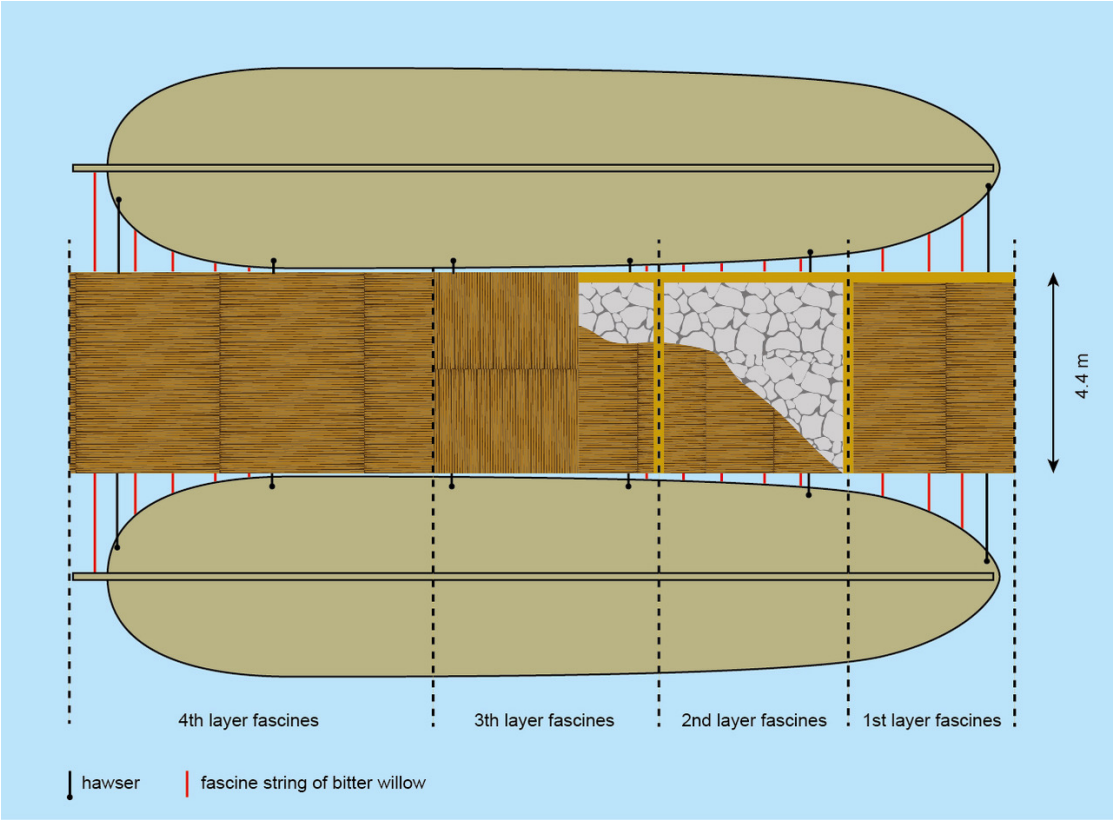


Fig. 4.5 Top view of suspended fascine mattress (sow) under construction

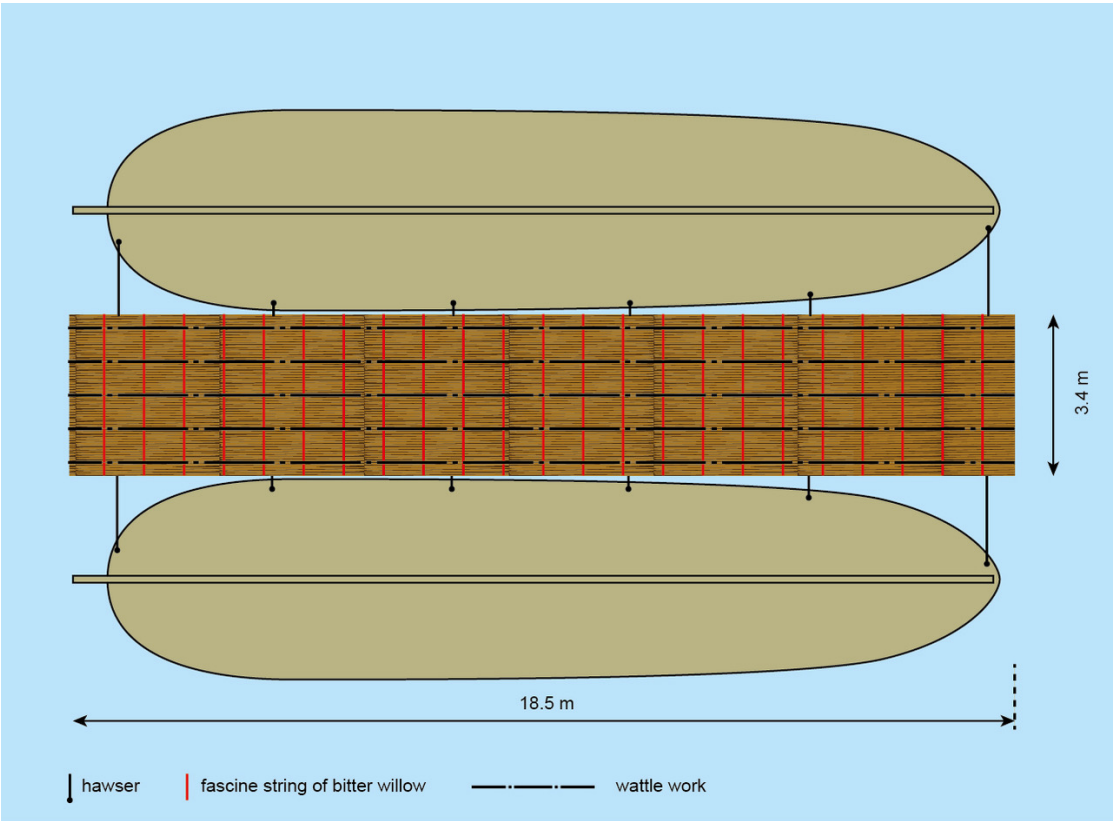


Fig. 4.6 Top view of suspended fascine mattress, ready to be immersed

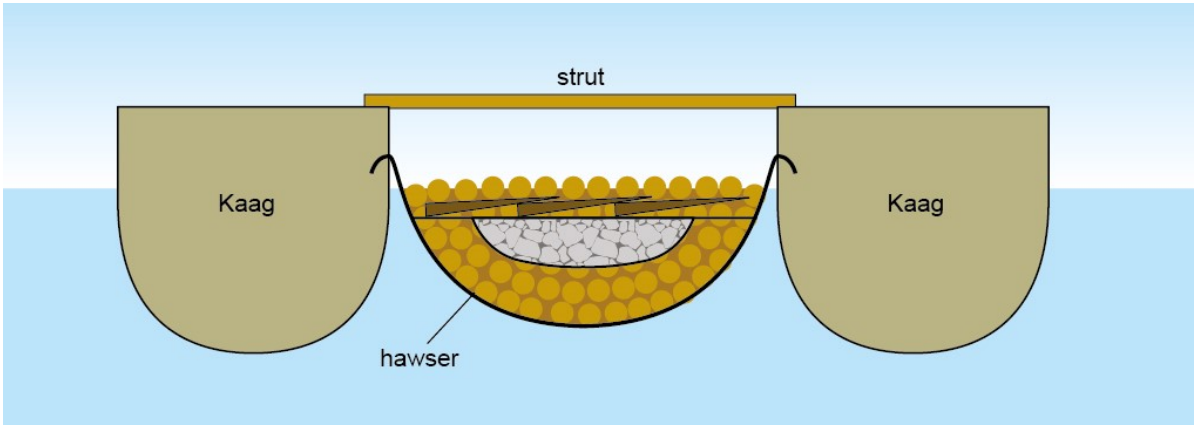


Fig. 4.7 Cross-section of suspended fascine mattress under construction

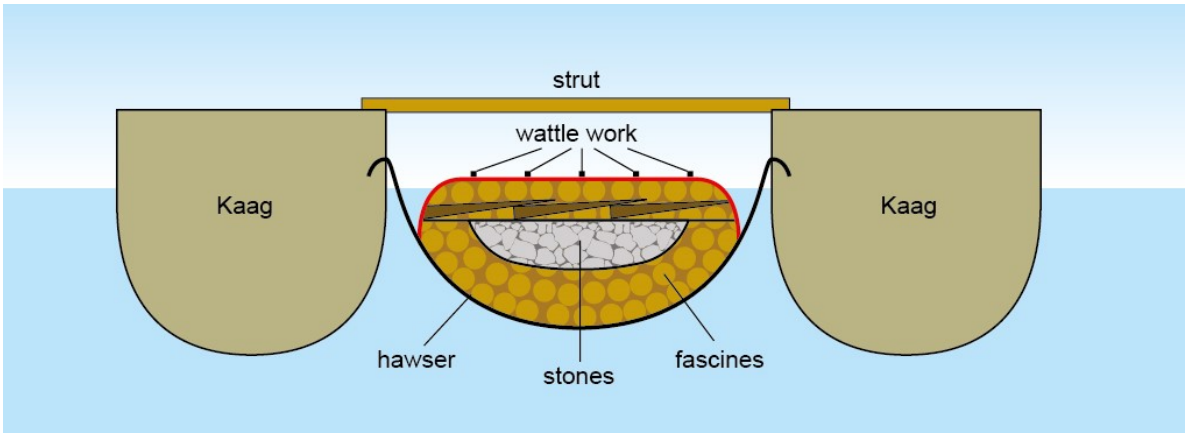


Fig. 4.8 Cross-section of suspended fascine mattress completed and tied together by strings of bitter willow ; it is ready for immersion

4.4 The floating fascine mattress

Somewhere between 1670 and 1750 the suspended fascine mattress, as described in Section 4.3, has been replaced by the floating fascine mattress. Since then, the floating fascine mattress is a vital part of closure works in an alluvial environment.
In this respect Fokker (1908) is quoted who notes (p. 79) about a flow slide in 1713 that it was decided to protect the foreshore at that location "by means of mattresses between ships". Later (in 1797) a "flat" fascine mattress is mentioned (p. 332).

In the past most dike breaches would not have been repaired and tidal channels would not have been closed without these floating fascine mattresses. Fascine mattresses have been applied in the Netherlands until the sixties of the 20th century. Nowadays, they are introduced again for ecological reasons, such as an environment-friendly under water slope protection. For instance in Laos by the Japanese Government (Fig. 4.9). The Japanese learnt the art of fascine mattress making and sinking from the Dutch at the end of the 19th century[38].

[38] Ir. C.J. van Doorn worked as a Harbour expert in Japan during the period 1872 – 1880. He introduced *rijswerkers* in Japan (Van Gasteren et al, 2000, 484). The art of making fascine mattresses is known in Japan as the 'method Westerwiel' (p. 381), named after one of the Dutch *rijswerkers*.

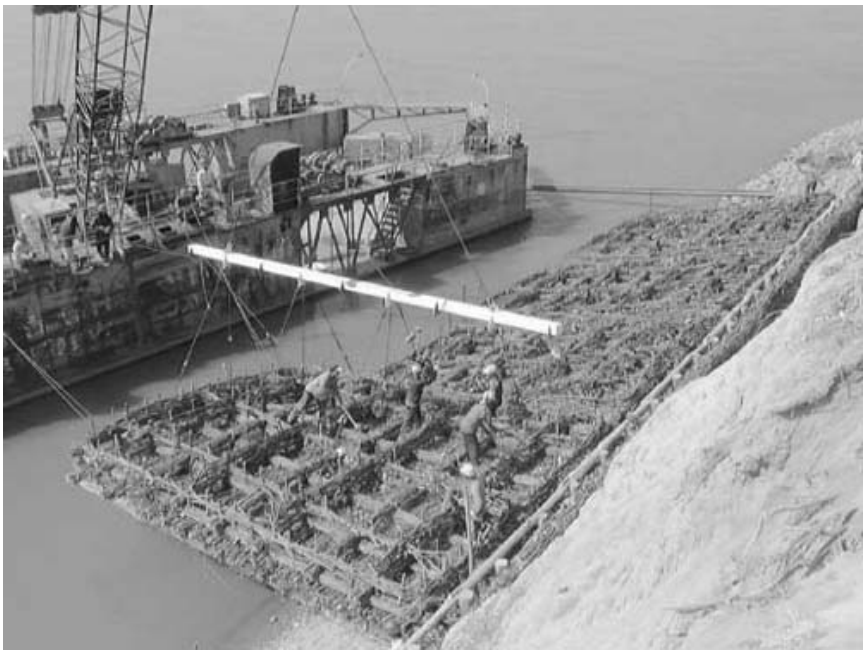


Fig. 4.9 Bank protection along the Mekong river in Laos in 2006 by placing fascine mattresses within context of Japan's Official Development Assistance; accessed on 1 Nov. 2023 at: https://www.mofa.go.jp/policy/oda/white/2006/ODA2006/html/column/cl02003.htm

Below, a short description[39] is given of construction, transport, positioning, ballasting and immersing of the floating fascine mattress.

- It consists of two or three layers of bundled brushwood (*rijshout*), so-called fascines, which are held together by an under and upper grating of so-called *'wiepen'* and ropes. A '*wiep*' is a string made of *rijshout* branches and is translated here as fascine string.
- The second layer of fascines is sometimes replaced by a layer of reed.
- Wattle work is made on part of the upper grating of fascine strings.
- In principle the mattress can have any desired size and shape but in most cases it will be rectangular and measure 20 by 40 m^2 or 40 by 40 m^2. Its thickness varies between 30 (2 layers) and 45 cm (3 layers) excluding the thickness of the fascine strings and wattle work.
- The mattress is constructed on a part of the mud flat, called the wharf ('*zate*'), which runs dry during Low tide.
- At High tide the mattress is in floating condition and is then towed to its final location; but in former centuries this transport required the energy provided by the tidal current, the wind (sailing) and manpower (rowing).
- The mattress is positioned at the planned location and kept in place by anchors.
- It now hangs by means of so-called sink lines between flat deck barges surrounding the mattress and loaded with rocks (or, in the past, clay clods).
- The mattress is then ballasted by manual labour distributing the said rock (or clay clods) over the squares of the upper grating (Fig. 4.10).
- At the turning of the tide when currents are nearly zero all sink lines are simultaneously loosened and the ballasted mattress then sinks to the bottom of the dike breach or tidal channel.

[39] An elaborate description of the fascine floating mattress is presented in Appendix D to this dissertation.

Finally, an additional layer of rock is dumped on the sunken mattress from flat deck barges which are shifted over the mattress.
So far, no reference has been found concerning the transition of a suspended mattress into a floating mattress. A possible theory about this transition is the following: to protect the toe of a dike against scour by wave attack a fascine toe protection is normally constructed on the shore during Low tide when the shore runs dry. This fascine toe protection (*kraagstuk)* is ballasted with stone. If, for one or another reason, the ballasting is not carried out in time, the fascine construction will start to float when the tide comes up.

Somebody must then have realized the potential of a fascine mattress which can float and can therefore be transported to, for instance, a dike breach and there ballasted and immersed. In the beginning the mattress will have been immersed gradually in shallow waters by ballasting it from one end only (Figure 4.12). Through the years, the construction will have been developed into its present state by adding wattle work (*tuinen*), bollards (*proppen*) and sink-lines (*zinkstroppen*). These sink-lines are required when the sinking operation has to be performed in a few minutes in deep waters at the turning of the tide when currents are nearly zero (Fig. 4.11).

Fig. 4.10
A floating fascine mattress being ballasted (Schelphoek, 1953)

The suspended fascine mattress is still being used in the 18th century (Schoorl, 1973, 291 and 294; Fokker, 1908, 79).

But at that time the floating fascine mattress is already in use. Bos (1969, 52) suggests that:

> "the first floating fascine mattress was made and immersed in Sliedrecht in 1678 and that some people consider it a Sliedrecht invention."

But Bos does not give any sources for his statemen! A more reliable source would appear to be Hunnius (± 1720, page nn), who states:

> "Not long after I came in the year 1675 in Oostburg[40], there was a flow slide … of considerable depth, in which were sunk complete fascine rafts by putting heavy stones thereon." [41].

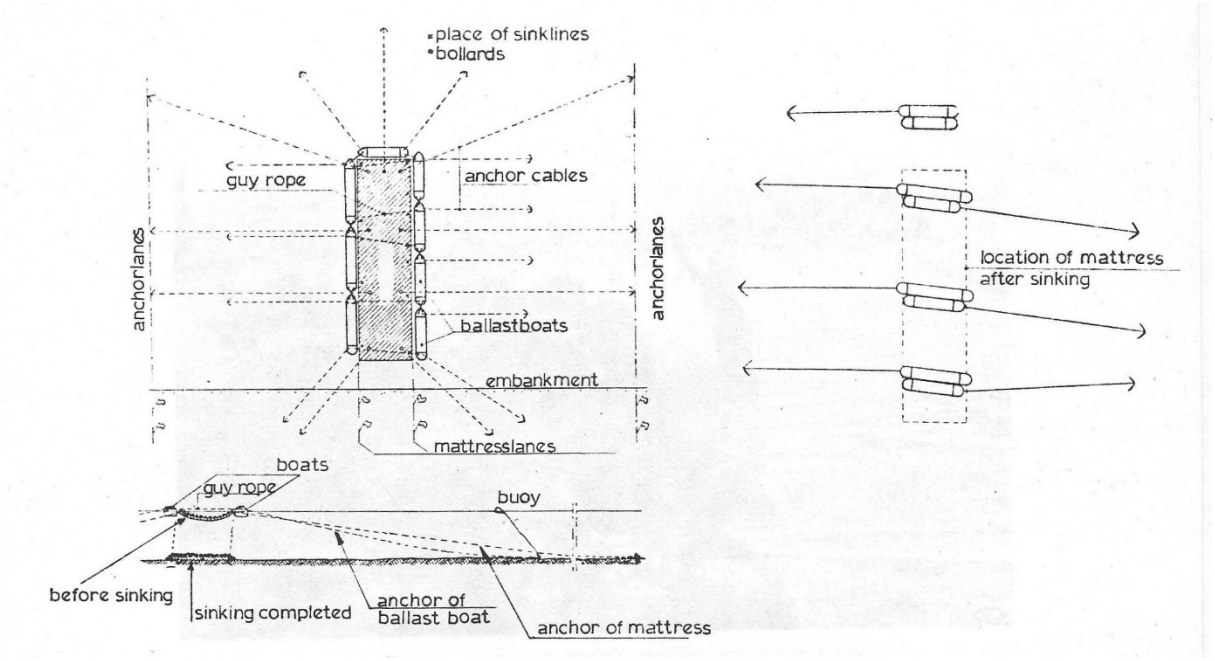


Fig. 4.11 Overview of the positioning and anchoring of a floating fascine mattress in deep waters, followed by its fast immersion at the turn of the tide; source: BWDB (1976)

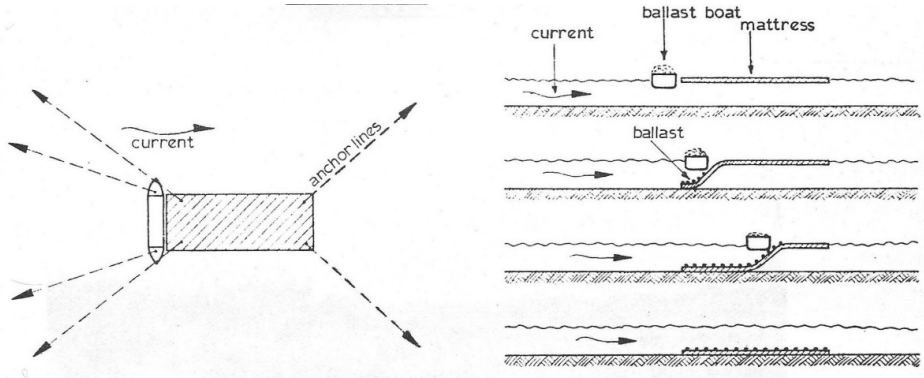


Fig. 4.12 Gradual immersion of a floating fascine mattress 'in the current' (*op stroom*), in shallow waters; source: BWDB (1976)

Fokker (1908), quoting from documents which were present in the Archives of the water board Polder Schouwen, mentions '*zinkwerk*' various times without being able to specify what is meant.

[40] A town located in Zeeuws-Vlaanderen.

[41] The original Dutch text: "*niet lange eer ik ten jare 1675 te Oosthurg quam had men bij liet wachthuijs van Riethem een val van een verveerlijke diepte, in de welke men deede sinken geheele rijsvlotten door swaare steenen, die men der op lei*."

However, his description (p. 202) of fascine mattresses in 1763 having dimensions of 78 by 60 ft (appr. 24 by 18 m^2) indicates without doubt a floating fascine mattress.

4.5 Pakwerken

Another important fascine construction is the pakwerk[42], a kind of fascine bund or quay wall.
In the past these *pakwerken* (pakwerks) were in most cases applied when a steep, nearly vertical face had to be constructed. As such they functioned as a quay wall and as wing walls for sluices and culverts.
In closure works pakwerks function as a closure bund placed on top of the fascine dam. The fascine dam is built up of fascine mattresses placed successively on top of each other, up to approximately 0.5 metre above the Low tide level (Fig.4.13). The pakwerk is then built during Low tide in situ on the fascine dam and the elevation of its crest will be that of mean high tide (MHT).

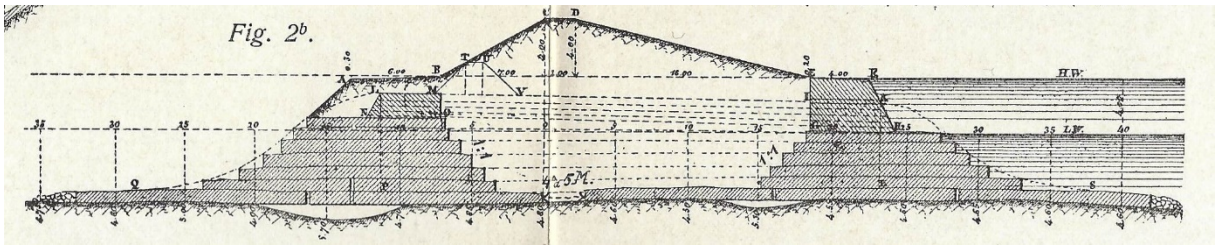


Fig. 4.13 Cross section over a closure dam: pakwerks have been placed on top of both fascine dams; a clay bund is subsequently placed on top of the sea side pakwerk, high enough to cope with summer storm surges; source: Storm Buysing, 1864, Plate XLVII)

The pakwerk placed on top of the fascine dam in the closure gap of the Sloedam (Neyt, 1872, 222) had a base 10 to 11 m wide and a 4.5 m wide crest (see also Ch. 8, Section 8.4, Figures 8.15 and 8.16).
The build-up of a pakwerk on top of a fascine dam will be more or less like the cross section shown in Fig. 4.14. Admittedly, this figure shows the cross section of a road embankment constructed on very weak soils but the build-up of the broad fascine bund (pakwerk) at the Sloe Dam, as described by Neyt , is more or less the same. One can see that an fascine bund consists of fascines, wattle work, pegs (to connect the various layers of fascines) and clay fill.
Each layer of the pakwerk is initially 60 cm thick and 40 cm after settlement.

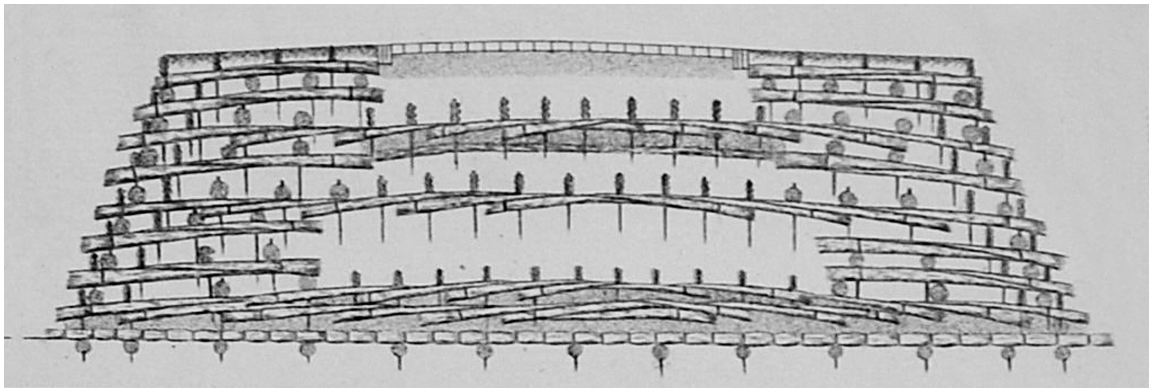


Fig. 4.14 Build-up of a broad fascine bund (pakwerk) (Storm Buysing, 1864, Plate I)

[42] There does not seem to be a good English translation for the fascine construction, which in Dutch is called *pakwerk*. Tyler (1906), a Coast Inspector, stationed in Sjanghai, used the word pakwerk (plural: pakwerks) for a Chinese construction which was very similar to the Dutch *pakwerk*. Probably, he got the name from J.C. de Rijke, at that time residing in Sjanghai. In this respect reference is made to Chapter 13, Section 13.3 of this dissertation.

A special feature is the anchoring of the fascine face in the clay fill[43] and the connection between both faces.

4.6 Baardwerken[44]

Baardwerken[45] are normally not used in closure works. But there is one closure discussed in this dissertation[46] where *baardwerken* are part of the closure works. Accordingly, this kind of fascine construction merits some explanation. Normally, (i.e. not necessarily for a closure) a *baardwerk* is applied if time to construct and place a floating fascine mattress in the tidal zone is lacking. Though this construction is not as robust as that of a floating fascine mattress, it is sufficiently strong and provides adequate cover to protect a bank against further scour by the current or erosion by wave attack.

It is beyond the scope of this dissertation to describe in detail the build-up of the *baardwerk*. Basically, it comprises some layers of fascines placed imbricate in one direction, perpendicular to the bank line, with some fascines placed in between in diagonal direction to prevent horizontal bending of the fascines by the current. Each fascine is fixed to the subsoil by only one peg (Fig. 4.15).

When all the fascines in the *baardwerk* have been placed (Fig. 4.17 shows the *baardwerk* in floating condition), the whole is ballasted by stones or clay clods. After immersion and settlement on the sloping bank, the *baardwerk* is fixed to the bank by rows of wattle work, parallel to the bank line (Fig. 4 16).

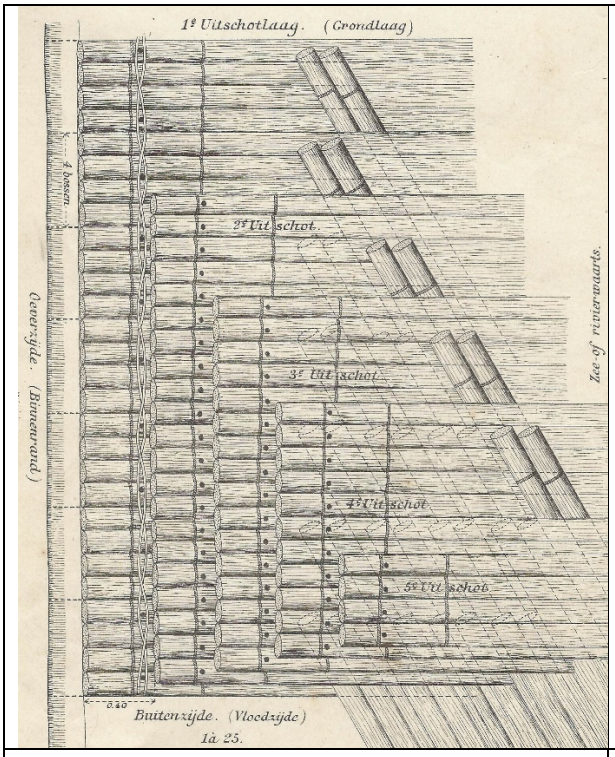


Fig. 4.15 *Baardwerk* without top layer

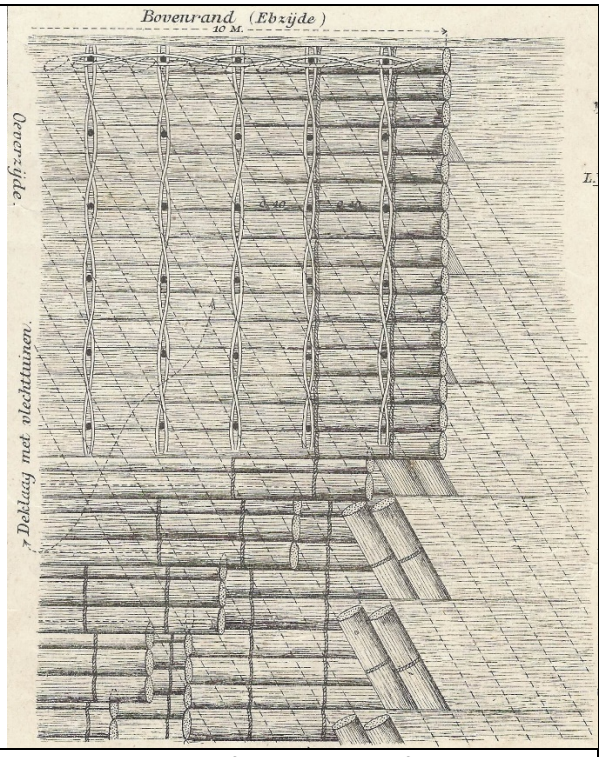


Fig. 4.16 Top layer fixed on part of *baardwerk* as well as rows of wattle work

sources of figures: Plasschaert (1902a), Atlas, figures 57 and 58, Plate V

[43] One could suggest: 'Terre armée avant la lettre'.

[44] For a detailed description of *baardwerken*, see for instance Plasschaert, (1902, 92 et seq.), Van Breen (1920, 195 et seq.), or Colijn (1921, 102 et seq.).

[45] Also here no suitable English translation could be found.

[46] I.e. the repair of the breach at the Nieuwerkerker polder in 1807, see Ch. 9, Section 9.6, Fig. 9.21.

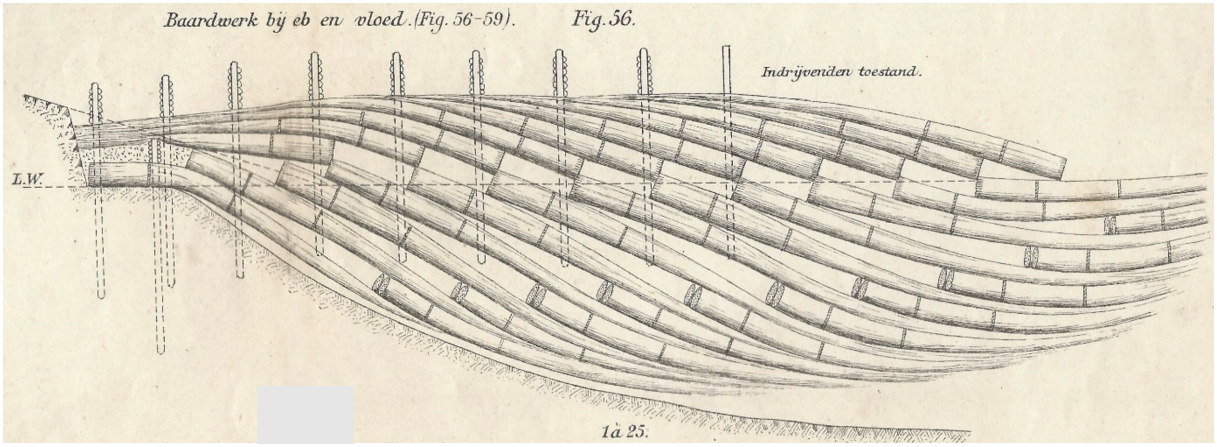


Fig. 4.17 Baardwerk in floating condition; source: Plasschaert (1902), Atlas, fig.56, Plate V

CHAPTER 5 Closure methods

5.1 Introduction

Before going into more detail about the methods, utilized for effecting closures by using simple means, a few comments must be made.
In principle only closures will be discussed which involve the closure of a flow gap (*stroomgat*).
A definition and delimitation about closures to be studied has been presented in Ch. 2, Section 1.4.
The emphasis in this dissertation will be on tidal closures. That are closures in which the tide[1] plays a dominant role. As far as known, river closures[2] by using simple means have mainly been carried out in the past in China[3] and Japan. In the Netherlands repairs of dike breaches along rivers are normally made after water levels in the river have fallen and the repairs can be carried out in a dry environment. This type of repairs is not considered a closure in the sense of this dissertation. Nevertheless one river closure, made along the river Lek near Wijck, in 1747 will be discussed in Ch. 9, Section 9.5. That closure was carried out because of the unusual negative effects of the flooding.
A scientific, technological approach is missing in most, if not all, tidal closures made during past centuries. One should for instance realize that formulae for the flow over weirs or sills are only being developed outside the Netherlands during the second half of the 19th century. The Dutch expressions for 'clear weir' (*volkomen overlaat*) and 'submerged weir' (*onvolkomen overlaat*) have come to the Dutch through the French[4].
Tidal calculations are only common after the reports have been published (1926 and 1920 respectively) about the consequences of construction of the Zuiderzee Enclosure Dam and about high water levels on the Nieuwe Waterweg (see for instance Mazure, 1937, 24 et seq.).
Finally, the relationship between, on the one hand, the size and relative density of stones remaining stable in high currents and, on the other, the prevailing current velocities as such, is published by Shields as late as 1936 and independently by Izbash in 1935.[5]
But, engineering is about the art of finding a solution for a problem based on the available knowledge and information. The early dike builders are able to find those solutions empirically, by trial and error and on the basis of experience obtained during earlier closures. And,… gradually they are able to develop methods to close tidal channels effectively without knowing much about the technical parameters such as discharge, water levels, head, 'rock' size, current velocities, flow pattern, eddies and scour in and around the closure site and their mutual relationship.
Nevertheless, a systematic approach to closure works is presented below[6] because closures using simple means have not only been effected by trial and error methods (roughly during the period 1200 – 1900) but also by methods developed using latest know-how on fluid mechanics and geo-technical engineering (roughly during the period 1900 – present).
In Section 5.2 the various purposes for making tidal closures are listed.
Section 5.3 deals with data collection, studies and surveys.
In Section 5.4 site selection and pre-closure operations are discussed.
Closure types are the subject of Section 5.5 while A few closure concepts are described in Section 5.6.
Finally, a Summary is presented in Section 5.7.

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[1] I.e. water level as well as current velocity and direction vary as a function of the tidal movement.
[2] Meant in this case is a river flowing in one direction, i.e. without any significant impact by the tidal movement.
[3] From CNCID (2005, 84) it would appear that they are nowadays, or until recently, still being carried out.
[4] *Deversoir complet* and *deversoir incomplet ou noyé* (Koechlin & Koechlin, 1924, 265 et seq.)
[5] See: https://nl.wikipedia.org/wiki/Sergei_Vladimirovitsj_Izbash.
[6] Part of the text comes from Chapter 7 of CIRIA, CUR & CETMEF (2007, 909 et seq.). As that particular text has been written by Van Duivendijk no quotation marks or further references are inserted in the text.

5.2 Purpose of tidal closures

Tidal closures within the context of this dissertation can be required for the following purposes:

a. to close a creek being part of a reclamation scheme;
b. to repair a dike, breached during a storm surge, after that surge;
c. to repair a dike seriously damaged by a flow slide in the shore near the toe of the dike or by warring parties;
d. to create a fresh water reservoir in an estuary fed by upland river discharge or in a bay;
e. to enable construction of an infrastructural link (road or railway) between two islands or between an island and the mainland;
f. to separate a tidal inlet or lagoon on the coast from the sea for safety or other reasons;
g. to construct a low dam solely to enhance accretion, followed later, if successful, by reclamation.

In Chapters 7 to 11 (incl.) an example will be given and analysed for most purposes mentioned.
The expression tidal closure is used here to distinct these closures from those in rivers.
In all tidal closures water levels and currents are determined by the tide[7]. This implies that during one tidal cycle (which, depending on the location, lasts approximately 12 or 24 hours) currents and water levels vary constantly and currents also change direction. The magnitude of the currents during the closure operation depends on (i) the tidal water levels at sea in the undisturbed situation, (ii) the dimensions of the closure gap and (iii) the area of the basin to be closed off from the sea.

5.3 Data collection, studies and surveys

It will depend on the magnitude of the closure, for what area, to what extent and for what period data are collected, studied and surveys carried out.
These activities in any case comprise the following:

- bathymetry of area and closure site;
- wave climate;
- tidal characteristics and its variation over the year;
- tidal currents through and near the closure gap;
- subsoil conditions: peat, clay, silt, fine or coarse sand including their erodibility;
- survey on the availability of construction materials: brushwood, bamboo, palm leaves, reed, clay, bricks, silt, fine or coarse sand, gravel, clay-clods and clay- sods, 'rock' [8];
- study on possibility or impossibility to make use of locally available waterborne equipment like fishing boats, flat deck barges;
- check on the availability of required plant such as manually operated pile driving equipment;
- study on the (hydraulic engineering) skills of the local building contractors and their labour force;
- accessibility of the closure site.

5.4 Site selection and pre-closure preparations

In most cases the horizontal alignment for the closure works is known from the start: on or near the spot where the dike has breached / damaged or where the tidal inlet or lagoon has to be separated from the sea. It has to be borne in mind that the magnitude of the closure operation is directly related to the area of the basin to be closed or to the location of a tidal meeting point.

[7] When tidal rivers are closed at their mouths also river discharge has to be taken into account.
[8] Here the word 'rock' not only means boulders and quarried stone but also bags/baskets filled with boulders, bricks, clay or sand.

In Chapters 7 to 11 (incl.) the reasoning behind the choice for a certain horizontal alignment regarding constructed closure works will be discussed where appropriate.

A closure is defined here as the last operation aiming at stoppage of the flow:

- between the tidal environment and the area to be reclaimed (reclamation scheme or inundated/flooded polder);
- between the tidal environment and the reservoir to be created or the lagoon to be separated from the sea;
- in the alignment of a future infrastructural link thus creating two different tidal basins or inlets.

The constructed closure is subsequently incorporated in the body of the dike or dam.
Closures in alluvial soils – especially when these soils consist of sand or silt – can only be made when, prior to increase of current velocities , the bed and slopes of the scoured breach, the bed of the creek, river or tidal inlet, is protected against scour in the region of increasing currents. This protection is in fact a filter which has enough weight (sink-fascine, ballasted fascine mattress, layer of clay filled bags, layer of rock or clay clods) or a good anchoring (cramp matting) to resist movement by strong currents and to prevent or delay scour. The extent of the area to be protected will be determined by site-specific circumstances. Further details will be given in Chapters 7 to 11 (incl.).
The last opening to be closed as part of the closure operations from now on will be called the final closure gap (*sluitgat*).
Prior to the start of closing this last opening various works have to be ready such as:

- protection of vulnerable submerged areas against scour, as outlined above;
- dike/dam sections outside the actual closure gap to be in an advanced state of completeness;
- availability and stockpiling of closure materials near the closure gap, like clay, bags filled with clay or sand, rock, sheet piles, bamboo, hurdles, etc.;
- temporary works, like a jetty across the closure gap;
- availability of small tools and equipment like spades, aprons, barrows, wheel barrows;
- availability of operational equipment like a narrow gauge railway (locomotives and side tip wagons);
- availability of a trained labour force;
- last but not least a detailed planning for the closure of the last opening (closure gap).

5.5 Type of closure

Depending on the length and depth of the closure gap, the tidal range, the materials to be used and the measures taken in advance, the closure can be gradual, speedy or instantaneous.
One speaks about a gradual closure when it takes more than one day to close the last gap. Speedy means that the closure is carried out within 4 to 8 hours, but always within one tidal cycle. An instantaneous closure will not take more than one hour.
It also has to be decided whether the closure will be a horizontal or a vertical closure, or a combination of both. Differences between these types of closures are spelled out in relevant literature; see for instance Huis in't Veld, Stuip, Walther & Van Westen (1984) and Van Duivendijk & Te Slaa (1987, 19 et seq.).
Here only some main features are highlighted. Schematic cross-sections are given in Fig. 5.1.

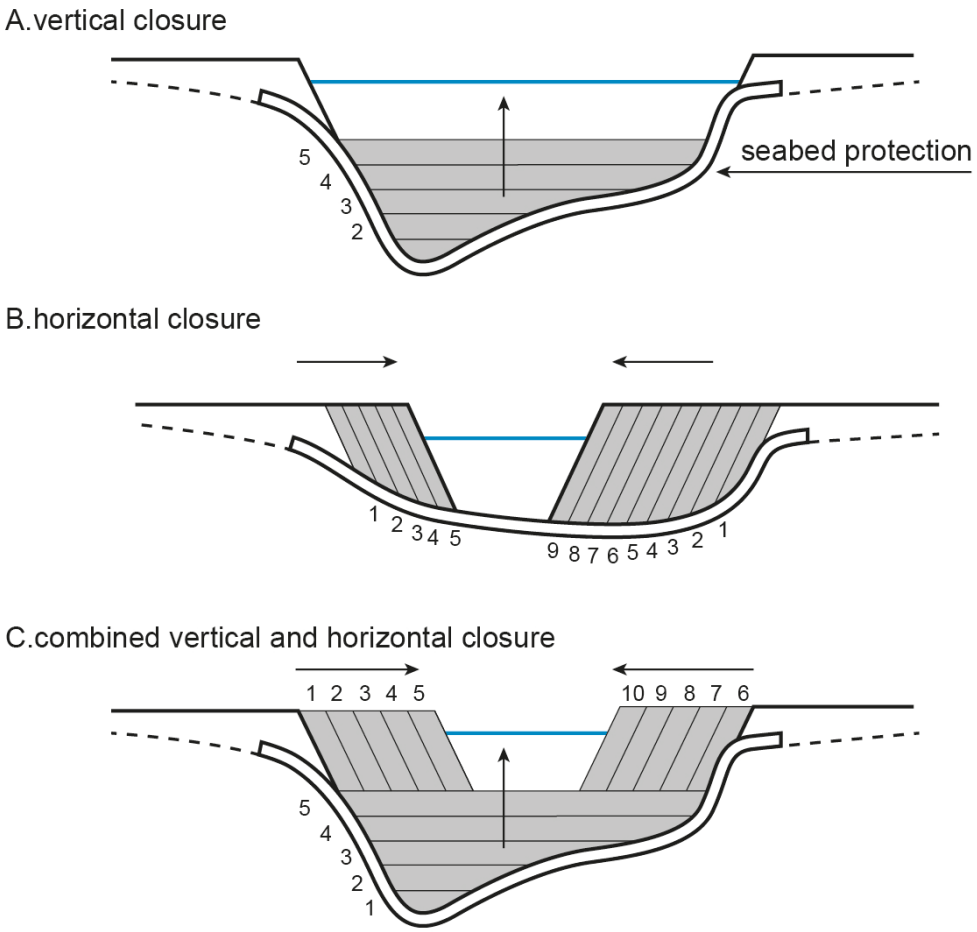

Fig. 5.1 Gradual closure methods; source: CIRIA, CUR & CETMEF (2007, 922) with adaptations

A horizontal closure is based on placing 'fill' in the closure gap from both ends. This 'fill' can be clay (clods) transported in carts, wheel barrows, aprons or head baskets, rock transported in carts, bags filled with sand/clay and carried by labourers[9].
The horizontal method has advantages and disadvantages. Advantages are the relative simple system of dumping and the possibility to use smaller rock sizes or boulders at the start of operation. A disadvantage can be a long walking distance between stockpile of material and place of dumping. Last but not least, one should mention the increase in current velocity as a function of the reduction in length of the closure gap: only for relatively small basins these velocities will not exceed the (approximate) 4 m/s limit at which a fascine mattress type of bed protection will start to collapse[10]. Also the dumping of clay clods or bags filled with clay or sand will not be successful unless a kind of cage has been made[11] in which these items are dumped.
Vierlingh (1920, 327) also points out repeatedly the difficulties experienced in placing standing *waesen* in the heavy current as part of a horizontal closure operation.
Horizontal closures are also effected by driving one, two or three rows of (sheet)piles in order to construct a 'wall' or a cofferdam across the closure gap. This is elaborated upon in Sub-section 5.6.3.

[9] The placing of a standing *waese* (see Ch. 4, Sub-section 4.2.1) can also be considered as a horizontal closure.
[10] This is, obviously, only an example. Other materials will start moving at lower current velocities.
[11] Reference is made to the closures discussed in Ch. 7 (Section 7.10, Dijkwater) and 11 (Section 11.3, Bangladesh).

A vertical closure is based on the principle of gradually raising the sill over the full width of the closure gap. The advantages of this operation are the possibility to carry out initially the dumping of closure material by using waterborne equipment and the limitation of the maximum current velocity to that of a broad-crested weir. Disadvantages are the need to use large rock sizes over the full length of the closure gap and the related porosity of the rock bund during the full period of closure. Moreover, after the bund has reached a certain elevation in relation to tidal water levels, it is no longer possible to utilize waterborne equipment and therefore other means of transport and for the dumping of rock or other closure materials have to be employed. For the type of vertical closures discussed in this dissertation, jetties crossing the closure gap, are used for transporting and dumping closure materials either by means of manual labour or by narrow gauge trains. In those cases there is no limit to the elevation to be achieved.
A completely different method of vertical closure is the piling up of fascine mattresses in order to construct a fascine dam (*rijzendam*). But also here the use of waterborne equipment limits the maximum elevation which can be reached and the closure then changes from a vertical into a horizontal closure as shown in Fig. 5.1 (C).
In order to prevent thru-flow, two parallel rows of piled up mattresses are placed having a dumped clay core in between. This closure method is used from the 18th until the 20th century in the Netherlands but also in Schleswig-Holstein.
In the 16th and 17th century, and possibly earlier, vertical closures are made in a similar manner by applying the suspended fascine mattress but this is only feasible for a rather narrow closure gap. Also Vierlingh, who describes the vertical closure method last mentioned and similar ones, advocates repeatedly the application of a clay core (*kleikist*) (see for instance Vierlingh, 1920, 342).

A combined closure is already mentioned above: one starts with a vertical closure which goes over in a horizontal closure. But it is also possible to start with a horizontal closure and close the narrow closure gap by a vertical method.
Finally, one can also do both at the same time as demonstrated in the closure of the tidal channel Sloe (Chapter 8, Fig. 8.13).
The closure experts from the past must have realized the advantage of the combined closure method first mentioned, i.e. start with a vertical closure by constructing a horizontal sill in the closure gap. and close the final closure gap by means of a horizontal closure over the sill. In a vertical closure with decreasing water depth over the sill the moment will arrive that the current velocity is not any longer dependent on the water level difference between the upstream and downstream side of the sill but mainly on the upstream water depth in relation to the height of the sill[12]. This implies that given a certain elevation of the sill, the supercritical flow over the sill reaches its possible maximum velocity at High tide. Provided, that maximum current velocity does not damage the constructed sill, one can then start the horizontal closure from both ends without any increase in the maximum current velocity, just mentioned.

5.6. Tidal closure concepts

5.6.1 General

From the texts presented in the Sections 5.1 to 5.5 (incl.) it follows that the successful implementation of a tidal closure depends on many aspects. It will be demonstrated in detail in Chapters 7 to 11 (incl.) for a number of closures. But it is considered useful to describe now already, in general and global terms, a few closure concepts as practiced during the past centuries. A closure concept is in fact a certain closure type as discussed in Section 5.5, adapted to the particular circumstances , like

[12] I.e. this is the moment of transition from a submerged into a clear weir.

tidal cycle, topography of the closure site, availability of construction materials, skill and size of labour force, tools and equipment.
Obviously, the repair of a dike breach is different from the closure of a tidal channel as part of a reclamation scheme or to make a connection between two islands.
The repair of a dike breach requires much improvisation and has to be done in a hurry in order to limit the damage. The repair mostly has to be carried out in winter when the days are short, the temperatures low and stormy weather to be expected, so during dynamic and uncertain conditions. Accordingly, the closure is not always successful.
Conversely, the damming of a tidal channel is a planned operation which can be done in spring or summer when days are long and storms seldom occur.
Below, the emphasis will be mainly on closure concepts for repair of dike breaches.
The three closure experts mentioned earlier (Vierlingh, Bommenee and Matelé) worked predominantly in the provinces of Zeeland and West-Brabant and in the area later called Zeeuws-Vlaanderen. Their statements reflect the approach of breach repair during (roughly) the period 1550 – 1750 in this part of the Netherlands.[13]
Normally, the first action is to protect both ends of the breach, the dike heads, (*dijkskoppen)* against scour, i.e. against a further widening of the breach.
The next aspect one has to consider is whether the size of the dike breach, given the area of the tidal basin flooded due to the breach and the local tidal differences, enables it to be closed at all. A repair of breached secondary dikes (*binnendijken*) in many cases is the first action to take[14]. If this does not limit the size of the tidal basin, it might be wise to construct one or more *vercortinghe*, before an effort is made to repair the breach. Nowadays, a *vercortinghe* would be called a compartmentalization dike (*delingsdijk).* See for instance Eekhoff (1835, 54), Gerhardt (1988, 216) and Matelé (1731, negenste Capittele) The latter goes even further and would consider the need for *hulpgaete*, i.e. local lowering of the dike at a few places to enable weirs (*overlaten*) to be constructed which would divert part of the tidal in- and outflow from the main breach. Admittedly, Vierlingh (1920, 325 et seq.) is very much against such practice[15].

5.6.2 Focus on closures using clay and fascine constructions and, to a lesser extent, stone

Vierlingh (p. 325 et seq.)) distinguishes three types of dike breaches:

(a) The breach runs dry at Low tide and in that case the bed level should immediately be heavily cramp matted (to prevent a further deepening of the breach). A few days later, when manpower, closure materials (clay clods and sods) , tools and equipment (carts, barrows, wheelbarrows) have been mobilized, the gap should be closed during one tide by horizontal closure.

[13] It is interesting to note that know-how about the closure of breaches in dikes in the second half of the 17th century has also been transferred to Italy. Cornelis Meijer, a Dutch hydraulic engineer, working in that country, and mentioned already in Chapter 4, published a book in 1689 having as title 'Nuovi Ritrovamenti' in which he explains how the Dutch would repair a breach by constructing a cofferdam around the perimeter of the scour hole. Probably, a breach in a river dike is meant but this could not be ascertained for sure. The original text, a translation in Dutch and his sketches are presented in Appendix F to this chapter.
[14] But this is not always a possible solution. For instance in West-Friesland the construction of secondary dikes could only take place after the authorities had given permission because it meant that taxable lands got lost (De Vries, 1876, 74).
[15] " *Een iegelijcken wacht hem van meer gaeten te maeckenne dan datter deur tempeest in dijcken en vallen, want één gat alleen can wel veul spels maecken*."

(b) The breach and the scour holes (at both sides) are not deeper than 1.5 to 3.5 m below Low tide[16]. It must now still be possible (according to Vierlingh, that is) to close the flow gap in the dike alignment using *liggende waesen* (i.e. sink-fascines). It is also possible to construct a *vingerling* instead, which is a bund around the perimeter of one of the two scour holes. But a *vingerling* is more costly (because of its length) and in the end you still have to close a last closure gap. Deep scour in this last gap should not happen if the breach is in a dike along a tidal river having a tidal difference of only 1.5 to 1.8 m and the water flows over meadows and salt marshes . But in Zeeland and Flanders the situation is different: tidal differences of (according to Vierlingh) 4 and 5 m do occur[17] and Vierlingh admits that *liggende waesen* is not the solution to the problem at hand because there is no slack water period. In that case he recommends to place timely a layer of fascines loaded with stones or clay clods on the base of the future closure gap in the perimeter bund. Fig. 5.2 , drawn by Verhoeven on the basis of Vierlingh's text, refers. This is a combined vert-hor. closure.
If, for one or another reason, construction of a *vingerling* is not feasible, the closure has to take place in in the original dike alignment. According to Vierlingh (p. 327) the only possibility, to make a closure in that case, is by placing standing *waesen*. Vierlingh emphasizes the need for a speedy fixing of the lower end of the placed *waesen*, by dumping from the land and sea side clay sods and clods against the *waesen.* This is a horizontal closure in two stages.

(c) The worst type of breach is the one at or near the location of a sluice. Because in that case the tidal flow from the start has a network of drainage channels at its disposal on the land side of the breach and an outlet channel through the shore (*voorland*) at the sea side.[18] Vierlingh (p. 328) suggests to select the highest ground level in this drainage network (sea side or land side) for the construction of a *vingerling* and to start by dumping a sill of stone with a clay core (2.5 m wide) in between. The sill should be 2 feet (0.6 m) high. The sill would be a good foundation for a closure of a last gap using *waesen*[19]. Also aprons *(stortebedden*) are mentioned at both sides to counteract scour by diving jets. Fig. 5.3, drawn by Verhoeven on the basis of Vierlingh's text, refers.

Vierlingh, though stiil rather vague, is much more specific about closure concepts than the aforementioned Adriaan Bommenee or Jan Matelé. At the time these last two gentlemen lived (first half of 18th century) the floating fascine mattress did already exist and could be used to prevent or stop scour at a closure site. But Matelé is not in favour of such bed protection. In his opinion the flow over a placed fascine mattress with wattle work on top is very turbulent and easily results in piping under the mattress. He advocates the dumping of a base consisting of clay clods because such a layer of clay is much more tight than fascines and therefore protects the underlaying soil very well against scour. Also the clay clods result in a smooth flow through the breach. The said base is in fact a *plemped* dike, i.e. consisting of clay clods or sods dumped from small boats. It can be considered a gradual vertical closure up to the level of Low tide (neap). The next step is the construction of a closure bund of clay during one neap tide (speedy vertical closure) by organizing as many boats as possible for transport of clay clods and sods.

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[16] Implicitly, it is suggested by Vierlingh that deeper breaches cannot be closed in the axis of the breach or by means of a *vingerling*. In that case, either the flooded area is left as it is and the polder is abandoned (it becomes '*drijvende*') or one tries to decrease the tidal prism by compartmentalisation.

[17] Many definitions of the length of a foot exist. Vierlingh may have meant the (relatively short) 's Hertogen-bossche foot which is 28.7 cm. In that case the tidal differences are still between 4 and 5.2 m.

[18] This case is very well demonstrated by the series of sketches showing the development in time of scoured channels due to the dike breach at Schelphoek in 1953 (Van der Velde & Gerritse, 1955, bijl. 3). See in this respect Ch. 9, Section 9.3, Fig. 9.8.

[19] Vierlingh does not state whether lying or standing *waesen* are meant.

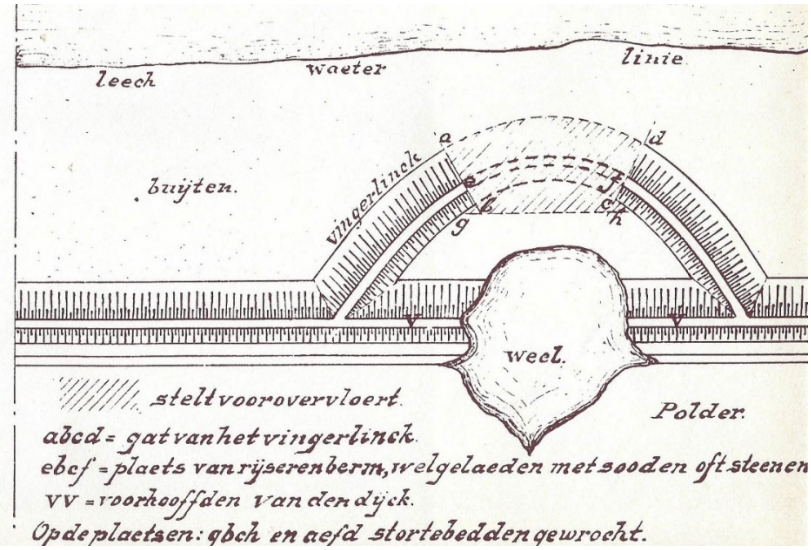

Fig. 5.2 Closure gap in perimeter bund; aprons have been made in advance (gbch and aefd), also a clay core bcfe; source: Vierlingh (1920, Fig. 62)

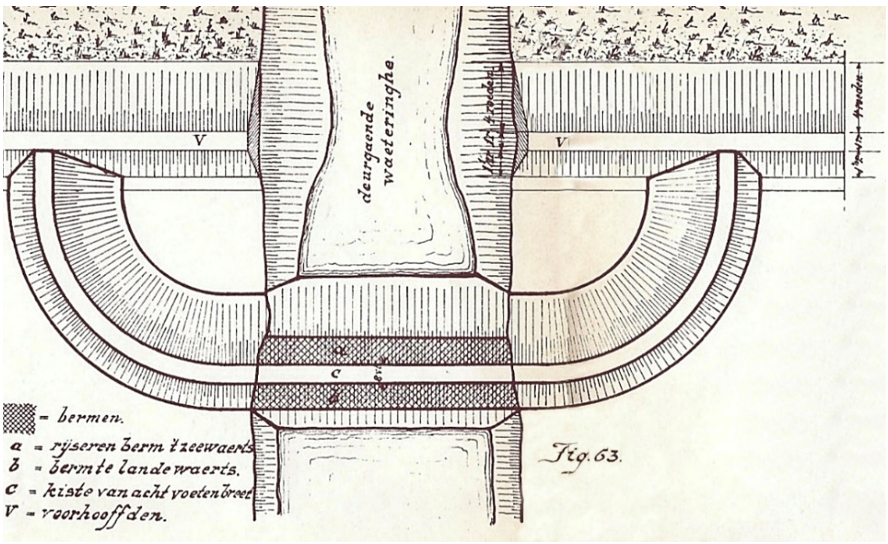

Fig. 5.3
Perimeter closure bund across deep channel;
legend:
a = fascine protection sea side,
b = ditto land side,
c = clay core 2.50 m wide,
v = dike heads;

source: Vierlingh (1920, Fig. 63)

Matelé gives very detailed instructions regarding logistics and also about rest periods, catering and payment of the labour force. Matelé also points out the need to work during these closures as much time as is available, i.e. also during Sundays and Saints' days: "may be the clergy does not like it but it is none of their business."[20] Also, the local authorities, in his opinion, have no say in the matter. The dike reeve and other members of the polder-board should decide what is good for works' progress.

[20] The principle of labour on Sundays and Saint's days to close a dike breach brings about an old argument. It is repeatedly mentioned in publications from the past about the repair of dike breaches. The opinion of contractors about this issue is not mentioned by Matelé and probably they had no say in the matter in the 18th century. But in 1868 G. van Oord resigned from the firm for which he worked as he refused to work on Sundays and founded his own firm (Van Oord, 2001, 17). Den Doolaard (1946, 24) mentions it as one of the 'difficulties' during the repairs of the bombed dikes on the island of Walcheren after the Second World War. But the situation also exists during the urgent repair works at Schelphoek, immediately after the Storm surge flood of 1953: the *rijswerkers* of Werkendam leave each week the closure works during two full days (departure Friday afternoon, return Monday morning early).

Bommenee (Gerhardt, 1988, 220) mentions the use of stakes and hurdles for the speedy closure. As soon as all the boats (*vletschuiten*) transporting clay clods and sods have arrived a row of vertical stakes is driven into the subsoil and vertical hurdles are used to retain the clay clods dumped by the boats, thus forming a kind of wall. Then a second bund is dumped and clay is brought in between using wheelbarrows. Apparently, a gradual vertical closure is meant which starts at a level below Low tide.

After 1750 the art of closing a dike breach by the method of stacking floating fascine mattresses on one another, in order to create an fascine dam, must have been developed[21]. On the one hand by closure experts like the ones just mentioned and on the other by the *rijshout* industry near the Biesbosch (e.g in Werkendam). Fokker (1908, 202, 208), on the basis of the old archives of the waterboard Polder Schouwen, mentions a low dam being built in the Oosterschelde between Schouwen-Duiveland at Flaauwers and the sandbank Roggeplaat during the years 1763 -1765 by placing (at least) seven layers of fascine mattresses stacked on one another.

Caland (1833, 93-121) describes this new method in detail for the closure of tidal channels as part of reclamation schemes. It can also be applied for dike breach repair provided the size of the breach is sufficiently large to allow for a full filling and lowering of the flooded area during a tidal cycle. If such a situation is not present, i.e. the breach is too narrow, the periods of slack water are too short to allow for the sinking of fascine mattresses. In that case *vercortinghe* (i.e. compartmentalization works) have to be made to decrease the area of the tidal basin (p. 121 et seq.).

In short the new closure method implies:

- a bed protection (*grondstuk*) of fascine matresses over a wide area (Fig. 5.4);
- construction of two fascine dams on top of this bed protection with a clay core in between up to the level of Low tide (i.e. a gradual vertical closure);
- followed by construction of a pakwerk in layers (also a gradual vertical closure) on both fascine dams up to the high water level of a neap tide (HWN);
- finally, a clay bund on the clay core on the landward side up to a level of at least 1 m above mean high tide (MHT), i.e. a speedy horizontal closure.

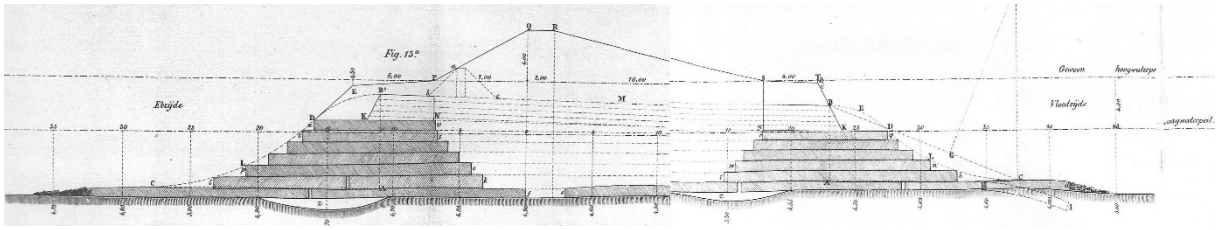

Fig. 5.4 Cross-section over completed closure bund comprising two fascine dams with clay core, two pakwerks with clay aprons in between, clay bund at landward side, dike body mainly on top of clay core; source: Caland (1833, Plate II, Fig. 13a)

In so far as known the first mention of stacking fascine mattresses to act as a wall against the tidal flow is by Fokker as indicated above. With variations it has been applied numerous times during the 19th and the first half of the 20th century. The last time it was applied was in Burghsluis to close a flow gap after the Storm surge of 1953. (Rijkswaterstaat, 1961, 491).

In Chapters 7 to 11 (incl.) a few case histories, where this method has been applied, will be presented.

[21] A possible problem with a fascine dam in a dike is its subsidence. Therefore one tries to place the fascine dam under the slope of the future dike body to prevent subsidence of the crest of the dike as demonstrated in Fig. 5.4 and also in Ch. 8, Fig. 8.10.

5.6.3 Closure by constructing a cofferdam or by (sheet) piling

Already in 1510/1511 a dike breach in the Spaarndammerdijk is repaired by driving one row of piles in order to construct a tidal barrier. The piles have to be placed as near as possible to each other. As the work progresses the row of piles is strengthened by dumping clay sods, continuously transported by boats (*vletschuiten*) to the *dijckschuyten*, moored alongside the section of piling which has to be strengthened. If closure of the last opening by means of pile driving turns out to be difficult two ships (*koggen*) will be sunk to close the last opening. (Van Dam, 1992 & 1998, 256-259).
Further details will be presented in Ch. 9, Section 9.2. But one comment should here be made already: Apparently, no bed protection or the like is required. The same applies to a closure in 1675-1676 in Scharwoude for which three rows op piles had to be driven (Thierry, 1930 and Schoorl, 1967). In fact the three rows of piles formed a double cofferdam. This closure is described in Ch. 9, Section 9.4.
A cofferdam comprising a scaffold-type structure with hurdles functions as a kind of retaining wall while it prevents the fill materials (which in principle can be anything) from being washed away by the current.
Dibbits (1950, 57) states that "North of the line Alkmaar- Hoorn the soil comprises predominantly sea clay'. Scharwoude is situated just South of that line. The relatively low current velocities may also have prevented scour.
By driving piles you still have some flow in between. But when sheet piles are used the situation is different. They are in fact used during the successive closure operations carried out at the Dagenham Breach (river Thames, East side of London) in the years 1716 – 1721. But the top of the sheet piles is kept low (only a bit above Low tide). Moreover, one constructs two temporary sluices in the still existing dike section to divert part of the ebb- and flood flow from and into the flooded area, from the actual breach (Perry, 1721). This closure is described in Chapter 16.
Sheet piles for closure operations are also used during construction of the Hindenburgdamm to the island of Sylt (Schleswig- Holstein), opened in 1927 but this is not a success (Huitema, 1947, 127-143).

5.6.4 Closures made by using a temporary bridge (jetty) across the closure gap

By the end of the 19th century contractors start to utilize side tip wagons on narrow gauge rail tracks to transport earth for the construction of dikes (Verhoeven, 1924, 191). In the 20th century these side tip wagons are also utilized in tidal closures. This method is applied If the gap to be closed is not too deep, say one metre or less below Low tide (Vierlingh, 1920, LV et seq. and Huitema, 1947, 129). Obviously one needs a jetty to carry the rail tracks across the gap. In the Netherlands the method is used for the first time during reclamation of (part of) the tidal inlet Zwin in 1872 (Ch. 7, Sub-section 7.10.1, Fig. 7.18) and later for other closures like the Dijckmeesterpolder, (Verhoeven, 124, 196 et seq.) described in Ch. 7, Section 7.10.2.
In Germany, Schleswig Holstein, the tidal channel entering the Pohnshallig Kooge is closed in this manner in 1924 (Hinrichs, 1931, 27).
It depends on the characteristics of the particular project whether or not the jetty just prior to the placing of the closure material is transformed into a cofferdam. This is for instance done at the first (failed) closure of the tidal inlet Dijkwater (Kalkwijk, 1955, 59-70) described in Ch. 7, Sub-section 7.10.2.
After the Storm surge flood in 1953 a closure bund of sandbags has been built near Kruiningen on the Rijksweg (Stuip, red. 1984, 29-30). De sandbags are carried to the spot, where the bags have to be placed, by labourers walking on a jetty across the closure gap. See Ch. 10, Section 10.10. In Bangladesh the jetties across closure gaps are transformed into a cofferdam and subsequently the cofferdam is filled with clay-filled bags dumped by labourers (BWDB, Interim Reports (1979, 1980 and 1982), see Ch. 3, Section 3.4, Fig. 3.14.

5.6.5 Instantaneous closures

In the foregoing sub-sections the closures mentioned are either gradual or speedy. But a few times in the past the closures were instantaneous. In those cases rather small old wooden ships are filled with stones and sunk in the closure gap at the turning of the tide. Such a closure is for instance made as part of the construction of the dike between Broek and Zevenbergen[22] in 1412. See Ch. 7, Sub-section 7.4.6. Generally speaking, this method is risky unless a good bed protection is in place and extent of flow under and between the sunken ships is small.

In England a completely different instantaneous closure is carried out in 1863. Sluices have broken down and through the access canal to the sluices an area (2 400 ha) is flooded by the tide. The tidal flow has to be stopped and a sturdy scaffolding is erected across the canal. A series of composite vertical rectangular beams are supported by temporary scaffolding. These beams have grooves. Wooden sliding panels thick 17.5 cm, wide 2.13 m and of varying height (up to 2.13 m) are constructed and will be placed between the said beams in the grooves. Aprons (*stortebedden*) are constructed at both sides of this cofferdam under construction. The final opening to be closed has a width of 27 m. First a kind of horizontal sill is being created by lowering gates of varying height. The first effort to close fails and the gates already placed are lifted. Difficulties encountered (scour, some badly positioned vertical beams) will be discussed later in Chapter 17. Ultimately, the closure takes place during a neap tide at the Low water slack period. All gates are lowered within a period of 20 minutes (Hawkshaw, 1863, 497-503).

5.7 Summary

1 Closure methods have been developed empirically, in the course of time, by trial and error.

2 Main materials used for a closure are clay, fascines and stone while predominantly manual labour is used for the production, transport and placing/dumping of these closure materials.

3 Depending on the physical environment, the tide and the available closure materials, the closure will be gradual, speedy or instantaneous in time, while it will be horizontal, vertical, or a combination thereof, in longitudinal section.

4 Closures in breaches dikes, which, in the Netherlands, have to take place in alluvial soils, during the period 1550 – 1750 are difficult to achieve due to lack of adequate closure techniques, organization and sufficient funding. Accordingly, many flooded polders remain flooded (*drijvende)* during long periods or are abandoned altogether.

5 when tidal amplitudes are moderate the use of (sheet) piling to construct a barrier or a cofferdam in order to stop the tide is a realistic possibility.

6 After 1750 the use of floating fascine mattresses to construct a fascine dam (vertical closure) followed by a horizontal closure by constructing pakwerks is the preferred method for closing tidal channels and to repair breached dikes in the Netherlands.

7 In the second half of the 19th century a temporary bridge (jetty) across the closure gap, to enable the use of a narrow gauge railway carrying side tip wagons, is introduced. This method of closing tidal channels in reclamation works is in use up to the fifties of the 20th century. Its variant, no transport of fill by railway but by labourers, carrying and dumping the fill from a temporary bridge, is actually applied after the storm surge flood in 1953 and in Bangladesh in the period 1979 – 1982.

[22] Sources: Hingman (1885), Vriend (1961), Van der Helm (1988), Leenders (2021).

Part II Closures in the Netherlands and other countries with significant input from Dutch[1] side

[1] During the study of relevant literature it became clear that also in other countries tidal or river closures have been made without any or limited input from Dutch side (manpower, methods, materials). These closures will be presented in Part III.

CHAPTER 6 Selection of closure projects to be studied

6.1 Introduction

In Chapter one, Section 1.4 of this dissertation a definition and delimitation is given of closures to be studied. This definition and delimitation will not be repeated here apart from one aspect: the definition of a flow gap.

Rijkswaterstaat (1961, 159) defines a flow gap as follows:

> "One speaks about breaches when the damage to the dike from the land side goes through the crest, thus decreasing the water retaining function of the dike,……, when the breach is deeper than the original ground level under the dike and the ground level in front of it, one speaks about a flow gap (*stroomgat*)."

It was also stated that this definition is not completely followed in this dissertation because:

> " the bed level of the breach or tidal channel must have an elevation low enough to allow for a continuous flow. In other words the breach or tidal channel does not run dry during low tide." (Polytechnisch Tijdschrift, 1954, 560b).

By applying the various criteria listed in the aforementioned Section 1.4, one arrives provisionally for Part II at an overall total of 71 closures (carried out by simple means) which may warrant further study and analysis. These closures, because of their nature and purpose have been classified in five different groups:

- twenty-seven closures as part of land reclamation projects, period 1263 – 1954 (Chapter 7);
- nine projects comprising closures of tidal channels as part of construction of dams in the tidal zone during the period 1751 - 1925 (Chapter 8);
- nine polders in which dikes were breached during the period 1508 – 1916 (Chapter 9);
- twenty-one repairs of dikes breached during the storm surge disaster of 1 February 1953[1] (Chapter 10);
- five closures in Ghana and Bangladesh in the period 1969 – 1985 (Chapter 11).

Given the many reclamation schemes realized during past centuries and the great number of storm surges in the past causing dike breaches, (27 + 9 =) 36 closure projects is, obviously, only a very small number. But, as already pointed out in Chapter one, Sub-section 1.5.3, only a few reports, books and dissertations about the archives from the past centuries contain information about the subject of this dissertation.

In this respect Prof. Ir. J.W. Thierry must be mentioned who, in his inauguration speech, pointed out that:

> "…….Much of this documentation has been scrutinized in view of the history of local law in the past as well as for the history of former governmental, regional and local institutions but oddly enough, not, or only to a small extent, for scrutiny of the history of hydraulic engineering works while these local laws and institutions in relation to the hydraulic engineering works can be considered to be of secondary interest." (Thierry, 1930, A 49).

Regrettably, at the time of writing, the situation has not improved very much.

[1] As the closures in the year 1953 after the storm surge disaster of 1 February, after a provisional selection, already numbered 21, it was decided to separate these from breaches which occurred earlier.

In order to make a selection of projects for shortlists to be studied and analysed one should determine in advance what topics are of interest. These topics are:

- the closure method used,
- particulars of the closure works like length profiles, cross sections, tides,
- the materials used for the closure (type, quantities),
- the labour input in terms of numbers, tasks, and period,
- the cost of the closure works,
- features related to fluid mechanics like tidal range, compartmentalization, current velocities, tidal prism,
- local circumstances which have a bearing on the closure works like accessibility, availability of labour, area flooded, ground level of flooded area in relation to tidal levels,
- information on the causes for failure of a closure and why this happened,
- contractual matters like type of contract, provision of materials.

In case essential information on the topics listed is lacking, a project will not be selected. However, it is always possible that for a certain project, the information is only available about one or two topics and that is still worthwhile to put this project on the shortlist.
Below, some further elaboration will be given about the selection of projects in the different groups.

6.2 Land reclamation projects

The selection of the 27 closures in land reclamation schemes has been based on the aforementioned topics, and also on:

- the availability of sufficient information in order to do some analysis;
- the method of closure with the emphasis on variety;
- the desire to report particularly about closures which took place in former centuries even when the information is rather sketchy;
- the aim to have a fair distribution of land reclamation schemes over the centuries.

For all practical purposes the selection has been mainly restricted to land reclamations which took place in the south-western part of the Netherlands in, what is now called[2], the province of Zeeland.

6.3 Dams in the tidal zone

When one speaks in the Netherlands about a dam it always has a water body at both sides. It is in fact an earth structure that crosses a tidal channel between two islands or an island and the mainland or it closes a tidal inlet[3]. The dams built in the past in the Netherlands and in Schleswig Holstein (Germany) have all been built in tidal waters.
As the aim of this dissertation is to describe and analyse certain hydraulic engineering works (i.e. at dikes and dams) constructed by applying simple means, the dams constructed in the Netherlands after 1930 are not discussed[4].
The dams discussed were built between 1751 and 1925, so over a period of less than 200 years.

[2] This last remark is made as some reclamation schemes, at the time they were constructed, were actually located in in the part of Belgium Flanders, which is now called Zeeuws-Vlaanderen.
[3] The former Zuiderzee can be considered as a tidal inlet and the enclosure by a dam should have been called *Afsluitdam.*
[4] For instance: Afsluitdijk (1932), Brielse Maas (1950), Braakman (1952), dams of the Delta project (1960 – 1971).

Information was only found about nine dams. There is no reason to decrease this number by further selection.

6.4 Repair of breached dikes in the period 1508 – 1916

In most cases the repair of breached dikes is quite different from the closure of tidal creeks in a reclamation scheme as described in Chapter seven.
First of all the breach mostly occurs in winter time and, consequently, as one tries to repair it as soon as possible, the work has to be caried out during short days, in cold and, sometimes, bad weather. In a reclamation scheme, however, one plans to close a major tidal channel before the end of June or, at the latest, by mid-July[5].
Then, in many cases the closure of a breach is characterized by lack of time resulting in improvisation which, inter alia, leads to limited planning, rather than the careful planning as is done for reclamation schemes. The improvisation and the limited planning might easily lead to a failure.
Obviously, nine polders in which dikes were breached and repaired during a period of about 400 years (1508 – 1916) is only a very limited representation of the many polders which, during that period, had to cope repeatedly with breached dikes. Either no technical records were kept about the repair of breaches or, in the course of time, such records, if made, were destroyed. At the best one finds some financial information without any technical data. Still it is thought that the repair works in the nine polders studied give a good impression of the manner in which repair works were carried out.

6.5 Repair of dikes breached during the storm surge disaster of 1 February 1953

6.5.1 Longlist of breaches

On 1 February 1953, a storm surge took place which resulted in unprecedented damage to the Netherlands and other countries in Western Europe. Apart from a large number of deaths, damage to agricultural lands, buildings, infrastructure and industry, there was much damage to the sea defences and river flood embankments.[6]
Rijkswaterstaat in its comprehensive Report '*Verslag over de Stormvloed 1953*' (Rijkswaterstaat, 1961, 159) lists breaches of dikes over a length of 47 653 m including flow gaps over a length of 5 683 m. According to the Rijkswaterstaat' definition given above (Section 6.1) it concerns 89 flow gaps. These 89 flow gaps are, apart from other damages to dikes, described in Part 3 of the Report (p. 157 – 340) and their locations are marked in the Report on two maps (Figures 3.2 and 3.3, arrows in red, between p. 158 – 159).
The repair works to the dikes are discussed in detail in Part 4 of the Report (p. 341 – 697). Fig. 6.1 in this dissertation (from another source) gives an overall picture of areas flooded and flow gaps.
A provisional selection (longlist) was made of the flow gaps to be studied and analysed within the context of this dissertation. This longlist is shown in Table 6.1 and presents particulars of 21 flow gaps (provisionally numbered B-1 to B-21).
The data of Table 6.1, in nearly all cases, originate from one source: the aforementioned Report. This Report is easily accessible and can also be consulted on the internet. It is based in turn on a great number of contributions from local waterboards, provincial civil servants, Rijkswaterstaat, the Royal Dutch Metrological Institute (KNMI) and scientific institutes like *Waterloopkundig Laboratorium* and

[5] For June/July there were two reasons: (1) possible bad weather during August and later on and (2) farmers were leaving the works because of work on their farm and fishermen were going to fish on the North Sea and the Atlantic Ocean. Also after the closure, the permanent profile of the dike still had to be established (before the end of September).
[6] Detailed information about this storm surge and the damages it caused can be found in: https://en.wikipedia.org/wiki/North_Sea_flood_of_1953 (accessed on 21 March 2024)..

Laboratorium voor Grondmechanica. So there is no need to provide elaborate descriptions of the 21 projects listed in Table 6.1.

The reduction from 89 flow gaps (Rijkswaterstaat) to, provisionally, 21 flow gaps (longlist) is mainly based on the following criteria for rejecting a flow gap for further study and analysis:

- depth of flow gap is such that it does not satisfy the definition valid for this dissertation;
- flow gap has been closed by utilizing (mainly) mechanical equipment like rock dumping barges, suction dredgers, tugs, floating cranes, draglines;
- flow gap has been closed by incorporating caissons, ships, pontoons;
- flow gap would have qualified for incorporation in longlist but essential information like length and depth of breach is lacking.

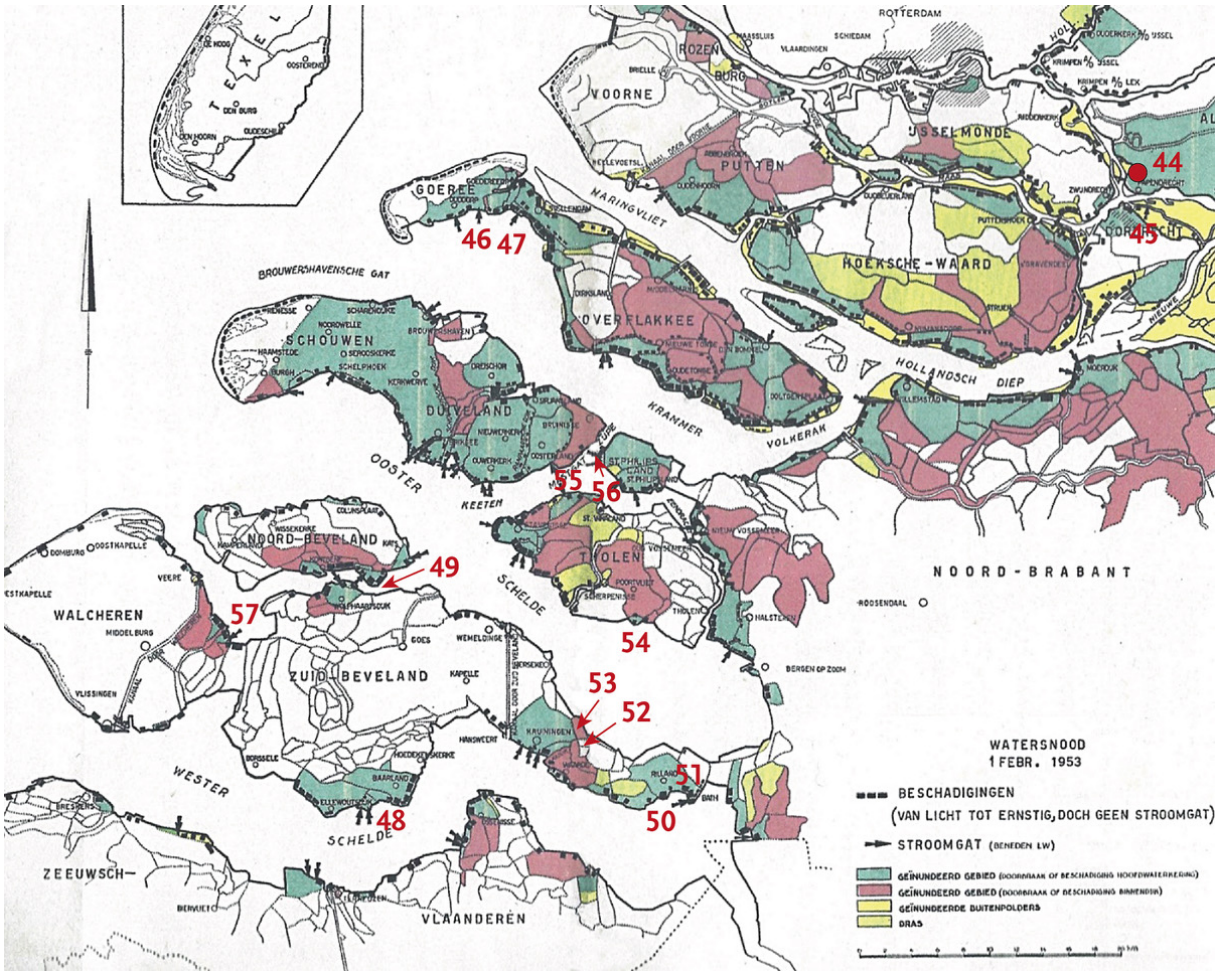

Fig. 6.1 Map of areas flooded in 1953 and breaches in dikes (flow gaps are shown by arrows); the project numbers of the finally selected projects as listed in Table 6.1 are also shown; source: Polytechnisch Tijdschrift (1954) with adaptations

Table 6.1 Longlist of breaches in dikes and appurtenant repair works in 1953 in the Netherlands repaired by applying simple means

No.	Name of Project, its location (province, island or town) and RWS ref. no. [7]	Tide: HW & LW in relation to NAP (m); and tidal range		Area of tidal basin (km^2)		Tidal prism during spring tide (10^6 m^3)	Length[8] of actual flow gap (m) and max. depth (m)	Source[9]	Closure method and other typical features
		spring tide	neap tide	spring tide	neap tide				
	1	2	3	4	5	6	7	8	9
B-1	Breach in dike Polder De Eendracht, island of Texel; no ref.	+ 0.66 - 0.95 = 1.61	+.0.42 – 0.73 = 1.15	appr. 2	0	< 0.5	170 NAP – 1.6 m	RWS (405)	no bed protection required, closure bund of sandbags
B-2 (44)[10]	Breach in Noorddijk Papendrecht, Alblasserwaard 275, no.9	+ 1.44 – 0.54 = 1.98	+ 1.21 – 0.54 = 1.75	5.4	5.4	not calculated, but maximum must be < 2.4	100 incl. 2 gullies of 20 m each at NAP – 2 m	RWS (423)	bed protection, then hor. closure using rock, sandbags and clay from both ends
B-3 (45)	Breach in dike Nieuwe Stadspolder, island of Dordrecht; 285, no 6	+ 1.44 – 0.54 = 1.98	+ 1.21 – 0.54 = 1.75	3.1	3.1	appr. 1	28 MLT – 6 m	RWS (458)	fascine dam and closure bund of sandbags
B-4 (46)	Breach in dike at Jillesweg, Polder Oudeland, Goeree 294, no. 274	+ 1.45 – 1.17 =2.62	+ 0.95 – 1.02 = 1.97	appr. 8	8	3.4	50 NAP – 2.25 m	RWS (463, 466)	fascine dam up to MLT + 0.50 m; subsequently dam of rock and sandbags
B-5 (47)	Two breaches in dike Kleine Zuiderpolder, but only one is a flow gap; South of Goedereede; 293, no. 259	+ 1.45 – 1.17 =2.62	+ 0.95 – 1.02 = 1.97	0.12	0.12	0.1	30 NAP – 3.5 m	RWS (467, 468)	first two closures in secondary dike; followed by closure of flow gap: fascine dam up to NAP – 2 m, then bund of sandbags high 5 m
B-6	Construction of new dike in order to close a flow gap and extend the existing harbour; Burgsluis, island Schouwen-Duiveland 302, nos 6 and 7	+ 1.58 – 1.47 = 3.05	+ 1.10 – 1.26 = 2.36	flow is through breaches to two polders incl. the Polder Schouwen (89.3 km^2). This last polder was inundated through three different breaches			50 at NAP – 7 m; 60 at NAP – 3 m	RWS (491, 492)	bed protection and fascine dam from NAP – 0.3 m (lowest point) up to NAP + 1.2 m (remainder of work with mechanical equipment)
B-7	Breach in *'inlaag'* Anna Frisopolder; North side island of Noord-Beveland 308, no 4	+ 1.62 – 1.59 = 3.21	+ 1.15 – 1.34 = 2.49	0.20	0.20	< 0.1	80 MLT – 1.5 m	RWS (568)	bund of sandbags, no bed protection
B-8 (48)	Eastern breach in dike Everingepolder; SW side Zuid-Beveland 309, no. 20	from NAP – 0.9 to + 2.20 = 3.1 m on day of closure		1.1	1.1	1.5	90; NAP – 6.4 m	RWS (574, 575)	core of pressed straw packs in closure bund of sandbags
B-9 (49)	Breach in dike Zuidvlietpolder; North side western Zuid-Beveland at Zand-kreek; 309, no. 5	+ 1.81 – 1.67 = 3.48	+ 1.27 – 1.36 = 2.63	0.49	0,49	0.65	25 NAP – 1.50 m	RWS (576, 577)	bund of sandbags against 'wall' of planks; maximum height of bund 4.8 m
B-10 (50)	Western breach in Reigersbergse polder, South of Rilland, Zuid-Beveland; 310, no.61	+ 2.79 – 2.28 = 5.07	+ 1.94 – 1.79 = 3.73	10,9	10,9	not calculated but max. less than 3	100	RWS (578 – 580)	perimeter bund of sandbags at sea side of breach on mud flat
B-11 (51)	Compartmentalization bund '4de weg'; inside Reigersbergse polder; 310, nos 64 & 65	0 to 1.5 m on road base		7.7	7.7	not calculated	n.a.	RWS (249, 578 – 583)	construction of bund by various means incl. 1.2 million sandbags; gradual hor. closure on road base at NAP + 1.20 to 1.50 m
B-12	Breach in dike Zimmermanpolder; Eastern Zuid-Beveland, at Westerschelde 310, no.58	+ 2.79 – 2.28 = 5.07	+ 1.94 – 1.79 = 3.73	2.4	2.4	not calculated but max. less than 1	60; NAP – 4.0 m	RWS (589 – 590)	perimeter bund of sandbags on mud flat at sea side of breach;
B-13 (52)	Breaches in Kadijk (secondary dike); dike between polders Kruiningen and Waarde 310, no.44	1.80 m on sill	appr. 1 m on sill	8	8	not calculated but in the order of 8 through the 13 breaches in these dikes	No detailed inf. and also not relevant	RWS (592 – 599)	perimeter closure bund on existing road made by dumping clay filled bags from jetty
B-14 (53)	Breach no. 13 in Lavendeldijk (secondary dike); dike between polders Kruiningen and Waarde 310, no. 45	+ 2.52 – 2.36 = 4.88	+ 1.73 – 1.76 = 3.49				100 NAP – 2.8 to + 0.8 m	RWS (594 – 595)	Over 60 m (shallow part) cofferdam formed by 2 rows of piles, remaining 40 m closed by means of a jetty transferred into cofferdam; sandbags were dumped from jetty into cofferdam

[7] For RWS read: *Verslag* (Report) *van de Stormvloed 1953* (Rijkswaterstaat, 1961); the relevant page in Chapter 3.3 of the Report is given as well as the reference number(s) of the breache(s) shown at that particular page; the latter are also shown in Figures 3.2 or 3.3 of the Report.

[8] Length in column 7 is always measured in the direction of the dike axis and/or perpendicular to the direction of the tidal flow.

[9] For each breach the relevant page numbers in the Report are given. In a few cases additional information was found in Polytechnisch Tijdschrift, 1*953 Ramp en herstel*; Uitgave B, nos 31-34, 14 August 1954 (Polytechnisch Tijdschrift, 1954).

[10] The numbers between brackets refer to the project numbers in Table 10.1 (Chapter 10) and Fig. 6.1.

No.	Name of Project, its location (province, island or town) and RWS ref. no. [7]	Tide: HW & LW in relation to NAP (m); and tidal range		Area of tidal basin (km^2)		Tidal prism during spring tide (10^6 m^3)	Length[8] of actual flow gap (m) and max. depth (m)	Source[9]	Closure method and other typical features
		spring tide	neap tide	spring tide	neap tide				
	1	2	3	4	5	6	7	8	9
B-15 (54)	Breaches in dike KLaas van Steeland-polder at bunker and sluice; near Poortvliet, island of Tholen 307, nos 53 & 54	+ 209 – 1.97 = 4.06	+ 1.45 – 1.56 = 3.01	21.5	21.5	10 (estimate) (thru 2 br.) 6 to 7 (on 6th Febr.) (through one breach)	64 (but above low tide) 16 NAP – 3.2 m	RWS (618 – 620)	first closure: anchored dam comprising clay bags; second closure dumping rock and clay bags; also by driving wooden beams in the sub-soil as support for the rock
B-16	Two breaches in 'Vierde Dijk', (sec. dike); island of Tholen	+ 1.75 – 1.60 = 3.25	+ 1.15 – 1.42 = 2.57	12.15	12.15	10.3	no information available	RWS (620, 623)	closure by placing 350,000 clay bags in breaches
B-17 (55)	Breach in dike Anna Vosdijkpolder; island of Tholen bordering Mastgat 306, no. 5	+ 1.74 – 1.63 = 3.37	+ 1.24 – 1.36 = 2.60	4.6	4.6	5.4	45 NAP – 3.50 m	RWS (620 – 622)	170 m long perimeter bund , inside polder, made up of clay bags wooden beams, stakes, wire and beet nets
B-18 (56)	Two breaches in dike Willempolder; former island St. Philipsland 305, nos. 3 & 4	+ 1.75 – 1.60 = 3.25	+ 1.15 – 1.42 = 2.57	7.5	7.5	14.4 (for two breaches)	46.5, below low tide 42.5, below low tide	RWS (627 – 628)	all four flow gaps on the island were closed by driving two rows of wooden beams to form a coffer-dam, wide 4 m; the cofferdams were filled with sand-bags or clay bags
B-19 (57)	Breach in dike Suzannapolder; island of Walcheren 307, no, 14	+ 1.81 – 1.67 = 3.48	+ 1.27 – 1.36 = 2.63	0.84	0	0.5	60 m NAP – 1.90 m	RWS (638 – 640)	perimeter bund long appr. 200 m built up of clay bags, the central gully was coffer-dammed by driving two rows of piles
B-20	Breach (one out of four in dikes Royale Polder / Schudde-beurs polder); Moerdijk, Hollands Diep 298, no. 79	+ 1.44 – 0.78 = 2.22	+ 1.12 – 0.78 = 1.90	Initially 60, after compartmenta-lization 16		not calculated as distribution over breaches unknown	50 NAP – 1.30 m	RWS (665, 668)	first bed protection made (2 fascine mat-tresses on top of each other); followed by bund of sandbags
B-21	Breach in dike Herenpolder; West coast of Noord-Brabant, along the Eendracht, 299, no. 9	+ 2.17 – 2.01 = 4.18	+ 1.48 – 1.56 = 3.04	Initially appr. 6.5, after compartmental-ization 1.4		Initially 25 but later less due to secondary dikes	45 NAP – 3 m	RWS (683 – 684)	fascine dam up to MLT, subsequently closure by means of a bund of sandbags.

The contents of Table 6.1 in the columns:

2 and 3: information about the tides

are based on information presented in Rijkswaterstaat (1954): *Tienjarig Overzicht 1941 – 1950 der Waterhoogten langs Zeeuwse stromen,… etc.* First the gauge station is selected which is the nearest to the site of the breach. For that particular gauge station the spring and neap tide levels are then taken which are valid for the beginning of 1951; see for instance RWS (1954, 36): the so-called end averages (*slotgemiddelden*) at Alblasserdam for 1951.0.
In some cases the average of two gauge stations is taken. If Rijkswaterstaat gives specific data for the day of closure such data are adopted.
Initially, it was thought that the *Getij Tafels (1953*) of Rijkswaterstaat would provide information on the expected tide levels on the day of closure. It turned out, however that, due to the limited number of tables presented in the *Getij Tafels* , only in a few cases such figures are available.

4 and 5: area of tidal basin during spring and neap tides

are, apart from the main source already mentioned, derived from various sources like introductions in regional archives to inventories of relevant polder archives.

6: tidal prism during springtide

is based on calculations using the digital program CRESS (www.cress.nl). But the available input does not always satisfy the requirements of CRESS for making the calculation. One gets for instance the message: 'The parameter area (8) is outside the indicative range, etc.' In that case the words 'not calculated' have been inserted in the particular cell of the Table and an estimate has been given instead. Sometimes, Rijkswaterstaat (1961) or Polytechnisch Tijdschrift (1954) gives the desired information.

7: Length and depth of flow gap

it must be realized that in most cases the actual flow gap is part of a much longer breach than the length indicated for the actual flow gap.

(formerly 12): cost of repairs

It would have been interesting to listen the cost of repair of a particular flow gap and correlate it to parameters such as tidal prism, tidal range, area of tidal basin, manpower involved, but no cost of repairs of individual flow gaps has been found. In all cases the financial figures presented comprise the cost of overall dike repair in a particular polder or polder district (*waterschap);* accordingly this column has been deleted.

6.5.2 Selection of projects for the shortlist from Table 6.1

Below, each project which has been listed in Table 6.1 and which has not been selected for the shortlist, will be summarized. Sometimes, information additional to that already shown in the said Table will be given. Also the reason(s) for disqualification of a project for the shortlist will be mentioned.

B-1 Breach in dike Polder De Eendracht, island Texel

Breach is closed by using sand and clay and, finally sandbags. No quantities given. Also tidal prism very small (and during neap tides even nil) due to high ground level in polder. This ground level is NAP + 0.40 m according to Hage (2015, 154). Moreover, tidal ranges are small (1.15 – 1.61 m) compared to tidal ranges elsewhere. As no quantities are given, tidal ranges are small and the closure did not require a special effort, it has not been selected for analysis.

B-6 Construction of new dike in order to close a flow gap and extend the existing harbour, Burghsluis, island Schouwen-Duiveland

Closure only partly by using simple means (fascine dam) over a height of 1.5 m. Current velocities must have been high in view of rapid development of deep scour holes, size of tidal basin and its low ground level: average NAP – 1 m, but near Burghsluis it is NAP – 0.5 m (Hage, p. 26). However, situation is too complicated (polder is flooded through three breaches, including the one at Schelphoek) and as sufficient data are lacking, it is not possible to make an analysis of current velocities and the like. Moreover, sand dam was placed by a suction dredger.

B-7 Breach in inlaag Anna Frisopolder, island Noord-Beveland

No cross-sections or quantities given. Not known whether or not a perimeter bund around scour hole was constructed. As ground level in polder is at NAP + 1.2 m (Hage, p. 72). there will be no inflow of water in polder during a neap tide. The necessary information to do an analysis is lacking.

B-12 Breach in dike Zimmermanpolder, Eastern Zuid-Beveland, at Westerschelde

Construction of a circa 200 m long, one metre high, perimeter bund on mudflat at seaside of breach. Input of manpower is known but cross-section of perimeter bund is missing as well as duration of closure operation. Ground level in polder is at NAP + 1.8 m (Hage, p. 88). The necessary information to do an analysis is lacking.

B-16 Two breaches in 'Vierde Dijk' (secondary dike), Island of Tholen

No information available regarding size (lengths, depths) of the flow gaps. Also duration of closure works and size of manpower missing. Ground level in polders varies between NAP + 0.5 and +1 m (Hage, p. 66). The available information is insufficient to do an analysis.

B-20 Breach (one out of four) in dikes Royale Polder / Schuddebeurspolder, Moerdijk

Apparently, closure of this 50 m wide breach was relatively easy. Two fascine mattresses were placed on top of each other which resulted in a sill at an elevation slightly below NAP. Subsequently, a closure bund, using sandbags was constructed having its crest at NAP + 2 m. No information has been presented about the duration of the closure process and the number of labourers involved. Also distribution of in-/outflow of tide over the four breaches not given. Ground level in polders varies considerably: NAP – 0.3 to – 1.4 m (Hage, p. 116 and 118). The available information is insufficient to do an analysis.

B-21 Breach in dike Herenpolder, West coast of Noord-Brabant along the Eendracht

More or less the same method of closure followed as for no. B-20 above. Also here no information on duration of closure process and number of labourers involved. Ground levels in this polder vary between NAP + 0.7 + 1 m (Hage, p. 110). The available information is insufficient to do an analysis.

6.6 Closures in Ghana and Bangladesh in the period 1969 – 1985

After the breaches, caused by the storm surge of 1 February 1953, had been repaired, simple means are not any more applied in the Netherlands for closing tidal channels. The dams to be built within the context of the Delta Project could not have been built anyhow by applying simple means. Moreover, manual labour is as much as possible being replaced by machinery; fascine strings are made by utilizing a machine and the fascines and reed in a fascine mattress are succeeded by poly-propylene filter fabric.
But, in 1969 an opening made in the sand ridge between the sea and a lagoon in Ghana had to be closed again and it turned out that this could only be achieved by using again simple means. Waterborne equipment required for the closure is either not available in the country or cannot reach this particular site. Only equipment used in road construction for dry earth moving is available while straw mats ballasted by sandbags will take over the function of fascines and stone for bed protection.
A few years later (1976), an assignment starts in Bangladesh (which has become independent in 1971) aiming at the introduction of a Dutch closure method for tidal creeks which is less risky than the local closure method. As Bangladesh at that time, and, at the time of writing, still is, pre-eminently a country of manual labour (e.g. in agriculture, infrastructural works and house building) it is logic to introduce also here labour-intensive methods for the closure of tidal creeks. This is successful and three creeks are closed (in 1979, 1980 and 1982) using a Dutch closure method adapted to local circumstances (materials, skills and suitable season). One of these closures fails a few day after completion but this is not due to the method used.
In the early eighties, an irrigation project is being carried out in Bangladesh, which, inter alia, requires the construction of an estuary dam and large sluice at the mouth of the Feni River. Though, in the summer of 1982, the construction of the large sluice is well under way, the consulting firm, responsible for design and site supervision of construction of the project, is not able to present a plan for closure of the tidal river at its mouth. After consultation of the financing institutions (incl. the World Bank) the Client[11] then decides to ask the Dutch consulting firm Haskoning (now RoyalHaskoningDHV) to design and supervise construction of the 1 330 m long estuary dam. On 28 February 1985, the last openings (overall length 775 m) in the closure bund are successfully closed, starting at Low tide, in 7½ hours by 10 000 labourers.

[11] Bangladesh Water Development Board (BWDB)

CHAPTER 7 Land reclamation schemes

7.1 Introduction

In Chapter six, the selection of closure projects was discussed and, more specifically , in Section 6.2 the criteria for selecting 27 closures in reclamation schemes was explained. The main particulars of these 27 schemes are presented in Table 7.1. The approximate location of these 27 projects have been marked by their project numbers on Fig. 7.1.

Table 7-1 Reclamation schemes (polders), selected for analysis.

project no.	name of polder and location (province, town, island) (name of major creek, if any)	year or date of reclaim-ation/ closure	tidal range (m) and /or levels of high and low tides		reclaim-ed area (km^2) and ground level	tidal prism during spring or neap tide (10^6 m^3)	width of creek(s) to be closed (m) and max. depth in m	closure method and other typical features	overall cost of reclamation (ditto but embankments only)	Sources[1]
			spring tide	neap tide						
	1	2	3	4	5	6	7	8	9	10
1 1a 1b	Oost- & West Hinkele-polders, Island Zuid-Beveland (Hinkele) Dam no. 2 Dam no. 4	 1263 & 1269	 3 3	 2 – 2.5 2 – 2.5	 1.20 2.35	all during neap tides 0.8 – 1.1 1.0 – 1.3 0.9 – 1.2 1.2 – 1.5	sills at level of mean low tide 150-200 150-200	closure method not known but probably identical to that used for reclamation of polder Groot Zuid-Beijerland		Obreen (1926); De Nooijer (1932) Dekker (1971)
2	Groote Waard;, new dike Broek-Zevenbergen	1410 - 1412	MHT = NAP + 0.70 m MLT = NAP – 1.30 m		39 MHT and lower	assessed at 4.65 for the situation during final closure (neap tide)	no information available	reference is made to Section 7.4	(£ 8 474 – 5 – 2)	Hingman (1885); Vriend (1961) Van der Helm (1988); Leenders (2021)
3 3a 3b	Groot Zuid-Beijerland; (Borrekeen, Dam A) (Grote Gat, Dam B)	 1631 & 1631	2.40	1.70	9.96	 not calculated 0.37 (neap tide)	 81 378	sills at level of mean low tide; followed by horizontal closure	f 246 876 (f 223 367)	Baars (1973); Baars (1973a)
4	Jonkvrouw Annapolder Island Noord-Beveland	1727	3.0 (est.)	2.5 (est.)	2.15 NAP + 1.2 m	0.18 (neap)	31 rods= 126 m & smaller ones	no data about closure method but inform. on constr. cost might be useful	(£ 6 726-7-0)	NL-MdbZA_3219_1 & _82 De Bruin & Wilderom (1961, 62-63)
5	Anna-Frisopolder, island Noord-Beveland	1747	3.0 (est.)	2.5 (est.)	1.29 NAP + 1.00 m	0.02 (neap)	17 rods = 69 m (one creek only)	no data about closure method but inform. on constr. cost might be useful	£ 4 199-14-9 (£ 3 593-16-3)	NL-MdbZA_3212_1 & _6 De Bruin & Wilderom (1961, 63 & 256)
6	Reigersbergsche-polder, island Zuid-Beveland (Vinkenisse Gat & Batse Kaaij)	1773	3.5	2.5 – 3	10.87 NAP + 1.50 m	2.47 (neap)	Vinkenisse Gat 242 & 359 Batse Kaaij 365 & 268	the two major tidal channels to be closed at both ends have been compartmentalized prior to closures	(£ 58 185-15-2)	NL-MdbZA_3157_2, _3a & _4 Wilderom (1968, 168-170)
7	Henriëttepolder; island St.Philipsland (De Luijster)	1776			1.09 NAP + 0.7 m		16 rods = 65 m	no data about closure method but inform. on constr. cost might be useful	(f 33 214)	NL-MdbZA_63_81 & _82 Wilderom (1964, 146-150)
8	Eendragtpolder, Oost Zeeuws-Vlaanderen (Zeeuws-Vlaanderen)	1777	3.5 (est.)	3.0 (est.)	2.24 NAP + 1.4 m	0.11 (neap)	20 small creeks , wide 4- 45 deep 0.7 to 1.6	no data about closure method but inform. on constr. cost might be useful	(f 59 897 for 4845 lineair metre of dike) (cost estimate)	NL-MdbZA_3733_18 and _3710_K10 & _K 60 Wilderom (1973, 170 and 224)

[1] A source in the Zeeuws Archief will for instance read as: NL-MdbZA_3219_1 & _82. This is the so-called ISIL-code, which in this case means: Country NL = Netherlands; town Mdb = Middelburg, i.e. the town where the archives are located; ZA = *Zeeuws Archief*; ; 3219 = access code for the particular collection; -1 or -82 are the numbers of the relevant files (identified as *pak, bundel* or *omslag*). ISIL stands for International Standard Identifier for Libraries and Related Organisations. In the Netherlands the ISIL codes are made available, if requested by an institution, by the *Nationaal Archief* (National Archives).

project no.	name of polder and location (province, town, island) (name of major creek, if any)	year or date of reclaim-ation/ closure	tidal range (m) and /or levels of high and low tides		reclaim-ed area (km^2) and ground level	tidal prism during spring or neap tide (10^6 m^3)	width of creek(s) to be closed (m) and max. depth in m	closure method and other typical features	overall cost of reclamation (ditto but embankments only)	Sources[1]
			spring tide	neap tide						
	1	2	3	4	5	6	7	8	9	10
9	Wilhelminapolder Island Zuid-Beveland (Schenge, Kapelle-kreek, Hondegat, etc.)	1809			15.70 NAP + 1.2 to + 1.7 m	not calculated	widest creek is Schenge (375)	bed protection in the seven main creeks followed by two fascine dams and pakwerks	f 630 216 (f 420 465)	NL-MdbZA_250 _128 and _294_1500 Wilderom (1968, 134-135)
10	Catharinapolder, Oost Zeeuws-Vlaanderen (Zeeuws-Vlaanderen) (Hellegat)	1846	3.5 (est.)	3.0 (est.)	4.15 NAP + 1.2 m	not calculated	no details but three creeks wide 30, 50 & 80 shown on map	no information traced	f 31 906 (f 26 500 for 770 linear metre of dike)	NL-MdbZA_3754_33 & _38 and _3710_K_28 Wilderom (1973, 155)
11	Mortierepolder Island Walcheren (old harbour access channel)	1846			0.5 NAP + 1 m			no information traced	f 5 600 (f 2 775 for 350 linear metre of dike)	NL-MdbZA_3016_7 Wilderom (1968, 96-97)
12	Perponcherpolder Island Zuid-Beveland (Schenge)	1847			1.96 NAP -0.3 to + 1.7 m		160	contractor may apply a bed protection in the Schenge ; no further details		NL-MdbZA _ 3155 – 3, _8 & _ K-1 Wilderom (1968, 135-136)
13	Anna Jacobapolder & Kramerspolder; on island St. Philipsland (Bruintjeskreek)	1847	+ 1.75 - 1.60 = 3.25	+ 1.15 – 1.42 = 2.57	8.7 & 1.5 NAP +1 and + 1.5 m	3.1 (neap)	300 south side; 600 north side; max. depth in axis southern dam 4.2 below mean high tide	bed protection specified for southern end of creek	the given unit rates might be useful for cost analysis	NL-MdbZA _ 3451 & _63_84; De Bruin (1953, 31) Wilderom (1964, 150-151
14	Willem III polder Oost Zeeuws-Vlaanderen (Beverskreek)	1861			2.36 NAP + 1.6 to + 1.8 m			the only reference to closure of creeks is when contractor plans to compartmentalize the Beverskreek	f 106 699 (contract sum)	NL-MdbZA _ 133.1_ 489 and _294_1579 Wilderom (1973, 155)
15	Zwinpolder, West Zeeuws-Vlaanderen (Zeeuws-Vlaanderen) (Grote Gat)	1864			2.35 NAP + 0.2 to 1.8 m			closure method could not be established	(f 39 000 for 800 linear metres of dike)	NL-MdbZA _ 3533_ 56 & _57; _292_1968_14c Wilderom (1973, 87)
16	Schengepolder, island Zuid-Beveland (Schenge)	29th July 1874	4.2	3.2	5.38 NAP + 0.9 to +1.6 m	1.7 (neap)	60, 160 & 310; resp. LW – 2, -2.5 & -4.5	fascine dam, closure bund of rock on top; first two closures with bund built up of earth failed due to piping	f 315 000 (f 240 000)	NL-MdbZA_3158 & _3160 &_3160_K_27 Wilderom (1968, 136-137)
17	Van Lyndenpolder, Oost Zeeuws-Vlaanderen (Zeeuws-Vlaanderen) (Hellegat)	1876			1.74 NAP -0.3 to + 1.8 m		two creeks wide 20 & 40	no information traced	(f 53 500 for 800 linear metres of dike)	NL-MdbZA _ 3756 _13 & _14 and _ 3710 _K-45 Wilderom (1973, 155)
18	Sluispolder, island of Tholen	1877			0.19		3 creeks deep 2.5, 1.7 and 1 m below MHT; widths 5, 3 and 1.5	the area was reclaimed by constructing a 250 m long dam in 23 days	(f 12 490)	NL-MdbZA _ 3421 _ 225 Wilderom (1964, 139)
19	Damespolder, island Zuid-Beveland	1884	AP + 2.5 AP -2.0 = 4.5 (est.)	AP + 2 AP – 1.8 = 3.8 (est.)	2.10 NAP + 1.7 m	0.28 (neap)	2 creeks wide 25 & 350 m, deep resp.AP – 0.5 & + 0.64 m	as the only documents found were a map and some profiles, closure method could not be established.	f 186 000; dike has an overall length of 3.8 km	NL-MdbZA _ 651.1_ 971 Wilderom (1968, 174)
20	Hedwigepolder Oost Zeeuws-Vlaanderen (Zeeuws-Vlaanderen) (Doelsche Gat)	1907	4.5 (est.)	3.5 (est.)	3.16 NAP + 2 m	0.42 (neap)	Some 12 creeks, max.width 130, deep 2.3 below MHT	as the only documents found were tender documents, a map and some profiles, closure method could not be established		NL-MdbZA _ 652.1 _ 681 Wilderom (1973, 152-153)
21	Hoogerwaardpol-der, (mainly) island Zuid-Beveland	1912	4.0 (est.)	3.5 (est.)	2.17 NAP + 1.0 to + !.7 m	0.19 (neap)	Some 23 creeks, max.width 160, deep 2.3 below MHT	as the only documents found were tender documents, a map and some profiles, closure method could not be established		NL-MdbZA _ 651.1 _ 874 Wilderom (1968, 175)
22	Dyckmeester-polder; in tidal inlet Braakman, Zeeuws-Vlaanderen (Hondegat & Geenenskil)	1 Aug. 1918 (both creeks)	MHT = NAP + 1.85		3.44 NAP + 0.7 to 2.2 m	not calculated but according to Wikepedia 2 (2 days after neap)	65 & 78.5; resp. MHT -2.5 & - 2.7	closure bunds filled from bridge by dumping clay from side tip wagons; mainly hor. closure	f 1 708 000 (f 900 000)	Verhoeven (1924) Huitema (1947, 129 et seq.)

project no.	name of polder and location (province, town, island) (name of major creek, if any)	year or date of reclaim-ation/ closure	tidal range (m) and /or levels of high and low tides		reclaim-ed area (km^2) and ground level	tidal prism during spring or neap tide (10^6 m^3)	width of creek(s) to be closed (m) and max. depth in m	closure method and other typical features	overall cost of reclamation (ditto but embankments only)	Sources[1]
			spring tide	neap tide						
	1	2	3	4	5	6	7	8	9	10
23	Kreekrakpolder, In former Kreekrak between isl. Zuid- Beveland and Noord-Brabant (Oude Woensdrechtse Kil)	1923	NAP + 2.5 NAP -2.0 = 4.5 (est.)	NAP + 2 NAP – 1.8 = 3.8 (est.)	2.57 NAP to NAP + 2 m	not calculated	initially width of creek is 480 but max. depth only NAP - 0.40	the final opening to be closed had a width of 65 m. Closure bund filled from bridge by dumping clay from side tip wagons	f 298 310 (contact sum) dike has an overall length of 2 400 m	NL-MdbZA _ 651.1 _ 975 Wilderom (1968, 174)
24	Hellegatpolder, Oost Zeeuws Vlaanderen (Hellegat)	13 Aug. 1925	MHT = NAP + 1.96 MLT = NAP - 2.04 (appr.)		1.35	not calculated	35 m at NAP and depth NAP – 0.96	closure bund filled from bridge by dumping clay from side tip wagons	f 264 800 (contract sum)	NL-MdbZA_612_ 184 to _191; various articles in reg. newspapers
25	Biesboschpolder; Z-Holland, bordering Nieuwe Merwede (Zuidgat) (Noorderdiep)	1926 4 August 1 Sept.	MHT = NAP + 1.20 m MLT = NAP – 0.75 m		9.25 NAP + 0.8 to + 1.0 m	not calculated	55 , NAP – 1.10 (sill) & 77 NAP – 1.10 (sill)	sill in Zuidgat comprises one fascine dam; sill in Noordgat comprises two fascine dams with clay core in between; closure bunds filled from bridge by dumping clay and sandbags from side tip wagons	f 1 317 118 (f 864 500) (employment project)	Verhoeven (1929)
26	Abraham Wissepolder, isl of St. Philipsland	25 June 1935	+ 1.75 - 1.60 = 3.25	+1.25 -1.42 = 2.57	0.6 NAP + 1.07 m	0.05 (neap)	30 NAP	last gap was closed by dumping clay clods from wheelbarrows	f 150 000 (employment project)	NL-MdbZA _ 3457_39 Wilderom (1964, 151-152)
27	Damming tidal inlet Dijkwater, Schouwen-Duiveland	26 August 1954 (failed)	3.2	2.4	1.80 NAP + 0.4 m	2.9 (spring)	± 60 NAP – 3.10	Sill at NAP – 1.50 m cofferdam filled from bridge by dumping clay from side tip wagons up to NAP + 2.50	± f 2 500 000	Van Duivendijk (1954); Kalkwijk (1955); Rijkswaterstaat (1961); CUR/RWS (1994)

Fig. 7.1 Approximate locations of the projects, listed in Table 7.1, marked by their project number

Through the ages people living in the deltas of Rhine, Meuse, Scheldt and Eems have lost and reclaimed lands along the rivers, estuaries and sea. There is a general belief that such land reclamation is difficult and requires special skills and tools. But, as will be discussed below, this is only true to a certain extent.
Land reclamation schemes (also called polders) were usually initiated by the owner(s) of the area to be reclaimed and that could be the central 'government', monastic orders, the nobility, the regional or local gentry, or the rich business men in the towns.
Prior to the start of the reclamation four administrative/financial/technical arrangements had to be finalized:

- permission (i.e. a charter, more recently called a concession) for the reclamation to proceed should have been received from the government (that could be a central government, the sovereign or his regional representative);
- the people initiating the reclamation (hereafter called the charter-party or concessionaire) must have signed an internal agreement as to the technical, financial and administrative arrangements required to jointly carry out the scheme;
- the charter-party must have made arrangements with the adjacent polders to carry out certain works in order to maintain the existing rights of the latter (like discharge of drainage water into the sea, estuary or river);
- last but not least it should have been established that the envisaged reclamation is technically and economically feasible.

In this chapter mainly the technical features of the closures will be discussed.
The first technical question to be answered should be: is the area 'ripe' for reclamation? Verhoeven (1924, 189) states that an area is considered 'ripe' for reclamation when the average ground level of the mud flat (also called salt marsh) has reached a level of mean high tide (MHT). He adds that the banks of the tidal creeks inside the salt marsh may have a slightly higher level: 10 – 20 cm.
One should add that the top layer of sediments (say 50 to 80 cm) of this tidal marsh (*schor* or *kwelder* in Dutch) must contain a high clay content, otherwise it is not interesting for agriculture. In most cases the clay layer, in the course of a number of years, has been deposited on a sandbank and when the clay layer has reached the said level MHT all kind of salt loving grasses and other plants are growing on the clay.
In the past, when pumping stations did not yet exist and windmills were costly and not always able to ensure at all times adequate drainage of the land, it was necessary to discharge drainage water from the polder at low tide through sluices. In this respect Huitema (1947a, 259) argues that the ground level of the area to be reclaimed at least has to be at MLT + 1.55 m. But this requirement is in any case not a problem in Zeeland with its high tidal ranges.
Prior to discussing a number of land reclamation schemes in greater detail, something must be said about the size of the schemes. Wilderom[2] , in his study of the *waterstaat* in Zeeland, counted 640 polders. But, probably, there are some double counts as some polders got lost due to shifting of tidal channels, flow slides or storm surges, while, later on, they were reclaimed again, having a completely different size and shape and also got another name. A quarter of these polders, 163, was created between 1300 and 1550 and their overall area is 20 000 ha. This implies an average size of 123 ha or 1.2 km^2. But many polders have an area between 50 and 75 ha and the smallest are only 4 ha. The polders reclaimed after the 16th century, in general, are larger.
If the keyword *bedijking* (embankment) is inserted in the search programme of the *Zeeuws Archief* (Zeeland Archives) one gets 360 hits, the keyword *overstroming* (flooding) results in 36 another hits, but there is some overlap. The research, based on these hits, led ultimately to selection for further

[2] See De Bruin & Wilderom (1961) and Wilderom (1964 – 1973)

analysis of 24 reclamation schemes in the province of Zeeland. This results, together with selected reclamation schemes located elsewhere[3], ultimately in the 27 reclamation schemes presented in Table 7.1 which have been carried out (including failures) during the period 1263 – 1954.
The aforementioned selection was, on the one hand, based on the availability of technical information (though, in most cases this is very incomplete) and on the other on interesting technical features. Part of the information retrieved (if available) for further analysis has been presented in Table 7-1 but in addition information was considered interesting when it concerned:

- contract conditions and specifications
- quantities, overall cost and unit rates
- compartmentalisation
- dike profiles and maps
- area of creeks inside the reclaimed area in relation to the overall area embanked.

7.2 Analysis of technical features

In the next sub-sections the analysis of a number of technical features will be carried out. This required a more detailed description of the three schemes realized during the 13th – 17th century as the available information is not always straight forward and needs some interpretation. It is, however, not considered necessary to present such a detailed description in Table 7.1 for all 27 schemes. Such a description would make reading rather cumbersome. Moreover, when the *Zeeuws Archief* is mentioned as source, one can find on the website general information about the scheme in the *Inleiding* (Introduction) to the collection concerned. Maps and technical drawings have been inserted to elucidate the text.
Following the subject of this dissertation, it was, initially, the intention to focus on methods used for closing tidal channels. But, apparently, such closures, in most cases have been easy and did not warrant much attention in the tender documents or in construction (progress) reports. This is illustrated by the contents of column eight in Table 7.1: information on closure methods used is missing for most reclamation schemes. But, as quite some information had been collected about other technical features like construction cost and unit rates, it was decided to present the analysis thereof. Moreover, information about the use of temporary bridges in closures is such that this topic could be discussed in some depth.

7.3 The East- and West Hinkelepolders (project no. 1)

7.3.1 Introduction

The study of literature made so far has not revealed any detailed technical report about the closure of a tidal channel prior to 1631 when the reclamation of the 'Groot Beijerland polder' was realized. Still, many such closures must have taken place as, during this period (12th – 16th century), reclamation of mud flats takes place frequently.
Below, as an example of an early tidal closure in stages, the damming of the tidal river Hinkele, is described. This tidal closure is selected because Dekker (1971, 168) states:

> "Of all hydraulic engineering achievements, performed by the Order of the Cistercians in the (County of) Zeeland, we are best informed about the damming of the Hinkele by the monks of the Abbey of Ter Doest."

Dekker continues by describing the complicated relationship between the Abbey , on the one hand, and the various Manors (*Heerlijkheden*) in the area on the other. The Abbey has the ownership of agricultural lands in the area and wants to protect its property against storm surges. But the Manors may have other (conflicting) interests, like navigation or drainage of their lands. Moreover, the Abbey

[3] This selection was based on the study of literature.

has the skills (or can organize the skills) to dam tidal channels as well as the money while the Manors are lacking both.

7.3.2 The Hinkele and its planned damming

Initially, the Hinkele is a rather wide tidal channel, between De Honte (later called Westerschelde) in the west and the Schelde (later called Oosterschelde) in the east. Accordingly, the Hinkele crosses the south-eastern part of the island South-Beveland (Zuid-Beveland) from west to east (Fig.7.2).
By the middle of the 13th century the Hinkele had already been closed at the east side (Dam 1). But no details about this damming are known.
At that time the actual tidal channel, according to Dekker (p. 168) has a width of 150 to 200 metres, but there are extensive mud flats (*schorren*) at both sides of the channel. These mud flats will be flooded periodically during high tides. The Hinkele is much wider at its mouth after the confluence with the Morlode.
The initial idea is to dam the tidal channel by building successively the Dams 2 and 3 (Fig. 7.2). A dam at the wide mouth of the tidal channel, apparently, is not considered to be technically possible.
In May 1263 the Abbey comes to an agreement with the Manors concerned about the construction of both dams. Dekker assumes that the storm surge of 1262 (Gottschalk, 1971, Vol. I, 202 et seq.) is important in arriving at this agreement.
It is decided to build Dam 2 in 1263 and Dam 3 in 1264. Dam 2 has indeed been built in 1263 but the construction of Dam 3 for one or another reason does not materialize. The polder reclaimed behind Dam 2 becomes known as the Oosthinkelepolder.

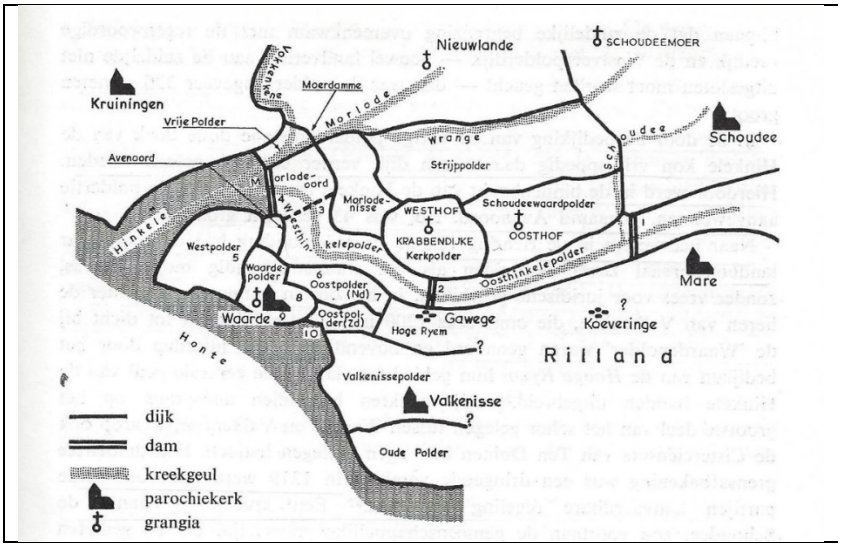

Fig. 7.2
The embankment of Waarde and Krabbendijke and the dammings of the Hinkele, 1186 – 1269;
source: Dekker (1971, 151)

7.3.3 Information available regarding the damming of the Hinkele

Dekker (p. XV-XX) mentions some six pages of sources, which he has consulted for his dissertation. But there are no details in his dissertation about the method of damming the Hinkele, which has been utilized by the Cistercians. Still it is possible to make a few observations.
In an Appendix (Dekker, 620 et seq.) a list of polders in South-Beveland, diked between ± 1150 and 1570, is presented. One learns that the Oost- and Westhinkele polders have areas of 306 and 600 g*emet*', respectively. By multiplying these figures by 0.392 one gets the areas expressed in ha: 120 and 235 ha.[4]

[4] According to the Meertens Instituut the so-called *Blooise* land measurement system was in use on the island of Walcheren and in the old polders of North- and South-Beveland. One *Bloois gemet* = 0.392 ha = 300 square rods.

But for damming purposes only the area of the tidal basin during a neap tide is relevant. The area of this basin is determined by the width of the tidal channel during neap tide (Dekker suggests 150 to 200 m) and its length. By comparing distances between villages on the 'Grote Historische Topografische Atlas, Zeeland 1856 – 1858' (1992) with those on Fig. 7.2 (no scale given) one arrives at the length of both tidal channels: East Hinkele 3.35 km and West Hinkele 3.83 km. The areas of both tidal basins can then be calculated. Table 7.2 gives an overview.

Table 7.2 Characteristics of East- and Westhinkelepolders

polder:	area of polder (ha)	length of tidal channel (m)	width of tidal channel during neap tide (m)	area of tidal basin A during neap tide (km^2)	reduced area of tidal basin A during neap tide (km^2)
Easthinkele	120	3 354	150	0.503	0.402
			200	0.671	0.537
Westhinkele	235	3 833	150	0.575	0.460
			200	0.767	0.614

But the ratio shown in the Table between area of tidal basin and area of polder (0.33 – 0.56) is not realistic as it assumes that the Hinkele has the same width everywhere along its length . After the construction of Dam 1, somewhere around the middle of the 13th century, the Hinkele must partly have silted up. Accordingly, its width and depth will have decreased in eastern direction. Conservatively, a reduction of 20 % is applied.

7.3.4 Further assumptions regarding a possible method used for damming

First, it is necessary to determine the local tidal difference during a neap tide.
As early as the 13th century, these differences are still rather low. Coen (2008, 56), on the basis of historical observations (gauge readings), states that the mean tidal difference at Antwerp goes up from 3.00 m in 1567 to 5.19 m in the period 1981 – 1990. It can therefore safely be assumed that the tidal difference during a neap tide at the mouth of the Hinkele in 1269 will be around 2 to 2.5 m. Possibly it is even less at locations west of Antwerp, along De Honte.
It is also assumed that the tidal difference during springtide level does not exceed 3.00 m and that for that case characteristics (i.e. width of tidal channel) as presented in Table 7.2 do not change.
It is further assumed that one starts the closure works by dumping clay clods[5] from flat-boats[6] up to a level slightly (20 cm) above mean low tide (MLT). This dumped earth mass is called the primary dam.
It is not known how the final damming is being made. For instance, the use of rock and stones (boulders, bricks) as well as fascines (*rijshout*) in the 13th century for damming purposes is not at all sure. The earliest reference to stone and *rijshout* found so far is Hingman (1885, 52 et seq.), who refers to the construction of the dike between Broek and Zevenbergen in 1421, where stones (mainly bricks) and *rijshout* were used in large quantities[7].
An important feature of this lack of knowledge is the non-existence of technical archives of water boards prior to (roughly) 1500.
The dissertation of Gottschalk (1983) as well as the dissertation of De Kraker (1997), both about parts of Zeeuws-Vlaanderen and starting in the Middle Ages, using the earliest available sources, only mention the use of *rijshout* after 1500.
Nevertheless, the use of both materials for damming purposes cannot be excluded completely.
For the time being it is assumed that a horizontal closure is made on top of the aforementioned primary dam, which was constructed more or less vertically, by dumping clay clods.

[5] The operation by water-borne transport is called *vletten* in Dutch.
[6] *Platbodems* like *hoogaarzen.*
[7] Reference is made to Section 7.4 below for more information.

BOX 7.1 Determination of the maximum water levels and current velocities during staged damming of the Hinkele

Mean low tide(MLT) is used as reference level. The level of the sill is then MLT + 0.20 m.
At neap tide, water levels will then be for a tidal difference:

- of 2.0 m: low water level MLT + 0.20 m; high water level MLT + 2.20 m
- of 2.5 m: ,, ,, + 0.00 m; ,, ,, + 2.50 m

Tidal difference at springtide is assumed to be 3.00 m.
If mean sea level MSL is fixed at MLT + 1.20 m then: high water level springtide is MLT + 2.70 m and, low water level springtide is MLT – 0.30 m.

In order to determine the values of various parameters in the closure gap the Coastal end River Engineering Support System CRESS[8] (www.cress.nl) is used. This is being done as follows: at left side of window select successively water movement, flow, tidal flow, flow through closure gap.
The following input parameters are now required:

- area of the tidal basin A = 0.402 to 0.614 km^2
- width of the closure gap B = 10 to 150 and 25 to 200 m
- tidal difference at the estuary De Honte Hz = 2.00 m, 2.50 m and 3.00m
- depth of sill (=crest level primary dam) in relation to MSL h = 1.21 m, 1.26 m and 1.51 m [9]
- discharge coefficient for a short free overflow weir c = 0.90
- period of tide T = 12.5 h

In CRESS, at left side of window, select successively water movement, flow, tidal flow, flow through closure gap.
Subsequently, parameter values, as given above, are entered. Based on four values of A and 3 of Hz it leads to 12 different tables giving for various values of B the values for

- water level difference in the basin H_B (m)
- maximum current over sill U_{max} (m/s)
- max head over sill Δh_{max} (m

Dam 2 East Hinkele, width of tidal channel 150 m, area of tidal basin A = 0.402 km^2

neap tide Hz = 2.00 m					neap tide Hz = 2.50 m				springtide Hz = 3.00 m		
B (m)	H_B (m)	U_{max} (m/s)	Δh_{max} (m)		H_B (m)	U_{max} (m/s)	Δh_{max} (m)		H_B (m)	U_{max} (m/s)	Δh_{max} (m)
150	2.00	0.28	0.01		2.50	0.34	0.01		3.00	0.34	0.01
100	2.00	0.43	0.01		2.50	0.51	0.02		3.00	0.51	0.02
50	2,00	0,85	0.05		2.50	1.02	0.07		3.00	1.03	0.07
25	1.97	1,68	0.18		2.45	2.01	0.25		2.96	2.02	0.26
10	1.51	3.23	0.66		1.81	3,71	0.86		2.29	3.92	0.97

Dam 2 East Hinkele, width of tidal channel 200 m, area of tidal basin A = 0.537 km^2

neap tide Hz = 2.00 m					neap tide Hz = 2.50 m				springtide Hz = 3.00 m		
B (m)	H_B (m)	U_{max} (m/s)	Δh_{max} (m)		H_B (m)	U_{max} (m/s)	Δh_{max} (m)		H_B (m)	U_{max} (m/s)	Δh_{max} (m)
200	2.00	0.29	0.01		2.50	0.34	0.01		3.00	0.34	0.01
150	2.00	0.38	0.01		2.50	0.46	0.01		3.00	0.46	0.01
100	2.00	0.57	0.02		2.50	0.69	0.03		3.00	0.69	0.03
50	1.91	1.14	0.08		2.49	1.36	0.12		2.99	1.37	0.12
25	1.81	2.18	0.30		2.35	2.58	0.42		2.87	2.63	0.43

Dam 4 West Hinkele, width of tidal channel 150 m, area of tidal basin A = 0.460 km^2

neap tide Hz = 2.00 m					neap tide Hz = 2.50 m				springtide Hz = 3.00 m		
B (m)	H_B (m)	U_{max} (m/s)	Δh_{max} (m)		H_B (m)	U_{max} (m/s)	Δh_{max} (m)		H_B (m)	U_{max} (m/s)	Δh_{max} (m)
150	2.00	0.33	0.01		2.50	0.39	0.01		3.00	0.39	0.01
100	2.00	0.49	0.02		2.50	0.59	0.02		3.00	0.59	0.02
50	2,00	0,98	0.06		2.49	1.17	0.09		2.99	1.17	0.09
25	1.95	1,90	0.23		2.41	2.27	0.32		2.93	2.29	0.33
10	1.38	3.39	0.72		1.65	3,87	0.94		2.10	4.12	1.07

Dam 4 West Hinkele, width of tidal channel 200 m, area of tidal basin A = 0.614 km^2

neap tide Hz = 2.00 m					neap tide Hz = 2.50 m				springtide Hz = 3.00 m		
B (m)	H_B (m)	U_{max} (m/s)	Δh_{max} (m)		H_B (m)	U_{max} (m/s)	Δh_{max} (m)		H_B (m)	U_{max} (m/s)	Δh_{max} (m)
200	2.00	0.33	0.01		2.50	0.39	0.01		3.00	0.39	0.01
150	2.00	0.44	0.01		2.50	0.52	0.02		3.00	0.52	0.02
100	2.00	0.65	0.03		2.50	0.78	0.04		3.00	0.78	0.04
50	1.99	1.30	0.11		2.48	1.56	0.15		2.98	1.56	0.15
25	1.86	2.43	0.37		2.28	2.86	0.51		2.80	2.93	0.54

7.3.5 Current velocities versus damming materials

If only clay clods are used (dumped or placed) and no rock, stones, bricks or *rijshout* are available, one can calculate (Schiereck, 2001, 58 et seq.) that clay clods, being cubes having a d_{n50} = 0.20 m (Storm Buysing, 1854, 534) and a density ρ = 1600 kg/m^3 (Caland,1833, 214 et seq.) will stay stable in current velocities of 1.3 to 1.9 m/s depending on the depth over the sill and assuming a Shields parameter Ψ = 0.03.
In Box 7.1 maximum water levels and current velocities in the closure gap have been determined for various stages of such horizontal closure during neap and spring tides.

[8] For equations used in CRESS , reference is made to Appendix N.
[9] These adjustments to MSL are necessary as the computer program does not work if the sill runs dry during a neap or springtide.

Bearing in mind that a maximum current velocity of 1.3 to 1.5 m/s would be acceptable, the last gap to be closed of Dam 2 during a neap tide, having a tidal difference of 2.00 m, should have a minimum length of approximately 40 m. This is valid for channel widths of 150 and 200 m. The 40 m long gap would have to be closed during one tide. The maximum current velocity would be 1.42 m/s (channel wide 200 m). Current velocities as calculated which exceed 1.5 m/s have been marked in red in Box 7.1. For Dam 4 the situation is slightly worse for the case of a 200 m wide channel: max current velocity 1.61 m/s in a 40 m wide gap. Still, it is not (yet) known:

(a) whether or not stones and *rijshout* were already being used in the 13th century for damming purposes; and/or

(b) whether or not sink-fascines filled with clay clods were an option at that time.

In this respect one should realize that Vierlingh wrote his *Tractaet van Dyckagie* 300 years later!

7.4 Construction of the dike between Broek and Zevenbergen in 1410 – 1412 (project no. 2)

7.4.1 Introduction

In the past, and also more recently, researchers[10] have tried to draw conclusions regarding the design and construction of the dike between Broek and Zevenbergen. It has been constructed during the period 1410 – 1412. It was located west of de dike surrounding the Grote Waard. An important reason for discussing it here is that 89 ships were ballasted and sunk in order to close major tidal creeks.

According to the Charter of 1 May 1410, the dike was required to protect the Grote Waard against future flooding. But it also would create a new polder which would offer opportunities for *moering* (i.e. digging of peat for the production of salt, so-called *selnering*) and agriculture.

The new dike would annihilate the drainage of the Grote Waard but the Charter stated: If the people of the Grote Waard would like to build sluices in the new dike to restore the drainage, they may do so at their own cost'.[11]

Normally, technical information cannot be found in charters and related administrative documents. But the accounts of a project in any case will give details of the cost of construction, the materials applied and quantities used. In this particular case the accounts are quite comprehensive: the main accounts and appendices[12] comprise together 642 folios (in most cases used double-sided).

Still, there is in these accounts:

- insufficient information about the horizontal alignment of the dike;
- no certainty about its exact length (could be anything between 7.0 and 7.3 km);
- nothing about the level of the land in relation to the tides and nothing about the range of the tide and tidal levels reached;
- no map showing number, location, width and depth of creeks and extent of mud flats;
- no specific information about the 89 ships which were sunk as part of the closure operations; the accounts merely describe a ship as (for instance): *schouw, schuit, pleit, bierschip, buis*, etc;
- no information about the size[13] and density of the 3 895 700 bricks/stones bought for use in the tidal closures to be effected.

[10] Mentioned in this respect are : Hingman (1885), Vriend (1961), Van der Helm (1988) and Leenders (2021).

[11] *"..of onse goede lude wten Grotenwairt tot enigerl te doen hadden sluysen te liggen in desen nyen dyck voir den Broek duer te wateren, dat sy dat ter alre tyt up horen cost doen mogen sonder wederseggen van ymand"* (Van Mieris, 1756, 140 -141).

[12] NL-HaNA_3.01.01_ 929 to _ 935.

[13] There are large differences in size between the various bricks used in the past. For instance a cloister-brick (*kloostermop*) can have a length of 30 to 38 cm, a width of 14 to 18 cm and a height of 8 to 12 cm. The small bricks made along the *Hollandse IJssel* have, however, dimensions of 18 by 9 by 4.5 cm.

Nevertheless, it is possible to 'read between the lines' and get an idea about the closure techniques applied.
Van der Helm (1988) has made extensive transcripts from (parts of) the aforenamed accounts and it are mainly his texts which have been used.
But first of all the horizontal alignment of the dike will be shortly discussed.

7.4.2 The alignment of the dike between Broek and Zevenbergen

Leenders (2021, 22 – 35) has made a serious effort to determine the alignment of the new dike. The alignment would start in the North near Broek. And as the new dike would pass in front of the sluices of the Grote Waard, the starting point must have been west of these sluices at Broek.

It is, however, not at all clear to what point (at an existing dike near Zevenbergen) the south end of the new dike would be connected. Even the length of the new dike is not exactly known because it is not clear what kind of rod (length measure) has been applied (Table 7.3). But at that time, and in this region, for dike works mostly the rod of Putten was used.
Leenders, has tried, on the basis of data on toponymy, found in the accounts and discussions, to determine the said alignment. It is, however, beyond the context of this dissertation to repeat the deliberations made and conclusions drawn by him regarding this possible alignment.

Table 7.3 Possible lengths of the sections of the dike Broek – Zevenbergen; source: Leenders (2021, 24)

No. of section	dike section	length	rods of Zuid-Holland	rods of Putten
		(rods)	(metres)	
1	north	514.5	2 022	2 093
2	centre	206.5	812	840
3	south	647	2 543	2 632
	sub-total	1 368	5 376	5 566
4	's heren keet	105.5	415	429
5	new Zevenbergse dike	310	1 218	1 261
	overall total	1 783.5	7 009	7 257

But one aspect is important to note: there are no reliable maps of that period. So nothing can be said about the size of the tidal basin east of the new dike, nor about the location of creeks or the width of the major tidal channel (Section 2, '*Hals van de Meren'*) crossing the alignment.
From the accounts it follows that the width of the '*Hals'* is some 800 m but it is not at all sure that it concerns a 800 m wide channel. Anyhow, such a major tidal channel would not be required to enable the tide to fill the relatively small area (39 km^2)[14] between the new dike and the western dike of the Grote Waard. Probably, the 800 m wide '*Hals*' comprised a mud flat, crossed by some creeks and one or two 'major' channels.
In Fig. 7.3 the possible alignment of the dike made in de period 1410 – 1412 is presented. Also the extent of the so-called '*Zwaluwse meren'* (in the east), '*Hals van de Meren'*, and '*Strijense Meren*' is shown. These four items have been plotted by Leenders on ,a present day, topographic map.

[14] According to Leenders (2021, 34), who in turn derived this figure from a soil study by Koornneef (1945) combined with contour lines on topographic maps made prior to 1980.

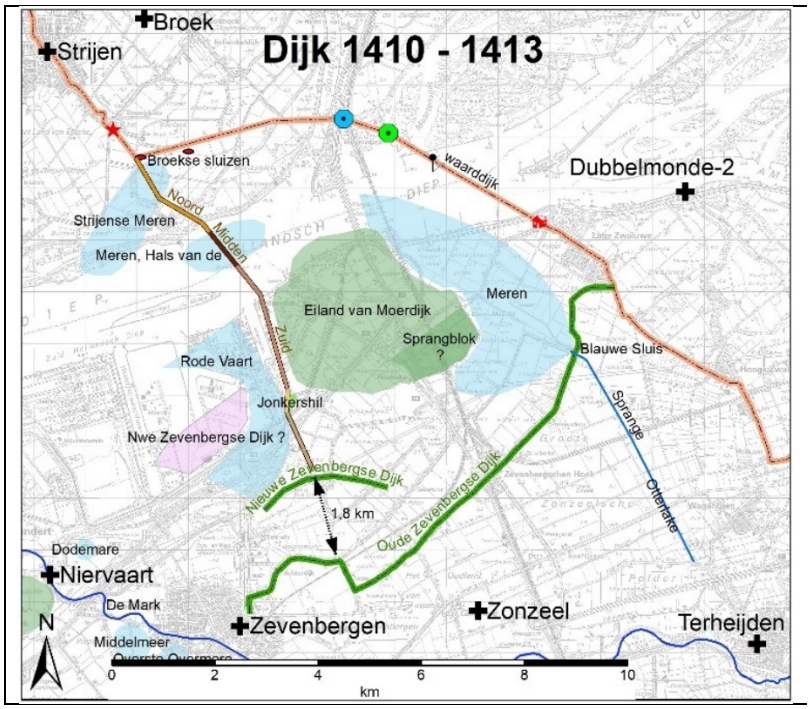

Fig. 7.3
Possible alignment of the dike Broek – Zevenbergen; drawing by K. Leenders (2021)

7.4.3 The subsoil

As stated in Sub-section 7.4.1, nothing is known about ground levels in the alignment of the new dike. But Leenders (2021, 34) argues that immediately west of the new dike (Grote Klundertpolder) prior to 1980 ground level was at NAP – 1 m, while the top of the peat layer was at NAP - 2.2 to -2.6 m. In 1410 that last level, in his opinion, must have been around the sea level[15] at that time (NAP – 0.40 m according to Leenders). The peat layer would have a small sediment layer on top. He also states that the peat layer must have had a maximum thickness of 4.5 m. Following this argument the scour of creeks and deeper tidal channels will have been limited by the resistance to scour of the peat.
But, as it turned out, a peat subsoil does not present an ideal foundation for a dike; after construction numerous slips and settlements occurred. Moreover, the dike had to pass many tidal channels.

7.4.4 Coffer damming

In the accounts much of the money goes to *kuipwerk*, which can be translated as coffer damming (see below). The following references to *kuipwerk* are quoted (Van der Helm, 1988):
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'The works of the first two groups[16] comprise predominantly the construction and heightening of the dike body and the execution of *kuipwerk* at both sides of the lakes.' [17]
'The third group[18] of works comprises *kuipwerk*…….'
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'The first *kuipwerk* started from June 1410 in the lakes. From September 1410 also *kuipwerk* was done near the Otterlaak. These activities were temporarily adjourned in December in view of the weather (winter) conditions.'
'Meanwhile the real *kuipwerk* for the creeks *(toter kenen*) had started.'
'Early November the *kuipwerk* had to be interrupted during a few weeks due to frost.'

[15] It is assumed that sea level means MSL. According to Schiereck & Visser (2021, 15) MSL in 1421 was at NAP - 0.30 m.
[16] These two groups are the dike sections North and South, i.e. 1 and 3.
[17] 'Lakes' is a direct translation of the Dutch word *meren*, but in the Middle Ages the word *meren* could also mean 'tidal channels'.
[18] I.e. the central dike section 2.

'…..However, the *kuipwerk*, in so far as possible, continued during the winter season.'

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'In the main accounts and in the appendices… are, as an indication of the nature of the *kuipwerk*, the following descriptions given:

- cuupwerk made….in order to heighten the heads, to reinforce, protect and maintain these,
- to lengthen the heads, to heighten, …'

'A gully is being closed by *kuipwerk,* clay clods and other materials.'

Van der Helm (1988) was not able to find in the literature a suitable definition for *kuipwerk* and could only provide a general and rather vague description.

As already stated above, it is thought that *kuipwerk* stands for coffer damming, which is in fact the construction of a *kistdam* by driving two parallel (double) rows of piles up to a certain point of time (e.g. at interruption by the tide becoming too high) when the two rows are connected to each other by means of a semi-circular row of piles[19] in front of the two rows. The piles are connected horizontally by means of walings, while both rows of piles are connected to each other by struts. The space between the two rows of piles will be transformed into a cofferdam by placing hurdles against the inside of the rows of piles and by filling the space in between by dumping clay clods. During the next Low tide the pile driving will continue.

The coffer damming will be required anywhere in the alignment of the dike where the ground level of the mudflat is below MHT or deeper, but not deeper than, say, slightly below Low tide and in shallow creeks. In the deeper parts sink-fascines (see below) are placed in two rows up to a level of MLT (Fig. 7.5).

7.4.5 Sink-fascines[20]

The following references in relation to the use of fascines are quoted (Van der Helm, 1988):

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'At the end of the Summer (1411), a start was made on the purchase of reed and *rijs* (fascines) for the *zinkstukken* (*wasen*)[21].'

'From November (1411), the delivery of bricks/stones started.'

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'The labourers have assisted in throwing (launching, dumping?) fascines and stones.'[22]

'Apart from the labourers, who…., …., *welemakers*[23] are mentioned.'

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'Overall, we counted 644 vimmen[24] and 3 638 sheaves[25] of *rijs* (fascines).'

'According to the accounts, the overall number of stones bought, numbered 3 895 700.'

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'Light timber was used in carpentry for…..the scaffolding from which wasen and stones were thrown (launched?).'

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[19] In this respect reference is also made to Schraver (1807a).

[20] For particulars of sink-fascines, please refer to Ch.4, Section 4.2.

[21] Van der Helm introduces the word *zinkstukken* in order to clarify the word *wasen*. But *wasen* are in fact sink-fascines.

[22] '*wasen ende steen….hulpen werpen*.'

[23] *Welemaker* means probably *touwslager* (rope-maker). Refer in this respect to Vierlingh (1920, 440/441) where the meaning of *wele* is given as *reep, kabel.*

[24] A small vim is 104 fascines, a large or thick vim is 156 fascines. This applies to '*Hollands rijshout'* (van Breen, 1920, 42).

[25] A sheave is normally used to indicate a quantity of straw or reed. It is assumed that here fascines are meant. Accordingly, the 3 638 sheaves would represent 35 small vims of fascines.

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'…. Specialized carpenters were recruited nearby and far away to work at the dike project. ….. They sharpened the piles, assisted in placing these, worked on the ships and made tools and *wazen*.'

'Other artisans were masons, blacksmiths, roofers, … and rope-makers.'

'As description of the work of the labourers the accounts state: assistance during sailing, unloading and loading of the ships, the throwing (launching?) of *wazen* and stones and other activities related to *kuipwerk*.'

A possible design of a sink-fascine is presented In Ch. 4, Box 4.2.

7.4.6 Ships

An overall number of 106 ships (Table 7.4) were bought of which 89 were sunk. In the accounts only the type, the purchase price and (sometimes) the size (in *cupes* or *korven*) are given. The ships of better quality (which' quality is reflected in the purchase cost) have most probably been used for transport of personnel, materials and foodstuffs (including beer!).

For those ships which were going to be sunk one would like to know the depth of the hold as this gives an indication of the height of water it will retain after it has been sunk.

In order to find this information some books about ships of the past centuries was consulted.[26] The result was not very successful. First of all it became clear that some types (e.g. kamuzer, zweenhals, bayart) are not mentioned in those books. Then, there are types which are only mentioned without any indication of size (e.g. hulk). Finally, of certain types (like schouw, schuit) many different variants did (and do) exist. Van Beylen (1970, 175-176) states also: 'As for most of the other types of the Middle Ages it is not known how these *schuiten* looked. The first identified images date from the 17th century.'

One would expect that all ships which were sunk were of the flat-boat type (*platbodem*).

Table 7.4 One hundred and six ships of which eighty nine were sunk

Type of ship	Account no. 929	Account no. 930	Account no. 932
schuit	9	8	9
schouw	5	17	3
pleit	5	10	1
maaschip (beytel)	–	–	3
hulk	3	–	
koggeschip	2	4	
lichtschuit	2	–	
lichtschip	–	1	
kamuzer	1	–	
schip	–	5	
(geboeyden) aelman	–	3	
boot	–	3	
ever	–	2	
pont	–	2	
bierschip	–	2	
haringbuis	–	2	
buis	–	1	
krayer	–	1	
zweenhals	–	1	
bayart	–	1	
totals	27	63	16

Source: Van der Helm (1988, 46)

From the scrutiny of many drawings of ships of the past in Le Comte (1831) and Schutten (2004) it is concluded that the average width of the 89 ships which were sunk must have been 3 to 4 m while the average depth of the hold must have been between 1.5 and 2.5 m. But most of the *schouwen* will

[26] In alphabetical order: Crone (1926), Le Comte (1831), Schutten (2004), Van Beylen (1970), Winschooten (1681).

have had a very small hold (i.e. less than 0.5 m) and must have been part of the group of the 17 ships which were not sunk.

7.4.7 Some suggestions regarding the closure method applied

In the accounts one speaks about the tendering (*bestadingen*) of two dike sections (Fig.7.3, nos. 1 and 3). It is assumed that these dike sections have there OGL (original ground level) at MHT (mean High tide) which is the normal average ground level (*maaiveld*) for a 'ripe' *schor* (salt marsh).
From the quotations in the foregoing sub-sections it would appear that the closure of creeks was established by coffer damming and dumping of sink-fascines.
One or more major tidal channels were closed by dumping sink-fascines, presumably with their length in the direction of the current and up to MLT (mean Low tide). It is thought that the average tidal range at that time and location was 2 m.[27]
These dumped fascines would form a sill on which anchored ships could be sunk during high tide (slack water) or during the ebb when the ships are grounded. These ships are placed in the direction of the current to limit the disturbing force, principally drag given by the projected area. Moreover, the overall number of ships sunk (89) is also an indication that this approach was practised. Still, some longer (and wider !) ships[28] may have been placed lengthwise on the, relatively narrow, sill.
In order to avoid under seepage, the two parallel rows of sink-fascines should be dumped with a clay core in between (Fig. 7.4). But, there is nearly always the problem of the triangular gaps left between the ships (e.g. base of 1 m, height ± 70 cm) which have to be closed rapidly during low tide.

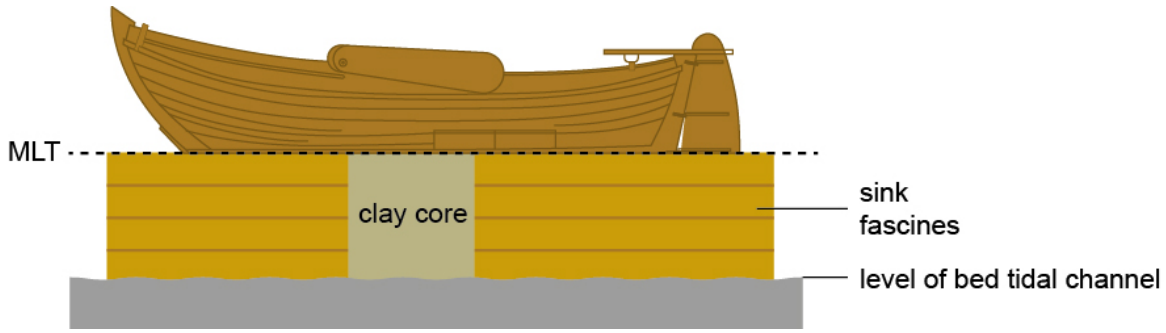


Fig. 7.4 sill of sink-fascines on which ships are sunk

The quotations also refer to heads. These texts are not very clear. A possible method of working could be the following: a dike head (*dijkskop*) is being advanced by building repeatedly a scaffolding from which two parallel rows of 4.10 m long sink-fascines are dumped with a 1.8 m wide clay core in between (Fig. 7.5).
According to the accounts, in 1412, Easter was on 3 April and Whit Sunday on 22 May (Van der Helm, 1988, 59). On Whit Sunday 20 ships were sunk. At that time 200 men were working on the dike. On 8 June 25 ships were sunk when the *Hals van de Meren* was closed (p. 39) and during five days 350 men were employed[29]. It is not understood why there are 3 weeks in between these two closure operations and not two or four weeks which one would expect.
In Box 7.2 an effort is made to calculate the possible maximum current velocity in the final closure gap.

[27] Schiereck & Visser (2021, 16).
[28] Like the Vlaamsche Pleit, having a length of 23 to 27 m and a width of 5 m (Crone, 1926, 177-178).
[29] Van der Helm (1988, 39).

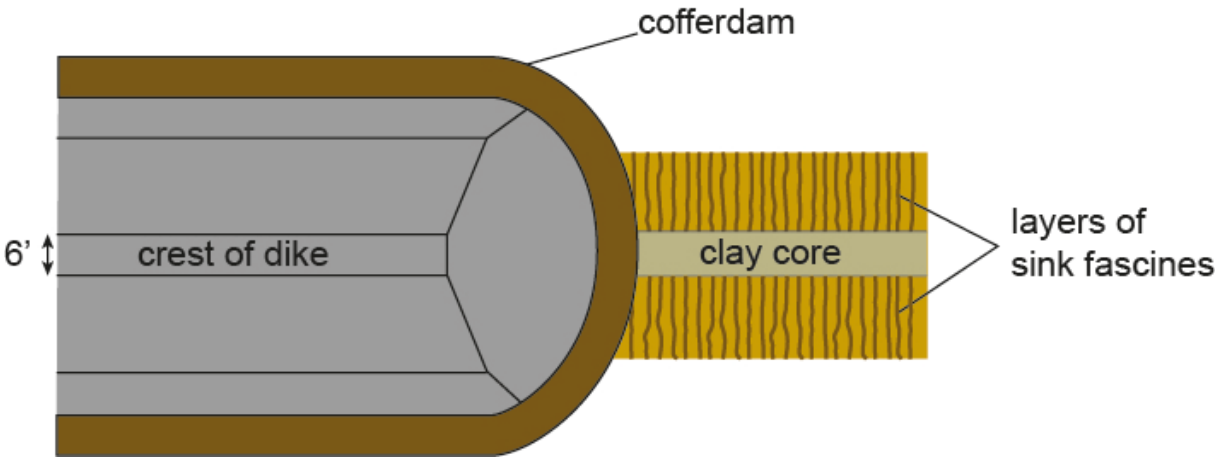


Fig. 7.5 Top view of dike head, protected by cofferdam and advancement of heads by dumping sink-fascines

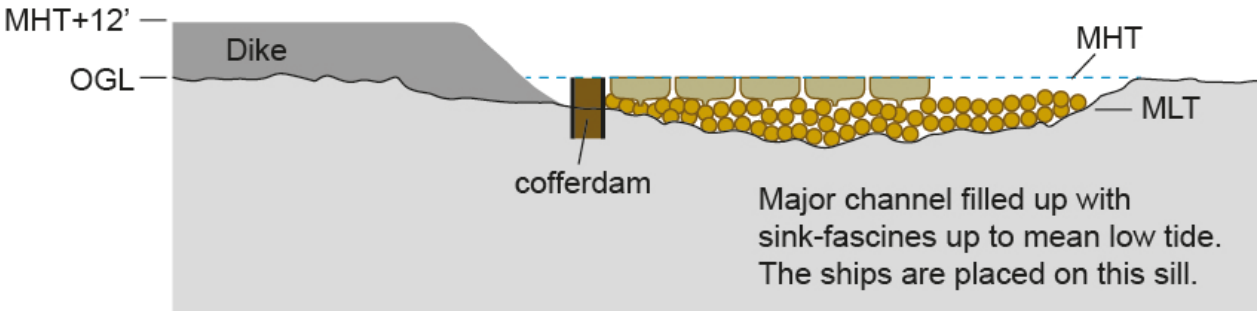


Fig. 7.6 Vertical length profile of dike head, cofferdam and ships placed on sill in tidal gully

Box 7.2 Calculation of possible maximum current velocity in final closure gap in dike between Broek and Zevenbergen

Mean low tide(MLT) is used as reference level. The level of the sill is supposed to be at MLT.
The average tidal range is 2 m (Sub-section 7.4.7). Then the tidal range at neap could be 1.7 m, i.e. from MLT + 0.10 to MLT + 1.80 m. The area of the new polder has been given as 39 km^2 (Sub-section 7.4.2). It is assumed that the area of the tidal basin during a neap tide is not more than 15 km^2. It is further assumed that the average width of a ship to be sunk is 3.5 m (Sub-section 7.4.6). On the day of final closure (8 June 1412) 25 ships are sunk (Sub-section 7.4.7) which implies a width of the final closure gap of 87.5 m.

In order to determine the values of various parameters in the closure gap the Coastal end River Engineering Support System CRESS[30] (www.cress.nl) is used.
Above the following values were determined for four parameters[31]: A = 15 km^2
B = 87.5 m
Hz = 1.70 m
h = 0.95 m
Moreover, c = 0.90 and T = 12.5 h
According to the calculation in Cress this results in:

-	water level difference in the basin	H_B	=	0.31 m
-	maximum current over sill	U_{max}	=	3.64 m/s
-	max head over sill	Δh_{max}	=	0.84 m

Tidal prism during final closure would be A x H_B =15 x 10^6 x 0.31 = 4.65 x 10^6 m^3 = 4.65 million m^3 .

According to the accounts 929 and 230 the overall cost of the project up to 11 June was £ 8 474 – 5 – 2 but, as the available funds amounted to £ 7 917 – 19 –, the project resulted in debts to various suppliers, contractors and labourers to a total of £ 566 – 6 – 2 (Van der Helm, 1988, 41).
This situation must have been the reason that the project accounts were closed abruptly on 13 June, only five days after closure. At that time the work was definitely not complete. For instance, the wooden sluice, to be installed in dike section 3, had been purchased in Tholen and transported to the site, but had not yet been installed. After June 1412, the project management could not find any

[30] For equations used in CRESS , reference is made to Appendix N.
[31] For the definition of these parameters, please refer to Box 7.1.

contractors/ labourers who were willing to dig ditches etc. in the new polder. Though it has not been stated explicitly also the dike in the final closure section 2 most probably was not yet complete at that time (i.e. five days after closure).
As late as 1431, a number of debts had still not been paid (Van der Helm, p.24).
In Table 7.7, the distribution of the expenses is given over the various items figuring in Tables 7.5 and 7.6. Apparently, coffer damming is the most expensive item (51.2 %) while the major closures, comprising fascines and stones to fabricate sink-fascines as well as the purchase, ballasting and sinking of ships, only come to 9.8 %. But, as said already, there is a great ambiguity in these figures because of non-technical grouping of different items under one heading and sum. A reason for this high cost has been no doubt that all coffer damming had to be done in dayworks while this also required much skilled labour like carpenters.

Table 7.5 Construction cost during the years 1410/1411 as detailed on account 929

account & no. of item	year and description	further details	cost in £-s-d groten
929 – 1	1410 tendered dike section 3	674 rods up to MHT + 6 ft.	£ 569 – 6 – 8
929 – 2	1410 ,, ,, ,, 1	514.5 ,, ,, ,, + 6 ft.	606 – 17 – 1
929 – 3	1411 ,, ,, ,, 2	206.5 ,, ,, ,, + 6 ft.	198 – 10 -
929 – 4	1410 ,, ,, ,, 1	515 ,, ,, ,,+12 ft.	442 – 17 – 11
929 – 5	1411 ,, ,, ,, 2	182.5 ,, ,, ,, + 9 ft.	98 – 10 – 10
929 – 6	1411 ,, ,, ,, 3	687 ,, ,, ,, + 9 ft.	326 – 9 – 2
929 – 7	1411 aprons	88 ,, 1 rod wide	44 – 14 – 7
929 – 8	1411 bunds on part of dike crest	88 + 613 MHT + 12 ft.	87 – 10 - 10
929 – 9	-- seepage dams, repair of slips		4 – 18 - 8
		sub-total	2379 – 15 – 9
929 - 10	coffer damming	1410 near Broek, Section 1	197 – 7 – 5
929 – 11	,,	1410 Section 3	216 – 2 – 3
929 – 12	,,	1410 – 1411 repairs	79 – 18 - 11
929 – 13	,,	Nov-Dec 1411 inside creeks	1642 – 15 – 5
929 – 14	,,	tools, wages and other exp.	640 – 3 - 10
		sub-total	2776 – 7 – 10
929 - 15	purchase of 24 ships and sinking	1411, also related expenses	121 – 1 – 8
929 – 16	purchase stone and fascines	1411 545 950 stones	29 – 11 – 7
929 – 17	miscellaneous	food, administration, office	372 – 18 – 7
	account 929, period 1410 - 1411	**overall total of expenses**	**£ 5 579 – 15 – 5**

Table 7.6 Construction cost during the year 1412 as detailed on account 930

account & no. of item	description	further details	cost in £-s-d groten
930 – 1	dikes and creeks, Section 4	105.5 rods up to MHT +6 ft.	£ 78 – 3 – 2
930 – 2	,, ,, ,, 5	310 ,, ,, ,, ,, +6 ft.	222 – 11 – 8
930 – 3	compartmentalization	also seepage dams	52 – 9 – 2
		sub-total	353 – 4
930 – 4	coffer damming until 9/5/1412	in the creeks	826 – 10 – 11
930 – 5	,, 1/5 – 11/6/1412	in the Hals van de Meren	541 – 4 – 8
930 – 6	,,	materials and tools	194 – 16 – 3
930 – 7	,,	wages of foremen	3 – 14 - 3
		sub-total	1566 – 6 - 2
930 – 8	purchase of 60 ships	also related expenses	163 – 17 – 10
930 – 9	purchase stones and transport	3 305 350 stones	384 – 14 – 11
930 - 10	fascines, alder-trees, etc	standing timber: oak, fir, pine	127 – 4 – 9
	miscellaneous	food, administration, office	198 – 12 – 3
	account 930, year 1412	**overall total of expenses**	**£ 2 793 – 19 – 3** [32]

[32] A few figures have been rounded off, resulting in a minor 'mistake' in the addition.

Table 7.7 Distribution of expenses presented in Tables 7.5 and 7.6

item	description	amount	number of rods	amount per linear rod	percentage of total
1	coffer damming	£ 4 342 – 14 – 0			51.2
2	tendered dike works on salt marsh	2 379 – 9 – 0	1395	£ 1 – 14 – 7	28.1
3	tendered dike works incl closure of some creeks	353 – 4 – 0	415.5	£ 0 – 17 – 0	4.2
4	fascines, standing timber & stones	541 – 11 – 3			6.4
5	ships	284 – 19 – 6			3.4
6	miscellaneous (including sluice)	571 – 10 – 10			6.7
	Overall total of expenses	**£ 8 474 – 5 – 2**			**100**

7.4.9 Failure of the project

Apart from lack of funds the project failed because by making the new dike in front of the sluices at Broek, the drainage of the Grote Waard was blocked. Obviously, this was not acceptable to its inhabitants. In a letter of 19 December 1413 (Van Mieris, 1756, 267) the Duke Willem, for a certain sum as promised to him, gives permission to the towns of Dordrecht and Sint Geertruidenberg and the inhabitants of the Grote Waard to remove the new dike in front of Broek in order to get the sluices operating again:

"….. Because ….we do not want to lose our land, nor that it will be damaged." [33]

It meant in fact that the new polder became again part of a tidal basin. In 1415 a charter dated 20 February (Van Mieris, 1756, 313) ordered that a sluice and a drainage channel near Wieldrecht had to be opened again which confirmed that the project had been abandoned.

7.5 The reclamation of the polder Groot Zuid-Beijerland in 1631 (project no. 3)

7.5.1 Introduction

The reclamation of this polder has been quite well documented and, moreover, the documentation has been the subject of (part of) a dissertation: Baars (1973) & (1973a) made a detailed study of all polders on the former island Beijerland called '*De geschiedenis van de landbouw in de Beijerlanden*' [34].
In Fig.7.7 the lay-out of the reclamation scheme is shown: surrounding dike, creeks and roads.
In Fig. 7.8 the hydraulic engineering features have been emphasized. The information in this figure has been derived by Van de Ven (2004) from Baars.

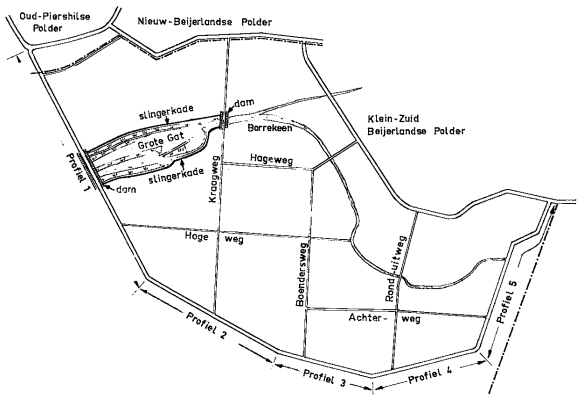


Fig. 7.7
Lay-out reclamation scheme (*polder*) Groot Zuid-Beijerland;
source: Baars (1973, 57)

Apart from the actual dissertation, Baars published also a collection of documents called '*Documenten en Bijlagen',* (Baars, 1973a). In this document much background information from

[33] "…..want wij onse lande om sommige lude onwillichede niet verloren, noch bescadicht en willen heben"'
[34] Translated as: 'The history of agriculture in Beijerland'.

numerous sources, which he used for his dissertation, has been quoted or summarized. Based on this documentation it is possible to establish and analyse the method most probably utilized to close the tidal creek called '*Grote Gat*' at its mouth (Dam B in Fig. 7.8).
For a reclamation scheme the dikes surrounding the scheme (i.e. the tidal marsh, *schor*) have to be constructed simultaneously with the dam(s) through the creek (s) present in the tidal marsh. Twice daily, the tidal waters will enter these creeks and, during spring tides combined with wind set up, may also flood the whole of the, not yet diked, tidal marsh.

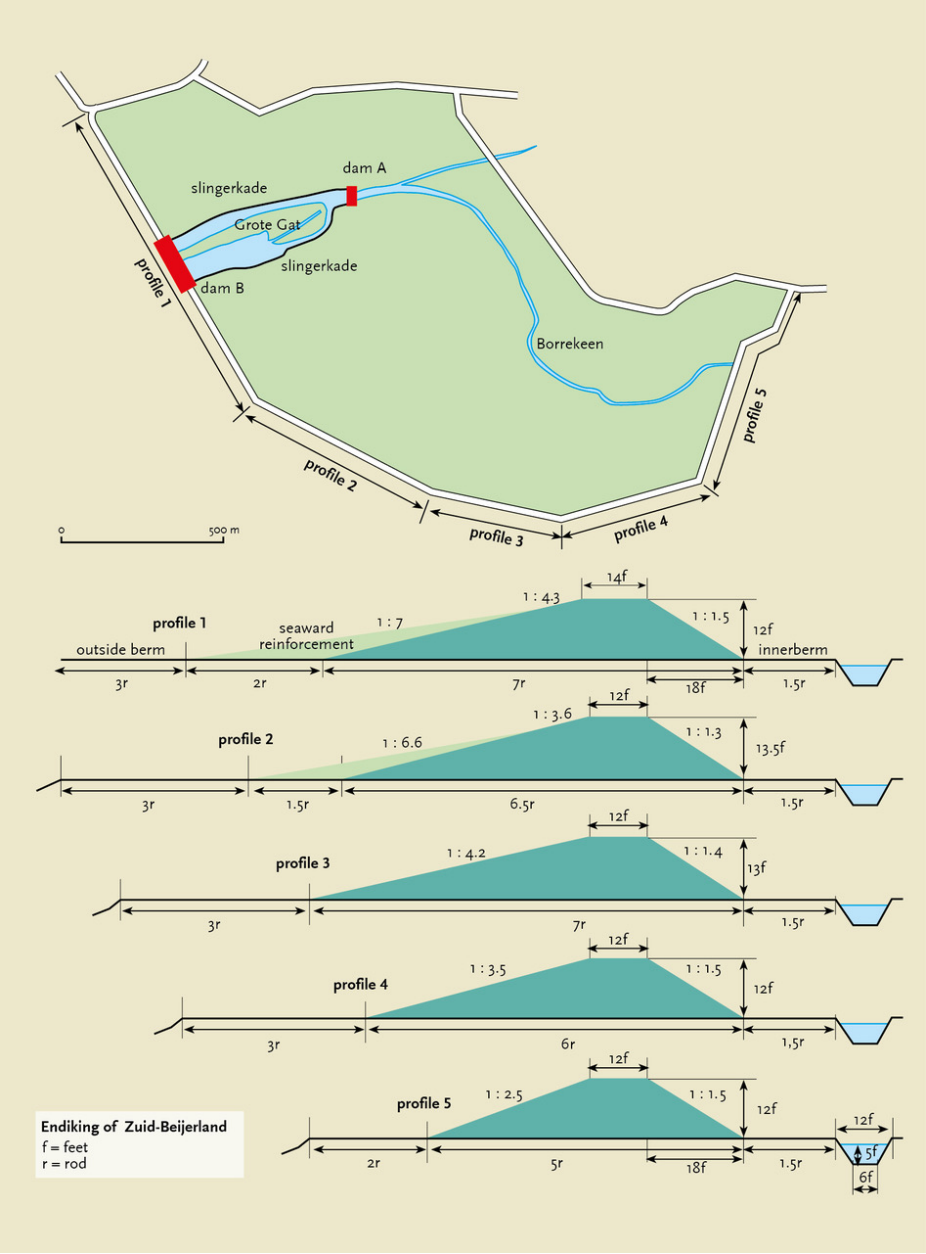


Fig. 7.8 Reclamation scheme Groot Zuid-Beijerland in 1631;
source: Van de Ven (2004. 137)

NB:
On Fig. 7.8, and sometimes in the text, the measurements are in rods (*roeden*) and feet (*voeten*). According to Baars (1973, 225) in the Beijerlanden 1 rod = 4.06 m=12 ft.; a foot = 0.338 m. This rod is known as the '*Putse roede*'

Diking the marsh is kept separate from the damming of the creeks as the latter is considered a work which can only be done by contractors used to work with fascines and the like in a marine environment. These marine contractors during past centuries could predominantly be found in places like Kinderdijk, Alblasserdam and Lekkerkerk, as already pointed out by Vierlingh (1920, 7).
Diking and damming will normally start in March/ April and has to be complete by the end of June in a given year.

Baars (1973, 27 and 45) lists the reasons for this timing:

- After June a lot of rain may fall while the winds are predominantly coming from westerly directions, resulting in wind set up (i.e.. higher water levels) and wave attack;
- for many labourers this diking is not they what they are normally doing: they are sailors, fishermen or farmers and they will leave the area as soon as they can resume work in their actual profession (e.g. catching herring at sea, growing crops and harvesting at their farms).

For the diking of the polder Groot Zuid-Beijerland some 100 contractors are employed (Baars 1973a, 166 et seq.). A contractor at that time is a small entrepreneur, actually a labour contractor without equipment. The contractors come from 30 towns/villages, mainly situated in Zuid-Holland and Noord-Brabant. The damming of creeks is done by a group of 18 marine contractors coming from Lekkerland, Lekkerkerk, Alblas and Alblasserdam (p. 169). They are supposed to start the work on 14 April and to have a labour force on site of minimal 300 strong and sturdy men ('*ten minsten dryehondert cloecke mannen*').

7.5.2 Sequence of operations for tidal closure works

The sequence of operations for the damming of the creeks has been specified in detail in the contract document (*bestek*) which was transcribed by Baars and, in as far as relevant, presented by him, together with a summary of the final account, in Document no. 49 (p. 165 – 170).
The said sequence is based on the wish to limit the area of the tidal basin, finally to be closed, as much as possible. Therefore the marine contractors have to construct first of all a dam across the tidal creek Borrekeen at location A (Fig. 7.8). such a dam in the past was called a '*vercortinghe'*. [35]
Meanwhile the dike contractors are constructing a perimeter bund around the remaining open tidal basin. It is specified that all creeks and gullies ('*kreecken ende rillen'*) crossing the perimeter bund shall only be closed after an order to that effect has been received from the works supervisors (dike reeve and his staff).
After completion of the dam across the aforementioned creek the marine contractors have to construct the small dams connecting Dam A with the perimeter bund (called *slingerkaden* in Fig.7.8) at south and north sides of the remaining tidal basin.
Finally, and only after the dam through the aforementioned creek has been certified as completed, the marine contractors are allowed to start and complete the closure of the *Grote Gat* at its mouth (Dam B). At that time the stage of completion of the main diking has to be such that the future polder is protected against summer floods. The marine contractors were able to have this last closure dam certified for completion on 7 July which implies a construction period of less than 3 months for all damming works.

7.5.3. Dimensions of closure works

It follows from the specification in the contract document that each of the two closures has to be carried out in two (*Borrekeen*) or three (*Grote Gat*) stages.
The first closure is at the site, marked on Fig. 7.8 as Dam A. The tidal channel has a width at low water level of 81 m. In the first stage the marine contractors are required to construct a closure bund, up to 0.34 m (= 1 foot) above local ground level (*maaiveld*). Its crest has a width of 10 m. Subsequently, as a second stage, on this crest (called '*zaete*') a small dam ('*kaede'*) is built. This dam has a width at its base of 8 m. and the slopes have gradients of 5 : 6 (land side) and 5 : 12 (sea side). The height is 1,7 m (5 ft.) and the elevation of the crest (wide approx. 2 metres) is then approximately 2 m (6 ft.) above the local ground level of the tidal marsh. Apparently, this elevation is required to prevent overtopping of the dam during summer storm surges in the situation of the tidal basin *Grote Gat* still being open but also to allow subsidence of the placed earth fill. From the specification it is

[35] In current Dutch: *compartimenteringsdijk, delingsdijk;* in English: compartmentalization bund.

clear that the designers have recognized the danger of bypassing (*achterloopsheid*) of the dam during storm surges and adequate counter measures (such as a lengthening of Dam A at both ends having a crest, gently sloping to the level of the *maaiveld*) are specified.
Finally, the specification mentions the possibility of the contractors needing various fascine materials mentioned as '*rijs, staecken, latten, welyen, capravens*, *ende anders'* for the construction of the closure bund. In that case the dike reeve will not only provide these materials free of charge but also direct their use in the closure works. In this respect reference is made to Sub-section 7.5.7, par. 3 below.
It is impossible to analyse this closure (Dam A) any further as adequate data are lacking (Document no. 49, p. 167).
The connections to be made by the marine contractors to the perimeter bunds (*slingercaeden*) around the tidal basin *Grote Gat* are, from a tidal closure point of view, not interesting. It is only recalled that the connecting small dams have a base wide 6 m, a crest wide 1.35 m and an elevation above local ground level of 2 m.
The *Grote Gat* has to be closed at its mouth (site of Dam B) , where it is flowing into the estuary of the rivers Rhine and Meuse, called Haringvliet. It has a length in the dike axis of 378 m at local ground level. The low water level is given as 2 m below local ground level.
As a first stage the marine contractors have to construct a primary dam (*berm*) up to the level of low water. It is assumed that is meant the low water level at mean low tide (MLT). This primary dam shall have a crest width of 40.6 m (10 rods) and its slopes should have a gradient of 1 : 1½ (Fig. 7.9).
The second stage comprises the actual tidal closure. It is made at the basin side on top of the aforementioned *berm*. This closure bund has a width at its base of 24 m, a 16.24 m wide crest[36] and an elevation of 0.34 m (= 1 foot) above local ground level (so its height is 2.40 m).
The third stage is a small dam (*caede*') placed on top of the closure bund. This small dam has a base of 12.20 m, a 2.70 m wide crest and an elevation of 2.70 m above local ground level (so its height is 2.40 m); The slopes have gradients of 7 : 18 (sea side) and 7 : 10 (land side).
Subsequently, the dike will be constructed, incorporating these three construction stages of the actual damming.

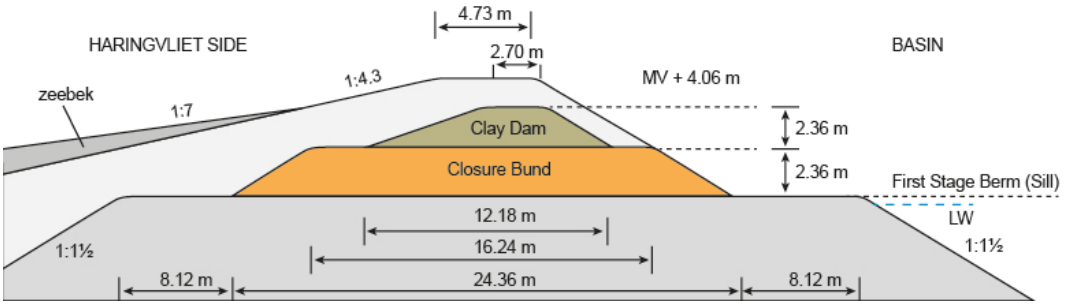


Fig. 7.9 Profiles of damming stages and final dike at Grote Gat, site of Dam B (see Fig. 7.8)

7.5.4 Analysis of the closure of the Grote Gat

Though the available information is limited, an effort will be made to analyse the closure of the *Grote Gat*. Apparently, neither quarried rock, nor erratic or river boulders are available for the damming. This implies that one will dump clay clods instead. Caland (1833, 214 et seq.), as follows from his calculations, takes for clay clods (*kloetelingen*) a density of ρ =1 600 kg/m^3 . Van Bleiswyk (1745, 61) lists a density ρ =1 821 kg/m^3 for 'natte gemene clay'. A clay clod has a size of 20 x 20 x 20 cm^3 (Storm Buysing, 1854, 534). In Sub-section 7.3.5 it was calculated that clay clods, being cubes having a d_{n50} = 0.20 m will stay stable in current velocities of 1.3 to 1.9 m/s.

[36] In the contract document the width of the crest is given as 4 ft. but this must be a mistake and should have been 4 rods.

Another possibility is the construction and placing of sink-fascines but, for the time being, it is assumed that such composite fascine constructions are not necessary here.
The technical feasibility and method applied for any closure in alluvial soils is predominantly determined by the maximum current velocities in the tidal flow.
In Box 7.3 maximum water levels and current velocities in the closure gap have been determined for various stages of closure during neap and spring tides.
From the description in the contract document one can conclude that the closure starts by dumping clay clods from flat-boats[37] (*platbodems* like *hoogaarzen*). This operation must have been more or less, what is nowadays called, a vertical closure. The restriction of the wet profile continues up to the level of low water, which is said to be 2.03 m (6 ft.) below local ground level (MV = *maaiveld*).

BOX 7.3 Determination of the maximum water levels and current velocities during damming of the Grote Gat

It is stated in the contract document that the low water level is 2.03 m (6 ft.) below local ground level. This local ground level is now used as the reference level MV (*maaiveld*). No data about the tidal cycle for the year 1631 at the Haringvliet closure site 'Grote Gat' are available. So far, it is only known that in the middle of the 19th century the mean tidal difference at Willemstad (situated a few km's upstream at the other side of the Haringvliet) is 1.98 m (Storm Buysing, 1854, 580, Tafel A).
It is now assumed for Grote Gat:: tidal difference at springtide is 2.40 m, and at neap tide is 1.70 m. Moreover, MSL is fixed at MV – 1.01 m.
Based on these assumptions, the following levels are established:

Neap tide	low water level MV – 1.85 m	Spring tide	low water level MV – 2.20 m
	high ,, ,, MV – 0.15 m	,, ,,	high ,, ,, MV + 0.20 m
crest level berm (called sill) first stage		MV – 2.03 m	
,, ,,	closure bund second stage	MV + 0.34 m	

The width of the tidal channel to be closed is B = 378 m. It follows from Fig. 7.7 that the area of the tidal basin A can be estimated to be in the order of:
B x 3B x ½ = 1.5 x $(378)^2$. Then A= 214 326 m^2 = 0. 214 km^2.
By using the Coastal end River Engineering Support System CRESS (www.cress.nl) one can carry out tidal calculations. Refer in this respect to Box 7.1.
Values have to be entered for A, B, Hz (tidal difference on the Haringvliet), h (depth of sill in relation to MSL), c (discharge coefficient for a short free overflow weir) and T (period of tide).
Subsequently, it is possible to calculate values for the following parameters:
H_B (m) = water level difference in the basin
U_{max} (m/s) = maximum flow velocity
Δh_{max} (m) = maximum hydraulic drop.
This exercise is carried out for situations at neap tide and at springtide.

Neap tide

In this case A = 0.214 km^2, Hz = 1.70 m, h = 1.02 m, c = 0.90, T = 12.50 hours. If the width of the closure is varied between B = 378 m and B = 25 m one gets the results presented in the left table.

Spring tide

In this case A = 0.214 km^2, h = 1.02 m, c = 0.90. The program does not function when, at low water levels the sill runs dry, while also h has to be < ½ Hz. Therefore it is decided to vary the tide between MV – 2.00 m and MV (instead of MV -2.20 m and MV + 0.20 m). Moreover, the duration of the tidal cycle is proportionally decreased to T = $\frac{2.00}{2.40}$ x. 12.5 h = 10.42 hours.
If the width of the closure is varied between B = 378 m and B = 25 m one gets the results presented in the right table.

Neap tide

B (m)	H_B (m)	U_{max} (m/s)	Δh_{max} (m)
378	1,70	0.06	0
200	1,70	0.11	0
150	1,70	0,15	0
100	1,70	0.23	0
50	1,70	0.46	0.01
25	1.70	0.92	0.05

Spring tide

B (m)	H_B (m)	U_{max} (m/s)	Δh_{max} (m)
378	2.00	0.09	0
200	2.00	0.16	0
150	2,00	0,22	0
100	2.00	0.32	0.01
50	2.00	0,65	0.03
25	1.99	1,29	0.10

If the latter is taken as reference level, the 'berm' or sill of this first stage of the closure is at MV – 2.03 m. From the results of tidal calculations presented in Box 7.3 it follows that maximum current velocities over this sill at this stage will not exceed 0.1 m/s. Therefore, clay clod dumping which, if desired, can be done around slack water, is technically feasible. Scour of the dumped clay clod mass is not expected. But, in view of the current velocities over the sill to be expected during the last days of the second stage, the central section of the 378 m long sill might have been protected against erosion by a reed layer fixed to the sill by straw cramp matting. See for instance Vierlingh (1920, 325). If this was impossible because of the loose packing of the poorly densified clay clod layer, a reed layer fixed to the berm by wattle work can have been applied instead.
During the second stage of closure a closure bund is constructed across the gap having a height of 2.40 m. This is a horizontal closure as water borne operations become less and less attractive because of available water depth and increasing current velocities. It is assumed that the bund is gradually made by daily constructing on average some 10 metres of bund from each side. It reads

[37] This dumping of earth fill from boats was called in the past in Dutch *vletten*.

'on average' because on some days production will be higher as it is possible to work in daylight during two tides. If one starts the construction of this bund 20 days prior to a neap tide, the gap remaining one week before the said neap tide (i.e. at springtide) will be some 150 metres wide and the maximum current velocities to be expected on that day will be 0.2 to 0.25 m/s. The last gap (28 m) will then be closed during a day of neap tide when maximum current velocities will not be higher than (roughly) 0,9 to 1.0 m /s.
Still, just placing each day 2 x 10 m of bund solely comprising of clay clods might be an option at the start of the second stage closure operation when current velocities are still low. But such method will only be feasible during the first two weeks.
It is suggested that this bund, having a height of 2.40 m and a width of 24 m (base) to 16 m (crest) is ultimately constructed as a pakwerk like the one constructed for the closure of the Sloe in 1871. (see Ch. 4, Section 4.5 for details).
That the closure bund, either in whole or in part, is constructed as a broad pakwerk might also be derived from the fact that its slopes have not been defined in the contract document (Baars, 1973a, 169). In case of earth fill, slopes would surely have been specified.
The third stage, as already described in Sub-section 7.5.3, comprises a small clay dam (*caede*) on top of the pakwerk. The contract document mentions specifically the cover of the slopes by clay sods, starting at the low water level and '*behoirlijcken in 't verbant geset ende geslooten'*. These sods have to be placed at the basin side up to MV + 0.34 m and at the sea side up to the crest of the small dam. This small dam has an elevation of MV + 2.70 m.
The dike in which these three closure stages will be embedded will have an elevation of MV + 4.06 m.

7.5.5 The use of fascines and the like

In Sub-section 7.5.3 the use of '*rijs, staecken, latten, welyen, capravens*, *ende anders'* for the closure works is mentioned and also that the dike reeve will decide when and where these materials will be used and how. Moreover, the dike reeve actually buys the materials and makes these available to the marine contractors free of charge. Probably, the dike reeve is afraid that without his final say in the use of these materials the contractors will try to incorporate as many fascines etc. in the works as possible. The reason is that such materials are never considered to be part of a lump sum contract and the quantities used are being paid separately. Vierlingh (1920) writes a number of times: '*rijswerckers (die) begeeren veel rijs te versmoren'* (pages 160, 339, 383).

7.5.6. The cost of constructing the dikes and closure works

Baars (1973, 61) summarizes in a table the cost of construction of the whole reclamation scheme including those for the dikes, materials and damming. The overall cost of the realization of the polder is f 246 876. The dikes take 2/3 of this amount (f 163 175). Purchase of materials (*rijs* etc., straw and iron) amounts to fl. 6 155. The two closures and the perimeter bund together cost fl. 28 684. In all the share of the dike works in the overall cost of realization of the polder is determined at f 223 291.

7.5.7. Concluding remarks

Though it is always possible that the suggested method of closure differs from the one which has actually been applied, the overall picture given must be more or less correct. In this respect three aspects should be highlighted:

1. It would appear that both tidal closures are realized without any significant protection of bed and sides of the tidal channels against scour.

2. No quarried rock or boulders are used, only clay clods and, possibly, sink-fascines for the first closure.[38]
3. Last but not least, the dike reeve and his staff must have been experienced men because, by taking the decision where, when and how fascines and the like are applied, the dike reeve in fact takes the responsibility for the final outcome of the closure works instead of leaving this responsibility to the marine contractors. In this respect, it should be realized that a tidal closure differs from other marine works in the sense that timing and logistics play a vital role. If a certain operation is not carried out during a planned period and completed at a certain time and/or the logistics (stockpiles of materials being at hand, sufficient labourers at work) are not in order, the closure will fail. Such a failure can never be prevented by the client, the designer or the daily supervisors. Only the contractor is in a position to take the necessary measures to render the closure successful. In the case of repair of dike breaches (like in 1953 after the Storm surge) the situation is different, the client (in 1953 Rijkswaterstaat) takes the responsibility and most of the work done by the contactors is being paid for through dayworks (*uitvoering in regie*).

7.6 The tidal prism of small reclamation schemes

In Section 7.1 it was stated: "… there is a general belief that such land reclamation is difficult and requires special skills and tools. But, as will be discussed below, this is only true to a certain extent." Reclamation schemes are, from a technical point of view, in most cases not difficult to achieve. On available maps one can see that there are numerous tidal creeks entering the area to be reclaimed but in most cases these creeks are of limited width and length (Figures 7.10 and 7.11). Also, the area will only be reclaimed when it is 'ripe' for reclamation, i.e. when most of the area has reached a level at MHT or slightly higher.

On a length profile taken in the axis of the dike, if available, it is clearly shown that such creeks are shallow: most creeks are not deeper than 2 m below MHT (Figures 7.12 and 7.13). Accordingly, they will run dry during low tide. Even a major creek like the Bruintjeskreek (Fig 7.14) is not deeper than 4.2 m below MHT, which is approximately 1 m below MLT. In this respect reference is also made to Van de Linde (1943, 101) who stated:

> "As far as I know, in the last 200 years, during reclamation works in the south-west of our country, there were no creeks deeper than 1 to 2 m below MLT."

[38] Possibly some more details about the quantities of fascines used are incorporated in the original account. This account has been summarized by Baars (1973a, 165-170). It will, however, require the input of, for instance, a historian to transcribe that document (provided it can be traced in in the archives).

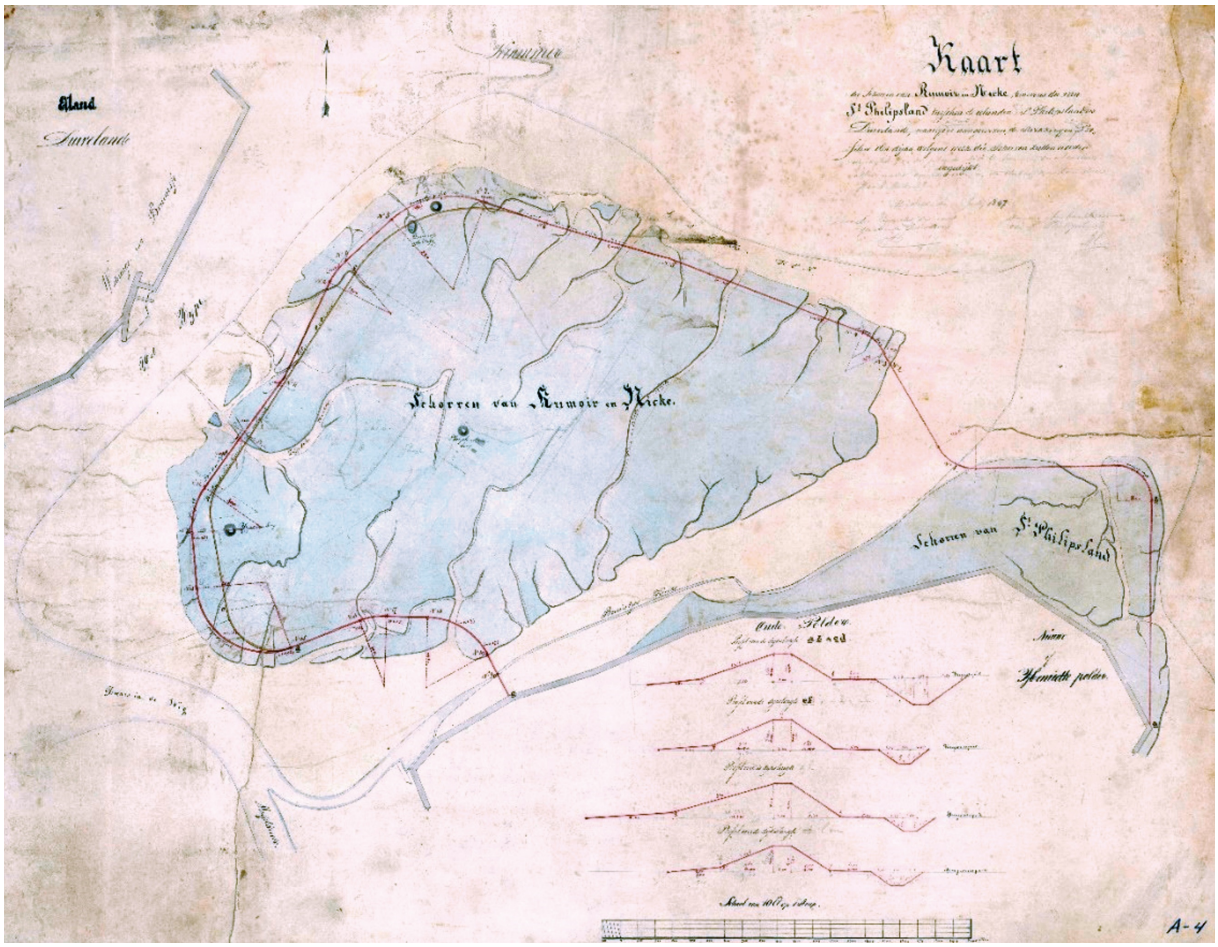


Fig. 7.10 Plan for reclamation of Anna Jacobapolder (Sint Philipsland) showing, alignment and profiles of dikes, numerous small creeks and the major tidal channel Bruintjeskreek; source: NL-MdbZA_3451_65.1

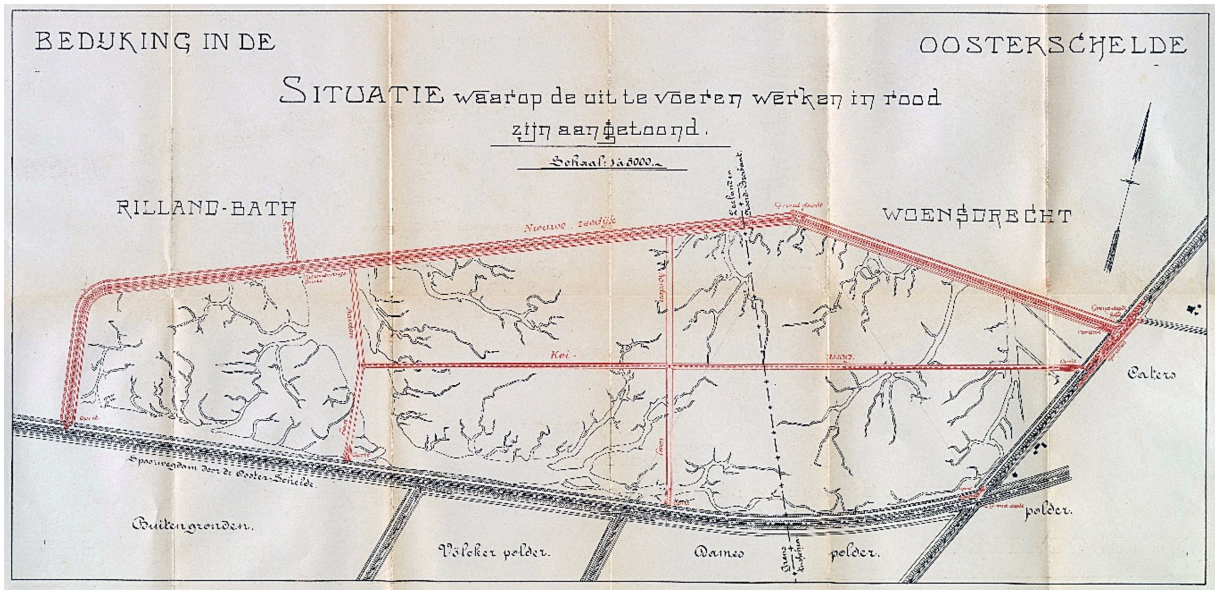


Fig. 7.11 Plan for reclamation of the Hogerwaardpolder showing alignment of new dike, creeks and future road network; source: NL_MdbZA_651.1_974

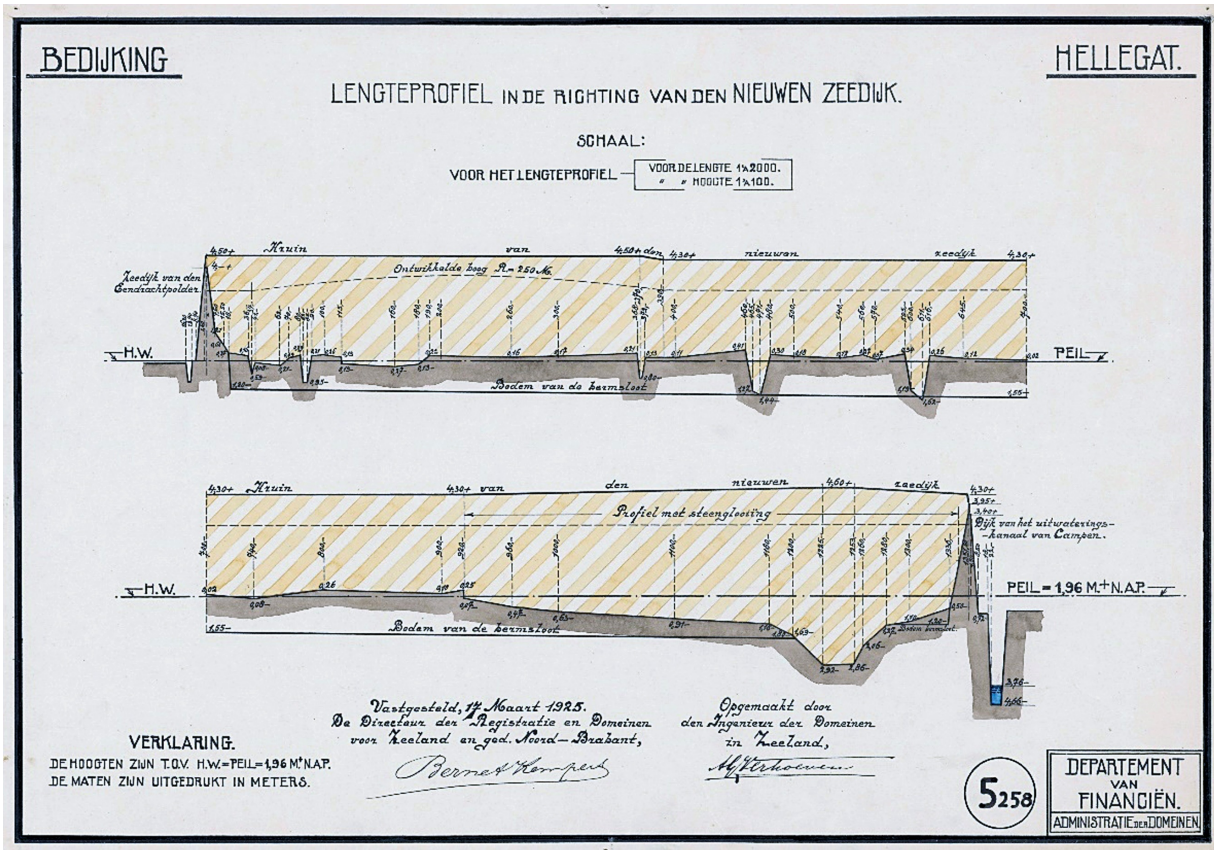


Fig. 7.12 Length profile of dike to be constructed as part of reclamation of the Hellegatpolder (Zeeuws-Vlaanderen); bed level deepest creek is MHT – 2.92 m, i.e. higher than MLT; source: NL-MdbZA_612_187

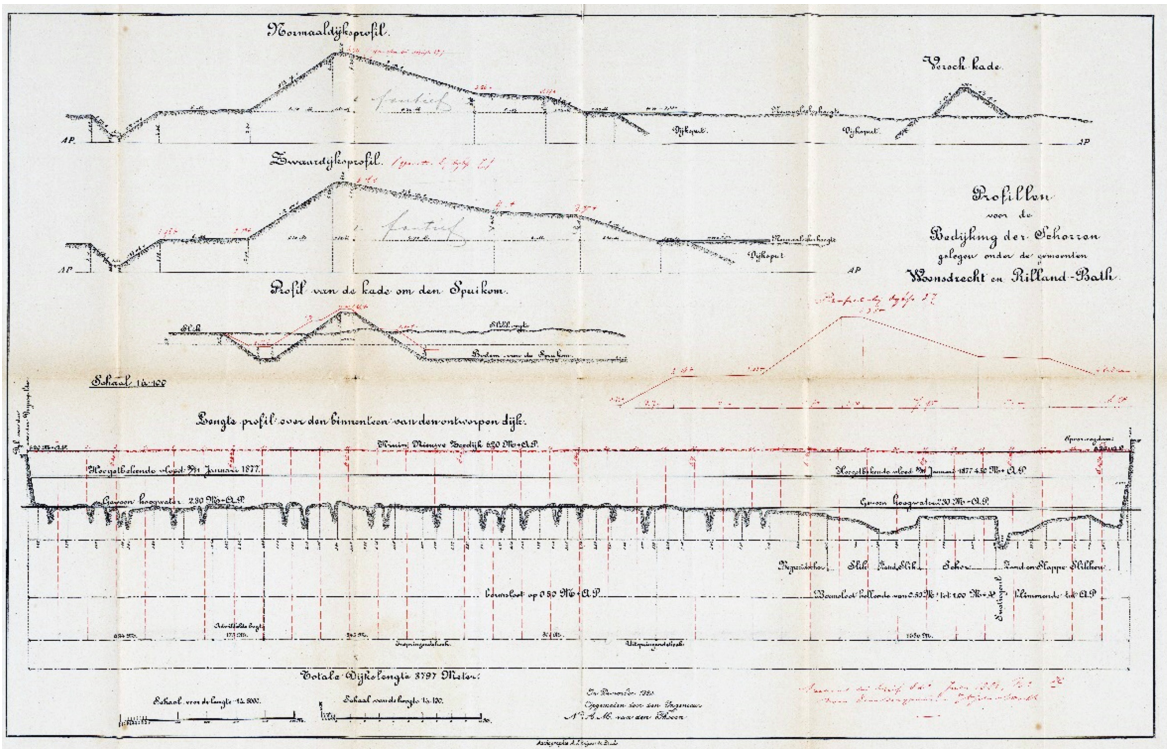


Fig. 7.13 Length profile and cross sections of dike around Hogerwaardpolder; deepest creek is the drainage channel (*suatiegeul*) having its bed level at MHT – 2.8 m, i.e. running dry at low tide; source: NL-MdbZA_651.1_971

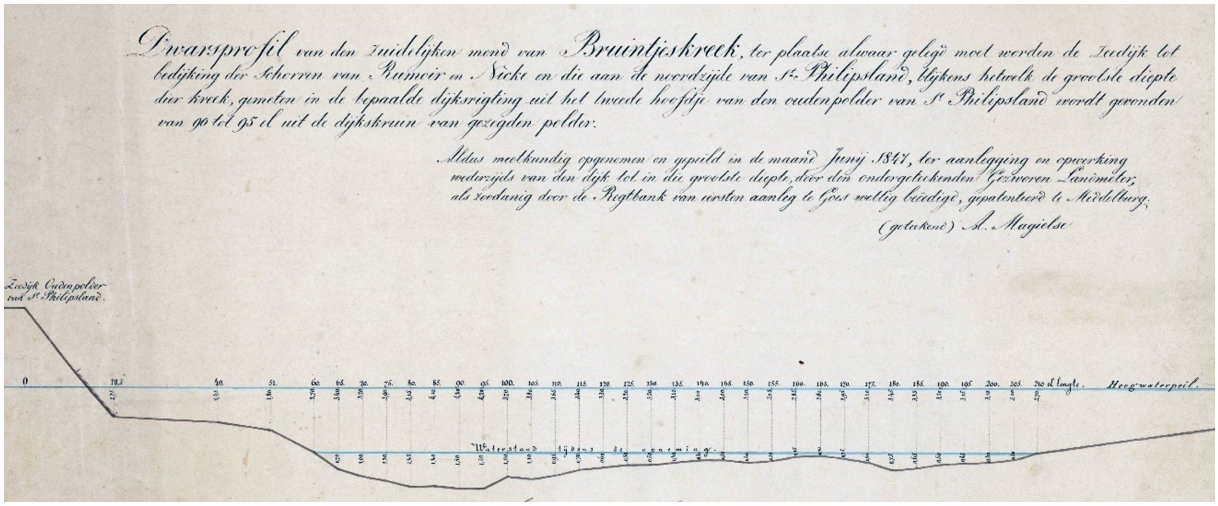


Fig. 7.14 Cross section over southern end of Bruintjeskreek (shown on Fig. 7.10); deepest bed level at MLT – 1 m (appr.); source: NL-MdbZA_3451_64

When reclamation of a 'ripe' salt marsh is being considered, a land surveyor will make a detailed map of the area. Based on this map the alignment of the surrounding dike will be determined. But the surveyor will also calculate the overall area inside the future polder which will become available for agriculture. To obtain the nett area (which will be taxed by the waterboard!) he deducts the areas comprising creeks, borrow areas, roads and road side ditches from the gross area. In Table 7.8 such figures are presented for some polders of Table 7.1. In most cases the creeks cover an area of less than 10 % of the gross area poldered.

Table 7.8 Area of creeks in relation to area poldered (gross)

pro-ject no.	name of polder	year of reclam-ation	areas (ha)				% of creeks in gross area poldered
			area poldered (gross)	area of creeks	area of roads, road side ditches & borrow pits	taxable area	
4	Jonkvrouw Annapolder	1727	218.73	25.32		193.41	less than 7
5	Anna Frisopolder	1747	105.74	1.56		104.18	less than 1.5
6	Reigersbergse polder	1773	1018.80	81.23[39]	14.90 (only borrow pits)	< 922.67	more than 8, maybe 12 to 15
7	Henriëttepolder	1776	134.50	4.89	p.m.	<129.61	3.6
8	Eendragtpolder	1777	223.89	5.72	3.39	214.78	2.6
-	Prins Hendrikpolder[40]	1907	282.60	25.49	pm	<257.11	less than 6
21	Hoogerwaard-polder	1912	216.81	23.80	9.03	183.98	11

Now in most cases the creeks inside the area to be reclaimed are widest in the alignment of the surrounding dike and from there on gradually become more narrow and shallow. One could say that the overall creek area c = ½ ($a_1xb_1 + a_2xb_2$ ….+ a_nxb_n) where a = width of a creek in dike alignment and b = length of that creek. The wet profile of a creek in the dike alignment during a neap tide is assumed to be a x h (h = tidal range at neap)[41]. One can then schematize the tidal volume in a creek as 1/3 x a x b x h.
The tidal prism during a neap tide for the whole area to be reclaimed is then 2/3 x c x h.

[39] Probably, this concerns only the two major tidal channels.
[40] This polder has not been included in Table 7.1.
[41] Obviously, this is a conservative approach because the said profile could be triangular or trapezoid; moreover, the tide at neap may not reach as high as the level of the salt marsh and also the depth of the creek may be less than h.

Most of the tidal prisms presented in Table 7.1, column 6, have been calculated following this approach. In general, these tidal volumes are small and, moreover, these tidal flows are entering and leaving the salt marsh through more than one channel.
The tender documents for construction of the dikes, etc. specify quality of the earth fill to be used when closing creeks as well as dimensions and slopes. They also stipulate that all tidal channels have to be closed at the same time and if the contractor wishes to use fascine constructions for closure of a tidal channel, he may do so at his own cost. For some projects from the 19th and 20th century the weekly progress reports have been archived. These reports contain all kind of information about the works under construction but references to possible difficulties during the closure of creeks are missing.

Table 7.9 Construction costs of reclamation schemes

project no.	**name of polder and location (province, town, island**	**year or date of reclam-ation/ closure**	**cost of completed scheme (€ of the year 2020)**			**construction cost of dikes (€ of the year 2020)**		**dike length vs. construction cost & reclaimed area (€ of the year 2020)**		
			overall cost (€)	ha re-claimed	cost/ha (€)	overall cost (€)	cost/ha	dike length (m)	cost/dike length (€/m)	dike length/ha reclaimed
	[1]	**[2]**	**[3]**	**[4]**	**[5] = [3] / [4]**	**[6]**	**[7] = [6] / [4]**	**[8]**	**[9] = [3] /[8] or [6] / [8]**	**[10] = [8] / [4]**
2	Groote Waard;, new dike Broek-Zevenbergen	1410-1412		3 900		6 191 276 [42]	1 587	7 009 or 7 257	883 or 853	1.8 or 1.9
3	Groot Zuid-Beijerland	1631	2 576 047	996	2 586	2 330 000	2 340	3 250	717	3
4	Jonkvrouw Annapolder Island Noord-Beveland	1727		215		480 780	2 236	4 401	109	20
5	Anna-Frisopolder, island Noord-Beveland	1747	254 319	129	1 971	217 632	1 687	2 413	90	19
6	Reigersbergsche-polder, island Zuid-Beveland	1773		1 087		3 342 574	3 075	14 082	237	13
7	Henriëttepolder; island. St.Philipsland	1776		109		343 940	3 155	2 570	134	24
8	Eendragtpolder, Oost Zeeuws-Vlaanderen	1777		224		596 522 (cost estimate)	2 663	4 844	123	22
9	Wilhelminapolder Island Zuid-Beveland	1809	4 947 228	1 570	3 151	3 300 672	2 102	7 190	459	5
10	Catharinapolder,Oost Zeeuws-Vlaanderen	1846	318 194	415	768	264 281	637	1 290	205	3
11	Mortierepolder Island Walcheren	1846	55 848	50	1 117	27 675	554	350	79	7
14	Willem III polder Oost Zeeuws-Vlaanderen	1861	109 589 (contr. sum)	236	464			3 500	31	15
15	Zwinpolder, West Zeeuws-Vlaanderen	1864		235		452 410	1 925	700	646	3
16	Schengepolder, island Zuid-Beveland	1874	3 499 942	538	6 505	2 666 622	4 957	2 500	1067	5
17	Van Lyndenpolder, Oost Zeeuws-Vlaanderen	1876		174		596 052	3 426	790	754	5
18	Sluispolder, island of Tholen	1877		19		135 700	7 142	250	543	13
19	Damespolder, island Zuid-Beveland	1884	2 366 887	210	11 271			3 800	623	18
-	Prins Hendrikpolder, at Slaakdam, St. Philipsland	1907	3 328 904	339	9 820			4 870	684	14
22	Dyckmeesterpolder; in tidal inlet Braakman, Zeeuws-Vlaanderen	1918	12 009 907	344	34 912	6 328 405	18 397	10 000	1200	29
23	Kreekrakpolder, In former Kreekrak between isl. Zuid-Beveland and Noord-Brabant	1923	2 359 783 (contr. Sum)	257	9 182			1 680	1405	7
24	Hellegatpolder, Zeeuws Vlaanderen	1925	2 094 702 (contr.Sum)	135	15 516			1352	1549	10
25	Biesboschpolder; Z-Holland, bordering Nieuwe Merwede	1926	10 872 073 (employment project)	925	11 754	7 135 965	7 715	8 260	864	9
26	Abraham Wissepolder, island of St. Philipsland	1935	1 539 342 (employment project)	60	25 656			1 000	1539	17

[42] Dike was 95 % complete; roads and ditches etc were never made.

7.7 Cost of constructing a reclamation scheme and the dikes (incl. closure of creeks)

In Table 7.9 the costs of a number of reclamation schemes are compared. First of all the cost figures of Table 7.1 have all been translated into guilders by applying £1 (*vlaams*) = f 6.00 and, subsequently, the guilders have been transferred into euros of the year 2020 by using the website: https://iisg.amsterdam/en/research/projects/hpw/calculate.php?back-link=1; accessed on 1 Nov. 2023. This transfer enables a comparison of the various reclamation schemes. But, apparently, there are large differences between the various schemes. This is due to various factors, such as:

- the length of the dike to be constructed in relation to the area to be reclaimed: the value of the parameter dike length/area reclaimed varies between 2 and 29;
- the number, depth and width of tidal channels to be closed in relation to the length of dikes on the salt marsh (see below);
- last but not least the dike profile: during the 19th and 20th century the dikes got a higher crest level and became wider than before while stone pitching was introduced on the seaward slope; consequently the cost of dikes per linear metre increased.

Accordingly, the cost of dike construction per ha reclaimed varies in the Table between € 554 and € 7 715[43].

7.8 Cost of dike construction

7.8.1 General

For some reclamation schemes of Table 7.9 details of the tender procedure are available. This renders it possible to check how expensive or difficult the closure is of tidal gullies and channels. In the following sub-sections three reclamation schemes are discussed. Tenders are always given in pounds Flemish or guilders per linear rod of dike. The rod used can be the rod of Gent or Putten. But below, all tenders have been translated into euros of the year 2020 per linear metre.

7.8.2 Jonkvrouw Annapolder (reclaimed 1727) (project no. 4)

The works are constructed through 28 different contracts. Most contracts concern construction of a dike section, long 30 or 40 m or a figure in between. The polder is shown on Fig.7.15

- three dike sections have a slightly different cross section because they are built on a barren and sandy subsoil[44]. Overall length 381 m, cost € 147/m^1;
- nine dike sections have a 'heavy' profile: base of 5 rods, crest 8 ft. wide, height 11.5 ft, accordingly cross-section is 44.67 m^2. Overall length 1 416 m, cost € 88 – 106/m^1. But some tenders are higher in price due to the presence of creeks crossing the alignment. The cost then increases to € 344/ m^1. But that is for the greater part due to the increase in the quantity of earth fill and its higher cost (waterborne transport);
- sixteen dike sections have a 'light' profile, the cross section concerned is 33.93 m^2. Overall length 2 604 m, cost is € 53 to 71/m^1. But also here creeks will increase locally the cost to € 182/ m^1.

7.8.3 Anna-Frisopolder (reclaimed in 1747)(project no. 5)

The works are constructed through 23 different contracts. Tenders are received on 23 March 1747.

- the first fifteen dike sections have a 'heavy' profile: base of 5.5 rods, crest is 8 ft. wide, height 10 or 11 (?) ft., seaward berm is 2 rods wide. Overall length is 1 490 m. A dike section in the contracts has a length between 85 and 126 m; cost varies between € 87 and € 99 the linear metre;
- the sixteenth dike section crosses a creek (shown on the map: Fig. 7.16); length of section is 69 m. Cost is € 114/m^1;

[43] The cost per ha reclaimed for the Dijckmeesterpolder (€ 18 397) is so high because of the extensive auxiliary works required to realize this reclamation scheme.

[44] Called *blick* or *blik: onbegroeide aan- en opgewassen zandgronden* (Danner et al, 2009, 32)

- the remaining seven dike sections have a total length of 856 m. Because it concerns dike sections at the east side of the polder the dike profile is lighter: base of 4 rods, crest is 6 ft. wide, height (?), seaward berm is 1 rod wide. Accordingly, the cost per linear metre of dike is also lower: € 65 to € 72.

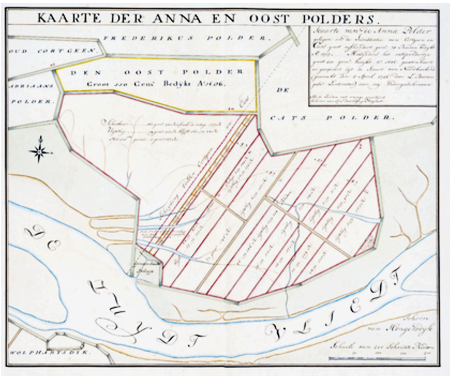


Fig. 7.15 Jonkvrouw Annapolder; source: NL.MdbZA_293_112

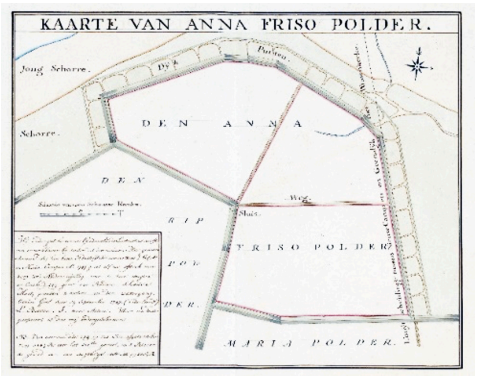


Fig. 7.16 Anna-Frisopolder; source: NL.MdbZA_293_106

7.8.4. Reigersbergse polder (reclaimed in 1773) (project no. 6)

This is a relatively large polder (Fig. 7.17) surrounded by 14 km of dikes. This explains the large number of contracts for construction of dikes and tidal closures: 114.
The works can be distributed over five categories:

- dike sections with or without closure of gullies and small creeks,
- closure of two major tidal channels,
- compartmentalisation bunds, and
- miscellaneous, like the sluice, office for the supervisors ('*de Heere tent*') and a quay wall;
- cost of roads, ditches and works supervision.

Here, only the first three categories will be discussed.

(a) <u>Dike sections with or without the closure of gullies and small creeks</u>

The overall length is nearly 12 km. Most dike sections have a length of ± 120 m. Tenders vary between € 96/m^1 and € 238/m^1 and the large difference will be due to the many creeks and gullies to be crossed. The average construction cost per linear metre of dike is € 170 and the overall total of tenders submitted is € 2 048 371.

(b) <u>closure of two major tidal channels</u>

These channels called Vinkenisse Gat and Batse Kaaij are secondary tidal channels between Oost- and Westerschelde. They have to be closed at both ends so tenders are received for four closures (Table 7.10).

Table 7.10 Particulars of tidal channels to be closed and provisions for compartmentalisation bunds; source: NL.MdbZA_3157_3a.

name of tidal channel	**location**	**length of closure**	**tender (€ of the year 2000)**		**bonus pay-ment**	**compartmentalization bunds**			
		m	€/ m^1	total in €	€	in tidal channel:	length (m)	Cost in €/m^1	total in €
Vinkenisse Gat	south side north ,,	241 359	425 352	110 425 126 368	5 745 5 745	Vinkenisse Gat	1 468	11.80	17 322
Batse Kaaij	south side north ,,	365 268	458 349	167 170 93 532	5 745 5 745	Batse Kaaij	1 205	9.44	11 375

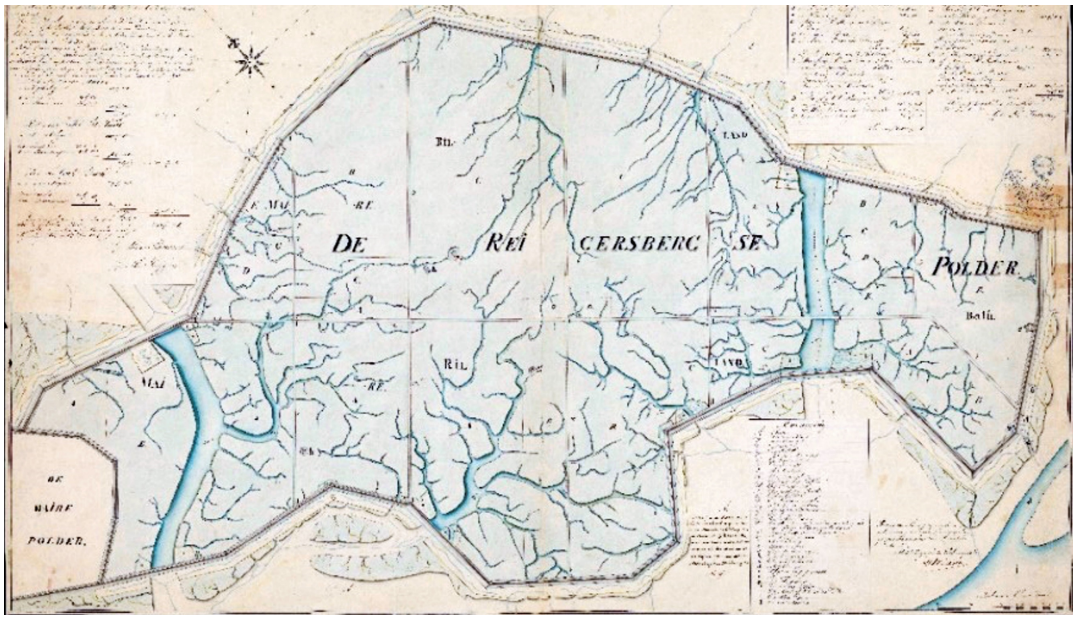


Fig. 7.17 The Reigersbergschepolder as surveyed in 1774 (source: NL.MdbZA_361.3_1945_029)

The average cost of these closure works is € 314/m[1]. That cost must be compared with construction of the dike on the ripe salt marsh: € 96/m[1].
Apart from the tendered earth works, contracts have been made for the purchase of fascine materials: 63 000 bundles of *Gelders rijs,* 36 000 bundles of thin flexible stakes (*gaarden*) to be used for wattle work, 8 000 stakes, 30 000 *Walcherse* stakes, 600 *weliën* (probably hawsers) each 12 m long, 800 hurdles and 8 000 bundles of straw (*gluy*). It is not clear whether these materials have been made available free of charge to the contractors or that they had to pay for it (probably the latter). It is also not known where these materials have been used and for what purpose. One would think: for the closure of the major tidal channels and to enable earth works at locations where the weak subsoil easily slides. It is however clear that the quantity of *Gelders rijs* is not sufficient to construct substantial fascine dams and pakwerken. According to Caland (1833, 206 et seq.) for 100 m^2 of a 50 cm thick fascine mattress, 1 500 bundles of *Hollands rijs* are required and an extra 373 bundles for fabricating the fascine strings (*wiepen*). Caland does not give figures for Gelders rijs but it is assumed that in that case 1 200 + 300 = 1 500 bundles are required. A bed protection of 100 m^2, thick 30 cm, would require 3/5 x 1200 +300 = 1 020 bundles. This implies a total area of bed protection of 63 000/1020) x 100 = 6 176 m^2. Say this protection is used to create a 10 m wide sill, then only 618 linear m of sill can be made while the overall length of the four openings to be closed is already 1 233 m. It is therefore thought that the major tidal channels have been blocked at both ends mainly by dumping and placing of clay fill after having constructed in each channel two or three compartmentalisation bunds.

(c) <u>compartmentalisation bunds</u>

In Table 7.10 also the overall lengths of compartmentalisation bunds are given. These bunds will have been used, first of all to close all creeks connected to the two major channels Vinkenisse Gat and Batse Kaaij. But that application would not require the quantities mentioned. Above, it has been suggested that in each of the major channels two or three compartmentalisation bunds are constructed. Given a length of these tidal channels of appr. 1 500 and 1 120 m a compartment would have a length of 375 m when applying three and two bunds respectively. These five bunds would have an overall length of ± 1 550 m. By using the computer programme CRESS it is calculated that the maximum current velocity u_{max}. in the southern mouth of the Batse Kaaij will be 0.87 m/s for a final closure gap of 20 m width and a neap tide having a tidal range of 2.5 m.

7.8.5 Concluding remarks

In this Sub-section 7.8, three reclamation schemes have so far been analysed. It would be possible to add another two or three projects but this would not result in spectacular findings. It was considered more useful to analyse the unit rates used in the construction of the reclamation schemes in the period 1764 – 1923 and to discuss the use of temporary bridges for closures in the 19th and 20th century.

7.9 Unit rates

7.9.1 Introduction

In a number of contracts unit rates are given for measurement purposes while sometimes also detailed cost estimates are presented[45]. It is interesting to compare these rates with one another as it gives an idea where the money is going and how construction techniques have developed in time.

7.9.2 Earth works

Up to the twenties of the last century dike fill was dug and placed by hand while most of time transport was by means of wheelbarrows. Normally in the unit rate, apart from digging and placing the dike fill, a transportation length has been included of 20 or 40 m (40 m is called a *handsloop*). The 40 m distance included in the unit rate is reduced when the fill has to be transported over sloping terrain.

Below, in Table 7.11, unit rates for earth fill are given as found for a number of reclamation schemes. The values in guilders of the year of contract or cost estimate have been transferred into euros of the year 2020[46]. Though the economic situation in the year of the contract will have its impact on the unit rate, the trend to pay gradually higher wages in time is clearly visible.

Table 7.11 Unit rates for digging, transporting and placing 1 m^3 of earth fill in dikes by utilizing wheelbarrows

project no.	name of polder	year of contract or cost estimate	20 m distance		40 m distance		additional 40 m	
			f	€	f	€	f	€
7	Henriëttepolder	1764			0.20	2.22		
10	Catharinapolder	1830			0.17	1.69	0.08	0.80
13	Anna Jacobapolder	1847			0.20	1.86	0.08	0.74
-	Leendert Abraham polder	1853			0.17	1.86	0.07	
15	Zwinpolder	1864			0.17	1.97	0.08	0.93
16	Schengepolder	1874			0.18	2.00	0.08	0.89
18	Sluispolder	1877	0.15	1.63	0.20	2.17	0.08	0.87
20	Hertogin Hedwigepolder	1907	0.15	1.87	0.20	2.49	0.08	1.00
21	Hogerwaardpolder	1912	0.18	2.05	0.24	2.73	0.10	1.14
23	Kreekrakpolder	1923	0.45	3.56	0.52	4.11	0.12	0.95

More efficient means of transport are also introduced (Table 7.12).

Table 7.12 Unit rates for digging, transporting and placing 1 m^3 of earth fill in dikes by utilizing hand-carts, horse-drawn carts and side tip wagons on a narrow gauge rail track

project no..	name of polder	year of contract	hand-carts and horse-drawn carts				side tip wagons	
			distance 300 m		distance 500 m		distance 1500 m	
			f	€	f	€	f	€
20	Hertogin Hedwige polder	1907	0.40	4.98	0.44	5.48	0.50	6.22
21	Hogerwaardpolder	1912			0.40	4.55	0.50	5.69
23	Kreekrakpolder	1923			1.00	7.91	1.30	10.28

But placing dike fill in tidal channels may also require waterborne transport (*vletten*) and this is not a typical 20th century phenomenon (Table 7.13).

[45] In the last case the year, the cost estimate was made, does not always match the year of reclamation.
[46] For the transfer of guilders of a certain year into euros of the year 2020 the website: https://iisg.amsterdam/en/research/projects/hpw/calculate.php has been used.

Table 7.13 Unit rates for digging, transport by ship and dumping 1 m^3 of clay clods or earth/sand in the dike body

project no.	name of polder	year of contract	clay clods from salt marsh		earth or sand	
			f	€	f	€
10	Catharinapolder	1830	0.70	6.97		
13	Anna Jacobapolder	1847	0.79	7.35	0.72	6.70
20	Hertogin Hedwigepolder	1907	0.50	6.22		
21	Hogerwaardpolder	1912	1.10	12.52		
23	Kreekrakpolder	1923	1.90	15.03		

7.9.3 Placing sods, summer and winter cramp mats

In the past the sea ward slopes of dikes were protected against damage by wave attack by a grass cover of sods (turfing) obtained from the salt marsh while in the tidal zone each year in spring a summer cramp mat was made and in the autumn a heavier winter cramp mat (Table 7.14)

Table 7.14 Unit rates for protecting 1 m^2 of the seaward slope of a dike

project no.	name of polder	year of contract	turfing		summer cramp mat		winter cramp mat	
			f	€	f	€	f	€
7	Henriëttepolder	1764	0.18	2.00				
13	Anna Jacobapolder	1847	0.04	0.37	0.09	0.84		
-	Leendert Abraham polder	1853	0.04 (thin)	0.44			0.10	1.09
15	Zwinpolder	1864			0.08	0.93	0.11	1.28
16	Schengepolder	1874	0.08 (thin)	0.89	0.07	0.78	0.11	1.22
17	Van Lyndenpolder	1876	0.08	0.89	0.07	0.78	0.11	1.23
18	Sluispolder	1877	0.08 (4cm)	0.87	0.08	0.87	0.12	1.30
20	Hertogin Hedwigepolder	1907	0.05 – 0.08	0.62-1.00	0.07	0.87	0.10	1.24
21	Hogerwaardpolder	1912			0.08	0.91	0.11	1.25
23	Kreekrakpolder	1923	0.10 – 0.12	0.76-0.92	0.16	1.22	0.20	1.53

7.9.4 Fascine works

Fascine works were considered to be expensive and, if at all possible, they were avoided. In the 19th century a protection layer of fascines ballasted with stones (rock) was generally applied at dike sections exposed to heavy wave attack (see for instance Ch.2 and Appendix A, Fig. A.5).
As regard the cost of a floating fascine mattress, Caland (1833, 206 et seq.) makes a detailed calculation of the cost of materials incorporated in a floating fascine mattress and the cost of its fabrication, transport and labour for ballasting. He comes to a cost of f 1.39/m^2 for a 50 cm thick mattress and f 1.25/m^2 for a 40 cm thick mattress. These costs are comparable to those given in Table 7.15 for the floating fascine mattresses of the Schengepolder. In both cases the cost of purchase and transport of clay clods or stone required as ballast is excluded.

Table 7.15 Unit rates for fascine works as part of dike and sluice works and for closing tidal channels

project no.	name of polder	year of comtract	fascine slope protection		fascine bund		floating fascine mattress		fascine apron	
			f/m^2	€/m^2	f/m^3	€/m^3	f/m^2	€/m^2	f/m^2	€/m^2
13	Anna Jacobapolder	1847	0.80	7.44					1.20[47]	11.16
-	Leendert Abrahampolder	1853	0.90	9.82	1.75	19.10				
15	Zwinpolder	1864	0.90	10.44	1.80	20.88				
16	Schengepolder	1874	1.00	11.11	2.50	27.78	1.15 1.70[48]	12.78 18.89		
17	Van Lyndenpolder	1876	1.00	11.14	2.50	27.85				
18	Sluispolder	1877			2.60	28.25				

7.10 Closure of tidal channels using temporary bridges

7.10.1 Introduction

In the chapters of Part I of this dissertation and in the current chapter various methods in use for closing tidal channels have been described. This included (Ch. 5, Sub-section 5.6.4), in the second half

[47] Thickness of fascines layer is 40 cm.
[48] The first amount is for a thickness of fascines in a layer of 30 cm, the second for a 45 cm layer thickness.

of the 19th century, the closure method using a temporary bridge (jetty) across the closure gap. In that case clay (clods) or sandbags were dumped from a train of side tip wagons, crossing the tidal channel on a narrow gauge railway track mounted on a temporary bridge. An early example is the first (failed) closure in 1872 of the *Groote Geul* in the tidal inlet Zwin as part of the reclamation of the Willem Leopoldpolder (1873). The temporary bridge as well as some side tip wagons are shown on a photograph (Fig. 7.18).
In a number of projects the side tip wagons are loaded, then pushed to their destination and tipped, solely by manual labour. But in other projects locomotives are used to pull or push the train of wagons. The tidal channels to be closed preferably should run dry during low tide but a depth of a channel slightly below MLT is also acceptable. In Table 7.16 some particulars are presented of this type of closures. For the projects of this Table always a train of side tip wagons on a narrow gauge railway track, with or without locomotives, was operated. But one can also refrain from transport of earth fill by train and have sandbags or clay bags carried to the temporary bridge and dumped solely by manual labour. This method has been practised in 1953 in the Netherlands[49] and in the period 1979 – 1982 three times in Bangladesh (see Ch. 11).
The basic difference between closure methods described earlier in this chapter and the method employing a temporary bridge is the construction above the level of Low tide.
The clay or sandbags are dumped on a 'sill' formed by a fascine mattress or the crest of a fascine dam (see for instance Fig. 7.19). In the past a pakwerk (hor. closure) was built on that sill in stages and that could cost a number of days. The advantage of dumping clay or sandbags from side tip wagons on the sill is the speed of the operation: you may reach a safe crest level in one tide.
Typically for the temporary bridge method is its simultaneous combination with the placing of hydraulic fill. In most cases one or more fascine mattresses form a narrow sill. There is a real possibility that the tide is going to flow through and under the fascine mattress(es) which may lead locally to piping, followed by settlement of the mattress and sliding or subsidence of the earth mass. The upcoming tide will then overtop the settled dam crest and the closure fails. The timely placing of hydraulic fill at the basin side of the closure, the latter being made by dumping of fill from the temporary bridge, will prevent such piping to develop.

Fig. 7.18
Temporary bridge across the *Groote Geul* in the tidal inlet Het Zwin, photographed on 25 October 1872; source: NL-MdbZA_296_331

[49] Breaches in Kadijk, between polders Kruiningen and Waarde; perimeter closure bund on existing road made by dumping clay filled bags from temporary bridge. See Chapter 10, Section 10.10.

Table 7.16 Closures done by dumping clay (clods) and/or sandbags from side tip wagons on a rail track crossing a temporary bridge constructed across the final closure gap

project no.	name of polder	date of closure final gap(s)	tidal range	final closure gap			no. of labou-rers	no. of rail tracks & number of wag-ons	locom-otives	vol. of earthfill dumped (m^3) & no. of hours	sources
				name	length (m)	bed level					
	1	2	3	4	5	6	7	8	9	10	11
-	Willem Leopold polder, Zeeuws-Vlaanderen	15 Aug 1872 (failed)		Grote Geul				1; 21	no		Middeburgse Courant, 3 Sept. 1872
22	Dyckmeesterpolder; in tidal inlet Braakman, Zeeuws-Vlaanderen	1 Aug. 1918	MHT = NAP + 1.85 m	Honde-gat Gee-nenskil	65 78.5	NAP – 0.65 NAP – 0.87	600	2; 48 1; 24 + wheel-barrows	no no (hor-ses in-stead)	1 218 in 12 h. 11 h.	Verhoeven (1924); Huitema (1947, 129 et seq.)
23	Kreekrakpolder, In former Kreekrak between island Zuid-Beveland and Noord-Brabant	1923	NAP + 2.5 /-2 (spring) +2/-1.8 (neap)	Oude Woens-drechtse kil	65	NAP – 0.40		1	yes	1 650	NL-MdbZA _ 651.1 _ 975
24	Hellegatpolder, Zeeuws Vlaanderen	13 Aug. 1925	NAP+ 1.96/ -2.04 (mean Tide)	Hellegat	35	NAP – 0.96	80 or more	2	yes	840 in less than 12 h.	NL-MdbZA_ 612_184 to 191; var. articles in regional newspapers
25	Biesboschpolder; Z-Holland, bordering Nieuwe Merwede	1926 4 Aug. 1 Sept.	NAP+1.2/ -0.75 mean tide	Zuidgat Noorder-diep	55 77	NAP - 1.10 NAP – 1.10	144 + 48[50] 158 + 35[51]	2; 89 2; 89	yes yes (5 nos.)	2 100 in 11.5 h. 1900 in 11.5 h + 7 000 nos. sandbags	Verhoeven (1929)
27	Damming tidal inlet Dijkwater, Schouwen- Duiveland	26 August 1954 (failed)	tides on day of closure NAP +1.18/-1.20	-	70	NAP - 1.50 (sill)	60	2; 40	yes (2 nos.)	1 000 in 5 h.	Van Duivendijk (1954); Kalkwijk (1955); Rijkswaterstaat (1961); CUR/ RWS (1994)

In the next Sub-section, some additional information will be given about the closures presented in Table 7.16.

7.10.2 Some particulars about the closures of Table 7.16

Willem Leopoldpolder

The Willem Leopoldpolder is an international polder. Its area in Belgium is 495 ha and in the Netherlands 134 ha. It therefore stands to reason that Belgium was in the lead for the supervision and construction of the reclamation works including construction of the 1.6 km long enclosure dam across Het Zwin. In the Netherlands information about the reclamation works could only be traced in regional newspapers (available through https://krantenbankzeeland.nl). It turned out that the first effort to close the final closure gap failed because of piping under the fascine bed protection, followed by subsidence of the crest down to 2 m below the water level which, at that particular moment, was MHT. Ten days later (25 August 1872) the width of the breach had reached 41 m and its depth was 4 m below MLT.

Dijckmeesterpolder (project no. 22)

On the drawings of the Dijckmeesterpolder only the level of MHT is given. But one can assume that the bed level of the final closure gaps (NAP -0.65 m and -0.87 m respectively) is not lower than MLT. It could be queried why the bridge across the widest closure gap has only one track instead of two like the other one. But that is probably because of lack of time: the bridge was only ready in the early morning at start of closure operations.

[50] The 48 men were involved in work related to the hydraulic fill production.

[51] ,, 35 ,, ,, ,, ,, ,, ,, ,, ,,

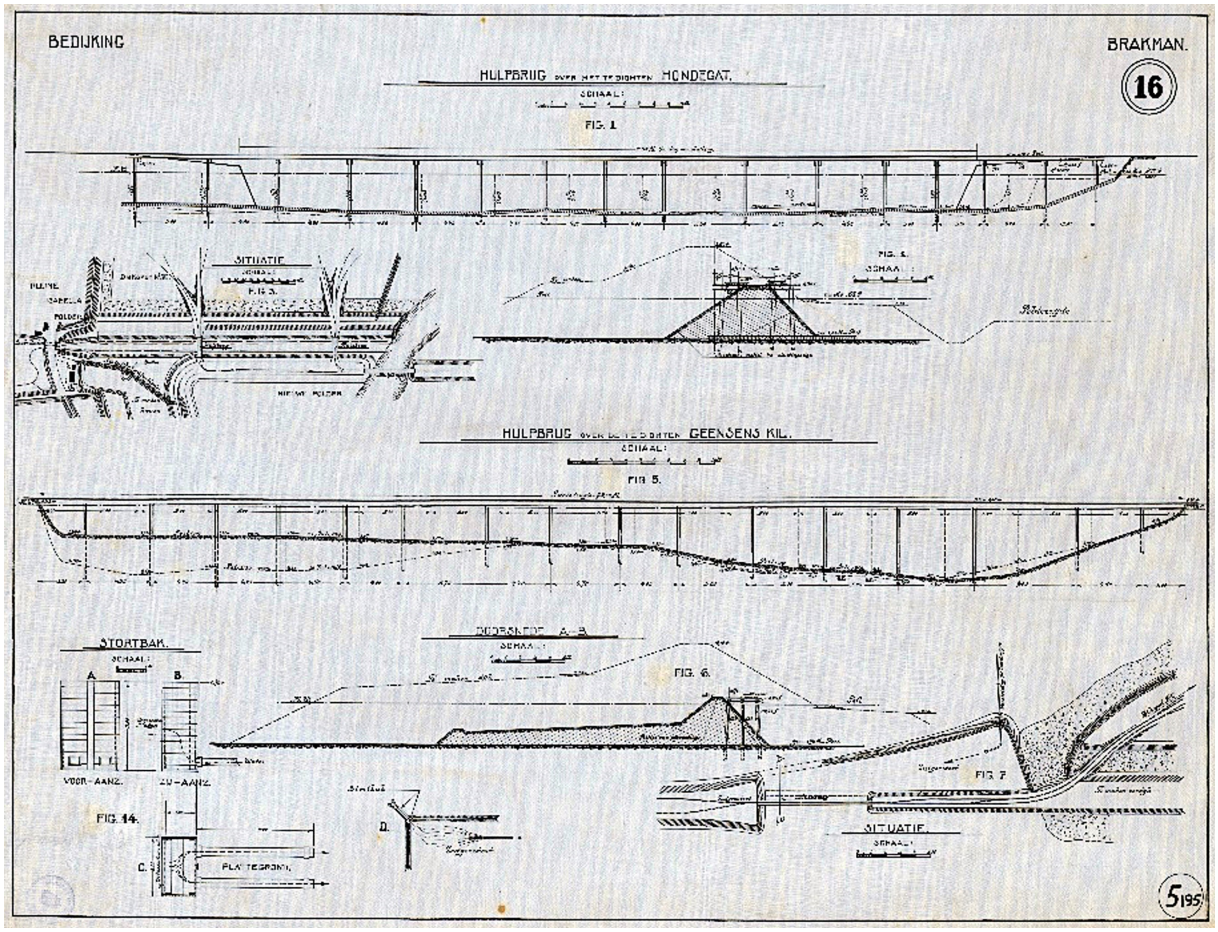


Fig. 7.19 Length profiles over the two creeks to be closed for creating the Dijckmeesterpolder; source: NL-MdbZA_612_132

Fig. 7.20 Dijckmeesterpolder; closure of the Geensens kil in progress on 1 August 1918 at noon; source: fotoalbum Bedijkings- en kanaalwerken Brakman

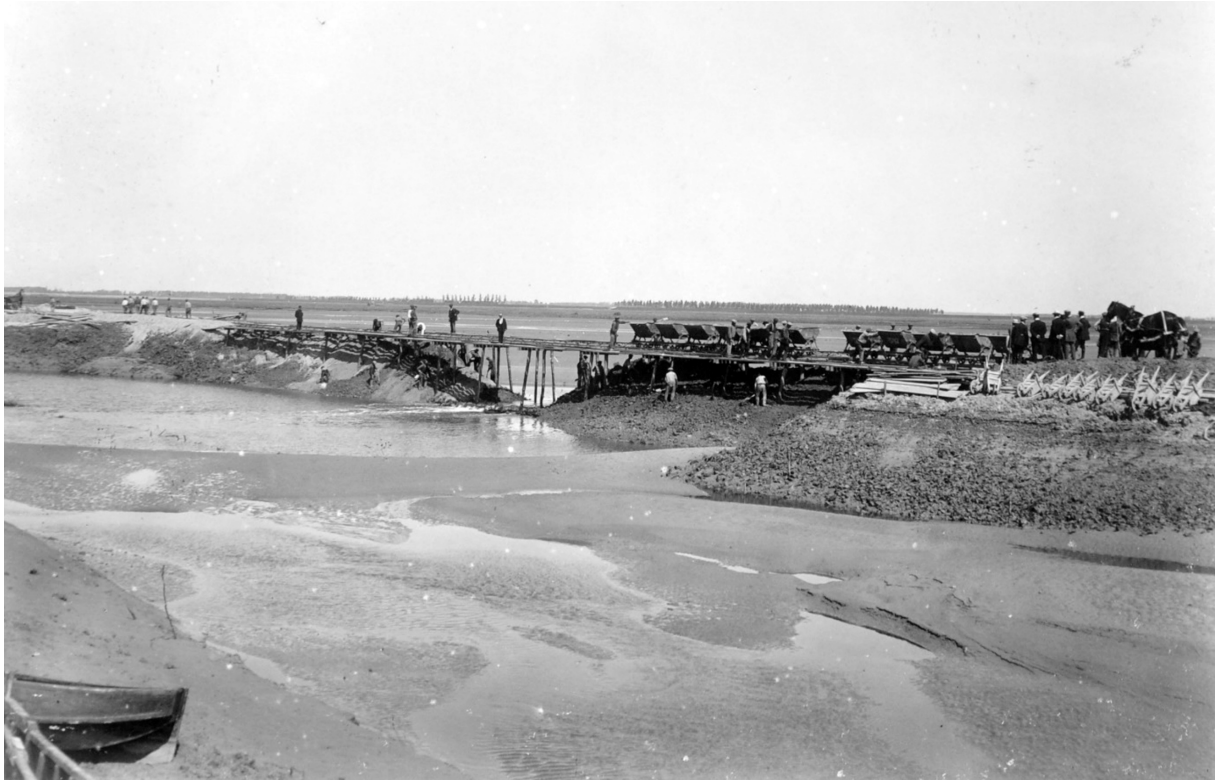


Fig. 7.21 Dijckmeesterpolder; closure of the Hondegat in progress on 1 August 1918 at 3 p.m. at low tide; source: fotoalbum Bedijkings- en kanaalwerken Brakman

Kreekrakpolder (project no. 23)

In the Kreekrakpolder locomotives are used for the first time in a closure to push or pull the side tip wagons. This can be derived from the unit rate in Table 7.12 where a distance of 1 500 m is mentioned. In Fig. 7.22 the plan of closure is presented. Note the low bund of sandbags in cross section E-F (sea side) made to allow placing of hydraulic fill, simultaneously with the actual closure works.

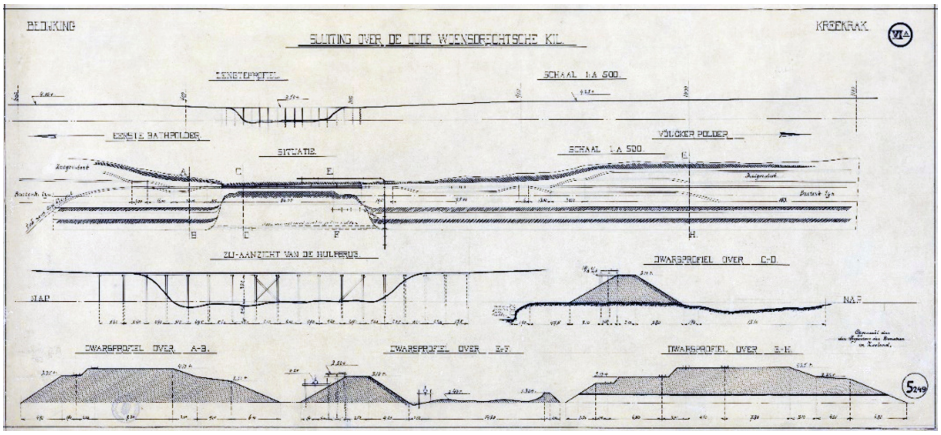


Fig. 7.22 Plan for the closure of the Oude Woens-drechtse Kil as part of the reclamation of the Kreekrakpolder; source: NL-MdbZA_612_180

Hellegatpolder (project no. 24)

The depth in the final closure gap in the Hellegatpolder is NAP – 0.96 m. As MHT = NAP + 1.96 m, also this tidal gully will run dry during Low tide[52].

[52] According to Coen (2008, 71 et seq.) the mean tidal range at Terneuzen during the period 1901-1910 is 4 m; so MLT must be approximately NAP – 2.0 m.

In September 1925, '*de ingenieur der Domeinen in Zeeland'*, Ir A.G. Verhoeven, was present at an excursion of German hydraulic engineers to both the Dijckmeesterpolder and the Hellegatpolder. But these hydraulic engineers must have known that the closure method, with the help of a temporary bridge, had already successfully been applied in Schleswig Holstein in the years 1921-1924[53]. On Fig. 7.23, showing the Hellegatpolder, note the black lines on the shore: so-called *achterkaden* (rear dikes), made to keep the borrow areas dry during high floods.

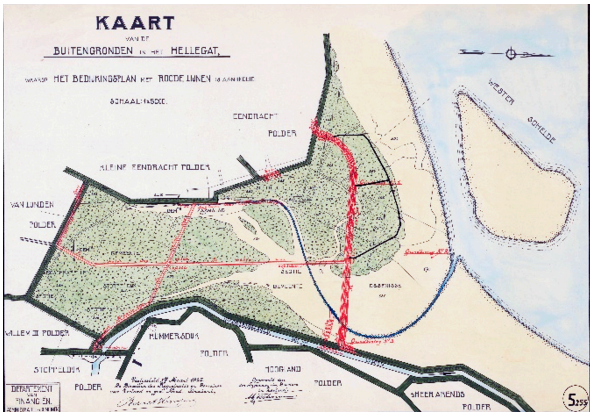


Fig. 7.23 Map of Hellegat showing future sea dike of Hellegatpolder source NL-MdbZA_612_184

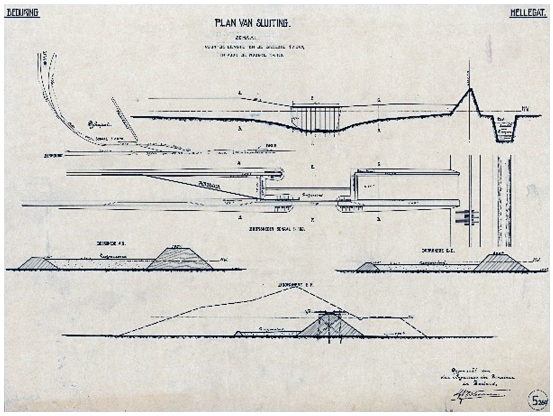


Fig. 7.24 Plan for closure of the tidal creek; source NL-MdbZA_612_189)

A length profile over the sea dike is shown in Fig. 7.12.

Biesboschpolder (project no. 25)

The Biesboschpolder is a relatively large polder (925 ha) and its major creeks are deeper than those of the polders mentioned above.

For closing the *Zuidgat* the contractor wanted to carry out a sand closure but that failed and, as a consequence, the maximum depth in the closure gap became NAP - 4.80 m. Subsequently, he constructed a fascine dam up to a level of NAP – 1.10 m. He then made a new effort by operating suction dredgers, flat deck barges and flat-boats loaded with clay and sandbags but this operation also failed. Finally, he decided to follow the proposal of the Client (*opdrachtgever*) TBD to utilize a temporary bridge. At the day of closure the work started at 5.30 a.m. by narrowing the 55 m wide opening (hor. closure). This continued until the current started to take the dumped clay away. Subsequently, the dam sections already made were strengthened and a vertical closure started for the central section of the dam. The last opening was closed at 11.20 a.m. (slack water) by dumping clay and sandbags. At that time 1 900 wagon loads had been dumped. From then on, the aim was to have continuously a dam crest which was higher than the increasing flood level. At 2.30 p.m. the tide was at its highest : NAP + 1.22 m. So far 2 400 wagon loads had been dumped including 54 wagons loaded with sandbags. Simultaneously with the dumping of clay and sandbags, a suction dredger placed hydraulic fill at the landward side (heel) of the dam while barges unloaded clay at the seaward slope of the dam. At 5 p.m. more than 3 000 wagon loads had been dumped and the dam crest had reached a level of NAP +2 m.

This left the so-called Noorderdiep to be closed. First two fascine dams were constructed with a clay core in between. The landward dam had a crest level of NAP – 1.10 m and the seaward dam a crest level of NAP – 1.30 m[54]. This Noorderdiep was the last of five creeks to be closed and not surprisingly

[53] Closure of a major creek during reclamation of the Pohnstallig Kooge (Hinrichs, 1931, 27, Bild 24)

[54] This practise of having a seaward dam being lower than the landward one had already been introduced nearly 100 years earlier by Caland, see Ch. 5, Fig. 5.4.

the tidal current velocities were high. For this closure were available 5 locomotives, 36 side tip wagons of ¾ m^3, 53 side tip wagons of 1 m^3, two suction dredgers, 75 flat deck barges and flat-boats. The narrowing of the 77 m wide opening started at 5.30 a.m. and it was agreed to stop the narrowing when the current velocity would become 3 m/s and start strengthening the dam sections already made. The closure works then followed the same pattern as those of the Zuidgat and here it is only recalled that the tide reached a level of NAP + 1.02 m at 12.30; the dam crest had a level of NAP +2.40 m at 5 p.m.

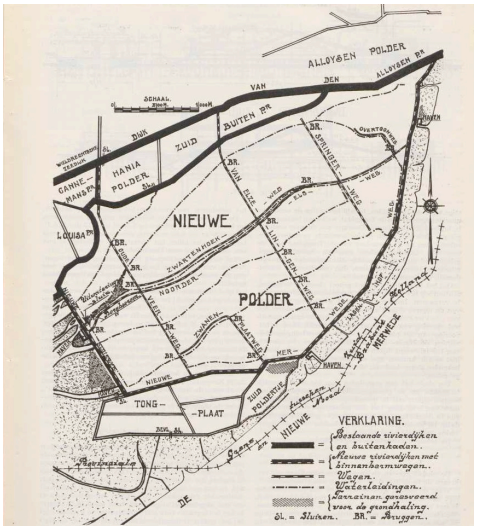


Fig. 7.25 Map of Biesboschpolder (Verhoeven, 1929)

Fig. 7.26 Closure Noorderdiep (at noon); piping is developing in region between fascine covered dike head and closure bund; leak is stopped by sandbags; source: Archives TU Delft, Fac. CEG

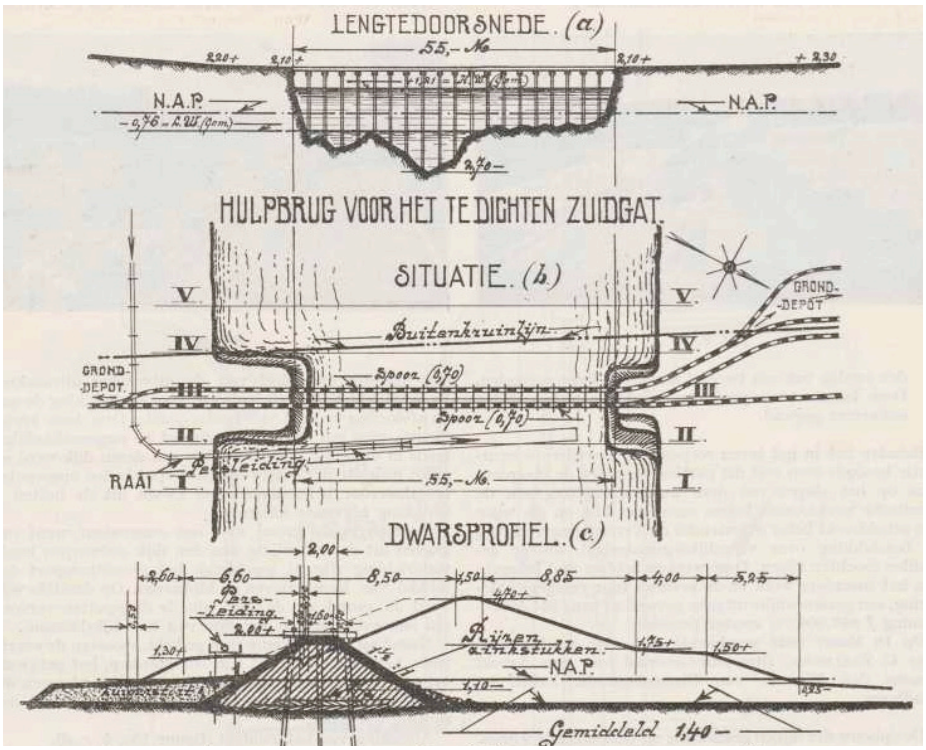


Fig. 7.27 Closure of *Zuidgat*, plan, length profile and cross section temporary bridge with fascine bed protection and closure bund; source: Verhoeven (1929, B 294)

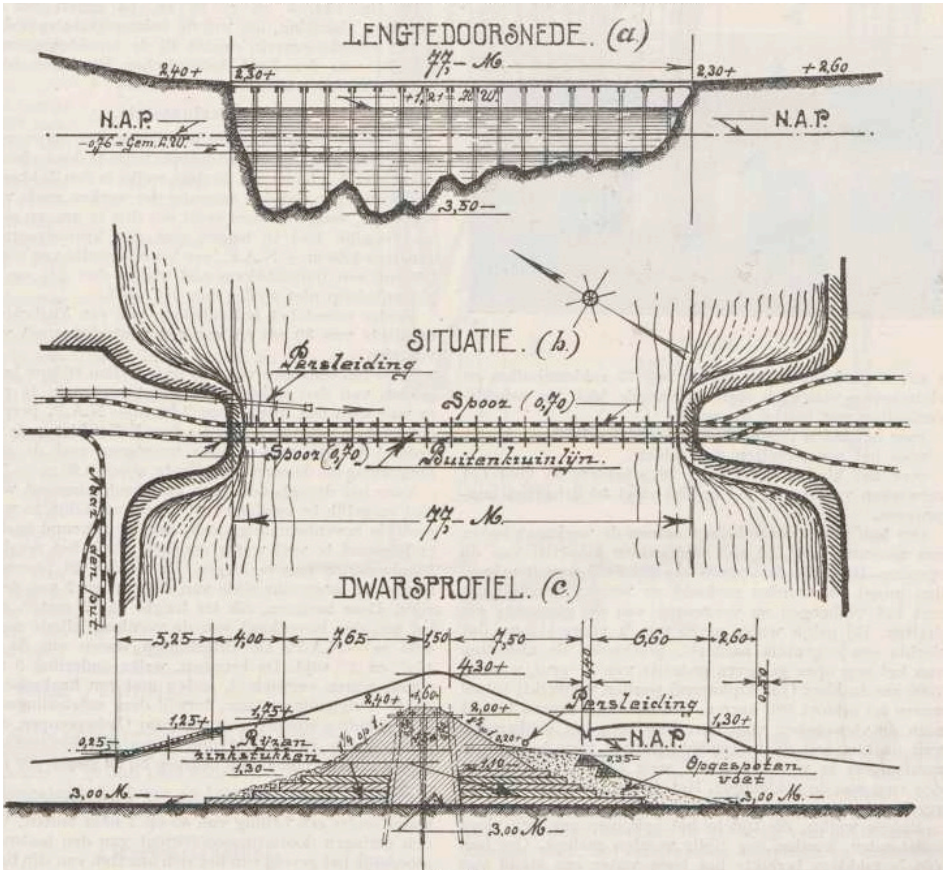
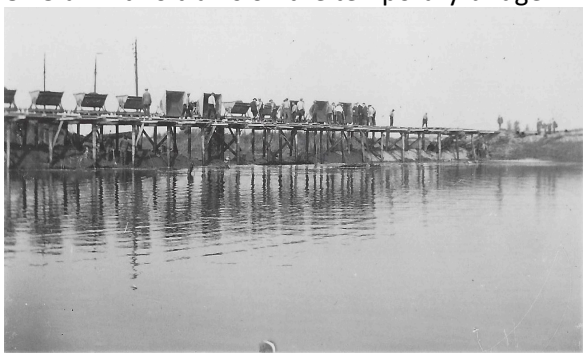


Fig. 7.28 Closure of the *Noorderdiep,* plan, length profile and cross section temporary bridge with fascine dams, clay core and closure bund; source: Verhoeven (1929, B 295)

8.00 a.m. the ebb is still flowing

8.15 a.m. two trains on the temporary bridge

9.15 a.m. overview of the 77m wide closure gap



9.30 a.m. the tidal flow has been stopped

Fig. 7.29 Biesboschpolder, closure of the Noorderdiep on 1 September 1926; source: TU Delft, Faculty Civil Engineering & Geosciences

Damming tidal inlet Dijkwater: failure of a tidal closure just after completion[55] (project no. 27)
In 1954 a tidal inlet in the south western part of The Netherlands, called 'Dijkwater', would be closed.

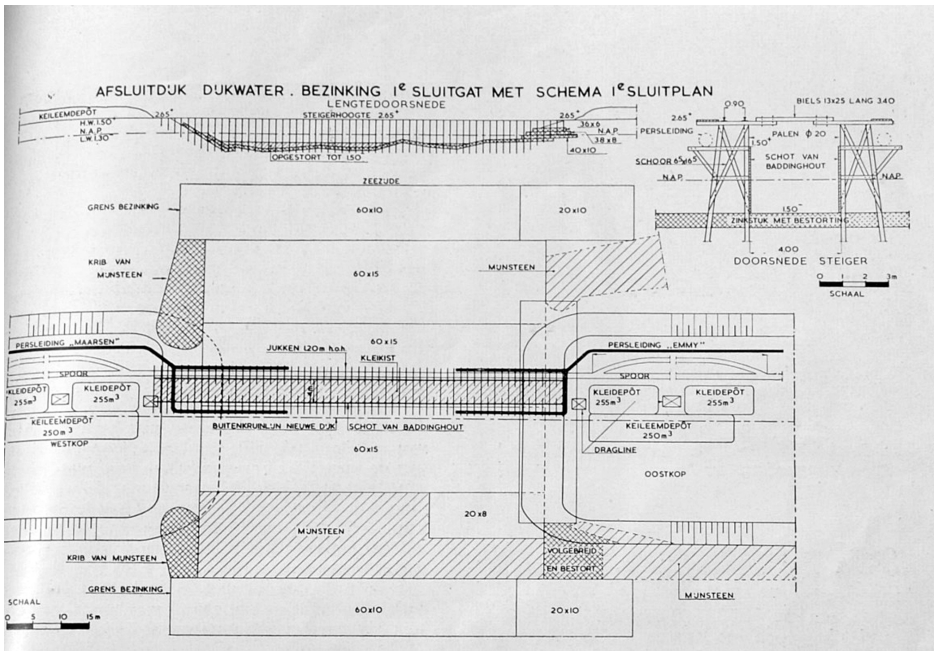


Fig. 7.30 Damming the tidal outlet Dijkwater; top view and profiles of the closure site (dike heads, temporary bridge, stockpiles of clay, etc); source: Kalkman (1955, 63)

The closure concept was as follows:
(a) protect bed of 55 m wide tidal channel by means of fascine mattresses;
(b) construction of sand body of future polder dike on the mudflats (which run dry during low tide);
(c) protect slopes of dike heads in and near axis of future closure bund at both sides of tidal channel by means of fascine bank protection works ballasted with stone (Fig. 7.31);
(d) construct a temporary bridge across the tidal channel and close this channel in one neap tide by transforming the bridge into a cofferdam filled with clay.

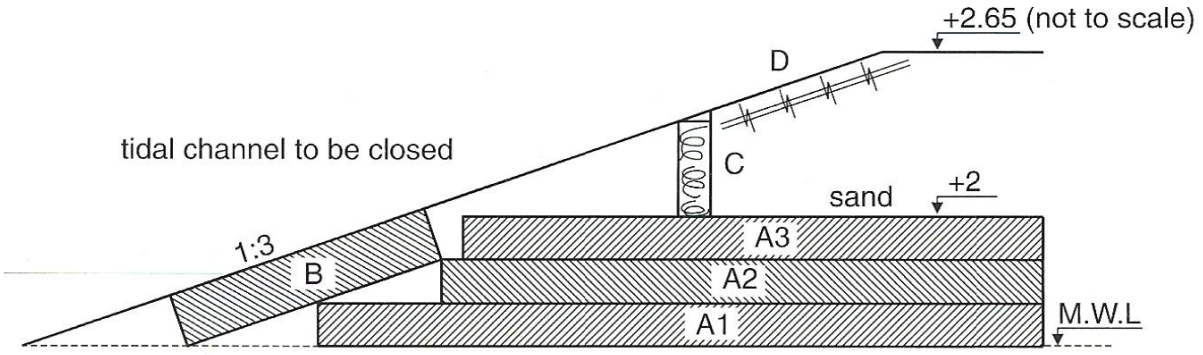


Fig. 7.31 Layer of three horizontally placed fascine mattresses (A1-A3) at east abutment, bordered by bed/bank protection mattress (B), reed 'wall' (C) and light slope protection of fascines (D)
Source: CUR/RWS (1994, 7-6)

On 26 August the closure works started at Low tide at 07.30 hours. At 11.00 hours a 70 m long cofferdam had been formed having a maximum height of 4 m by nailing boards to the bridge piles and bracings and

[55] The following text has been copied from the Manual on the use of Rock in Hydraulic Engineering (CUR/RWS, 1994, 7-5 et seq.). This text has been written by Van Duivendijk in his capacity of co-author of that Manual.

dumping clay inside the cofferdam from side tipping narrow gauge railway wagons crossing the bridge from both sides (Fig. 7.32).
At one side of the channel it had been neglected to remove the aforementioned bank protection B prior to start of closure. When outside high water reached its peak it was roughly 1.5 m higher than the water level in the closed-off tidal inlet. Meanwhile, the water had started to flow through the fascine layer, thick 0.3 m from outside to inside and the adjacent layer of three mattresses (A1-3 in Fig. 7.31). The head difference had become 1.5 m over 4 m distance. It took the sand of the adjacent sand body with it. Within 30 minutes a 40 m wide breach had formed and all works for the closure which had been prepared during a period of 5 months could be considered as total loss. Ultimately, the length of the breach became 140 m. A new attempt was made 1½ months later at considerable cost by constructing a combined vertical and horizontal closure using shales (a waste product of Dutch coal mining operations).

Dumping of clay in cofferdam

Placing hydraulic fill land side

One day after failed closure

Fig. 7.32 Dijkwater, closure site, 26 and 27 August 1954 (photographs obtained in 1954 from various sources)

7.11 Summary and Conclusions

1 In order to analyse closures in land reclamation schemes 27 projects were selected for discussion.

2 The closures were effected between 1263 and 1954.

3 The information regarding the tidal closures in three schemes, constructed during the 13th – 17th century, was rather sketchy or lacking and some interpretations were required while some assumptions had to be made.

4 The average area of a reclamation scheme between 1300 – 1550 is not more than 1.2 km^2; after the 16th century the constructed reclamation schemes are larger.

5 From the information available about some 20 reclamation schemes constructed between 1727 and 1912 it could be concluded that closure of tidal channels must have been relatively easy. This is reflected by the maximum depth of tidal channels (in most cases not deeper than MLT) and by the moderate tidal volumes (in so far as calculated) which were lower than 4 million m^3.

6 No direct ratio could be established between the various parameters determining the cost of a reclamation scheme like area reclaimed and length of surrounding dikes.

7 The cost of closing a tidal creek (incl. full dike profile) per linear metre turned out to 2 to 4 times the cost per linear metre of dike on a 'ripe' salt marsh.

8 The unit rates for transport of dike fill clearly demonstrate the significant impact of the means of transport (wheelbarrows, horse-drawn carts or side tip wagons pulled by locomotives) on the height of the unit rate.

9 From the contract documents for the various reclamation schemes it could be concluded that fascine works (more specifically floating mattresses and pakwerks) only played a role in a few closures.

10 Between 1872 and 1954, six closures were made in the Netherlands by dumping clay (clods) and/or sandbags from side tip wagons, crossing a temporary bridge across the closure gap. This enabled closures to be achieved within one tidal cycle.

CHAPTER 8 Dams through tidal channels in the Netherlands and Schleswig Holstein

8.1 Introduction

What kind of dams are discussed in this Chapter has been formulated in Ch. 6, Section 6.3.
The aim of the study of the dams through tidal channels is basically to discover and describe the problems encountered during construction of these dams and, more specifically, the closure of the tidal channel(s) to be crossed.

Each and every dam serves a purpose. In the Netherlands and in Schleswig Holstein the nine dams (numbered 28 to 35 incl.), discussed below, presented in Table 8.1 and their location shown in Fig. 8.2, have been built aiming at accretion and/or improvement of transportation (by implementing a rail or road connection) or navigation.
As will be seen below in most cases the objectives navigation and transportation were achieved. The objective accretion by building a dam in most cases is also successful but the means, i.e. an overflow dam (*overloopdam*), was throughout a failure. All these low dams (4 nos. in Table 8.1) breached within a few years after completion (sometimes a second time after repairs). Ultimately, three of these dams were rebuilt as a non-overflow dam (nos. 28, 33a and 34) while one was abandoned (no. 32) all together. It is surprizing that people did not learn from these successive failures: no.28 in 1757 and 1765, no.33a in 1863, no.32 in 1881, 1882 (twice), no.34 in the period 1911 – 1919.
The basic reason for all these failures was the lack of knowledge at that time about the magnitude of the flow during a storm surge. In nearly all cases the axis of the overflow dam more or less follows the well-known narrow zone of the tidal divide. But one overlooks the fact that during a storm a surge may occur at one side of the low dam while the wind may considerably lower the water levels at the other side of the dam. At that last side the mud flat may even run dry. The overtopping of the low dam will then result in a plunging jet (*duikende straal*) on the downstream side of a dam as for instance demonstrated in Fig. 8.1. The plunging jet erodes the unprotected berm at the 'downstream' side of the dam which causes this side to slide and collapse.

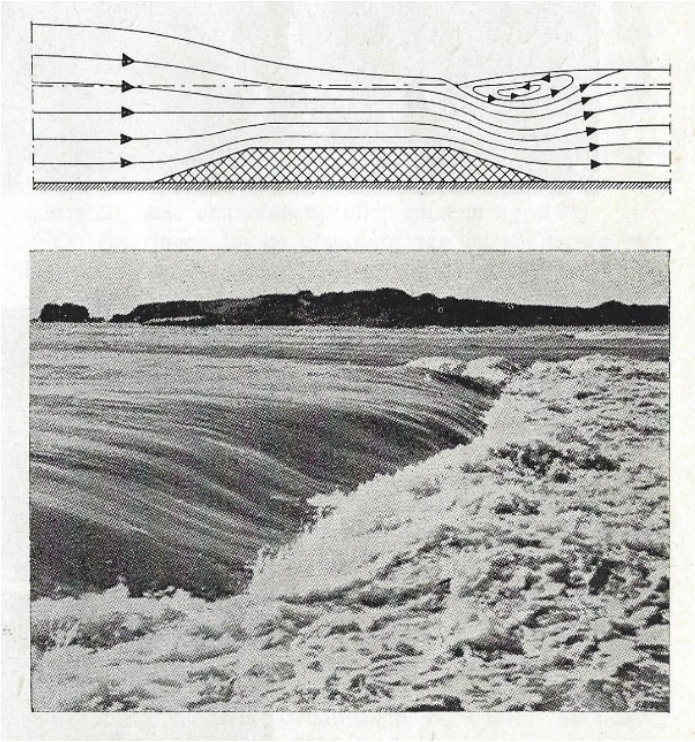


Fig. 8.1
Plunging jet;
source: Dibbits (1956, B.114)

flow over a low dam with plunging jet

plunging jet in the closure gap of the Nolledijk with surface eddy (Walcheren, 1945)

Table 8.1 Dams built in the period 1751 – 1925 in the Netherlands and Schleswig-Holstein

pro-ject no.	name of dam, its location, (province, town, island) & country	purpose of dam	length (m) & crest level	date of final closure	tidal range at start of works		max. diff. in water levels between both sides of dam during closure operation (m)	width of tidal channel to be closed (m) & max. depth (m)	length (m) & bed level of final closure bund (m)	sources	some typical features (like tidal prism in 10^6 m^3)	cost of construc-tion (in guilders at time of construct-ion)
					spring tide	neap tide						
	1	2	3	4	5	6	7	8	9	10	11	12
28	Statendam; between the islands Goeree & Overflakkee, NL	transportat-ion link, navigation, defence & accretion	two sections: 2480 & 1265, crest at MHT	1751 (breached in 1757 and 1763[56])	on average 1.60 m			2 480 and 1 265 MLT – 0.50		Teixeira de Mattos (1941) & (1941a)		contract sums total f 76 805
29	Kreekrakdam , damming the Oosterschelde between Zuid-Beveland & Noord-Brabant, NL	railway link (road link only made 60 years later)	3640 & NAP + 6.28 m	20 July 1867	4.25 m	2.9 m	0.35	200 & 400 NAP – 2	400 NAP – 2	De Stoppelaar (1867) Stoompost (1868); Neyt (1872); Burgers (2017);	bed protection, two fascine dams up to NAP + 1 m	lowest tender: f 1.5 million
30	Sloedam, damming the Sloe between Walcheren & Zuid-Beveland, NL	railway and road link	965 & AP + 6 m	12 July 1871	4.2 m	3.2 m	1.04	365 AP – 10.9	140 AP	Neyt (1872); Van de Linde (1938)	36.5 (Z → N) 9.5 (N → Z)	cost excl. auxiliary works f 1 100 000
31	IJdam, enclosing the tidal inlet IJ, at Schellingwoude, near Amsterdam, NL	navigation & transport--ation link	1360 & AP + 3.5 m	30 Sept. 1872		0.36 m	approxi-mately 1	approxi-mately 300 AP -6	25 (?) AP – 8	Dirks (1871); Watson (1875); Van de Ven (2008); Bakker (2020)	difficulties during construction due to large subsidence of subsoil	f 4.5 million (incl. the navigation locks)
32	Dam from Frisian coast to the island Ameland; between Holwerd & Buren, NL (overflow dam)	to further accretion for reclamation	8400 & AP + 0.54 m	1878 and 1882 breached late 1881, 1882 (2x)	MHT = AP + 1.03 m MLT = AP - 0.93 m = 1,96 m (as stipulated in KB of 29 Dec. 1874, no. 7)		0.05 to 0.10 (estimate)	2 x 150 average depth MLT – 0.6	50, 45 and 60 MLT – 2 MLT – 2.5 and – 4.5	Teding van Berkhout (1860) & (1867) Rijkswater-staat (1892)	ultimately, breaches were not repaired and project was abandoned	roughly f 1 million has been spent
33-a 33-b	Damming the Slaak, between island St. Philipsland & N-Brabant, NL **(a)** overflow dam (Slikdam) &, after failure, **(b)** a non-overflow dam (Slaakdam) at a more southern site	**(a)** to further accretion for reclamation **(b)** transporta-tion link & to further accretion for reclamation	**(a)** ± 2580 & AP + 2.45 m **(b)** 2770 & AP + 5.85 m	**(a)** 1858 (failed in 1863) (**b**) May -June 1884	MHT = AP + 1.45 m MLT = AP – 1.4 (?) m		0.25 (estimate)	(**b**) 86, 100 & 110 at AP, max. depth AP – 1.01, -0.47 & - 0.9		NL-MlbZA _ 63_ 85 De Bruin (1953, 32)	(**a**) the Slikdam failed in 1861, was repaired, but failed again in 1863 and project was abandoned and (**b**) con-structed instead	(**b**) Contract Sum f 334 700
34	Norstranderdamm between the island Nordstrand and the coast of Schleswig-Holstein, Germany (initially an overflow dam)	to further accretion for reclamation	overflow dam 3400 & crest at MHT + 0.5 m; later, a non-overflow dam 4000 & crest at NN +6.60 m	overflow dam June 1907, but breached repeatedly; a non-overflow dam was finalized in July 1935	MHT = NN + 1.24 m MLT = NN – 2.06 m		no data	300 & 200 MHT - 5.0 MHT – 3	263 and 174 MHT – 4.8 MHT – 3	Müller & Fischer (1936) & (1936a)	Initial cross-section was strengthened various times until a definite profile could be built which turned out to be sustainable	no inform-ation available
35	Hindenburgdamm; between the island of Sylt & the coast of Schleswig-Holstein, Germany	railway link only	11 000 & NN + 6.30 m	September 1925	average 1.7 m		more than 1m at largest opening to be closed	200 to 1000 MLT - 0.30 to MLT – 3.0	440 and 2 x 150; various, deepest was HT – 7 to -9	Hinrichs (1931); Gripp et al (1937); Huitema (1947)	tidal gullies partly closed by sheet piling, partly by instantane-ous closures using sliding panels	appr. f. 11.1 million

The tendency to build a low overflow dam for furthering accretion rather than a non-overflow dam was, on the one hand a condition imposed by the Rijkswaterstaat[57] and, on the other, the wish of the dam builder to keep cost of construction as low as possible.

Admittedly, a non-overflow dam across a tidal channel, in most cases, furthers accretion because of deposition of the sediment load in the flow, the latter at that location having a very low tidal current velocity.

[56] In both cases repaired.

[57] It has not been ascertained why, either, this condition was imposed, or, the desire for a low overflow dam was expressed.

Many reclamation schemes have been successfully carried out at one or both sides of these dams[58]. In the next sections some additional information and elucidation will be given about the various dams presented in Table 8.1.

Fig. 8.2 Approximate locations of the damming projects, listed in Table 8.1, marked by their project number; the small map (upper right) shows the west coast of Schleswig-Holstein with the dams numbered 34 and 35

[58] For examples reference is made to Table 7.1: Damespolder (1884), Hoogerwaardpolder (1912), Kreekrakpolder (1922) and Prins Hendrikpolder (1907) (the last one not in the Table).

8.2 Statendam between the islands Goeree and Overflakkee (project no. 28)

The available information has been summarized by Teixeira de Mattos (1941) and (1941a, 340 et seq.), to which reference is made. The information given below has been based partly on Teixeira de Mattos and partly on the sources given by him.

During the first half of the 18th century, repeatedly (e.g. in 1728, 1734 and 1742), the idea crops up to connect the islands Goedereede and Over Flakkee. At the request of the *Staten van Holland*, the surveyor Cruquius produces a detailed bathymetrical map (parts of it reproduced on Figures 8.3 and 8.4) and makes a cost estimate for a non-overflow dam (f 422 282). This is considered to be too costly and it is then proposed to construct a 5 km long dam having its crest at MHT and slopes of 1:3, covered by sods and cramp matted. The cost would be f 70 000. But the accuracy of this figure is doubted by the *Staten*.

Ultimately, the *Staten van Holland* in February 1751 decide to construct an earth dam, having slopes 1 : 2, protected by fascines. The crest would be at MHT and, accordingly, its average height would be 2 m. It was considered that the dam would be advantageous to accretion, navigation (deeper fairway in the Goereese Diep) and defence.

The construction of the dam is tendered on 25 May 1751 in two sections (Fig. 8.5), 2 480 and 1 265 m long[59] and the accepted tenders are f 45 882 and f 30 922.50 respectively[60]. The works are complete in October of the same year. No length profile of the dam is available but it is assumed that the bathymetrical situation did not differ much from that given on Cruquius' map of 1734 (Fig. 8.3). On that map one tidal channel is marked: *het Afgebaackende Vaar Water met Vol Zee over den Hals* (the demarcated fairway at MHT across the *Hals*). According to Fig. 8.4, the average tidal range is five *Rijnlandse* ft (= 1.57 m) while a vessel having a draught of 1.5 m and 0.3 m keel clearance would require a depth of 1.8 m. One may therefore assume that the tidal channel would have a maximum depth of MLT – 0.5 m. As the dam is being built more or less on the tidal divide the closure of this tidal channel must have been easy.

But the dam is low and it is vulnerable to drift-ice! And, as to be expected, the dam is seriously damaged by drift-ice in Jan. 1757[61]. Dams of ice, 2.5 to 3 m high on the dam, cause four breaches (overall length appr. 200 m) and part of the fascine protection on the slopes of the dam is destroyed over a length of 600 m[62]. Repairs are carried out in dayworks but a storm surge in March 1757 results in new damage to the dam and a new breach widens to 45 m. Still, the dam section in the *Hals* has less damage than the section through Scharrezee as the former is higher and has more gentle slopes. It is therefore decided to adapt the cross section of the dam through Scharrezee and protect the slopes and crest of vulnerable dam sections by placing stones from Vilvoorde (slopes) and Norway (crest) while elsewhere cramp matting is applied.

The accretion, since 1752, has been considerable at both sides of the dam and the '*Staten*' are of the opinion that this merits the cost of repairs and annual maintenance of the dam.

Then, in February 1763, there is again considerable damage by drift-ice and storm surges, and the *Staten* now hesitate to, once again, spend considerable funds as, moreover, improvement of the fairway through Goereese Diep has not materialized[63].

Subsequently, it is decided by way of a charter to transfer the ownership of the dam, and the right to reclaim polders at both sides of it, to a group of 21 citizens.

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[59] De lengths are given in rods; it is assumed that the *Brielse* or *Voornse* rod was used, which is 3.89 m measuring 12 ft. of 0.324 m each.

[60] Source: Resolutiën van de Heeren Staten van Hollandt ende West Vrieslandt, NL-HaNA_ 3.01.04.01_192 (year 1751, 1076 – 1077).

[61] Resolutiën…, NL-HaNA_ 3.01.04.01_202 (year 1757, 465-470).

[62] In this respect it is observed in the Resolutiën that cramp matting is normally not damaged by drift-ice.

[63] Resolutiën… NL-HaNA_ 3.01.04.01 _ 214 (year 1763, 794-795).

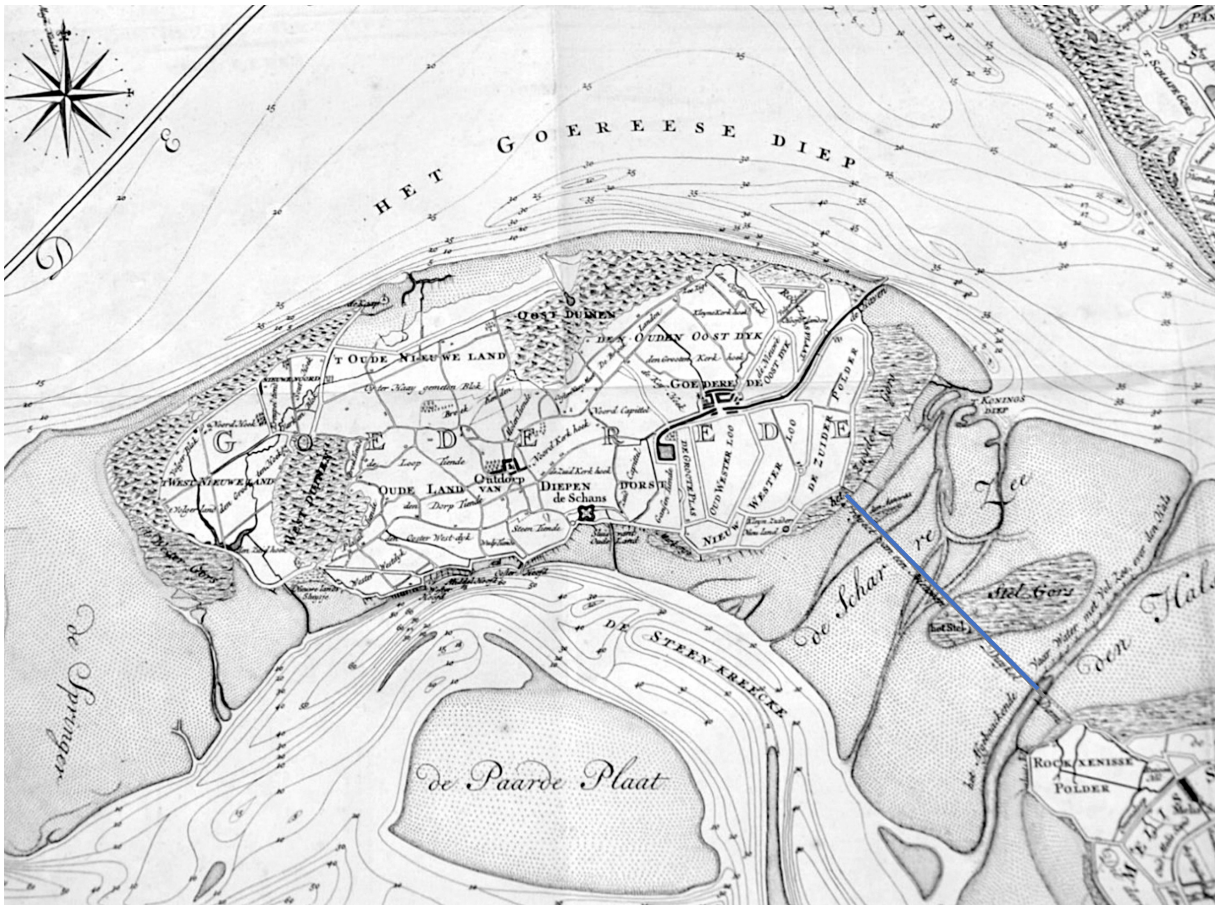


Fig. 8.3 Part of map made by Cruquius in 1734 showing the proposed dam (marked in blue) between the islands Goederede and Over Flakkee;
source: Resolutiën…NL-HaNA_ 3.01.04.01 _ 171 (year 1734, map inserted at page 512)

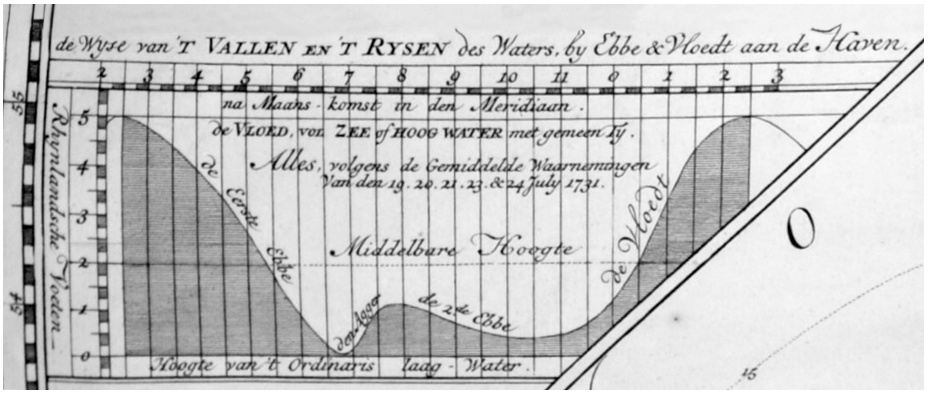


Fig. 8.4 Part of map made by Cruquius in 1734 showing the average tide 'at the harbour' (Goedereede?)

One of the conditions[64] is a significant heightening (1 m) of the dam and slopes of 1: 6 to prevent as much as possible damage in future[65].
The accretion continues and polders are reclaimed in 1769 and 1780. As for these polders sea defences have been constructed the Statendam now becomes a secondary dike.
Meanwhile, during this period, the Goereese Diep and the fairway in front of Hellevoetsluis have considerably deteriorated. The *Staten* request the Inspector-General C. Brunings for advice. In his

[64] Another condition is to maintain the prohibition to drive carts, wagons and the like on the crest of the dam as long as this crest is protected by a cramp mat.
[65] Resolutiën… NL-HaNA_ 3.01.04.01 _217 (year 1765, 351 – 355).

report[66], dated 21 April 1781, Brunings, inter alia, recommends to restore the connection between the Grevelingen and the Goereese Diep/Haringvliet, either by constructing in the Statendam a 1 200 m long weir having its crest 2 to 3 ft. below the MHT in the Brouwershavense Gat[67] or by digging a 150 m wide canal between both tidal waters, having its bed level at the lowest low tides and provided with sluices. The *Staten* postpone a decision on the matter and, according to Teixeira de Mattos (1941a, 351), the problem is hushed up.
Because of the construction of reclamation schemes at both sides of the Statendam, the latter lost its function as a separation between the tidal inlets Haringvliet and Grevelingen and it has been removed. The Damweg (N 215) follows its former alignment.

Fig. 8.5
Horizontal alignment of Statendam as constructed in 1751; source: Teixeira de Mattos (1941, map no. 4)

8.3 Kreekrakdam[68] [69] (project no. 29)

At the request of the emperor Napoleon, *'s Lands Inspecteur in Zeeland*, A. Schraver makes a design for damming the Sloe between the islands Walcheren and Zuid-Beveland. In January 1811, he travels to Paris to submit his report (comprising surveys and a design of the dam) to Napoleon and is then requested to make also, during his stay in Paris, a design for damming the Oosterschelde (Kreekrak). The horizontal alignment of the Kreekrakdam is partly visible on Fig. 7.11 in Chapter 7 and on Fig. 9.34 in Chapter 9.
But, for a long period nothing happens. In 1846, a contractor in Zeeland, Dirk Dronkers, obtains a charter to construct a railway Vlissingen-Middelburg-Maastricht. This would imply dams and/or bridges to cross the Sloe and Oosterschelde (Kreekrak). The latter would be a combination of dam and bridge, approximately in the line Rilland-Bath – Woensdrecht[70]. Subsequently, the conditions of the charter are amended repeatedly, while also the charter changes hands a number of times. In 1859 the charter is revoked.
In accordance with the *Spoorwegwet*[71] of 1860, a railway connection of Middelburg and Flushing with the mainland is expeditiously required. The construction of the dam[72] and a railway between Bergen op Zoom and Woensdrecht is (re-)tendered on 26 July 1866. Lowest tender is f 1 499 000.

[66] Resolutiën… NL-HaNA_ 3.01.04.01 _242 (year 1781, 659 – 671).
[67] Mouth of the tidal inlet Grevelingen.
[68] Dam through the Oosterschelde between Zuid-Beveland & Noord-Brabant.
[69] The text has been drafted using information found in the sources listed in Table 8.1, no. 29, column 10 and in various articles in regional newspapers, available through https://krantenbankzeeland.nl.
[70] At that location the fairway in the Kreekrak was silting up; the tidal divide is approximately at that spot.
[71] Railway Law, concerning the design and construction of a comprehensive railway network in the Netherlands.
[72] The idea of a central bridge section in the dam has meanwhile been abandoned.

Apparently, this figure is much higher than the cost estimate. In the Summer of 1866 the dam section on the shore at the Noord Brabant side is constructed. In March 1867 the works for the tidal closure of the 400 m wide channel start and the closure is a fact on 20 July 1867. Neyt (1872, 222) recalls that maximum tidal difference between both sides of the dam through the Oosterschelde during construction has been 35 cm only (highest level at south side). Given the fact that the closure concept comprised two fascine dams with a wide clay core in between, providing overall for final closure of a 60 m broad sill (in the direction of the current) at a level of MLT + 2.00, horizontal closure (by means of 1.5 m high pakwerk) must have been easy (Fig. 8.6).
An attempt has been made to obtain more detailed technical information. After that abortive attempt the following statement was found: "….neither in the library of the Royal Institution of Engineers, nor … in any magazine (is) a description of this important work to be found…" [73]

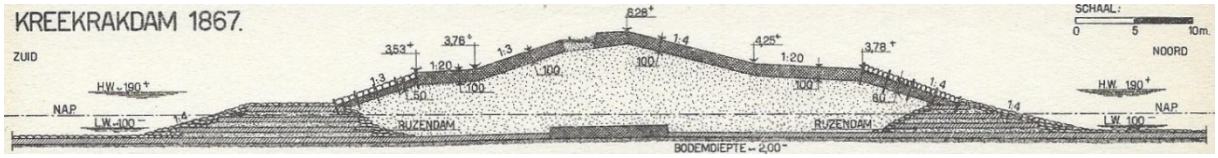


Fig. 8.6 Kreekrakdam, built in 1866 – 1867; initially only railway link, no provision for vehicular traffic; source: Cross sections Afsluitdijken I, (Delft University of Technology, Faculty of Civil Engineering and Geosciences)

According to Van de Linde (1936) a good road on the dam, connecting the provinces Zeeland and Noord-Brabant, has only been constructed 60 years later.

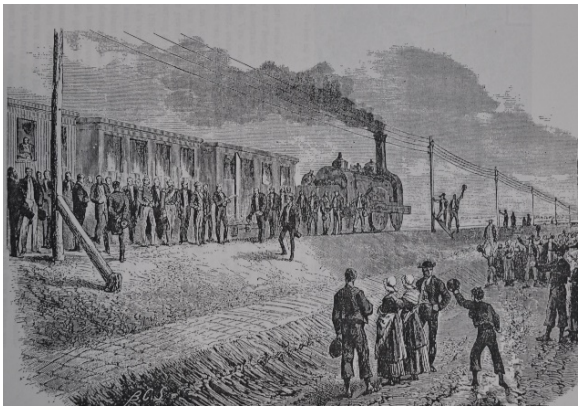


Fig. 8.7
Official Welcome at the Kreekrakdam of the first train which entered the province of Zeeland in 1868;
source: NL-MdbZA_219_91

8.4 Sloedam, damming the tidal channel Sloe between Walcheren & Zuid-Beveland (project no. 30)

The inducement for building the Sloedam is the same as for the Kreekrakdam which has already been discussed in Section 8.3. In 1810, during the period of French dominance of the Netherlands (1795 – 1813) and long before construction of a dam, 's Lands Inspector in Zeeland, A. Schraver, at the request of the emperor Napoleon, made surveys and a design. He presented his report to the emperor in Paris in January 1811. This design is shown in Fig. 8.8. This design is a classic one: a base of two parallel fascine dams, comprising fascine mattresses, having a clay core in between and a crest level at MLT.

[73] Statement made by Repelaer van Driel (1918, 382 et seq.) within the context of an obituary of Mr. L. J. Kesper (1839 – 1918) , who was resident engineer during construction of the Kreekrakdam.

A design made by Banque Suisse in 1857 follows the same principle but here the crest level of the base is higher (at AP)[74].

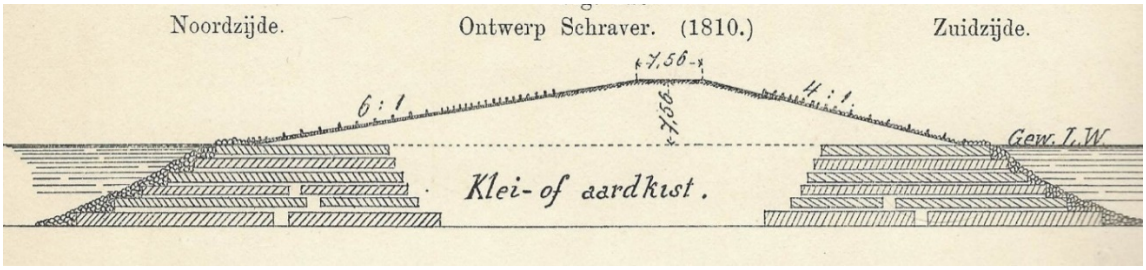


Fig. 8.8 Design of Sloedam in 1810 by '*s Lands Inspecteur in Zeeland*, A. Schraver, for the emperor Napoleon; source: Plasschaert, (1898, Atlas, Plate 30, Fig. 213)

Ultimately, as a consequence of the *Spoorwegwet* 1860, the matter had to be solved expeditiously but the issue, a dam or a bridge across the Sloe, or a combination of both, had to be solved first. Especially the city of Middelburg was pressing for a dam and it succeeded. A provisional design is then made by the Staatsspoorwegen in 1864 and it is still based on the aforementioned two parallel fascine dams (Fig. 8.9).

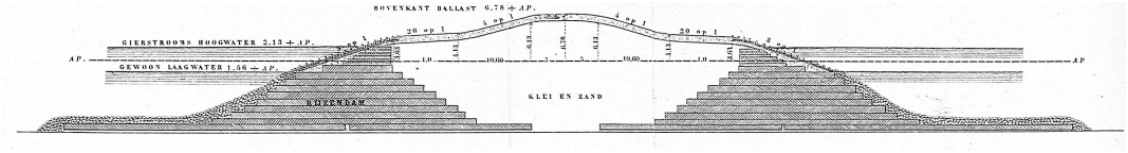


Fig. 8.9 Provisional design of Sloedam made by Staatsspoorwegen in 1864; source: Neyt (1872, Plate 21)

But the design finally adopted and built shows only one fascine dam (Fig. 8.10). Neyt (1872, 218) gives a number of examples of works where two fascine dams were applied or considered[75] and he shows that the volume of fascine works required for making one fascine dam is considerably less than in the case of two fascine dams. The former is cheaper and saves time! Neyt does not discuss the stability of the fascine dam at High tide. A single fascine dam solely comprising fascine mattresses, stone and clay clods may become unstable during High tide. This is due to uplift, seepage and differences in water level between both sides of that dam. But at the Sloe there is tidal movement at both sides of the closure bund and the highest water level difference measured during the period of closure is 104 cm (Table 8.2). So what is possible at the Sloe closure is not necessarily feasible elsewhere in Zeeland with its high tidal ranges when a deep dike breach has to be repaired or a tidal creek in a reclamation scheme has to be closed.

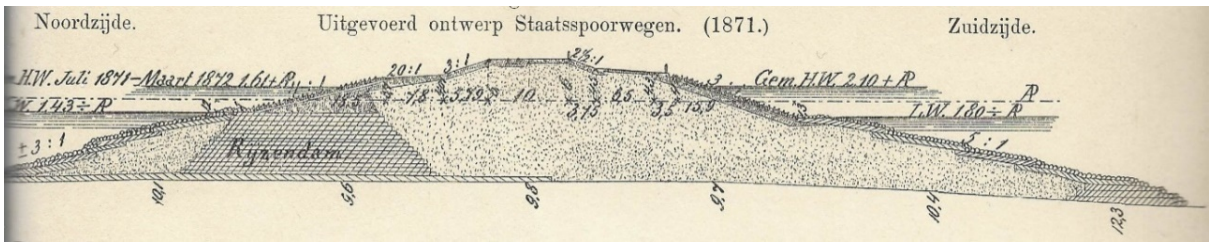


Fig. 8.10 Design of Sloedam as constructed in 1871; source: Plasschaert (1898, Atlas, Plate 30, Fig. 212)

[74] For a definition of the Dutch Ordnance Datum AP (now NAP) see for instance: https://www.kwaad.net/Dutch_Ordnance_Datum.html (accessed on 1 Nov. 2023).

[75] Mentioned are: Schraver (1810, for the closure of the Sloe), Caland (1833, 93 et seq.), Kreekrakdam (1867), abutments for the railway bridge at Moerdijk, Beijerinck (design for a dike enclosing the Zuiderzee).

It is decided to dam the Sloe where it is at its narrowest (Fig. 8.11). This also allows the shortest distance to be had for the railway line between Middelburg, Arnemuiden and Goes.

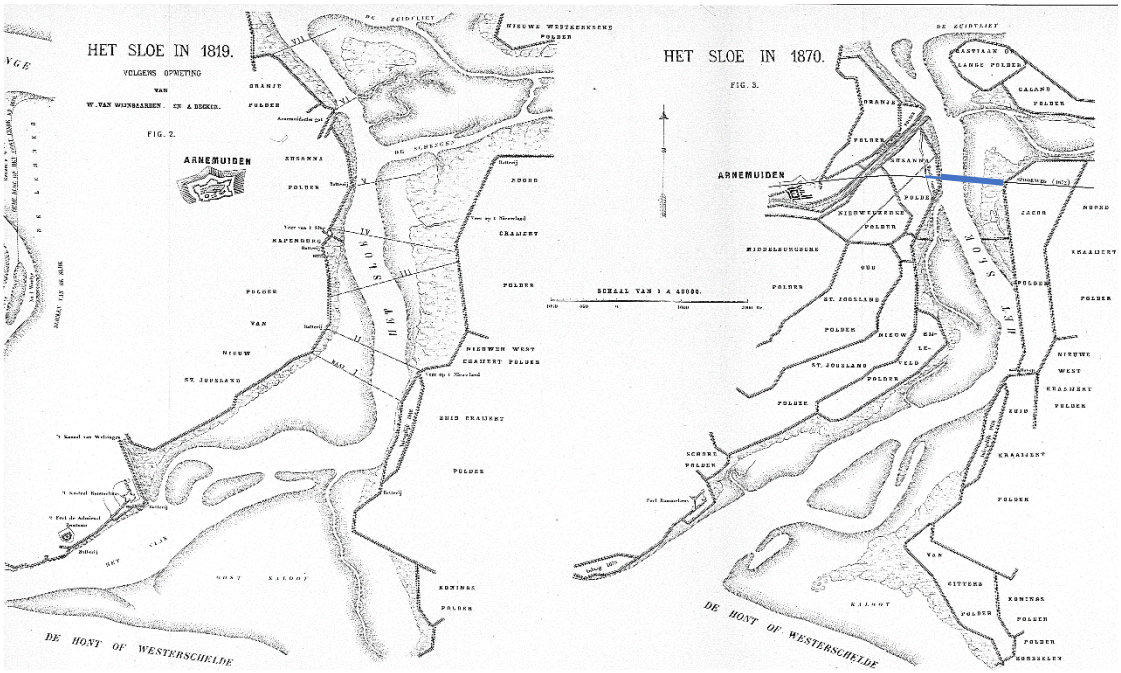


Fig. 8.11 The Sloe tidal channel in 1819 and in 1870; the Sloedam is marked by a blue line; source: Neyt (187, Plate 20, Fig. 2 and 3)

The Sloedam will have an overall length of 1 000 m. Over a length of 405 m (east side) the ground level is above the tidal zone (salt marshes, *schor*). A section of 230 m (also east side) is on the shore between High and Low tide. The actual navigation channel below Low tide has a width of 365 m.
The subsoil in the axis of closure consists of loosely packed sands at the Zuid-Beveland side down to the deep centre of the navigation channel (AP – 10 m). From there in the direction of the island of Walcheren the subsoil consists of clay and slightly silty clay (*spier*).

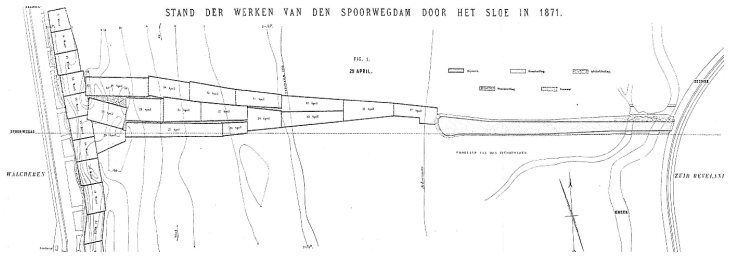


Fig. 8.12 Overview of slope and bed protection mattresses placed for construction of Sloedam; source: Neyt (1872, Plate 23, Fig. 2)

The hydraulic engineering works start on 7 March 1871 by protecting the steep foreshore at the western (Walcheren) side over a length of 375 m by placing toe protection mattresses (*kraagstukken*) down to the deepest point of the navigation channel. This work is complete on 23 March. Because of expropriation difficulties on the east side the work comes to a standstill until 11 April.
The bed protection works then start in easterly direction in the axis of closure from the edge of the mattresses already placed (west side) over a width (in direction of the current) of 25 m. A second and a third strip of mattresses follow such, that the bed protection gets a width of 82 m in the centre of the navigation channel and of 18 m at the toe of the earth dike, meanwhile built on the eastern salt marsh (Fig. 8.12). On 30 April all 16 bed protection mattresses have been placed but not without difficulties.

First of all, one should realize that for towing and ballasting the mattresses only rowing boats and sailing-vessels are available[76]. One of the fascine mattresses, during the rising tide, gets afloat and then breaks adrift in the current and runs aground some 3 000 m to the North on the foreshore. The last and largest fascine mattress to be sunk (20 m by 151 m) during ballasting partly turns upside down in the current[77].
In 1871 the flow profile of the Sloe in the axis of the railway line has an area of 2 310 m^2 below AP. During a springtide ± 36.5 million m^3 flows from Westerschelde to the North (Veergat and Zuidvliet) while ± 9.5 million m^3 flows in opposite direction. There is at that time only one tidal divide in the Sloe: at 1 200 to 1 800 m south of the actual dam axis[78].
Mean high tide in the undisturbed situation is AP + 1.79 m and mean low tide AP – 1.80 m. During closure operations the high tide goes up to AP + 1.92 m.
The construction of the fascine dam on the bed protection starts at the beginning of May and on 14 June the crest of the fascine dam is everywhere above Low tide. This fascine dam has a base, wide 30 m, while its crest is approximately 20 m wide.
This is basically a vertical closure. The deepest point on the bed protection was AP – 9.6 m while the crest of the fascine dam is now at AP – 1 m. In fact the actual closure works start now by constructing a pakwerk (i.e. a fascine bund, comprising fascines, clay clods and stone) up to the level of a summer storm surge (AP + 3.4 m). This is a gradual horizontal closure and it is the most risky and difficult operation in the construction of the dam as the head over the remaining closure gap (Table 8.2) goes up from 27 (15 June) to 104 cm (5 July)! The flow profile during High tide then has a width, perpendicular to the current, of 140 m.
Neyt only states that the bund functions as a weir. There is nothing in his detailed publication (33 p.) about weir formulae, clear/submerged weir stages, sub- critical and super-critical flow or a hydraulic jump (*watersprong*). But, in his publication there are 10 pages of tables about water levels and current velocities (both apparently measured, not calculated), so it is possible to draw some conclusions.

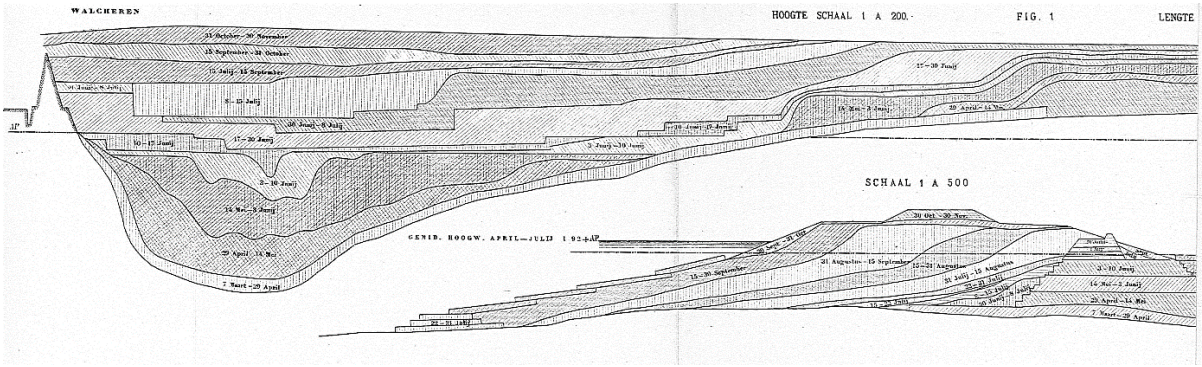


Fig. 8.13 Build-up of Sloedam; on top, longitudinal section showing various stages of the closure; below, the construction stages in cross section; the initial pakwerk has been clearly shown; source: Neyt (1872, Plate 22, Fig.1)

[76] Stuip (ed.) (1984, 11).
[77] Because of this accident Neyt recommends, depending on the width of a mattress, a maximum proportion between length and width of a fascine mattress. For example a 10 m wide mattress should be no longer than 60 m and a 30 m wide mattress should be maximum 60 m long (should probably read 160 m, note Van Duivendijk). In this respect it is remarkable that, 80 years later, during the reclamation of the Zuid-Sloe polder in 1949- 1950 (Verhoeven, 1951, 579 et seq. and Huitema, 1949a, 80 et seq.) again long (63 m) and narrow (3.2/4.8 m) fascine mattresses are applied and also here mishaps occur.
[78] The dam, designed by Schraver, was projected 2 500 m south of the railway embankment (Van de Linde, 1938, 57).

Table 8.2 Water level differences between both sides of the Sloedam closure gap during various stages of closure; source: Neyt (1872, 222)

dates in 1871	largest difference in water level between the two sides of the damming works		current velocity per second		notes
	higher at north side during ebb or pre-flood from the Veergat (cm)	higher at south side, during flood (cm)	during ebb or pre-flood in direction north to south (m)	during flood in direction south to north (m)	
19 April	-	6	-	-	On 19 April no set-up of the water level at the side of the island of Walcheren could be observed; the largest difference in water level, during the damming of the Oosterschelde, was only 35 cm, higher at south side than at north side
6 May	-	7	-	1.8	
14 ,,	6	8	1.33	-	
20 ,,	20	9	2.00	1.96	
30 ,,	30	-	3.00	-	
2 June	50	14	3.00	2.00	
9 ,,	55	20	-	-	
15 ,,	-	27	-	-	
19 ,,	-	40	-	3.18	
25 ,,	-	50	-	-	
30 ,,	-	80	-	3.33	
5 July	-	104	-	-	
8 ,,	-	82	-	-	
11 ,,	-	61	-	-	
13 ,,	-	62	-	-	

There is also a sketch, copied in Fig. 8.14, showing the flow (with hydraulic jump) over the fascine bund on 5 July which, according to Table 8.2, is the day having the maximum head over the pakwerk. On that day current velocities have not been measured but, assuming the horizontal dimensions on Fig. 8.14 are correct and the whole drawing is drawn to scale, the upstream water level must have been H = 2.1 m above the crest of the dam and the water level on the crest 2/3 H = 1.4 m. This implies that current velocity = $(1/3 \times H \times 2 \times 9.81)^{½}$ = 3.7 m/s (ignoring a possible discharge coeff. which is ≠ 1.0).

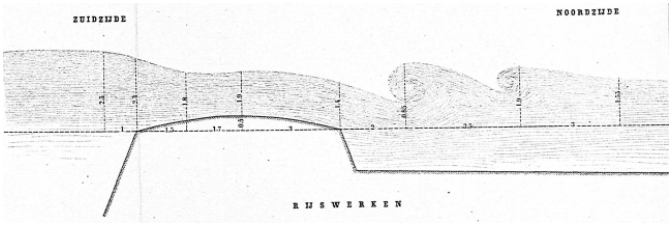


Fig. 8.14
Flow over free weir formed by non-completed pakwerk in closure gap of Sloedam on 5 July 1871 during max. head over weir;
source: Neyt (1872, Plate 22, Fig. 3)

The max current velocity measured on 30 June is 3.33 m/s. In Table 8.3 part of the Table about observations of that day has been copied.
Subsequently, the current velocities have been calculated as follow from differences in water level between the upstream and downstream side and (in the case of a free weir) as follow from the height of the nappe over the crest of the fascine bund. As could be expected the discharge coefficient μ varies.
In the beginning of July, when the head over the non-completed dam is appr. 100 cm or more, rocks (40 to 60 kg) on the pakwerk start to move and disappear in the current, unless they have been fixed between stakes driven into the pakwerk (Neyt, p. 223).

Table 8.3 Observations made on 30 June 1871 during closure operations of the Sloe, columns [1] – [7]; lowest level in remaining gap to be closed is AP; source Neyt (1872, 236)

column [1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]
time of observation (a.m.)	in m relative to AP		cm head of		current velocity in m/s		1/3 x figures in [3] (m) = 1/3 H	free weir u = (2/3 x gH)^{½}	disch. coeff. μ = [7] / [9]	submerged weir u = {2 g x [5]}^{½}	disch. coeff. μ = [7] / [11]
	north gauge	south gauge	north to south	south to north	north to south	south to north					
10.25	+ 0.98	+ 1.71		73		2.86	0.57	3.34	0.85		
10.30	1.08	1.88		80		3.33	0.63	3.49	0.96		
10.40	1.22	1.96		74		3.33	0.65	3.57	0.94		
10.50	1.32	2.03		71		3.33	0.68	3.65	0.92		
11.00	1.48	2.09		61		3.33	0.70			3.45	0.96
11.10	1.57	2.07		50		3.33	0.69			3.13	1.06
11.20	1.68	2.00		32		2.50	0.67			2.5	1
11.30	1.72	1.94		22		2.50	0.65			2.07	1.21
11.40	1.73	1.89		16		2.00	0.63			1.78	1.12

According to the tables in Neyt (p. 236-237) presenting water levels on 3 to 6 July incl., the maximum differences in water level between both sides of the dam vary between 97 and 104 cm. In all cases a free weir is formed and the maximum current velocity u_b = 4.04 m/s.
A (low) value for the density (ρ_r) of the rock is assumed: 2 200 kg/m^3 and the mass M of a stone at 40 kg.
Now the velocity -type stability criterion of Isbash[79] is applied:

When the ratio $u_b^2/(2g \times \Delta \times D) > 0.7$ the stones are unstable in the current:

$Dn = (M/\rho_r)^{0.33} = (40/2200)^{0.33} = 0.26$ m; $D = Dn/0.84 = 0.31$ m

$\Delta = (\rho_r/\rho_w) - 1 = (2200/1016) - 1 = 1.1654 \rightarrow \Delta \times D = 0.3613$ m

$u_b^2/2g = (4.04)^2/19.62 = 0.8308$ m

Isbash: 0.8308 / 0.3613 = 2.3, so the stones are not stable in the current
If a less pessimistic assumption is made: ρ_r = 2700 kg/m^3 and M = 60 kg the outcome is 1.46 which means that also these stones are unstable.
On 12 July the closure bund is everywhere above mean high tide (Figures 8.15 and 8.16). The dam is complete that same year (21 Dec, 1871).
Neyt (p. 224 – 230) also gives details about the calculation of unit rates for earthworks and fascine works. A summary is presented in Table 8.4. For overall cost of the dam see Table 8.7 in Section 8.10.

Table 8.4 Unit rates for construction of the Sloedam in 1871; source Neyt (1872, 224 – 230)

material	equipment	activities	quantity	unit	unit rate	
					f of 1872	€ of 2020
sand	bucket dredgers	dredging, transport, dumping	155 100	m^3	0.33	3.69
sand	wheel barrows	loading, transport, making level	105 000	m^3	0.39	4.36
sand clay clay	ships	digging, loading, transport, dumping; note distinction between top layer of clay clods and lower layer	44 000 49 800	m^3	0.40 0.65 0.90	4.47 7.26 10.06
clayey sand	side tip wagons & locomotives	digging, loading, transport, dumping	208 000	m^3	0.59	6.59
fascines, reed, stone, clay clods		fascine dam fascine bund (pakwerk)		m^3 m^3	4.80 2.85	53.64 31.85
sods	ships	digging, loading, transport, placing		m^2	0.07[5]	0.84

Neyt (p. 224) is of the opinion that the speedy and successful realization of the project is due to the combination of (a) ideal circumstances regarding earth hauling, (b) diligence of the contractor, (c) good weather conditions and…(d) good fortune.[80]

[79] For the symbols used see the List of Symbols.

[80] It is assumed that nobody had expected a head over the closure bund as high as one m. The head at the closure of the dam through the Oosterschelde has not been more than 35 cm! May be this was also expected here.

One could also say: the client and the contractor have had a lot of luck. The project could easily have ended in a great disaster. On 22 July (ten days after closure) the difference in water level between both sides of the dam is 1.29 m!

Fig. 8.15 The completed fascine bund (pakwerk) on the Sloedam during high tide (July 1871); source: Zeeuws Archief, Zeeuws Genootschap, Zelandia Illustrata, deel III, nr 326-33

Fig. 8.16 The completed fascine bund (pakwerk) on the Sloedam during low tide (July 1871); source: Zeeuws Archief, Zeeuws Genootschap, Zelandia Illustrata, deel III, nr 326-35

8.5 IJdam near Schellingwoude[81], closing off the tidal inlet IJ from the Zuiderzee[82] (project no. 31)

The IJdam has been constructed to enable a constant water level in the Noordzeekanaal. The concessionaire, the Amsterdamsche Kanaal Maatschappij awarded the contract for construction of the Noordzeekanaal to an English contractor, Henri Lee & Sons, which in turn subcontracts the construction of the IJdam to the Dutch contractor J.C. van Hattum from Sliedrecht. Initially, a 3 km long dam is planned but in November 1865 the Netherlands' Government approves a much shorter alignment: the IJ will now be dammed west of Durgerdam. The dam will have a length of 1 360 m between the Waterlandsche Zeedijk and the Paardenhoek, a point at the corner of the so-called Oostelijke Afsluitdijk, constructed in 1828-1832 in order to create the Oosterdok (Fig. 8.18).

In Figure 8.17 two cross sections of the dam are shown. The ordinary section shows two fascine dams while in the section of closing one fascine dam is shown. The tidal closure as such, in view of the small tidal range, must have been easy. Note the 'Assumed settlement' (4 m) in relation to the original ground level (OGL), ('surface of bottom').

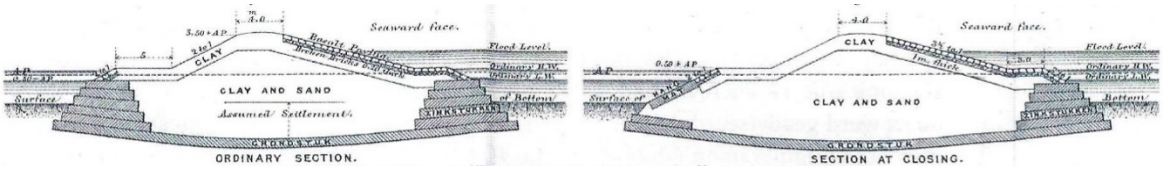


Fig. 8.17 Cross-sections of the IJdam near Schellingwoude; source Watson (1875, Plate 11, Fig. 1 & 2)

Construction starts in 1866 by placing fascine mattresses (so-called *grondstukken*), thick 75 cm, over the full width and length of the base of the dam. Apart from providing protection against scour, the mattresses are also required to strengthen the weak foundation of the dam. Still, much subsidence takes place and the profile of the dam has to be reconstructed and modified repeatedly during its construction and after its completion. Ultimately, the volume of dike fill placed is 2.5 as much as originally shown in the cross section. Construction is also considerably delayed because of various problems cropping up during construction of the navigation locks (the Oranjesluizen).
According to Watson (1875, 158-170) the mean tidal range is 14 inches (36 cm)[83], but during storm surges the water level may rise 3 to 4.5 m. Watson mentions a final closure gap of 328 yards (300 m) but elsewhere (Dirks, 1871) it is stated that, prior to the completion of the locks a 25 m wide gap is maintained in the dam to facilitate inland shipping. It is assumed that Watson means the 300 m wide navigation channel (Fig. 8.18) and that the 25 m wide gap is the actual final closure gap having a bottom width of 25 m.

This gap is closed in June 1872 but because of on-going subsidence the dam nearly breaches by the end of September during a storm surge. The low closure bund starts to overtop at a level in the Zuiderzee of AP + 0.78 m. On 1 May 1873 the dam, over a length of 130 m is still not complete. After new subsidence in the autumn of the same year and reconstruction by adding berms at both sides of the dam, it is finally considered to be complete in March 1874.

[81] The text has been drafted using information found in the sources listed in Table 8.1, no. 31, column 10 but in this connection Bakker (2020, 68), is quoted who, in turn, cites Kingma (2000, 4): "About the technical side of the development of the (Noordzee-) Kanaal little has been written."
[82] The latter is now called IJssel Lake.
[83] Storm Buysing (1854, 581) lists 34 cm (MHT = AP + 0.07 m, MLT = AP – 0.27 m) as mean tidal range for Amsterdam.

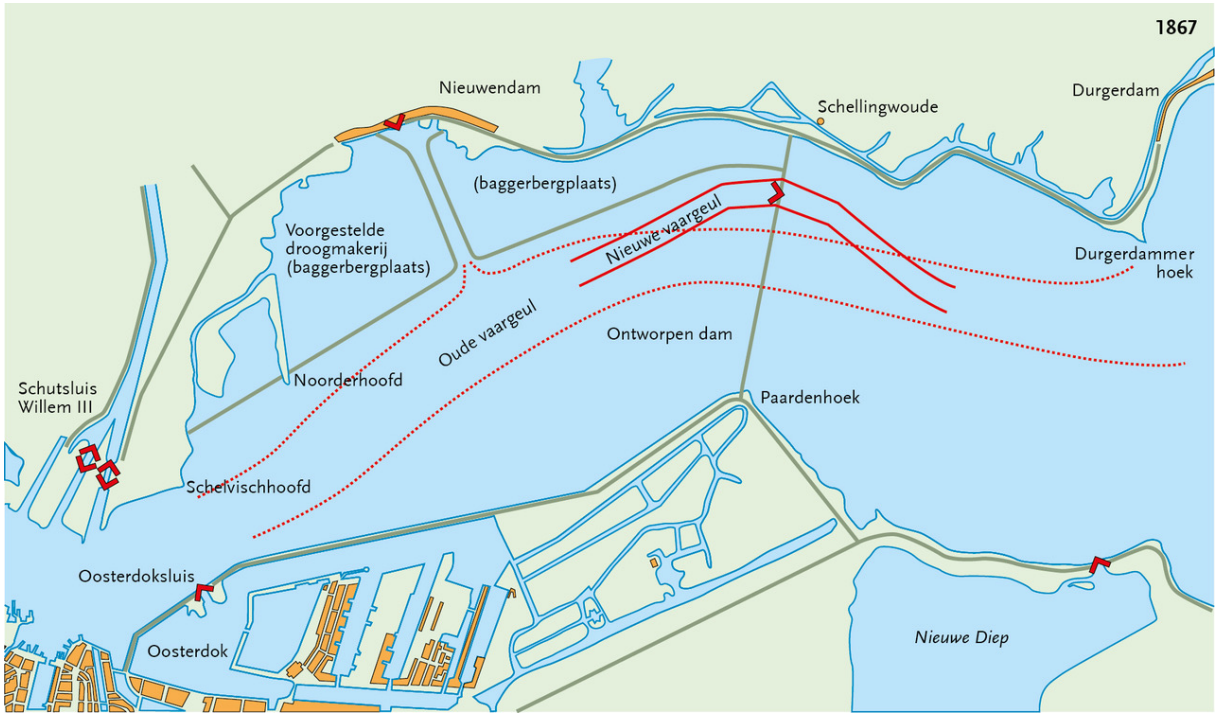


Fig. 8.17 Alignment of the IJdam near Schellingwoude; source: Van de Ven (2008, 18)

Fig. 8.18 The IJdam under construction in August 1872, as painted by Johan Conrad Greive (1837 – 1891); the ships in the background are passing the Oranjesluizen; note the low closure bund made in June 1872;
source: Beeldbank Stadsarchief Amsterdam, 010097015339.

8.6 The Dam between the island Ameland and the Frisian coast[84] (project no. 32)

In 1845 the governor of the province of Friesland, M.P.D. baron Van Sijtzama, asks ir. B.P.G. van Diggelen and W. van Peijma to give their opinion about a dam between Ameland and the Frisian coast to further accretion followed by reclamation. Van Diggelen, at that time, is working on a plan for reclamation of the Zuiderzee and parts of the Waddenzee. This plan is presented in 1849[85].
Van Peijma, who is well known in Friesland because of his book about the Frisian dikes[86], subsequently writes two long letters to Van Sytzama. In March 1846 the latter asks Van Diggelen and Van Peijma to collaborate as much as possible while Van Peijma is also asked to have a more direct involvement in the studies. Van Peijma, in his report[87] dated 23 September 1843, remarks that, to his satisfaction, Van Digggelen agrees about the need for a dam; but both are of the opinion that each should write his own report[88].
Van Peijma proposes to build a dam having, on average, a height of 4.5 m (Fig. 8.20) in the zone of the tidal divide (Fig. 8.22). West of the dam, at a distance of 50 – 60 m and parallel to the dam, a bund (*hoofd*) will be required. Every 100 m this bund will be connected to the main dam by a cross-bund. This system, in his opinion, will further the accretion in front of the dam.

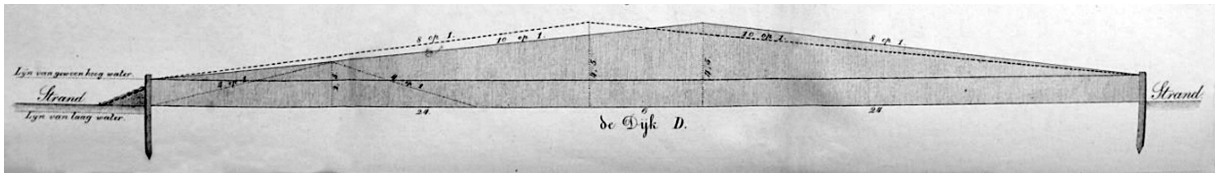


Fig. 8.20 Cross-section of the dam as proposed by Peijma (1850)

But, if this is considered to be too costly, he suggests to only build two bunds, parallel to each other, having a distance to each other of 300 m and cross-bunds every 500 m. See also Fig 8.21.

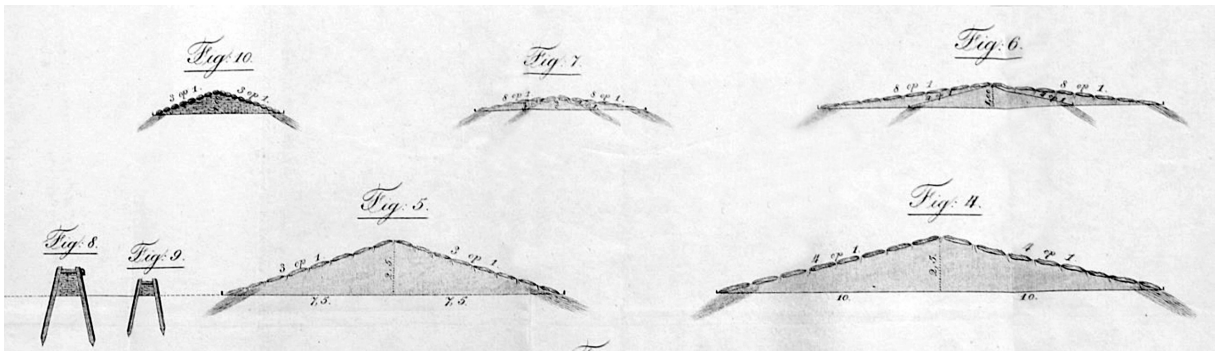


Fig. 8.21 Various types of bunds; in this respect Van Peijma refers to Caland (1833,273) and to Storm Buysing (1854)

[84] Apart from the references given in Table 8.1, various documents have been consulted in the National Archives and the Frisian Archives (*Tresoar)*.
[85] '*De Zuiderzee, De Friesche Wadden, en de Lauwerzee, hare bedijking en droogmaking*', beschouwd door B.P.G. van Diggelen, ingenieur van 's Rijks Waterstaat (1849).
[86] Van Peyma, W. (1827); '*Verhandeling over de beste wijze voor aanleggen van zeedijken, en de hervorming derzelve'.*
[87] The above mentioned letters and the report have been published in 1850 under the title '*Verslag over de aansluiting van Ameland aan den Vrieschen wal en de opslijking van het wad'.*
[88] "*Wat voor het overige onze meer bepaalde beschouwingen aangaat, zoo is het ons voorgekomen, dat wij den besten weg inslaan, met deze te geven, elk voor zich en alzoo in afzondelijke rapporten, zoo als eigene opmerking dezelve verschaffen konde – algemeene opmetingen daarvan uitgezonderd – zonder wijziging en schikking van denkbeelden naar elkanders, mogelijk niet altijd overeenkomende, wijze van zien, op deze en gene punten."* (Van Peijma, 1850, 25-26).

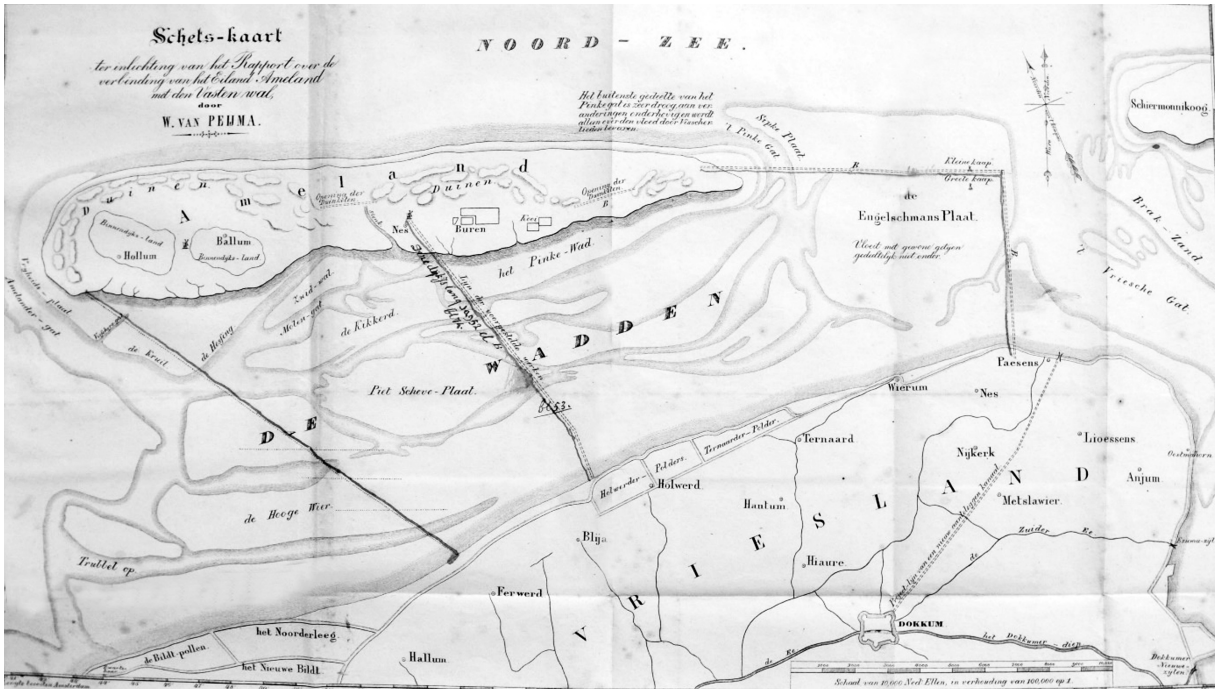


Fig. 8.22 Proposed alignments of dam and dikes of the future polder; source: Van Peijma (1850)

After the publication of the books by Van Diggelen (1849) and Peijma (1850) it takes another 10 years before action is taken. In September 1860, Jhr. mr. P.J.E. Teding van Berkhout (hereafter called TvB) requests the central government for a concession to reclaim the so-called *Friesche Wad* between Ameland and the Frisian coast. In October of the same year he provides details of his request[89]. He follows more or less the ideas of Peijma (1850). A first connection between Ameland and the Frisian coast will be made on an alignment following the line of the tidal divide. Initially, this will be a low, 8.4 km long dam which, gradually, depending on the rate of accretion, will be widened and heightened (in layers of 30 cm) until the crest has its level at MHT. When sufficient accretion has taken place, dikes will be built to a total length of 23 km at west and east side (Fig. 8.23) of the dam. These dikes will enclose an area of 16 000 ha. After subtraction of 2 000 ha for roads, ditches and remaining gullies, 14 000 ha will be available for agriculture.
The following details are given of tide, ground level and dam to be built: MLT = MHT (*volzee)* – 1.80 m. Creeks, crossing the alignment, have a maximum depth[90] at MLT – 0.60 m. The average ground level of the *wad* in the alignment of the dam is MHT – 1.25 m. Fascine dams, wide 10 m and having a height of 1.15 m (i.e. up to the local level of the *wad)* , will be made through the creeks. The dam having a width at base of 10 m will have a width at crest level of 6 m.
TvB estimates the overall cost of construction of the dam at f 1.521 million. Also estimates are given for the two dikes (total f 1.694 million), the works required for parcelling of the polder, water storage and drainage (total f 2.211 million), roads, culverts and bridges (f 90 000) and, finally, 5% for administration and 3% for contingencies. This brings the overall cost (excluding interest during construction) at f 5.96 million. The overall construction period could be 20 years or less.
During the following years there is much discussion about the desired profile of the dam and about a possible financial contribution by the central government and by the province of Friesland.

[89] Teding van Berkhout (1860).
[90] Actually the text reads: 'two Amsterdam feet' which is 0.5662 m.

Fig. 8.23 Alignment of dam, of eastern dike and of alternative alignments western dike; source: Teding van Berkhout (1867)

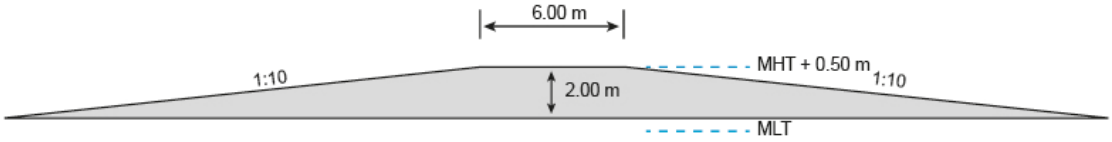


Fig. 8.24 Cross-section of dam as laid down in Royal Decree (16 October 1869, no. 17); slopes and crest are protected by fascines

This results finally in a Royal Decree (16 October 1869, no. 17) which, inter alia, fixes the cross section of the dam as shown in Fig. 8.24. Subsequently, TvB transfers the concession to a company (hereafter called the Company) founded by him for reclamation of the Frisian mud flats[91]. The Company at start has a capital of f 300 000, later on increased to f 600 000, while the Company will receive a subsidy of f 200 000 from the central government and the same amount from the province of Friesland.

However, it is felt by technical staff in the central government as well as in provincial technical circles that the dam would be too high to further accretion and too low to function as a non-overflow dam. It results in a new Royal Decree (2 October 1870, no. 12) lowering the height by one metre to MHT - 0.50 m and fixing the width of the crest to 34 m while at both sides of the crest slopes would be at 1: 14. The principle is similar to that shown in Fig. 8.25. Only the dimensions and levels are different.

Works then start in May 1871. The idea is that rows of wattle work (*tuinen*) lengthwise on the crest of the 34 m wide crest (crest is initially much more narrow) will stimulate accretion. In June 1872 a connection between Ameland and the coast at Holwerd is established but this dam is still far from complete.

[91] *N.V. Maatschappij tot landaanwinning op de friesche wadden*.

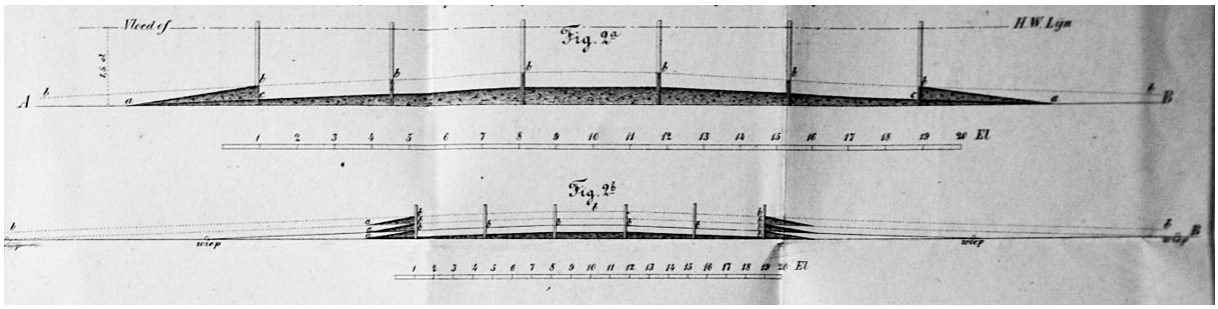


Fig. 8.25 A possible cross-section of the dam when silting up in layers of 30 cm is left to nature; source: Teding van Berkhout (1867)

The results of the accretion, however, are disappointing and, on the basis of a pilot scheme of Rijkswaterstaat, a new Royal Decree (29 December 1874, no. 7) is issued prescribing a minimum profile: a 0.60 m wide crest, slopes of 1:3 or more gentle, the elevation of the crest to remain the same: MHT – 0.50 m. Now, slopes and crest have to be protected by fascines, ballasted by stones. In the same Decree the concessionaire is given permission to increase the height of the dam if the Company considers this to be desirable. Because of this new Decree the Frisian government decides to suspend for the time being payments from the subsidy to the Company.
In October 1878, the dam has the desired profile (or higher) over its full length. The Company now starts to widen and heighten the dam. In September 1881 the crest of the dam is everywhere at MHT or higher (Fig. 8.26).
During a heavy storm surge on 14/15 October 1881, at 3.2 km distance from Ameland a breach is formed, long 75 m, which, after the storm surge of 29/30 April 1882, lengthens to 156 m. The same storm leads to two new breaches (long 62 and 45 m). These two breaches are repaired during the summer of 1882. In the breach, first mentioned, a fascine dam is placed up to the level of MLT.
Then a third storm surge (25/25 October 1882) occurs. It causes two new breaches (long 105 and 115 m) while the fascine dam in the old breach of 1881 is destroyed (Figures 8.27 and 8.28).
In 1887 these three breaches have still not been repaired as the Company has spent all its funds.
The investigations in 1883 by experts from the central government reveal that the breaches are mainly due to the absence of an apron (*plasberm*)[92] at the east side of the dam. During storm surges from western directions, water set-up leads to overtopping of the dam while at the east side the *wad* runs dry because of the same storm surges. This leads to a plunging jet falling on a non-protected sea-bed resulting in erosion. This explains also the gully along the east side of the dam (some cross-sections in Fig.8.26 show this gully).
The dam crest is at MHT + 0. 25 m at both ends of the Dam, decreasing to MHT in the middle. The stone protection of the dam is sometimes meagre.

Table 8.5 Particulars of the breaches (surveyed in 1883); source: NL-HaNA_2.16.07_ 142

location of breach	distance from centre of breach to Ameland	length of breach at low tide	depth below low tide			notes
			in the axis of the dam		at the east side of the breach	
			on average	largest		
northern breach	3 320 m	75 m	3.6 m	6.0 m	18.6 m	tides are most probably as follows: MHT = AP +1.04 m; MLT = AP -0.92 m
middle breach	5 855 m	105 m	5.3 m	6.2 m	5.9 m	
southern breach	6 340 m	115 m	6.3 m	8.3 m	9.7 m	

[92] The word *plasberm* is used in the reports but the word *stortebed* should have been used instead.

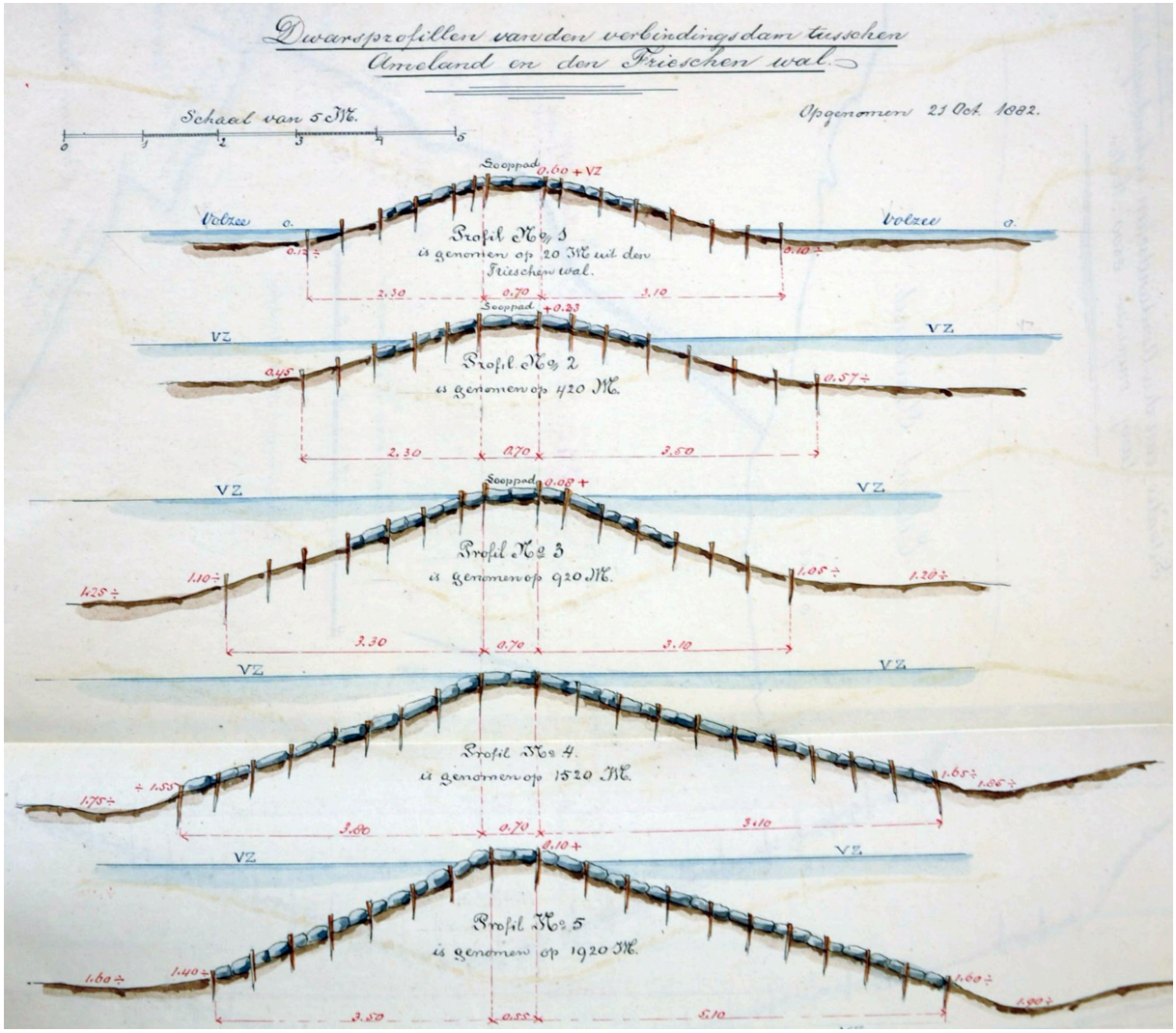


Figure 8.26 As-built cross-sections of the dam in the section Frisian coast – 1 920 m distance from this coast, surveyed in October 1882; source: Rijkswaterstaat (1892)

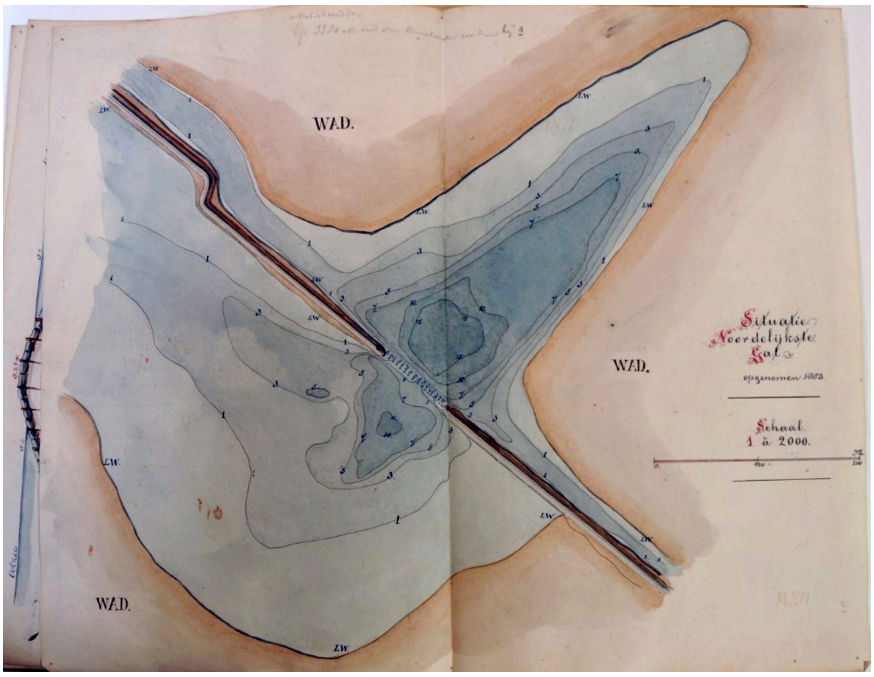
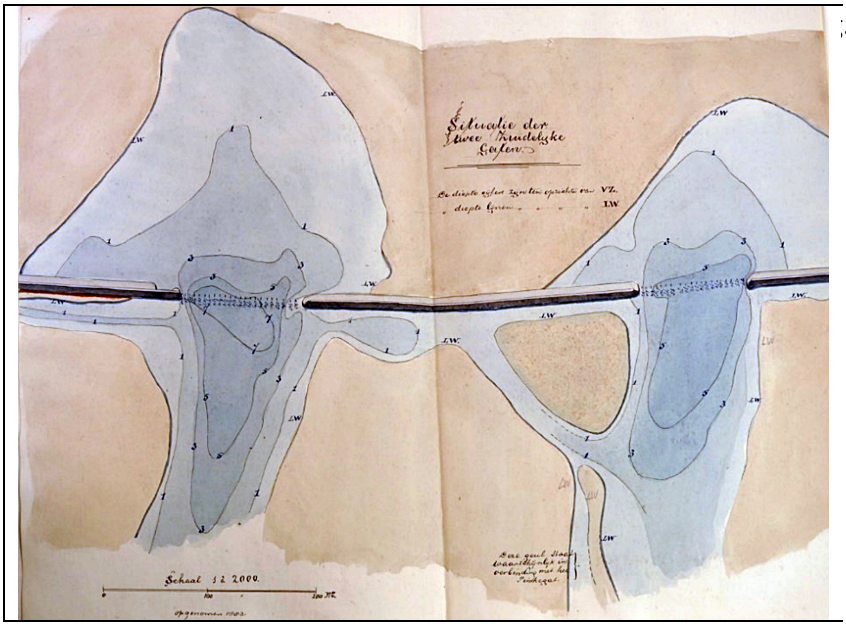


Fig. 8.27 Situation of the northern breach as surveyed in 1883; source: Rijkswaterstaat (1892)

In the design of remedial works as made by the Chief Engineer of Rijkswaterstaat the following is recommended:

- the dam to have as cross-section: height of crest MHT + 0.25 m; width of crest 0.75 m; slopes: east side 1: 4, west side 1: 3;
- an apron (*plasberm*) comprising a fascine mattress (*kraagstuk*) ballasted by stone at the eastern toe of the Dam having a width of 5 to 6 m decreasing to 3 m near the coast; its top should be at MLT + 0.50 m; (Fig. 8.29);
- in order to decrease overtopping of the dam at the most vulnerable spots the crest of the dam should be at MHT + 0.50 m over a length of 400 m where the apron in the deep gully would be replaced by a 15 m wide fascine mattress;
- the northern breach should be repaired (Fig. 8.30); the two other breaches to be left and a new dam section, long 1 350 m, to be made along the perimeter of the scour holes (*wielen*);
- cross-bunds to be constructed, at the east side to further silting up of the gully and at the west side to act as beach-heads;
- cost of construction is estimated at f 960 000.



. 8.28
Situation of middle and southern breaches as surveyed in 1883;

source: Rijkswaterstaat (1892)

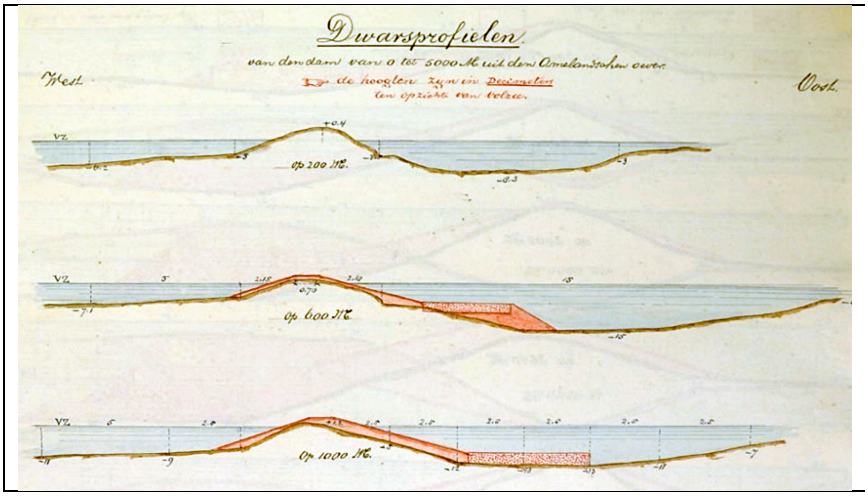


Fig. 8.29
Proposed remedial works (in 1884) at various distances from Ameland;

source: Rijkswaterstaat (1892)

Subsequent discussions with the director of the Company (i.e. TvB) result in some changes in the design and cost estimate which brings the estimated cost down to f 730 000. The remedial works would take three years to construct (1885 – 1887).

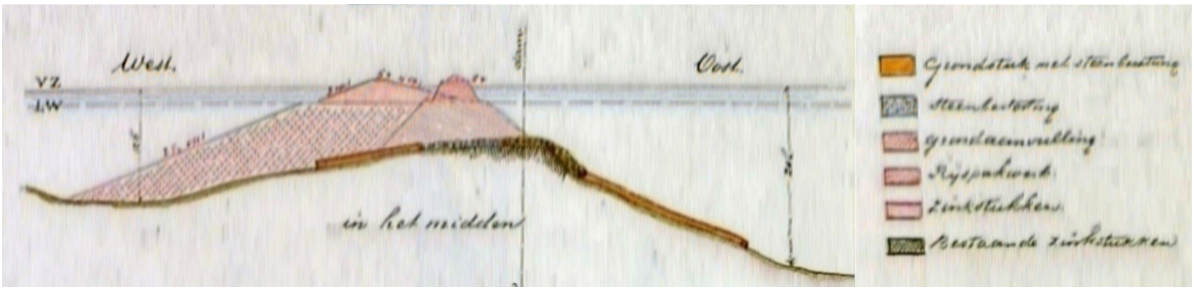


Fig. 8.30 Proposal for closure of the northern breach; source: Rijkswaterstaat (1892)

After various discussions about these works and about the transfer of management and maintenance of the dam to the central government, an agreement (dated 25 June 1884) between the central government and the Company is signed. In the same year a bill is presented to the House of Representatives to ratify the agreement. But as support of this bill is lacking, it is withdrawn.
A new agreement is now negotiated between the parties and it is signed on 7 December 1886. A new bill is presented to the House of Representatives but also this bill cannot meet approval of the Representatives and it is rejected on 10 June 1887 (30 to 20 votes).
The surveys of 1887 show some changes in the breaches but outside the zone of the breaches regular maintenance has been carried out. Overall, the condition of the dam is found to be more or less the same as in 1883.
In 1888, the Company asks ir. C. Lely for advice. He starts his study by the end of October and submits his report[93] by the end of the year. Lely recommends not to repair the dam. It is not at all sure that it will further accretion. Moreover, silting up of the *wad* may take many years and meanwhile the dam has to be maintained and, repeatedly, damage by storm surges will have to be repaired. Such expenses, without hope for benefits in the near future, are not attractive for a private company. Lely proposes to reclaim straightaway a significant part of the new polder, always provided that first an elaborate programme of soil borings and tests has been carried out which show the reclamation to be worthwhile[94].
In 1889 the Company comes with a proposal based on Lely's report. The Government is requested to pay during six years the interest over a capital of f 12 million (which comes to f 2 880 000) and to give a subsidy of f 75 for each ha reclaimed (based on 11 850 ha, this comes to f 888 750). Accordingly, the Government would invest f 319/ha while the Company expects to sell the reclaimed area for f 900/ha. But the Government only offers a subsidy of f 200 for each ha reclaimed. The negotiations then have to be broken off because the Government falls.
In 1890 the new Government is prepared to maintain the offer of a subsidy of f 200/ha reclaimed land but this subsidy will only be paid after completion of the reclamation and its certification by the Government.
From now on the Company is only able to carry out some maintenance on the still existing dam sections because its funds have been exhausted.
In 1893 (and again in 1895 and 1898) one of the water boards along the coast of Friesland asks the Minister of Water affairs, Trade and Industry (Minister of W, T and I) to revoke the concession but the Chief Engineer for Hydraulic Works in the province of Friesland still believes that the Company will act and is able to stall a decision.
In 1999 the Company states its intention to carry out some works (including reclamation of 500 ha) which would cost f 410 000 and requests a subsidy of f 136 666. Accordingly, the Minister of W,T and I requests the provincial government (*Gedeputeerde Staten*) for advice (*bericht en raad*).

[93] Lely (1888).
[94] This comprehensive report is ready in February 1890 and shows satisfactory results.

Though the Chief Engineer, mentioned earlier, still has a positive opinion about the Company, the provincial government considers it time to take action and rejects the plans of the Company in the strongest terms[95] [96]. In 1903, the Company (which is in the process of being liquidated) does not repair the damage caused by the storm surge in the beginning of October. The Minister of W, T and I, by letter dated 13 November 1903, then summons the Company to repair the damage within six weeks. In default of action, the concession will be revoked in accordance with Art. 16 of the concession. Subsequently, early 1904, the concession is indeed revoked.

A few remarks should be made about this case history of the dam from Ameland to the Frisian coast. First of all it is surprizing that, apparently, Rijkswaterstaat did not have any idea how a cross-section of an overflow dam furthering accretion should look like. Repeatedly, width and elevation of the crest as well as the slopes are changed by Royal Decree.
Secondly, by specifying the cross-section of the dam, the Government is also to a large extent responsible for the damage to the dam. But it did not draw the consequences of this responsibility, i.e. a financial contribution to repair the damage. While the Company invests f 600 000, the subsidy by the central government is only f 200 000 but the Government insists in having an important say in the matter.
Finally, as explained in Section 8.1, The basic reason for failure of this type of dam was the lack of knowledge at that time about the magnitude of the flow during a storm surge. With that knowledge one would have constructed an apron at the downstream side of such dams. Now, such apron was only proposed in 1884 (Fig. 8.29) though also the Slikdam (Section 8.7) had already breached twice for that reason (in 1861 and 1863).

8.7 The dams across the Slaak, connecting Sint Philipsland to the mainland (project no. 33)

By a deed of conveyance, dated 25 June 1853 (1), the Government gives on a long lease (*erfpacht*) to A. van Haaften, Sliedrecht, an area of water, sand and mud flats, situated between the island Sint Philipsland and various sea dikes bordering the province Noord-Brabant. It is Van Haaften's intention to reclaim, together with landowners in the area, as much as possible of the area covered by the concession.

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[95] "…; *de vraag mag intusschen onzes inziens gesteld of eigenlijk niet tenslotte gesteld of dat alles zich niet laat resumeeren tot het pasklaar maken …ten behoeve van een nieuw doel van de in hoofdzaak waardelooze ruïnes van het oorspronkelijke werk met een vooruitzicht stellen op landaanwinning van zo'n beperkte omvang dat inderdaad de vraag gesteld mag worden of daar tegenover eene subsidiering van rijkswege op zoo groote schaal wel te verantwoorden is. ……..*
Is niet het hele verloop van de handelingen der Maatschappij eene lange lijdensgeschiedenis waarbij men na 29 jaren na steeds geld in de zee te hebben geworpen, tenslotte even verre staat als in het begin. Neen niet even ver want die onzalige stukken dijk ten kwade vers in de zee geworpen maar sedert bijna twintig jaren schromelijk verwaarloosd, hebben juist tot noodzakelijk gevolg dat het rijk vleugellam zou zijn, want nooit (werd) een ernstige poging tot drooglegging der wadden beproefd. De bestaande concessie en de daarmee verbonden werken, hoe totaal nutteloos die voor het ogenblik mogen zijn in de praktijk, zouden dan wellicht nog eene ernstige remschoen blijken; een lastig bezwaar om de uitvoering van het werk eens eindelijk op te dragen aan eenen nieuwen concessionaris van wien althans met grond mocht worden verwacht dat hij minstens beter dan de tegenwoordige berekend zou zijn voor het werk dat hij op zich nam. Intusschen in elk geval, er bestond een middel om af te komen van eenen concessionaris, die niets was dan een sta-in-den-weg. Het middel was zo eenvoudig mogelijk, het bestond in de intrekking eener concessie……..
……
Wij gevoelen niets voor eene verbinding van Ameland met den vastenwal, als hier wordt voorgeslagen, wij gevoelen er niets voor dat in plaats van verdronken land eene verdronken maatschappij boven water worde gehaald. In het kort, wij gevoelen niets voor de opportuniteit van uitgaven waarvan niet (.?..) van haren omvang het openbaar belang wordt bevorderd en wij kunnen dus niet anders dan U voorstellen het gedane verzoek te wijzen te hand." (concept Kabinet, nr 333, gedateerd 9 november 1899, was getekend Gedupeerde Staten).

[96] Note that the quotations in footnote 95 are taken from a draft letter. The letter which has been sent to the Minister may have had a different text.

As part of his obligations Van Haaften must construct and maintain a dam (known as the Slikdam, alignment shown in Fig. 8.32) across the tidal channel Slaak. Purpose of this dam is the permanent link between the provinces Noord-Brabant and Zeeland and it is also supposed to further accretion. By Royal Decree of 14 October 1853, no. 68, Van Haaften obtains permission to construct the dam, provided certain conditions are adhered to. These conditions, laid down in a decree of the Minister of Internal Affairs of 20 October 1853, no. 158, enable Van Haaften, at his request, to apply smaller dimensions to the dam than originally specified by Rijkswaterstaat.
But one should add that Rijkswaterstaat had already fixed the level of the crest of the dam at MHT + 1 m (De Bruin, 1953, 32) and, apparently, this level was not changed.
The dam is constructed in 1858, but according to the report dated March 1859, made by the Chief Engineers of Waterstaat in Zeeland and Noord-Brabant, only a thin layer of clay is supposed to prevent erosion of the bund (*berm),* mainly built up of sand*,* and the permanent connection mentioned above does not seem to be guaranteed. Already during the Winter of 1858/59 it appears that the dam is too weak to allow the passage of carriages as they disturb the cohesiveness of the clay layer and, subsequently, overtopping waves will erode the dam crest.

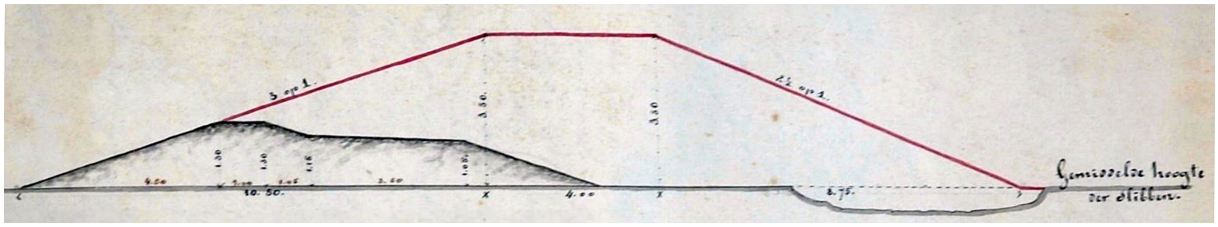

Fig. 8.31 At left cross-section of the Slikdam as constructed; the line in red indicates the heightening of the dam as proposed in 1859; source: NL-MdbZA_6.2_3954

In 1859 it is proposed by the Chief Engineers of Waterstaat in Zeeland and Noord-Brabant to heighten the dam by 2 m (Fig. 8.31) but this heightening never materialized.
In November 1861 the dam breaches at three locations. These breaches are repaired by the concessionaire. In January 1863 the dam breaches at two locations and is heavily damaged at other locations.
After these breaches and the subsequent washing away of most of the dam, several years of negotiations between Van Haaften and the provinces of Zeeland and Noord-Brabant take place to arrive at an agreement regarding the reconstruction of the dam at the cost of the concessionaire. These negotiations are not successful.
De Bruin (1953, 32) states that the dam was not re-opened for traffic as Van Haaften had come to the conclusion that the dam was too low (MHT + 1 m).
In 1869 a committee, having four members, is formed to decide how to continue with the plans for reclamation. The members of the committee recommend to the Government to withdraw the long lease of Van Haaften and get the reclamation done by the State. But the members do not arrive at a joint recommendation on the manner in which the reclamation should be done. The two members representing the islands Sint Philipsland and Schouwen-Duiveland want to have a non-overflow dam, not only because this is in the common interest of the northern islands of Zeeland but also to further a substantial and fertile land reclamation. The Chief Engineers of Waterstaat in Zeeland and Noord-Brabant, however, consider it doubtful whether such a non-overflow dam will further accretion. A high dam, in their opinion will disturb the existing pattern of accretion, as it will change direction and power of tidal flows. The House of Representatives then asks the opinion of T.J. Stieltjes, a well-known expert in the field of hydraulic engineering and the latter declares that a non-overflow dam is

much better for accretion than other works (like a network of bunds and ditches) and that automatically reclamation of a polder would be followed by other polders[97].
It takes a period of ten years before the Government is able (in March 1880) to withdraw the lease of Van Haaften and to come to an agreement with the other landowners.

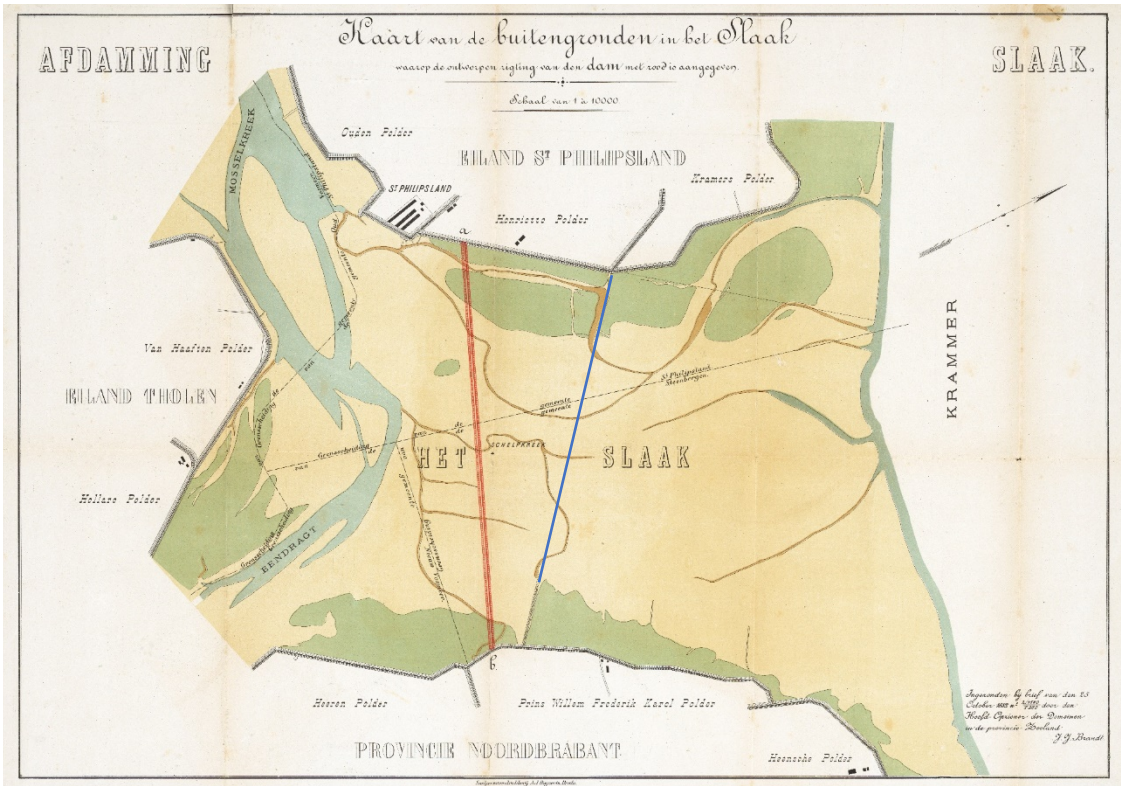

Fig. 8.32 Map of the Slaak showing the alignments of the Slikdam (blue) and of the Slaakdam (brown); source: NL-MdbZA_63_85.1

In December 1883 construction of the Slaakdam is tendered and the Contract Sum is f 334 700. The contract documents, inter alia, stipulate that the contractor has permission to place fascine mattresses in the gullies if he wishes to do so but he will not receive additional payment. In fact the vertical alignment (Fig. 8.33) shows as deepest point AP – 1.01 m, while MHT = AP + 1.45 m and then MLT = AP – 1.85 m[98]. Even if the value of MLT is only AP – 1.40 m the tidal channels will run dry and closure will not have caused problems.

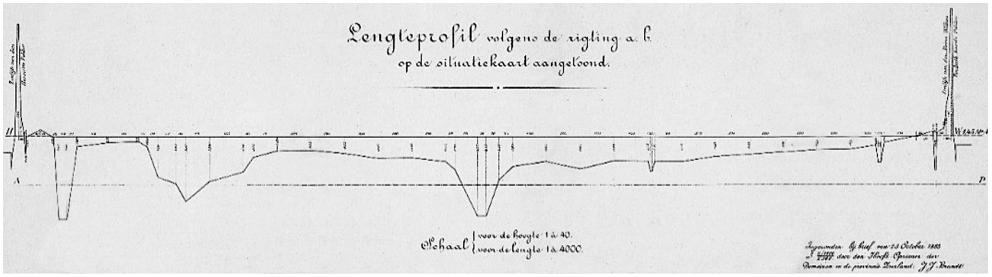

Fig. 8.33 Longitudinal section over the Slaak in alignment of Slaakdam; source: NL-MdbZA_63_85.3

[97] Session of the House of Representatives of 18 December 1872 (Official Parliamentary Reports, 1872 – 1873, pages 848 – 849).
[98] The value of MHT is given on the longitudinal section (Fig. 8.33). For the average tidal range 3.30 m is taken. This is the value at Zijpe opposite St. Philipsland (Storm Buysing, 1854, 579). But this might be too pessimistic.

Thanks to good weather, construction of the dam does not meet any problems. In May 250 labourers are placing earth fill and in June the crest of the dam is everywhere above MHT and the stone pitching on the slopes has started[99].

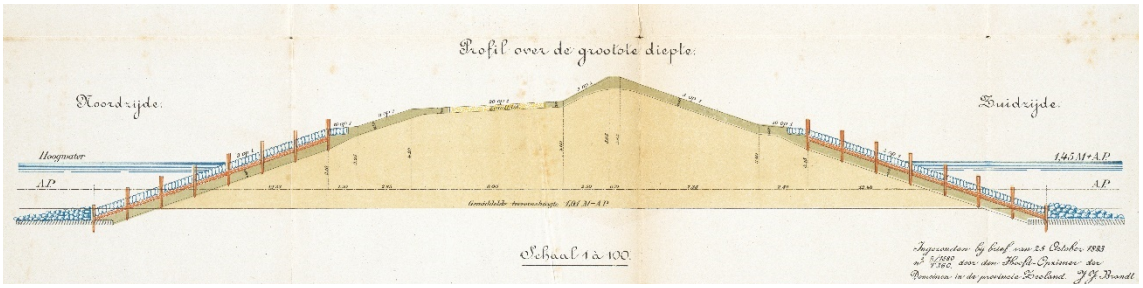

Fig. 8.34 Cross-section of Slaakdam; source: NL-MdbZA_63_85.2

In the contract documents a number of unit rates are given. For the time being these are for reference only. Later on, these will be compared with those presented in Chapter 7 and in Section 8.4 (Table 8.4) of this chapter.

Table 8.6 Unit rates in the contract documents for construction of the Slaakdam

voor een M³ grond ontgraven, opladen en lossen en op 4 meters afstand werpen	*f*	0,10
voor een M³ grond te vervoeren over 40 meters afstand	"	0,08
voor een M³ grond vervoeren over 10 M. verder	"	0,02
voor elken meter klimming wordt de afstand van vervoer met 5 M. verminderd		
voor een M³ kleigrond aan te voeren	"	0,60
voor een M² steenglooijing van Doornikssche-, Lessinesche- of Bazaltsteen dik 25 c. M. met bevloering van puin en krammat.	"	3,20
voor een M² kraagstuk dik 30 c. M.	"	1,30
voor een M² dunne bezoding.	"	0,06
voor een M² winterkrammat	"	0,11
voor een M² zomerkrammat	"	0,07
voor een scheepston Doornikssche- Lessinesche- of Bazalsteen	"	3,60
voor een scheepston afval van Doornikssche-, Lessinesche- of Bazaltsteen	"	3,20
voor een scheepston puin	"	2,00
voor een uur werkens voor een aard- of rijswerker.	"	0,14
voor een uur werkens van een timmerman, smid, metselaar of verwer	"	0,18
voor elken strekkenden meter meerder gemaakten dam, met inbegrip van de steenglooijingen, perkoenrijen en plasbermen dan in het bestek is opgegeven zal worden bijbetaald	"	150,00

After completion of the dam, the area north of the dam starts to silt up rapidly. In 1906 the average elevation is NAP + 1.2 m and the State (i.e. Ministry of Finance, *Directie der Registratie en Domeinen*) decides to reclaim the area (Fig. 8.35). In 1907 approximately 339 ha is reclaimed at a cost of f 267 400.[100]

[99] Algemeen Handelsblad of 7 May and 7 June 1884 respectively.
[100] Source: NL-MdbZA_3456, *Inleiding*.

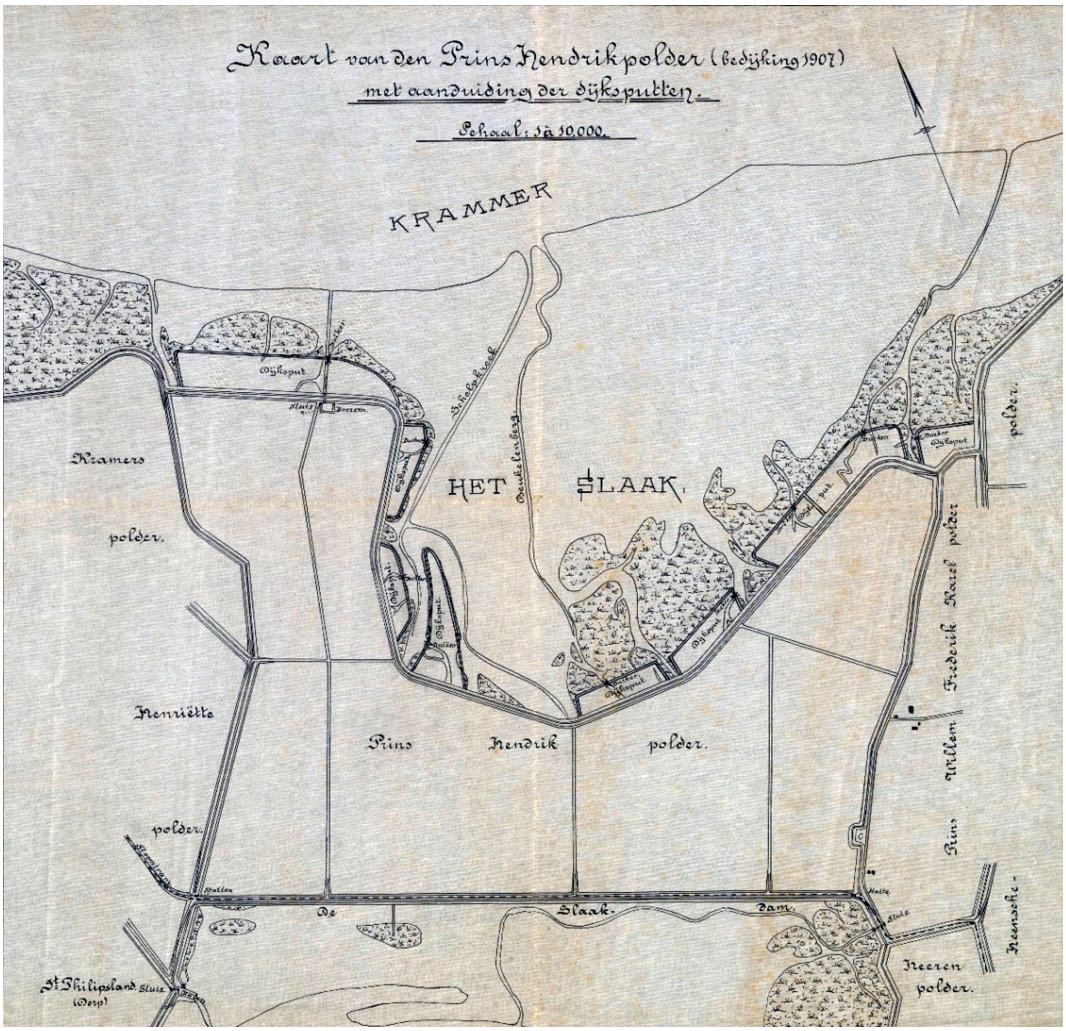

Fig. 8.35 Prins Hendrikpolder north of Slaakdam, the borrow areas (*dijksputten*) for the dike fill (*specie*)have been clearly marked; source NL-MdbZA_612_194

8.8 Nordstranderdamm, connecting Nordstrand to the mainland (Schleswig-Holstein) (project no. 34)

The island Nordstrand is situated on the North sea coast in Schleswig-Holstein near Husum. Throughout its history Dutch diking experts have played a major role in reclamation of its polders and sea defences. In this respect Jan Claesz Rolwagen[101] and his son Claesz Jan as well as the Family Indervelde are mentioned (Baars, 1982a, 39 et seq.).

Usually, a distinction is made between Alt-Nordstrand and Nordstrand. The former is the island which existed until 1634 when, during a storm surge, the island (which still suffered from the damage caused by a storm surge in 1625) is for the major part destroyed. It remains without dikes[102] for a period of 20 years.

Jan Claesz Rolwagen was *dijkgraaf general* of the duchy Holstein (Belonje, 1933, 211) and also adviser to the Duke of Holstein. In 1612, he writes a letter in that capacity to the Duke about the need to strengthen the dikes of Alt-Nordstrand. He presents in that same letter a plan to connect Alt-Nordstrand to the mainland by constructing a dam. The dam will cause accretion at both sides of it, which, ultimately, will lead to reclamation of new polders.

[101] It is said that Jan Claesz got the surname Rolwagen because of his introduction in Schleswig-Holstein of the wheelbarrow for transport of dike fill (Baart de la Faille & Jacoby,1928 , 492).

[102] In the province of Zeeland one used to call an island, being in that situation, *drijvende* (floating).

In 1620 his son, Claes Jansz, together with his brother-in-law Jacob de Mol, writes a report in which the need for construction of the dam in emphasized. But the aforementioned storm surges torpedo the implementation of the plan.
During the years after the disastrous storm surge of 1634, various efforts are made to repair the dike breaches but to no avail[103]. Meanwhile the situation on the island regarding the remaining sea defences gets progressively worse. Then, in 1652, three sons of the dike reeve of Holstein, Francois in der Velden, Quirinus Benedictus (a priest) and Johannes come to Nordstrand and start reclamation[104] of the first '*koog*'[105]. It is reclaimed that same Summer (600 ha).
Though storm surges repeatedly result in dike breaches and flooding, by 1691 some 2 280 ha has been reclaimed. Still, Alt-Nordstrand had an area of 21 500 ha[106]. The process of land loss and land gain continues through the centuries. Nowadays, Nordstrand has an area of 5 743 ha and the nearby island Pellworm (which was part of the island Alt-Nordstrand) of 3 744 ha.
In 1904 it is suggested to connect Nordstrand to the mainland by a low dam[107]. The dam will be first of all built to further accretion but the protection of the dike along the hattstedter Marsh, a road link (in due course) and a higher value of lands on the island, are also mentioned as benefits. The municipality on the island accepts that this dam will not be a non-overflow dam (too expensive).The dam will follow more or less the line of the tidal divide but one knows that the wind may cause differences in water level between both sides of the dam.
The subsoil of the *wad* has at the surface a 50 cm thick layer of clay; under the clay is a layer of peat, 1 to 1.5 m thick, and thereunder sand. The crest of the dam will be at MHT + 0.50 m. MHT = NN[108] + 1.24 m. On the wad the dam has a height of 1.3 m. The dam will cross two creeks (das Rott and Pohnsbucht, Fig. 8.36). The two dam sections have lengths of 2 890 and 980 m respectively and are separated by Pohnshallig[109]. Cross-sections of the dam are presented in Fig 8.37.

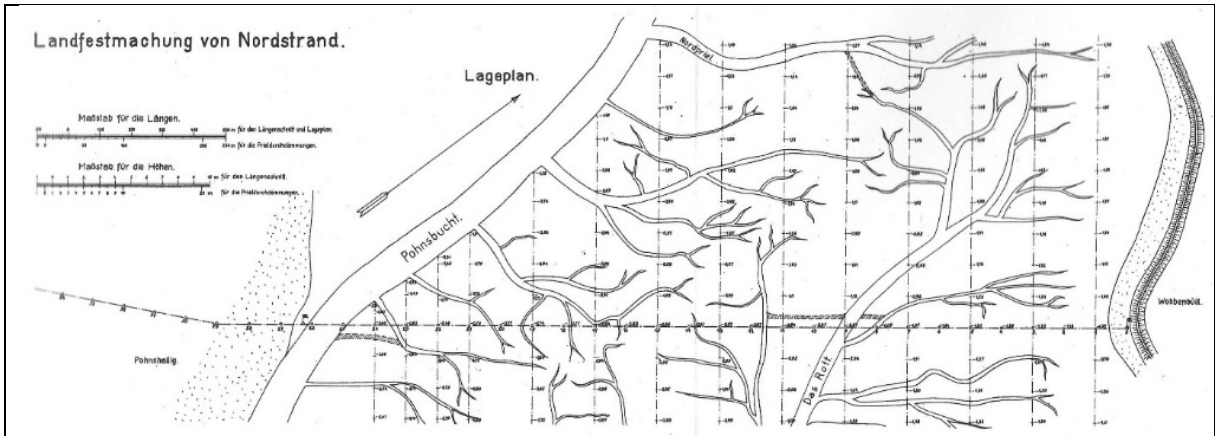

Fig. 8.36 Part of the alignment of the dam between the island Nordstrand and the mainland; Source: Müller & Fischer (1936a, 198)

[103] Müller & Fischer (1936a, 1 – 28). The difficulties appear to be of a technical, financial, cultural and institutional nature but a discussion thereof is beyond the scope of this dissertation.
[104] According to Baars (1982a, 44), the Inderveldes had a labour force of a few hundred experienced Dutch men working on the reclamation.
[105] A small polder in Germany is called a '*koog*', but in the Netherlands it also means *buitendijks land, bedijkte kwelder* (Danner et al, 2009, 76).
[106] The information in this paragraph has mainly been derived from Müller & Fischer (1936a, 48 – 66).
[107] The information about the dam has mainly been derived from Müller & Fischer (1936a, 191 – 215 and 243 – 251).
[108] NN is the abbreviation for Normal Null, the German Ordnance Datum between 1879 and 1992.
[109] A *hallig* is an inhabited area on the *wad* above springtide but without dikes.

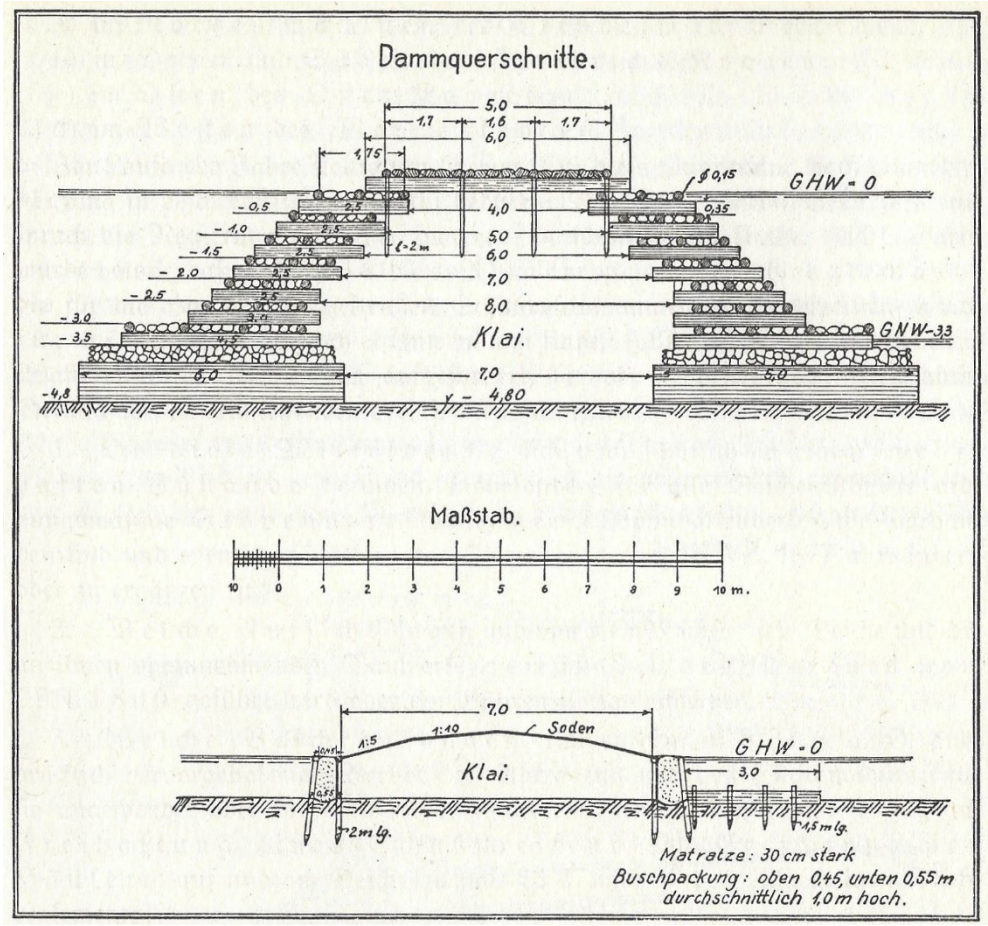

Fig. 8.37 Cross-sections of Nordstranderdamm built in 1906/7; the upper cross-section is applied in the creeks, the lower one on the *wad*; source: Müller & Fischer (1936a, 199)

Dam construction starts in April 1906 and is complete in June 1907. During this period it is decided to omit the apron of fascines as shown along the earth fill dam on the drawing[110] while the width of the clay core is reduced from 7 to 3 m. The clay core is contained by so-called '*schlickfänger*'. A *schlickfanger* consist of densely packed fascines between two rows of stakes.

In April 1909 the *wad* north of the dam is already at a level of MHT. But a levelling of the crest of the dam shows that substantial subsidence has taken place due to compaction of the aforementioned layer of peat in the subsoil and settlement of the clay core. The clay core is also decreasing in volume because of wave action. It is stated that wave action on the *wad* is higher than expected!

In 1913 a design for reconstruction of the dam is made. It is ready for implementation in May 1914 but, because of the First World War, the work on the earth fill dam section can only start six years later. During the War a number of storm surges result in considerable damage to the dam and the small number of available labourers can only carry out limited repairs. In fact, the dam over its full length is completely destroyed except for the sections through the creeks (170 and 260 m). These sections were already rehabilitated in 1914, as part of the planned reconstruction. They comprise fascines on a layer of clay with a rock-filled cofferdam on top. Moreover, wooden sheet piles (thick 5 cm) have been driven in the centre of the dam (Fig 8.38). It is thought that the sheet piles have saved the dam sections through the creeks and a new profile is designed for the vulnerable earth fill sections of the dam (2 200 m) incorporating sheet piling as well (Fig 8.39). Also *schlickfänger* (660 m in all) are placed in 50 m long sections perpendicular to the dam to prevent the progressive enlargement of tidal gullies already present along the toe and heel of the dam.

[110] In the design shown the apron is placed at the south side but the central government thought it should be at the north side.

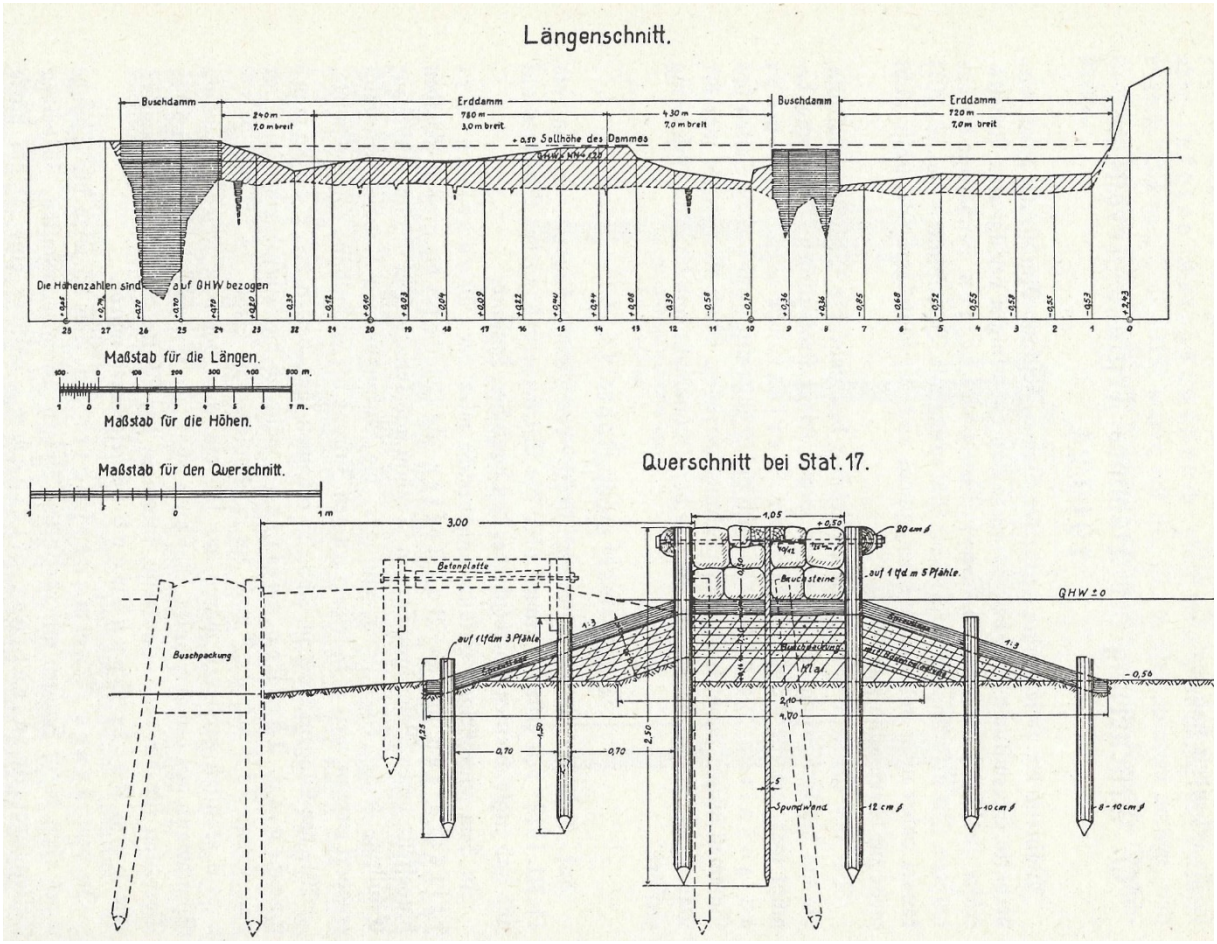

Fig. 8.38 Length profile of Norstranderdamm after storm surge of October 1921 and reconstructed cross-sections of dam through creeks and on the *wad* (the latter in dotted lines) according to the 1914 design; source Müller & Fischer (1936a, 210)

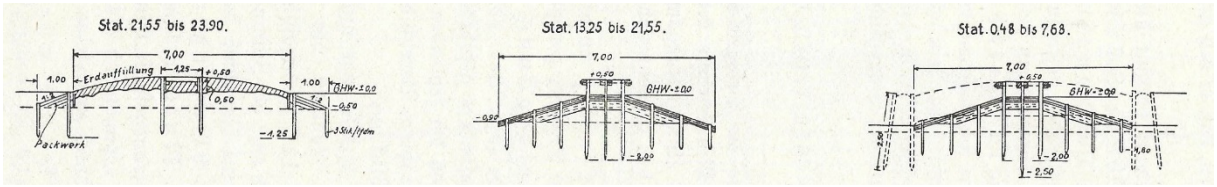

Fig. 8.39 Cross-sections of dam on the *wad* (1920 design); source: Müller & Fischer (1936a, 212)

Because of administrative difficulties, the construction of the earth fill dam, having the new profile, is delayed. The storm of 23 October 1921 practically destroys most of the dam sections already reconstructed (1 268 m). The storm surge from the North-West, first of all, brings heavy wave attack which loosens the sheet piling. Subsequently, north of the dam the water reaches a level of MHT + 2.17 m while the *wad* at the south side is still dry. The loosened sheet piling floats away on the current, the earth fill is eroded by the waves and current and the fascine layer is completely disturbed. In summary, 1257 m of earth fill dam has been completely destroyed.

So, once again, the design has to be modified(Fig. 8.40). The sheet piling gets a length of 4 instead of 3 m. Stone pitching, thick 25 cm, is applied on slopes and crest and rows of stakes must ensure that stones are not moving during wave attack. In fact, the storm surges of 30 August and 18/19 December 1923 are very well sustained by the dam.

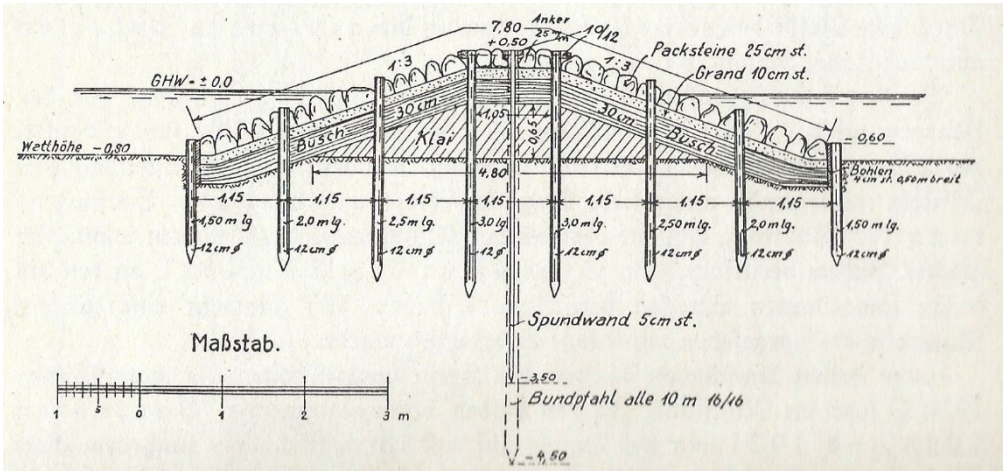

Fig. 8.40 Cross-section of dam on the *wad* in 1922; source: Müller & Fischer (1936a, 214)

The case history of the Nordstranderdamm has been described in such detail because the many mistakes made are similar to those already discussed in foregoing sections. Obviously, part of it is lack of knowledge. But, apparently, one does not learn from mistakes made. Or, is it, because these mistakes were not published and therefore not known to other engineers?
In June 1925 a request is received by the authorities for construction of a non-overflow dam. A design is ready by May 1926 but for the time being there is no budget for construction.
This non-overflow dam, having its crest at NN + 6.60 m, is built in the period July 1933 – July 1935. As it cannot be considered to be a structure built by using simple means, it will not be discussed.

8.9 Hindenburgdamm, connecting the island Sylt to the mainland (Schleswig-Holstein) [111] (project no. 35)

The Hindenburgdamm has been built in the period 1923 – 1927. Its construction has been very difficult. This is partly due to local circumstances like unsuitable dike fill (fine sand, silt), too shallow depths for (suction) dredgers and water level differences (up to 1 m) occurring between both sides of the dam during construction. But also the construction methods must be mentioned. It was, for instance, thought that tidal channels without bed protection could be closed by narrowing the opening by driving wooden sheet piles (thickness 8 cm). Huitema (1947, 120) describes this operation as follows:

> "When the sheet piling had reached the East channel the difficulties started. The progressive narrowing of the channel resulted in its progressive deepening by scour right in front of the head of the sheet piling. The depth became 7 to 9 m below MHT and the scour shifted to the same degree as the head of sheet piling progressed. The length of the sheet piles had to be increased from 5 to 10 m. It was then decided to first drive sheet piles on the higher ground (*wad*) at the other side of the channel. Subsequently, the closure by sheet pile driving started simultaneously from both ends. By the middle of October the gap had been closed but this 440 m long section had taken three months."

The closure of two tidal channels during the next year (1925) was done in a different way. In the channels the top of the sheet piling, over lengths of 150 m, was kept at a level of MLT – 0.05 m, while every 2.5 m a steel H-beam (DIN 15) was driven and strengthened by raking struts. At both sides of this 'sill' rock was dumped in order to have 2 x 5 m wide apron which prevented scour (Fig. 8.41). When the works had progressed sufficiently the 150 m wide openings left were closed by sliding 8 cm thick wooden panels (*schotten*) into the grooves formed by the H-beams. Subsequently, rock was dumped at both sides of the panels.

[111] Most of the information about the Hindenburgdamm has been derived from Huitema (1947, 117 – 123)

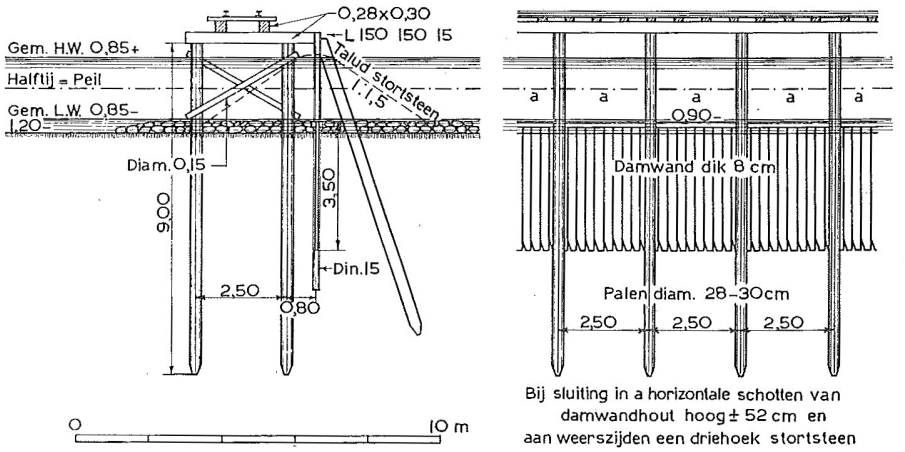

Fig. 8.41 Hindenburgdamm, opening in the sheet piling through the Hollandergat; cross-section and front view; Source: Huitema (1947, 121)

To give an impression of the magnitude of this project some quantities are mentioned (Huitema, p. 122 et seq.). Hydraulic fill placed by suction dredgers amounted to 1 750 000 m^3, fill transported by trains on a narrow gauge railway to 1 850 000 m^3. Sheet piling placed over a length of 9 300 m. The construction of this project is, however, so messy that an analysis for this dissertation is not worthwhile. Also, placing of 3 600 000 m^3 of fill utilizing dredgers and trains cannot be considered to be simple means. Nevertheless, it is also stated that pile driving for the temporary jetty (required to drive the sheet piling) and most of sheet pile driving proper, was done by pile driving crews and not by utilizing mechanical means.

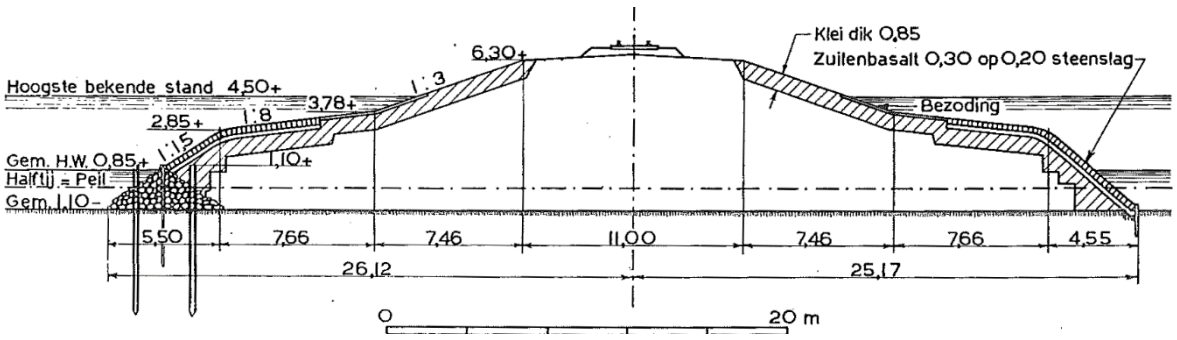

Fig. 8.42 Cross-section of the Hindenburgdamm; note the steep slope in the tidal zone; source: Huitema (1947, 123)

8.10 Comparison of construction costs

An effort has been made to compare the costs of construction of the various dams discussed in this chapter (Table 8.7). Because of the large differences between the cross-sections of these dams and the local or particular circumstances, this exercise in most cases is deemed to fail. Also, data are sometimes lacking. For instance the cost of construction for the IJdam excluding the navigation locks could not be traced.

Table 8.7 Comparison of the costs of construction of the dams discussed

no	name of dam	year of completion	length (m)	cost of construction		cost per m[1]	height (m) (from OGL to crest)
				f of period	€ of 2020[112]	€ of 2020	
28	Statendam	1751	3 745	76 805	865 062	231	aver. 2
29	Kreekrakdam	1867	3 640	1 500 000	16 223 817	4 457	max. 10
30	Sloedam	1871	965	1 100 000	12 291 605	12 737	max. 17
31	IJdam	1874	1 360	4 500 000	49 999 169[113]	36 764	≥ 10 m
32	Dam to Ameland	1881	8 400	1 000 000	11 567 123	1 377	aver. 1.5
33b	Slaakdam	1884	2 770	334 700	2 259 126	816	aver. 7
34	Norstranderdamm	1935	4 000	no information available			aver. 6
35	Hindenburgdamm	1925	11 000	11 100 000	87 806 620	7 982	aver. 7.4

Nevertheless, it is possible to make a few comments:

- When the costs per linear metre of Kreekkrakdam and Sloedam are compared one can see that the ratio of costs is 4457/12737 = 0.35 while the ratio of the squares of the maximum heights 10^2 / 17^2 = 0.346. This comparison is only possible because both dams have narrow crests and cross-sections are approaching triangles.
- The costs per linear metre of the overflow dams Statendam and Dam to Ameland are € 231 and € 1 377 respectively and give the impression that the latter has a high cost. But the Statendam started as a low dam only protected by fascines (without stones) while the Dam to Ameland had a protection layer of fascines covered by stones. Table 8.8 can be made by making use of the unit rates in Table 8.6.

Table 8.8 Comparison of construction costs of overflow dams Statendam/Dam to Ameland per linear metre of dam

type of work	unit	rate (f)	Statendam		Dam to Ameland	
			quantity	cost (f)	quantity	cost (f)
earthworks	m^3	0.10	10	1.00	7	0.70
fascines	m^2	1.30	10	13.00	8.8	11.44
stones	m^2	2.50			8.8	22.00
totals				14.00		34.14
Transition of guilders of 1883 into euros of 2020:				€ 171		€ 417

Obviously, this comparison only gives an indication of the ratio of costs between the two overflow dams. Moreover, the cost of Statendam in Table 8.7 is without maintenance and that of the Dam to Ameland includes various intermediate design changes and 20 years of maintenance. If the maintenance is taken at 4 % /year, then the amount of € 171 changes into € 308. The ratio of costs Statendam/ Dam to Ameland is then 0.74. The same ratio in Table 8.7 is 0.17. But it has been assumed that unit rates for both dams are the same and, as Ameland is at a much greater distance from the places of origin of the material used for fascines than the area around the Statendam, different unit rates should have been used for this item in Table 8.8.

- A last comment must be made about the construction cost of the Slaakdam. It is tendered in 1883 and built in 1884. It is a non-overflow dam and cost of construction is € 1 538 /m^1 in euros of 2020. Let us assume that the Dam to Ameland (built in the period 1871 – 1881) is tendered in 1883 for a slightly higher cost (because of deeper creeks and remoteness of the area). The construction cost then would have been 8 400 (length) x 1 600 = € 13 440 000. This is f 1.1 million in guilders of 1883. So, for a slightly higher cost than the f 1 million spent on a failed project, Ameland would have had a permanent connection to the mainland.

[112] For the transfer of guilders of a certain year into euros of the year 2020 the website: https://iisg.amsterdam/en/research/projects/hpw/calculate.php has been used.

[113] Incl. cost of three navigation locks and sluice.

8.11 Summary and Conclusions

1 A dam in the Netherlands or Schleswig Holstein is an earth structure that crosses a tidal channel between two islands or an island and the mainland, or it closes a tidal inlet.

2 Between 1751 and 1925 nine dams have been built by applying simple means.

3 The study of these dams had as aim the discovery and description of the problems encountered during construction and, more specifically, during the closure of the tidal channel(s) to be crossed.

4 The nine dams were built either to further accretion or to enable a transport link to be made. In some cases the dam also had a function in navigation and/or defence.

5 Four of these dams were in first instance built as low overflow dams in order to further accretion. All these dams failed as no measures had been taken to allow for significant overtopping during storm surges. Ultimately, two of these dams were transferred into non-overflow dams, a third one was built as a non-overflow dam at a different location, the fourth one was abandoned altogether.

6 It is noted that all four low overflow dams just mentioned turned out not to be sustainable. Obviously, part of it was lack of knowledge. But, apparently, one did not learn from mistakes made. Or, was it, because the responsible engineers were reluctant to publish their mistakes, and, as a consequence, others did not know about it?

7 For completeness' sake it must be emphasized that dams did not only fail because of overtopping. Subsidence of the foundation (IJdam, Slikdam), lack of maintenance after completion (Statendam, Slikdam) and, last but not least, a meagre dam profile in order to save on construction cost (Slikdam, Dam to Ameland) also played a role in causing damage ultimately resulting in failure.

8 The closure of the tidal channel(s) crossing the alignment of six out of nine dams must have been easy and did not require special techniques. This was either due to a limited tidal range (IJdam) or to the moderate width and shallow depth (in relation to tidal levels) of the tidal channels concerned.

9 The closure of the major tidal channels crossing the alignments of two dams was done by applying well-known and proven techniques: bed protection against scour, then gradual vertical closure up to the Low water level by stacking fascine mattresses on top of each other, thus creating one or two fascine dams, finally followed by a gradual horizontal closure by constructing pakwerks up to the level of the prevailing High tide.

10 Given the application of the said closure techniques (with two fascine dams) at the Kreekrakdam, the success of the closure was also due to the moderate maximum head of 35 cm over the broad-crested weir, formed by the two fascine dams and the clay core in between, during construction.

11 The aforementioned closure technique (with one fascine dam only) applied at the second dam (i.e. the Sloedam) has only just been successful; this near failure was due to the non-expected high head in the tidal flow across the dam under construction: 104 cm.

12 The last dam, which can be considered as having been built partly by simple means, is the Hindenburgdamm. Its construction has been very difficult. This was partly due to local

circumstances like unsuitable dike fill (fine sand, silt), too shallow depths for (suction) dredgers and water level differences (up to 1 m) occurring between both sides of the dam during construction. But also unsuitable construction methods played a role. The last openings were closed by a speedy lowering of a number of wooden panels in the grooves formed by vertically driven steel H- beams.

13 An effort has been made to compare the construction costs of the nine dams (in so far as known) with one another. It turned out in most cases that circumstances during construction of these dams differed so much among the dams considered that no generally valid parameters regarding construction cost could be determined.

CHAPTER 9 Repair of breached dikes up to and including the year 1916

9.1 Introduction

For a first introduction to the topic of this chapter please refer to Ch. 6, Section 6.4. In that section the difference is described between the closure of tidal creeks in reclamation schemes and of tidal channels developing through breached dikes. Another difference is the area of the tidal basin fed by a breach. In Table 9.1 three basins have an area smaller than 15 km^2, four basins vary in size between 90 and 253 km^2, average 160 km^2.

In Table 7.1 the reclaimed area (which is equal or larger than the tidal basin of the scheme concerned) is smaller than 10 km^2 for 26 out of 29 schemes. In the Table the polder to be reclaimed to the west of the Groote Waard has the largest area: 39 km^2.

Finally, as has been observed in Chapter 7, areas to be reclaimed, usually, have a great number of tidal creeks and gullies conveying the tidal flow while flooding of a polder takes usually place through one or a few breaches only.

Relevant to the repair of breached dikes are the events resulting in a breach:

- a storm surge: nos. 36, 40, 41, 42 and 43 in Table 9.1;
- dike seriously damaged:
  - by a flow slide in the shore near the toe of the dike, or
  - by water pressure: no. 38,
  - or a bad foundation: no. 39;
- destructive actions by warring parties: no. 37.

The selection of the eight breach repair projects in Table 9.1 has been based, first of all, on the availability of data. But, in nearly all eight selected cases the data base is still incomplete. A second criterion for selection has been the applied closure method. Finally, a third criterion was the desire to spread the closure projects over the period studied: 1200 up to 1916. This has not been completely successful. Now the earliest breach discussed is that in the Spaarndamse dijk which took place in 1508.

Fig. 9.1 Repair of a breached dike near Zierikzee (1715); source: Stadhuismuseum, Zierikzee

During the Middle Ages, documents of a technical nature concerning the repair of a breach either were not prepared or got lost in the course of time. The information on the early projects listed in Tables 7.1 and 9.1 has been based in most cases on financial accounts studied by historians and hist. geographers.

Table 9.1 Repair of breaches in dikes, selected for analysis

No. of pro-ject	name of project, location (province, town, district, island); Cause of breach	A = date of breach B = date of final closure	tidal range (m)		area of tidal basin (km^2)		tidal prism during spring tide (10^6 m^3)	length of actual flow gap (m) and max. depth (m)	length (m) & bed level of final closure bund (m)	sources	closure method and/or other typical features	cost of construc-tion In guilders of the period
			spring tide	neap tide	spring tide	neap tide						
	1	2	3	4	5	6	7	8	9	10	11	12
36	Breach in dike between Haarlemmer Lake and tidal inlet IJ, Spaarndamse dijk, near Halfweg, storm surges	A = Oct – Nov.1508 B = Middle of August 1510 (3rd effort)	probably less than 0.60	0.34 (?)	appr. 150	appr. 150	appr. 32	120 MSL – 7 to -9	approx. 250 to 300	Van Dam (1992); Van Dam (1998)	perimeter row of piles; fill placed at both sides; mainly waterborne transport	
37	Flow gap at Leverhuyzen, near Zierikzee; sluice removed and dikes breached as part of military inundation, then a major breach developed	A = 29 Sept. 1575 B = 27 June 1578	Probably more than 2.5		90 (used to be to more than 100 prior to its land losses)		not calcul-ated	142 14 to 17 m below low tide	no informat-ion	Cau (1878); Pot (1925) &(1929); Uil (1986)	project only interesting because of similarities with development of breach at Schelphoek (1953)	
38	Breach in dike at Scharwoude, Noord-Holland; dike section was pushed in landward direction by the water pressure during a storm surge	A = 5 Nov. 1675 B = 23 Jan. 1676	appr. 1	0.32	253.5	253.5	not calculated as the parame-ter area (= 253) is outside the indic-ative range of CRESS	initially 100 and after repair and new storm surge 23 and 54 in perimeter bund, deep 11 to 14	345 MSL – 3 to -11 m	Schoorl (1967); Thierry (1930)	perimeter cofferdam comprising 3 rows of piles	
39	Breach in the Lekdijk Bovendams; Wijck, prov. of Utrecht; slip in the land side slope after high river stage	A = 28 Febr. 1747 B = 25 March 1747	n.a.	n.a.	n.a.	n.a.	n.a.	55, deepest point 11.7m below flood plain level	980 m (195 dike rods) flood plain level and partly crest of low sum-mer dike	Woertman (1747); Van Hengst (1898); Van Vliet (1961);	perimeter cofferdam comprising 2 rows of piles, hurdles, clay fill	
40	Breach in dike Nieuwerkerkepolder, island Walcheren Sluice pushed out of dike body during storm surge	A = 30 Sept-ember 1807 B = 25 November 1807	3.40 or less		1.12	1.12	1.2	23 at MLT 3.5 m below MLT	19 m on sill (= crest of fascine dam) at MLT + 1.5 m	NL-MdbZA_ 3017 _ 1 & _361.1_ 134; Wilderom (1968, 85 et seq.)	fascine bed protection, subsequently hor. closure by cofferdam	f 14 068 (cost estimate)
41	Breach in dike Kruininger polder, east of Hansweert; island Zuid-Beveland storm surge	A = 15 Jan. 1808 B = 1 June 1808	5.0	3.5	13.23	13.23	29.5	initially 240 in sea dike and 135 in secondary dike; depth not given	1130 long perimeter dike Its bed level is at local ground level which is MLT +1.9m	Waterst.-kundige Onderwer-pen in Zeeland (ca 1800); Van Hoek (1808, 214 et seq.) De Bruin (1953a)	fascine dam up to MLT + 5.0 m; then bund of clay clods having crest at MLT + 5.65, which is HWS + 1.8 m	Contract sum was f 520 000
42 42a 42b	'English' polders: First: Oosterschelde, near Bath; Second: bad workmanship	A = Febr. & Sept.1853 B = May 1856 A = Febr. 1858, Dec 1862, Dec 1863 B= 1857, Summers of 1862, 1863 & 1864	4.2	3.2	5.8 ↓ 4.5 7.0 ↓ 2.85		no calcul-ations made	as it concerns a large number of breaches and the like, details will be presented in Sub-section 9.8	no infor-mation available	Ortt (1853) Muller (1862); Chamuleau (2014); NL-MdbZA_ 219, Inlei-ding, Bijlagen, 3.1.1	the reclamation works for these two polders failed repeatedly; the reasons have been analyzed and presented already by others.	after the damage by the storm of 1906, the rights and properties were sold in 1906 for f 175 000; the total loss was ± f 8.2 million
43	Dike breaches south of Edam; storm surge	A = 13/14 Jan. 1916 B = 24 March 1916	0.50		140		49 (appr.)	Katwoude 120; deep-est 11-12; Waterland 47, deep 7.5	bed levels at NAP -0.50 (Kw.) and NAP (Water-land)	Bos (1916); Reigersman (1917); RWS (1916)	fascine dams up to NAP & NAP -0.50 m; then clay dam incl. fascine bund having crest at NAP + 2 m	no informat-ion

Fig. 9.2 Approximate locations of the projects concerning repair of breaches in dikes, listed in Table 9.1, marked by their project number

9.2 Repair of a breach in the Spaarndamse dijk in 1510 [1] (project no. 36)

During the period 1510 – 1515 the Spaarndamse dijk repeatedly breaches or is seriously damaged. Van Dam (1992) has analysed the labour input for the repair of the largest three breaches during the period 1510 – 1515. This includes the breach at Het Hart (location of the present village Halfweg).

[1] The basic data used in this Section have been derived from Van Dam (1992) and (1998).

Here, the dike breaches in 1508, is being repaired in the spring of 1509 but breaches again on 27 September 1509.
Because of the breach, the tidal inlet IJ becomes connected to the Spieringermeer, the old Haarlemmermeer and the Leidsemeer (Fig. 9.3). The total area of these lakes comes to approximately 150 km^2. But also the areas bordering these lakes are inundated.

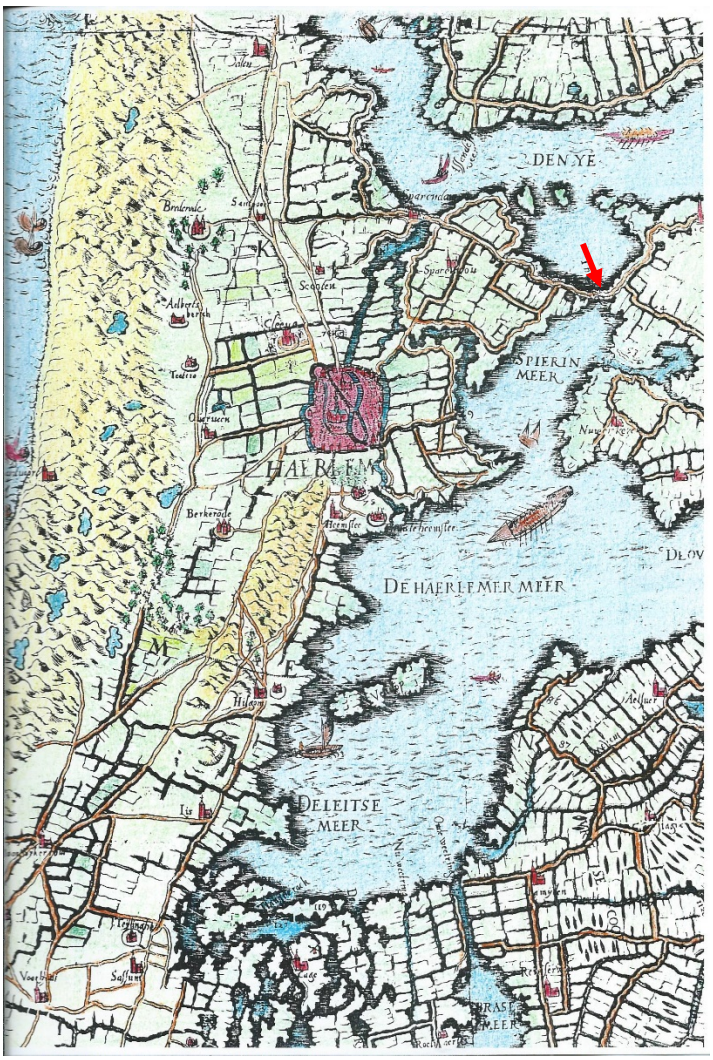

Fig. 9.3
Map (1575) of the surroundings of Haarlem, showing the tidal inlet IJ as well as the lakes mentioned in the text; the breach at Het Hart has been indicated by the arrow;

source: detail taken from the map of Noord-Holland made by J.J. Beeldsnijder (NHA, *collectie prenten en tekeningen van de Provinciale Atlas* Noord-Holland)

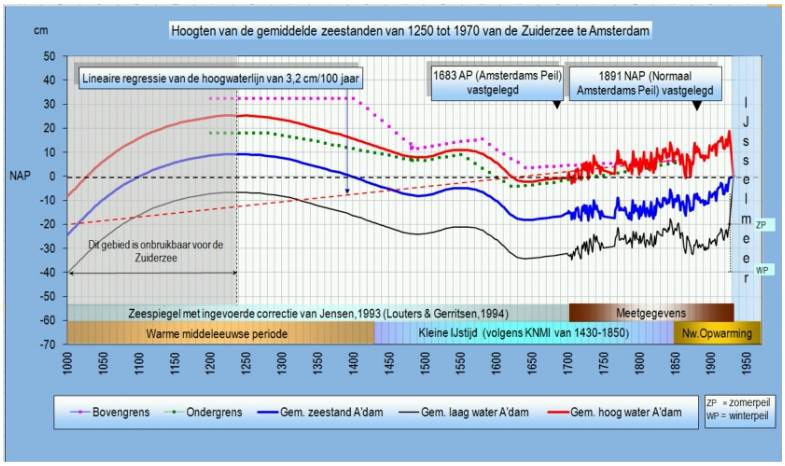

Fig. 9.4 Average water levels (MLT, MSL and MHT) in the tidal inlet IJ at Amsterdam; accessed on 1 Oct. 2021 at: https://www.zuiderzeehoorn.nl/pg-27093-7-70602/pagina/waterstaatgeschiedenis.html

The length of the breach of September 1509 is given as 120 metres while its depth is 7 to 9 metres. The tidal range in the beginning of the 16th century at Het Hart is not known but, according to Storm Buysing (1854, 581) the values are the same for Amsterdam, Zwanenburg and Spaarndam: MLT = AP – 0.27 m and MHT = AP + 0.07 m. But these values are for the middle of the 19th century. In Fig. 9.4 one can see that in 1510 in Amsterdam MLT = AP – 0.23 m and MHT = AP + 0.09 m. These levels are adopted for Het Hart.

By using the Coastal and River Engineering Support System CRESS[2], a few tidal calculations are made. Based on information given already above one can assume:

A = area of flooded area = 150 km^2

B = width (= length) of gap = 120 m

H_z = tidal difference at Het Hart = 0.32 m or 0.50 m (the latter due to wind set-up or springtide)

h = depth of flow gap in relation to MSL: take 7, 8 and 9 m

c = discharge coefficient for a short free overflow : 0.90

T = period of the tide: 12.5 h

Now the parameters:

- water level difference in the basin H_B (m)
- maximum current in gap U_{max} (m/s)
- max. head over gap Δh_{max} (m)

can be calculated for various values of h and B (Table 9.2)

It is noted that the calculation using CRESS cannot be made for values of h < 8 m. That does not matter because it concerns a horizontal closure (see below). It is in any case clear that the large size of the tidal basin together with the modest tidal range result in a nearly non-varying head Δh_{max} and a ditto max. current velocity U_{max} for various values of the parameters h and B. This implies that, given a certain H_z, the value U_{max} stays practically the same regardless of the value of B.

Table 9.2 Tidal calculations for various values of h and B by using CRESS

INPUT						OUTPUT		
H_z (m)	c	T (h)	A (km^2)	h (m)	B (m)	H_B (m)	U_{max} (m/s)	Δh_{max} (m)
0.33	0.9	12.5	150	8	120	0.15	1.53	0.15
					100	0.13	1.55	0.15
					50	0.07	1.60	0.16
					25	0.03	1.62	0.16
					10	0.01	1.62	0.16
					5	0.01	1.62	0.16
				9	120	0.17	1.50	0.14
					100	0.14	1.54	0.15
					50	0.07	1.60	0.16
					25	0.04	1.61	0.16
					10	0.02	1.62	0.16
					5	0.01	1.62	0.16
0.50				9	120	0.21	1.90	0.23
					5	0.01	1.99	0.25

In order to demonstrate that the type of closure, as described below, is technically feasible, various assumptions have to be made which can be incorrect in part.

[2] For equations used in CRESS , reference is made to Appendix N.

Van Dam (1998, Bijlage 10) transcribed two texts[3] which describe in rather vague terms how the closure of the breach should take place. But it is apparently the only technical source available as regard the closure method. It is clear that it concerns a horizontal closure by driving a row of piles, thus narrowing the breach. For the last opening ships can be ballasted and sunk, if required.
In Text no. 1 it is mentioned that piles have to be driven as near to each other as physically possible. The row of piles has to start "at the deepest point of the breach from the Amsterdam end" [4]. It is thought that it only concerns one row of piles and not piles becoming part of a cofferdam. The piles will be connected at the top to each other by horizontal beams, one at each side. The beams are connected to each other by bolts every 3rd or 4th pile. Every 8 to 12 m a bracing is to be made but it is not clear whether it is a bracing in the plane of the piles or perpendicular to the row, probably the latter. Nothing is said about the horizontal alignment of the row of piles. Is it the intention to go straight from one dike head to the other or along the perimeter of the scour hole (*wael)*? Probably the former but it could also be an alignment somewhere in between to avoid the deepest spots.
After having driven the row of piles over some distance, one should start dumping earth from ships up to the water level. The dumped earth body (earth is dumped simultaneously at both sides of the piles) must have a width at the water level of 8 rods (30 m) and 'above' (probably is meant the crest of the dike) 4 rods (15 m). At both sides of the piles '*dijckschuyte*' are anchored. The (approximately) 100 ships bring the earth fill to these '*dijckschuyte*' and the dumping of the earth fill is done from the latter. Dumping of clay clods is mentioned for the time that the opening has become narrow.
A number of 's*chortcleen*' (aprons) must be arranged to transport earth fill to the gap and dump it.
In the event that closure becomes difficult a kogge[5] (already bought) and long rafts, (carrying the piles to be driven as support to the kogge during the sinking operation) must be on stand-by at both sides of the row of piles.
It is also stated that only limited time is available to construct the dike before autumn and therefore 1 600 hundred labourers should be employed rather than 800.
Van Dam (1992), on the basis of the wage-sheets and related accounts, has made an analysis of the labour input and also of the wages and places of origin of the labour. The figures in Table 9.3 have been copied from her article (Bijlage 2, Tabel 1).
The breach in the Spaarndamse dike at Het Hart is repaired during a period of 13 weeks (11 July – 3 Oct. 1510). Apparently, the carpenters, sailors, rafters/boatmen and hand pile-drivers, are only required during a short period. It implies in fact that the row of piles is complete in a period of six weeks. The actual pile driving takes five weeks. From the number of hand pile-drivers one may conclude that two gangs are involved during nine days and one gang during 20 days. This comes to 38 gang-days. The horizontal length of the row of piles is not known. It might be 120 m but also as much as 235 m (i.e. alignment along the perimeter of the scour hole) . So the output of a gang could be as low as 3 m a day or as high as 6 m a day. Accordingly, If the piles have a diameter at the top of 25 cm, a gang will drive 12 to 24 piles/ day.
These figures are compared to the output of the hand pile-drivers at the Hindenburgdamm (Chapter 8, Section 8.9). When sheet pile driving was very difficult the output by one gang was 2.5 m/day. When it was easy (no tidal differences, shallow sections) the output of one gang was as high as 22 m/day (Huitema, 1947, 120 & 122). This comparison is only made to check whether certain assumptions made for the closure of the year 1510 are realistic. A direct conclusion can, given the available information about the closures of 1510 and 1925 (Hindenburgdamm) not be drawn.

[3] Source: Manuscripts filed in NL-LdnRAL-0501: 1263-1264 Boeck van allerley stucken 't Heemraetschap angaende, 1286 – 1574.
[4] " …*van't diepste eynde als van Amsterdam comende op de wael…"*.
[5] A kogge is a type of merchant vessel in use until the end of the 16th century. Its carrying capacity was 200 to 250 tons and its length was between 15 and 30 metres.

The earth fill for the new dike is loaded into the ships by the diggers in the borrow areas. Transport to the work site is by more than 100 small ships. Van Dam (1992, 457) mentions various types[6]. One of these types is the Ever (Fig. 9.5).

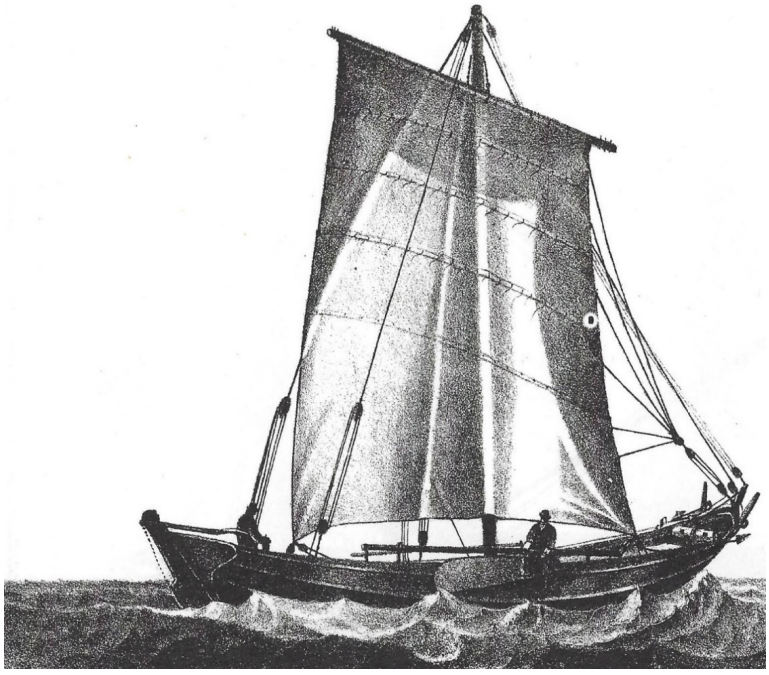

Fig. 9.5 Ever; source: Le Comte (1831, Pl. 43)

Transport of the earth fill from the ships to the dike under construction is done by barrow bearers but the earth fill is also transferred directly from the ships into the '*dijckschuyte*' mentioned earlier. It follows from the figures in the Table that there is a peak in the transport of dike fill during weeks 3 to 6 (incl.) and that is exactly during the actual closure of the breach when there will be high losses of dike fill because of the horizontal eddies, created by the flow through the narrow closure gap. After the actual closure of the breach the number of boatmen and diggers/spreaders goes down, roughly, by 50 % (weeks 7 to 13).

Table 9.3 Labour input during repair of the breach at Het Hart, Spaarndamse dijk

week no.	number of days	diggers & spreaders	barrow bearers	boat-men	carpenters	sailors	rafters / boatmen	hand pile-drivers	totals
1	3	202	35	95	8	3	10	122	475
2	6	269	87	241	10	3	10	84	704
3	6	431	118	286	9	4	10	36	894
4	7	473	113	265	9	3	10	40	913
5	7	442	76	320	9	4	10	33	894
6	6	396	74	345	9	3	10		837
7	5	221	56	142			5		424
8	6	218	48	176					442
9	6	181	40	152					373
10	5	200	43	135					378
11	5	175	45	106					326
12	6	211	51	150					412
13	4	177	54	162					393
14	6	17		25					42
15	2	15		27					42
16	5	14		23					37
17	5	13		15					28

Source: Van Dam (1992, 473)

[6] Such as: "*damlopers, evers, aalmannen, utrechtse schuiten en roeischuiten*".

In Fig. 9.6 a possible cross-section of the new dike is shown, based on an average depth below MSL of 8 m and a crest level of MSL + 2.50 m. The volume of earth fill in this dike per linear metre is 392.25 m^3 of which 336 m^3 is below MSL. If the loss during the actual closure operation (weeks 1 – 6) is 50 %, then the volume of earth fill becomes 560.25 m^3/m^1. For a dike long 125 m, the overall volume of earth fill is 70 000 m^3. This volume of earth fill has to be dug by the diggers[7] in 10 467 man days. This means a production of 7 m^3/man day. According to Visser (1946, 176), the production per manhour for digging is ¾ to 1.5 m^3 incl. throwing the dug earth over a distance of 2 to 3 metres (known as *smakken*). The actual production depends also on the type of earth fill. As the diggers work 13 hours (nett) during a day the required production can easily be achieved.

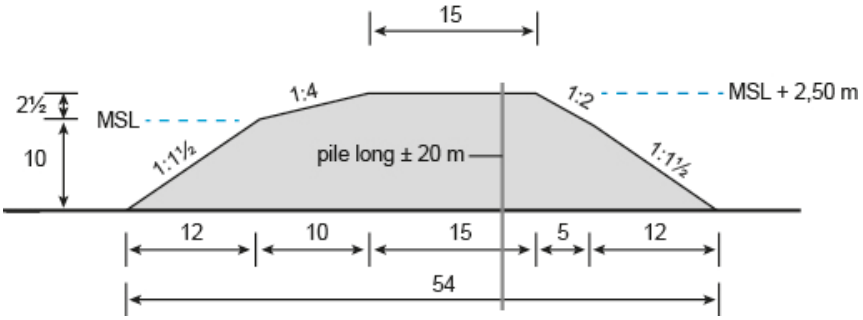

Fig. 9.6
Possible cross-section of the dike made at Het Hart (now Halfweg) at the south side of the tidal inlet IJ;
all measures in metres

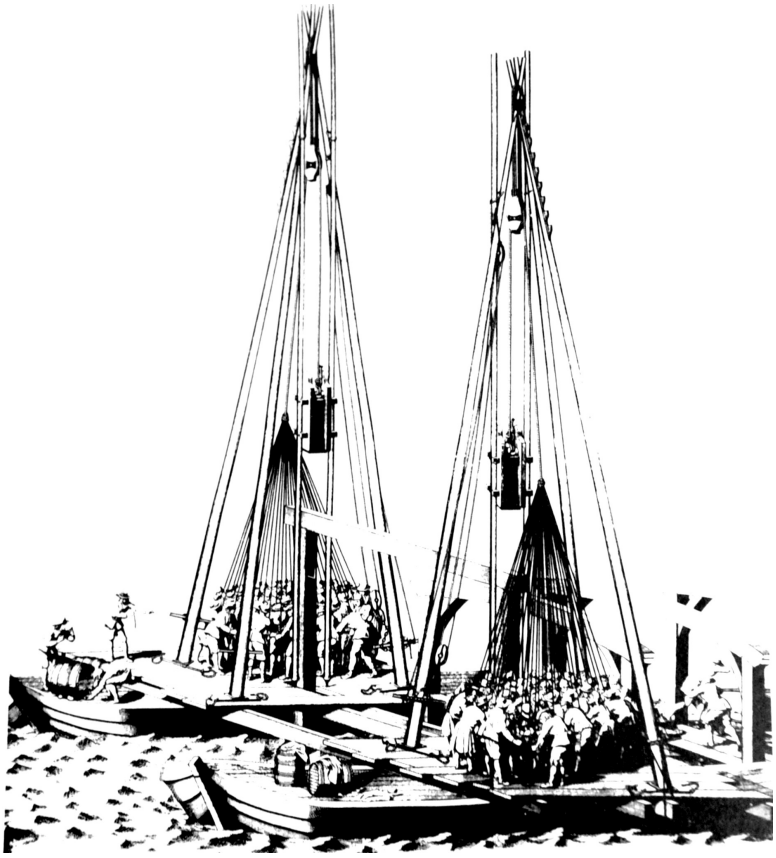

Fig. 9.7 Pile driving in Holland during the 18th century; source: Van der Horst & Polley (1736)

[7] The overall number of man days of the diggers/ spreaders during 13 weeks is 20 935. For this calculation it is assumed that there are as many diggers as there are spreaders.

9.3 Breaches made near Zierikzee in 1575 prior to the start of its siege by the Spanish army [8] (project no. 37)

One of the breaches made near Zierikzee in the autumn of 1575 illustrates very well certain remarks made by Vierlingh (1920). According to Verhoeven, Vierlingh distinguishes three types of breaches (p. LIII):

- those comprising a deep scour hole in the dike's foundation (*zate*),
- those where a thru-going creek has developed, for instance in the case of a drainage channel in the polder leading to a sluice having a tidal outlet channel at the seaside,
- those without any scour in the dike's foundation.

Obviously, the second type of breach is the most difficult to close. This is very well demonstrated by the development of the breach at Schelphoek in 1953. During five days the wide and deep access channels to the breach develop following the existing network of a drainage channel (to a new drainage pumping station) and its side drains (Fig.9.8).

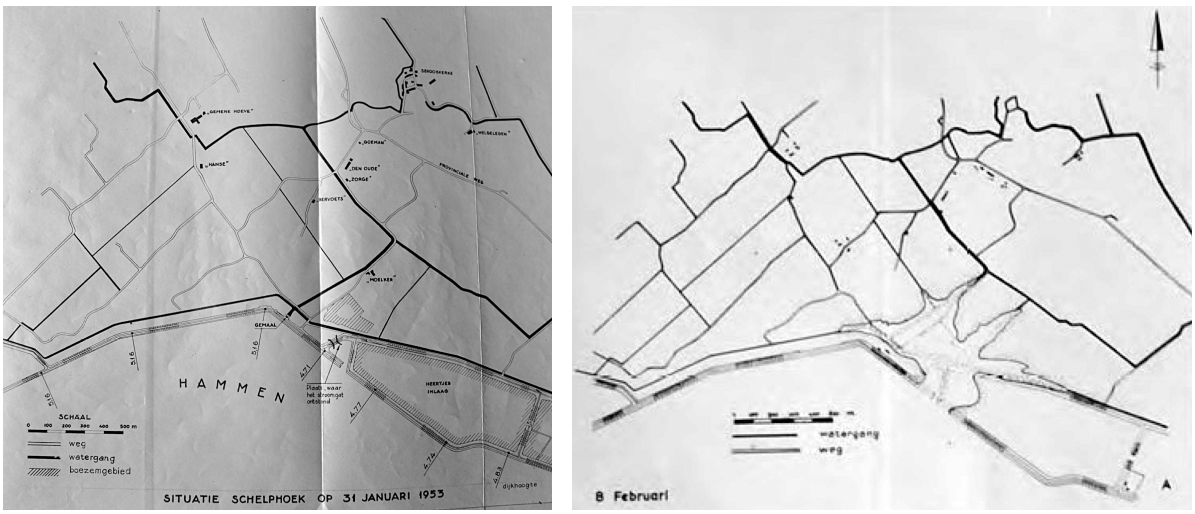

Fig. 9.8 Development of breach and scoured channels at Schelphoek in 1953; at left the situation on 31 Jan. 1953, at right situation on 8 Febr, seven days after breach;
source: Van de Velde & Gerritsen (1955, 105 & 107)

A similar development takes place in 1575 near Zierikzee. As, so far, nobody has commented on this phenomenon, it is worthwhile to present it here.
In the autumn of 1575 the Spanish army is able to reach the island Duiveland and a few inhabitants of Zierikzee together with soldiers of the '*Staatse*' garrison decide to breach the dikes in order to inundate the polder Schouwen and to prevent the Spanish from invading the town.
One breach is made south of the town at Borrendamme (Fig. 9.9) in the dike along the Oosterschelde while the second breach is made by ordering a carpenter to remove the sluice in the Leverdijk , north of the town in the dike along the Dijkwater (the tidal channel between the islands Schouwen and Duiveland). As one is afraid that the Spanish army will easily bridge the flow gap, created by the removal of the sluice, a second gap is made in the Leverdijk which, ultimately has a length of 38 rods (= ± 142 m) and a depth of 8 to 9 fathoms (= 15 to 17 m) below Low tide. 450 years later, the deep and 50 to 125 m wide tidal channel, called Kaaskenswater, still exists and it has a length of 875 m. Moreover, bordering the Kaaskenswater is a deep, nearly circular, scour hole, known as the Ronde Weel (diameter 150 m) (Fig.9.10).

[8] Most information in this Section has been derived from Cau (1878), Pot (1925) & (1929) and Uil (1986).

Soon, the Spanish army is able to construct a flood-free perimeter bund at the southern breach near Borrendamme on the mud-flat in front of the dike but cannon fire from the fleet of *'geuzen'*-ships on the Oosterschelde prevents completion and soon it is breached.

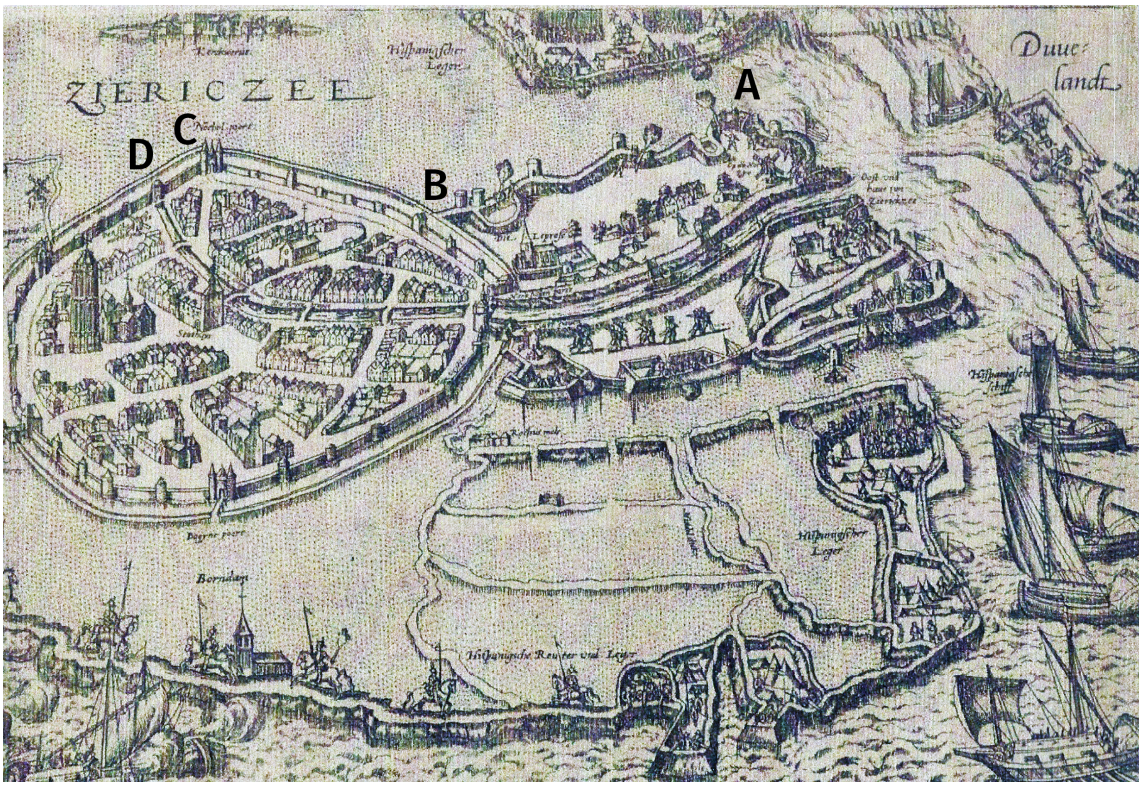

Fig. 9.9 Figurative map of the town Zierikzee at the time of the siege (1576); the village Borndam (Borrendamme) is shown (below, left) as well as the inundated area around the town; source: KZGW, Middelburg

In the beginning of July 1576 the town is taken by the Spanish army after a siege of nine months. Then a mutiny starts among the troops originating from Spain, Germany and Wallonia. Gradually, the troops leave the flooded and emaciated island. In November 1576 also the commander of the troops from Wallonia, Mondragon, leaves the island.
Now it is time to reclaim the flooded island but there is no money and the magnitude of the scour in the tidal channel, Kaaskenswater, landward of the breaches in the Leverdijk is enormous.
Why did this happen? The answer is simple. Uil (1986, 56) states that probably Kaaskenswater comes from 'Kadewater' i.e. a widened and deepened moat along the *kade* (bund) shown in Fig. 9.9 and marked A-B. This moat is connected to the moat around the town. Thus, the tide entering through the breaches in the Leverdijk finds a deep wide channel at A which it can follow to point B and then flows through the moat around the town down to point D (Fig 9.9) where it will leave the moat and flow into the polder Schouwen (which has an area of 90 -100 km^2 at that time).
In February 1577 a start is made at repairs of the breach at the dike near Borrendamme while ships transport earth fill to construct a temporary bund (Fig. 9.10) between the town gate Nobelpoort (C on Fig. 9.9) and the Leverdijk. This bund is required to minimize the area of the tidal basin, 'fed' by the breaches in the Leverdijk, as much as possible. Everything goes well until the evening of Sint Jan, 23 June 1577, when a storm surge breaches the temporary bund which results in the creation of the aforementioned scour hole Ronde Weel. Also the repair works at Borrendamme are in jeopardy: all stockpiles of fascines, reed and straw as well as the wheelbarrows of the labourers float away (Pot, 1929, 138).

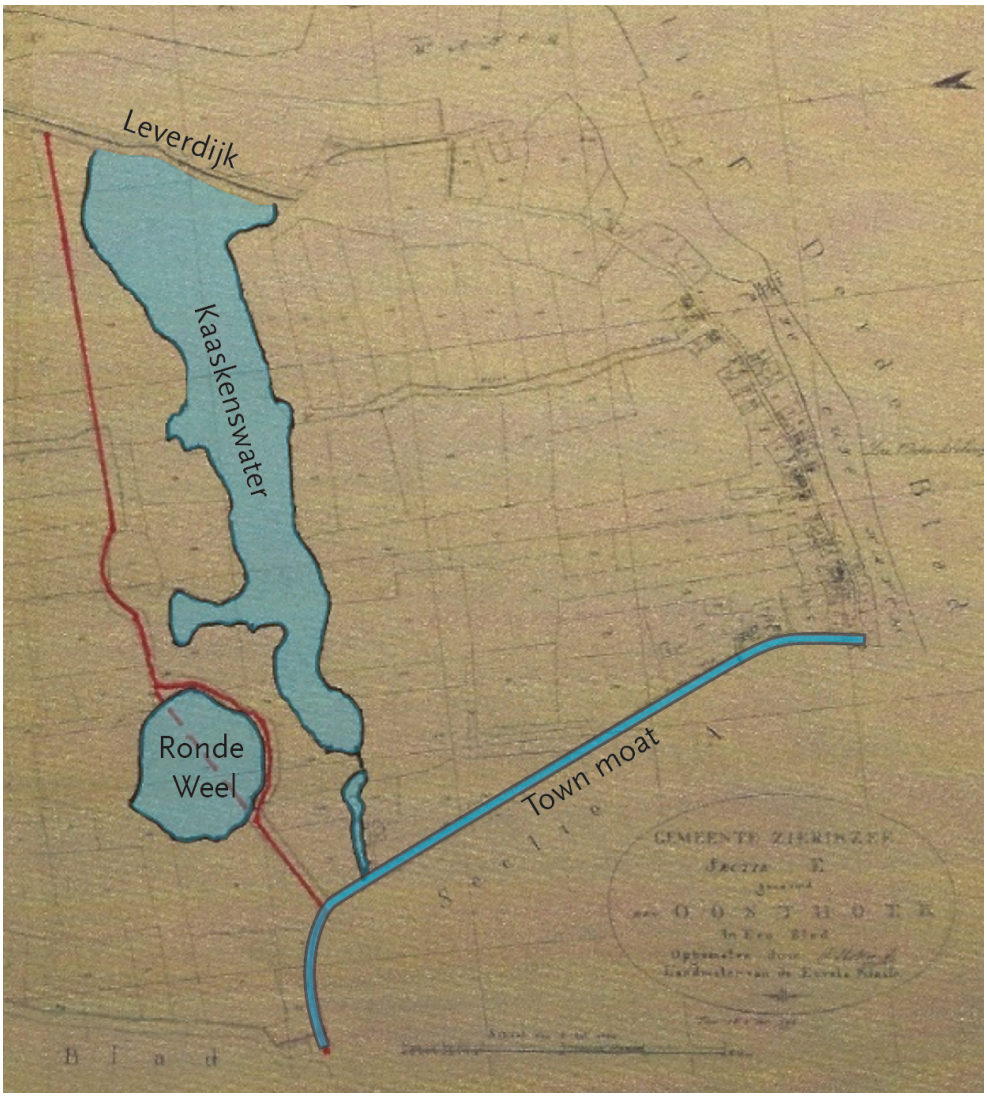

Fig. 9.10 Part of cadastral map (19th century) showing the area between Zierikzee and Leverdijk; the borders of Kaaskenswater and Ronde Weel have been clearly marked. The red line shows the horizontal alignment of the temporary bund (dotted line is section breached in June 1577)

Apparently, later in the year, the breach at Borrendamme and the temporary bund are successfully repaired. In the large breach in the Leverdijk, early 1578, a *zoute berm* is constructed which probably means a dam built up of sink-fascines[9] up to the level of MLT. It is said that for construction of this (19 m wide) dam 90 ship loads of fascines and 10 ship loads of reed were required. Subsequently, ships transport earth fill to the breach for a final closure bund to be made on top of the dam of sink-fascines and Cornelis Claes notes that closure in this manner would not have been possible without the temporary bund, mentioned earlier (Pot, 1929, 139).
The final closure is effected on 10 June, but breaches on 12 June. The breach is closed again on 27 June 1578 (Pot, 1925, 71). This is nearly three years after the wilful breaching for military purposes.

[9] On 27 March 1578 it is decided by the town's council that the inhabitants have to carry materials and stones to the breach. On 26 April it is ascertained that the breach is now considerably more narrow than before. On 7 June it is decided to start the final closure operation. During the days of closure all craftsmen must be available for the closure works (Cau, 1878, 109).

9.4 Repair of a breach in the sea dike near Scharwoude in 1675-1676 [10] (project no. 38)

During a storm surge on 5 November 1675 the dike at Scharwoude (marked as 19 in Fig. 9.11) slides away on its base. This happens because the dike is light in weight due to much peaty material in the dike body. A flow gap is created, 100 metres long and 9 metres deep.

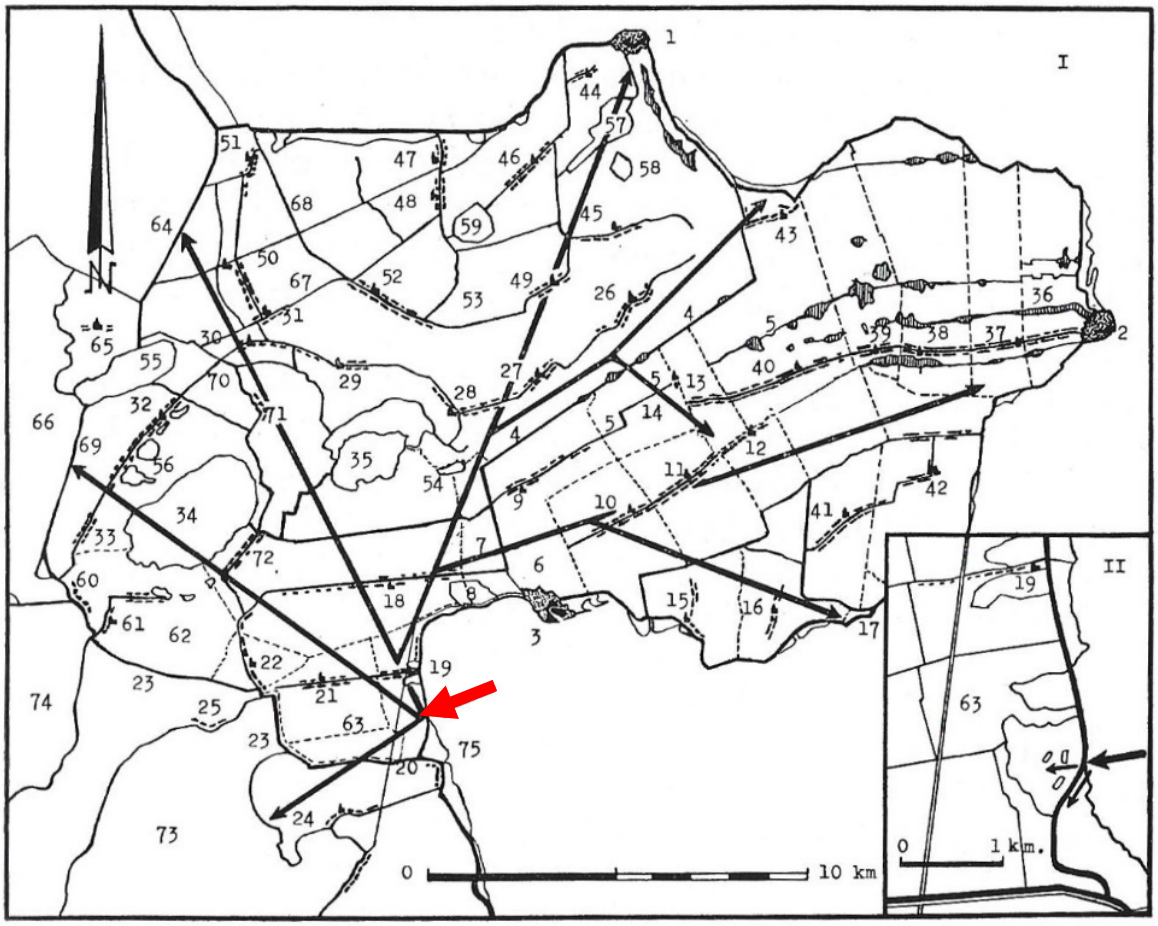

Fig. 9.11 The breach near Scharwoude in 1675 in the dike bordering the Zuiderzee; the black arrows indicate the extent of the flooding; 1 = Medemblik; 2 = Enkhuizen; 3 = Hoorn; 19 = Scharwoude; source: Schoorl (1967, 101)

Immediately, a closure is attempted by driving at the seaside one row of piles along the perimeter of the scour hole *(wiel).* But ten days later another, even higher, storm surge destroys part of the work[11]. Now one tries to drive two rows of piles in order to construct a cofferdam. The distance between the two rows is nearly seven metres and the space in between is filled up with hay, straw and '*andere ruigte*' (literally: other shagginess, like underwood, brushwood, brambles).
At the time this cofferdam is provisionally complete a third storm surge (on 5 December 1675) destroys most of the work done and the new cofferdam is breached at two places. Breaches are 23 and nearly 54 metres long and some 11 and 14 metres deep. The water levels during the storm surge are again 30 to 40 cm higher than before. In a hurry a secondary dike, long 14 km, and not yet breached, is heightened up by more than one metre by 4 800 labourers in a period of two weeks. Still other secondary dikes have breached because of the high water levels during the storm surges and the overall area of the flooded polders and lakes is now 253.5 km^2.

[10] Most information in this Section has been derived from Thierry (1930, A 53) and Schoorl (1967,100 et seq.).
[11] What remains of this single row of piles has been marked on Fig. 9.12 as '*stael vant eerste werck*'.

The plans for the closure are now made by the secretary of Huisduinen who is also inspector of dike works in that village, by a surveyor (Dirk Pieterszn. Abbestee, who also made the drawing shown in Fig. 9.12) and a foreman from Amsterdam who advises on the pile driving operations. The works are resumed on 12 December. Because of the aforementioned scour holes in the alignment of the breached cofferdam, the alignment of the new cofferdam has to be longer and its length is 345 m (Fig. 9.12).

The closure is complete on 23 January 1676, seven weeks after the third storm surge. It comprises partly two rows of piles (part of first cofferdam) and partly three rows of piles, together forming one cofferdam. This cofferdam last mentioned has a total width of 13.65 m. The landward side compartment of the cofferdam is filled using fascines, stone and earth. The seaside compartment is filled using sea-weed. Fig. 9.13 shows the repair works including pile driving.

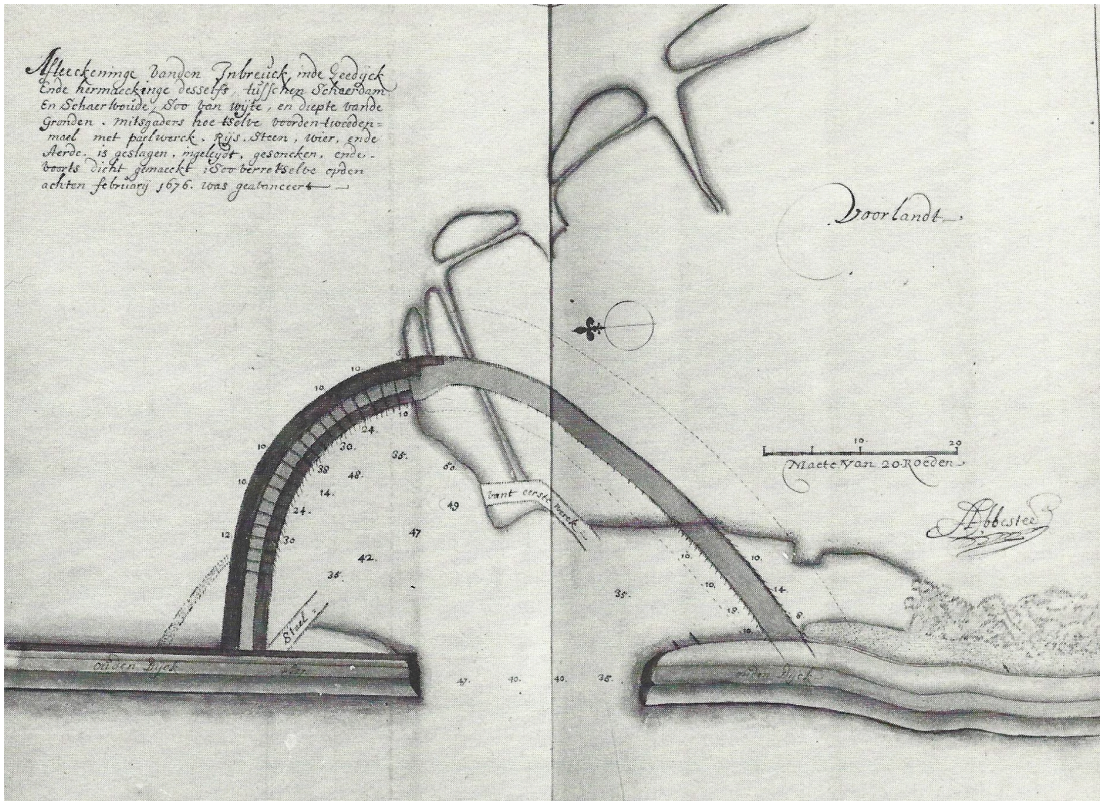

Fig. 9.12 Successive efforts made to repair the breach in the sea dike near Scharwoude in 1675 - 1676; the alignment of the first row of piles has been marked '*stael vant eerste werck'* (foundation of the first works); at the left the abandoned part of the alignment of the first, breached cofferdam is visible, The licht-gray coloured part of the completed cofferdam at the right is what was left of the first breached one; at the left part of the cofferdam the struts between the rows are clearly visible; one *Zijpse* rod = 3.829 m; one Amsterdam foot = 28.3 cm; source: Schoorl (1967, 104)

Assuming a three days break in the working period 12 Dec – 23 Jan. 40 clear working days are left (incl. Sundays). During this period three rows of piles are driven and some repair work to the first cofferdam. The second cofferdam (left part in Fig. 9.12) has a length of 130 m. Three rows of piles come to 390 m plus some 50 m repair works at the first cofferdam. Thus 440 linear m of piles driven in 40 days, i.e. 11 m a day by two piling gangs. This figure matches satisfactorily with those calculated in Section 9.2.

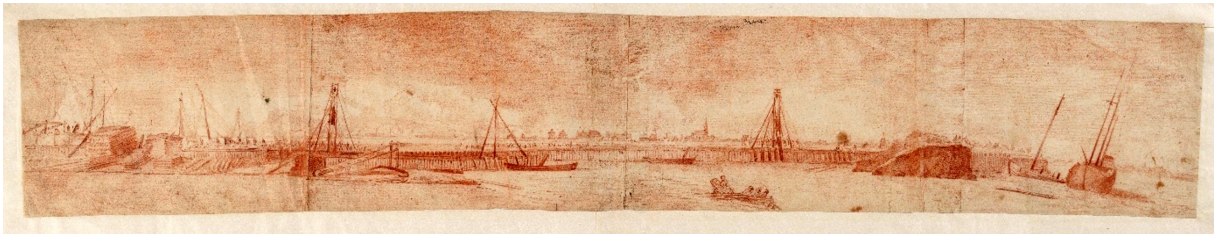

Fig. 9.13 A view of the repair works in progress (1675), two pile drivers can be seen as well as construction materials stocked at the left side; in the background the town of Hoorn; source: drawing by M. Withoos, Hoorn, Noord-Hollands Archief/Collectie van tekeningen en prenten, NL-HlmNHA_359_4307

This closure is probably a gradual, horizontal closure[12]. Apparently, no bed protection was required!! In this respect three comments can be made:

- Dibbits (1950, 57) states that "north of the line Alkmaar- Hoorn the soil comprises predominantly sea clay". Though Scharwoude is situated just south of that line there still could be an erosion-resistant subsoil.
- An effort to make hydraulic calculations using CRESS is not possible because 'the parameter is outside the indicative range'. But given the large area (253.5 km^2) and the modest tidal range (32 cm[13]) one can assume a constant level of NAP – 0.16 m in the flooded area; maximum current velocity during an average tide will not be more than 1.8 m/s and that only during a short period.
- Another possibility is to vary the input parameters in CRESS in such manner that a fair impression is obtained of the value of the output parameters. When the input parameter A (= area of tidal basin) is outside the indicative range the highest acceptable value is adopted (marked in red).
  Input and output parameters for the Coastal and River Engineering Support System CRESS have already been defined in Section 9.2 to which reference is made.

For the average tidal range of 32 cm the values for U_{max} do not vary much and this is due to the large size of the tidal basin which implies a nearly constant water level around MSL.
The first calculation is for the initial 100 m wide breach. The second and third for the case of the first breached cofferdam having the reported 23 and 54 m wide breaches and depths of 14 and 11 m. Then follow calculations for a gradual narrowing of the flow gap and two different depths. Finally, H_z is varied to see what happens in that case with the three output parameters.

Table 9.4 Tidal calculations for various values of h and B by using CRESS

INPUT						OUTPUT		
H_z (m)	c	T (h)	A (km^2)	h (m)	B (m)	H_B (m)	U_{max} (m/s)	Δh_{max} (m)
0.32	0.90	12.5	99	9	100	0.20	1.41	0.13
			253	14	77	0.10	1.55	0.15
			218	11	77	0.09	1.56	0.15
			198	10	100	0.12	1.53	0.15
			139	7	100	0.12	1.54	0.15
			198	10	50	0.06	1.58	0.16
			139	7	50	0.06	1.58	0.16
			198	10	25	0.03	1.59	0.16
			139	7	25	0.03	1.59	0.16
			198	10	10	0.01	1.59	0.16
			139	7	10	0.01	1.59	0.16
0.50			178	9	100	0.15	1.94	0.24
1.00			178	9	100	0.22	2.78	0.49

[12] It is also possible to complete the rows of piles first and then start filling the spaces between the three rows over the full length of the cofferdam from below like a vertical closure.
[13] According to the source given for Fig. 9.4, the tidal figures in Fig. 9.4 are also valid for Hoorn, which is near to Scharwoude. For Hoorn in 1675/1676 MLT = NAP – 0.32 m and MHT = NAP.

9.5 Breach in the Lekdijk Bovendams near Wijck in 1747[14] (project no. 39)

So far, no breaches in river dikes have been discussed and also the breaches to be discussed in the next sessions will not be about river dikes. The reason is simple: when a river dike breaches, normally one waits till the river has gone down and the water has left the flood plain (*uiterwaard*). Subsequently, in nearly all cases a new dike section is constructed in the flood plain along the perimeter of the scour hole developed by the flow through the breach. For construction of the new dike section the clay, available in the flood plain, is used and no special materials (like fascines, stakes, piles, sheet piles or stones) or methods are required. Accordingly, this process does not merit any particular attention in this dissertation.

But a breach in the Lekdijk Bovendams[15] is a different case! The Lekdijk Bovendams is defined as the dike along the North side of the river Lek between Amerongen and Klaphek. The importance of this dike for the provinces of Utrecht, Zuid- and Noord-Holland is well known and various hydraulic engineers during the 18th century, like Lulofs, Brunings, Van Bolstra and Van Velsen, have repeatedly told the authorities about the enormous damages a breach in this dike would cause[16].

This is not the place to go in too much detail but a breach within 24 hours could result in the flooding of 170 km^2 and within 10 days of 1 700 km^2 of the most fertile and populated, but also lowest[17], areas of the Netherlands[18].

According to the various experts in hydraulic engineering of the 18th and 19th century, a breach in the dikes along the river Lek can be due to one of the following reasons:

- scour along the toe of the dike by a shifting river channel (assuming such scour has not been prevented by spurs);
- water levels during a river flood locally higher than crest of river dike, resulting in overtopping;
- ice jamming in river and against dike as well as related high water levels;
- undue settlement of crest and landward slope of dike because of the presence of layers of peat and '*welzand*' in the foundation of the dike and in the dike body proper. *Welzand* is also called *loopzand*, most probably fine sand layers between clay layers are meant which, during high river stages, lead to piping.

On 28 February 1747 the Lekdijk breaches just downstream of Wyck[19] (Fig.9.14) at a place called The Noord, due to undue settlement of the crest and slope of the dike (as just mentioned). It is sheer luck that the river flow through the breach is hampered by the bund around a 'summer' polder in the flood plain in front of the breach and by a grove of willows at the landward side of the breach.

In the days that follow the water flows to the city of Utrecht and the surrounding polders through rivers and streams like the Vaartsche Rijn, Cromme Rijn, Minstroom, Vecht and Leidsche Rijn. The water flows through the canals in Utrecht and threatens to disrupt bridge abutments. Many villages and towns are flooded (Woertman, 1747, 44 et seq.). See also Fig. 9.14.

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[14] Most of the information in this Section has been derived from Woertman (1747), Van Hengst (1898) and Van Vliet (1961).

[15] A distinction is made between the Lekdijk Bovendams and the Lekdijk Benedendams. They have been separated by the damming of the Hollandse IJssel at its mouth at Klaphek in 1285. Together they have a length of 49.4 km between Amerongen and Schoonhoven.

[16] This is a summary of the statement made by the *Commissie tot onderzoek der beste rivier-afleidingen* (Commission for the study of the most suitable river diversions) in their Report (1827, 182) to H.M. King Willem I. In fact the Commission saw it as its task to, again, elaborate on this matter. It did this also to justify the large amounts of money which would be needed to enlarge the safety of the dikes along the river Lek.

[17] Some of these polders have a ground level of AP – 5 m.

[18] Report Commissie…. (1827, 183).

[19] Wijk bij Duurstede.

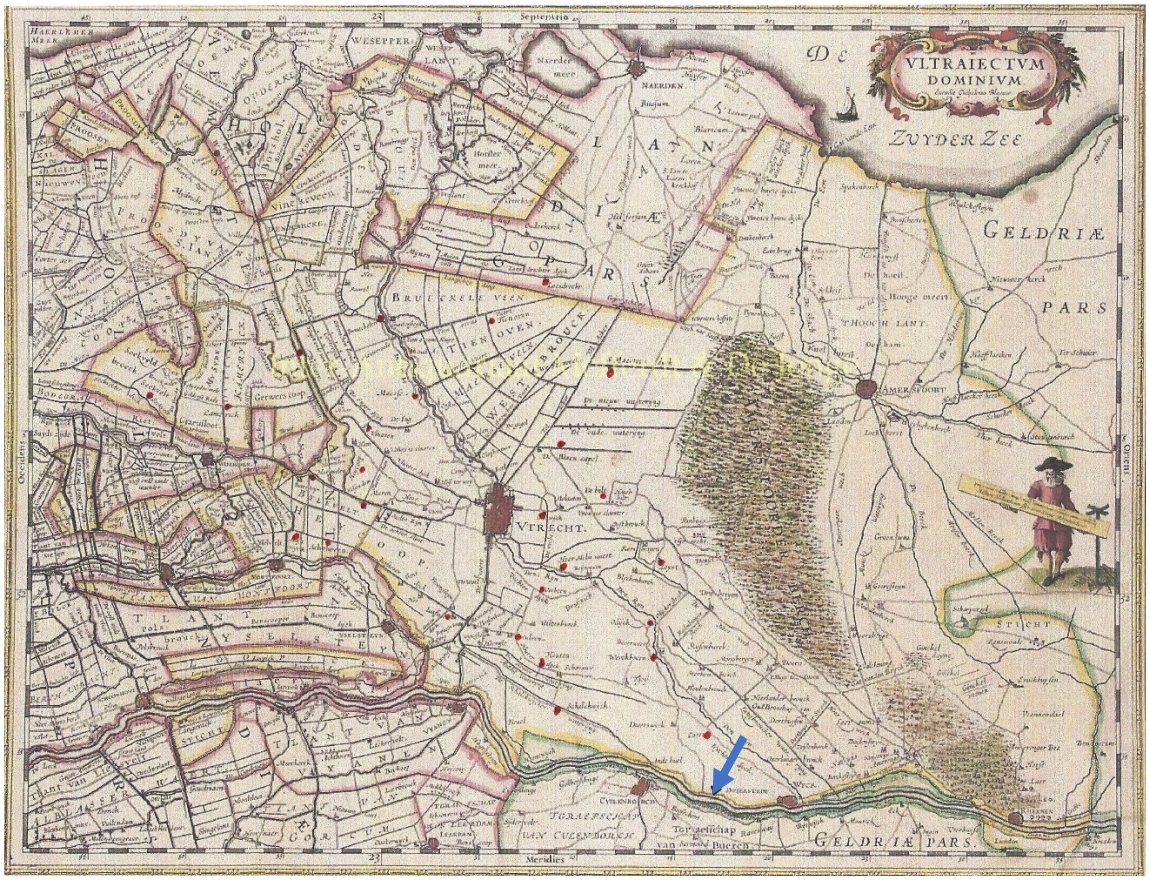

Fig. 9.14 Map showing the location of the breach in the Lekdijk in 1747; some of the towns and villages flooded have been marked by a red dot; source of map: Willem Blaeu (1635)

The breach and its closure by a cofferdam as well as the design of the new dike, constructed in 1747, has been described in Woertman (p. 63 – 66).
The length of the breach is 11 rods[20] (55 m). A scour hole (*wiel*) develops having a depth of 36 to 42 feet (10 to 12 m).
Works start on 6 March (i.e. one week after the breach occurred) by constructing a semi-circular perimeter bund (partly constructed as a cofferdam) having a length of about 195 rods (980 m). This bund is placed partly on the (remains of) the summer dike enclosing the aforementioned summer polder in the flood plain and partly directly on the flood plain proper as a cofferdam. Accordingly, its height varies between 5 and 8 ft. (1.40 and 2.25 m). Its crest width is 1.1 m for the low part and 1.4 to 1.7 m for the high part. First, piles (long 4 or 5 m), at an equidistance of 1 m, are driven in two rows. At regular distances at the river side bracing-piles are placed (Fig. 9.15) while bracing-piles are stored for use at the landward side in case water levels in the river increase again. For the functioning as a cofferdam hurdles are placed while the actual filling is by clay from the flood plain. Against leaching of the clay, wet hay is applied. The cofferdam is ready on 25 March. Apparently, the works have started when the river Lek had gone so far down that the nappe of water over the crest of the summer dike has a thickness of only 25 to 50 cm. On 10 March parts of the crest are already dry while on other (scoured) parts the nappe has a thickness of 40 cm. In any case on 11 March the nappe seems to be only 10 to 15 cm thick (this conclusion is based on the text in Woertman, p. 53 and 56). That a cofferdam is being built, instead of waiting for the river to leave the flood plain and

[20] Here the 'Lekkendijk' rod is meant which is 5.035 m and is divided in 18 feet (of 0.2797 m) while a foot has 10 thumbs (*duimen*) of 2.79 cm.

then start straight away on constructing a new dike, is because one is afraid that a spring flood will cause new flooding. A cofferdam, similar to the one just described is shown in Fig. 9.15.

Fig. 9.15 Cofferdam built for the breach at the Spees in 1855 (southern bank Nederrijn near Opheusden); source: Atlas van Stolk, Rotterdam, etching made by Herman and Mari ten Kate

A new dike section will be placed between the breach and the cofferdam and it will also follow a semi-circular alignment having a length of 61.5 rods (310 m) which is only 1.5 rods more than the old dike, which' alignment, apparently, was also more or less semi-circular.
Borings show that the new dike will be founded on a layer of very good clay, thick 4 to 4.5 m, which at some places is interspersed by a 30 cm thick layer of sand.
In Woertman (p. 66) the height of the new dike is given as 41 feet which is 11.4 m and that seems to be much too high. Probably, a maximum height of 41 feet is meant where the new dike crosses the river side slope of the scour hole.
In November 1824 crest levels of the Lekdijk Bovendams are measured. The dike at Wijk bij Duurstede has a crest level of AP + 8.98 m (Van Vliet, p. 54). In 1765 and following years crest levels have been increased by 28 to 42 cm. It is therefore assumed that the crest level of the dike at Wijck in 1747 has been somewhere around AP + 8.60 m.
When the flood plain level is at AP + 2 m, the dike has a height in the flood plain of 6.6 m.
Width of the crest of the new dike is 7.5 m and its slopes are 1: 3. There is also a berm at the landward side, wide 2.8 m and high 40 cm.
Fig. 9.16 shows the changes in time of the cross-section of the dike at Vreeswijk.
In a letter dated 7 July 1747 to the *Staten van Holland*[21], the three waterboards (*hoogheemraad-schappen*) of Rijnland, Delfland and Schieland state that, because of the breach of 1747, the water "was up to their necks… (and) a complete ruin had to be feared".

[21] Resolutiën Staten van Holland, 11 July, 1747, p. 409.

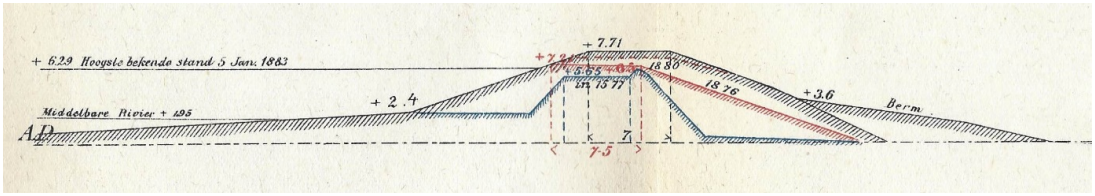

Fig. 9.16 Changes in time of profile of Lekdijk near Vreeswijk; source: Beekman (1911, cross-sections of dikes)

They also note (in a letter dated 14 April 1747[22]) that during river floods in 1744 and 1747 the water levels in the Lek were 18 feet (5.65 m) higher than the ground level in the reclaimed areas of Rijnland. As already stated above, during the years 1765 – 1768, the dike profile has been improved and the responsible authorities have tried to maintain the revised dike profile until 1828 (Van Vliet,1961, 54). But the Report of the Commissie (1821) shows that, once again, measures have to be taken to safeguard the lowlands of Utrecht and Holland. A further discussion on this topic is outside the scope of this dissertation. It is only recalled that, because of bureaucratic delays and disagreement on technical issues and 'who should pay', the required works have only been realized in the period 1880 – 1881 (Van Vliet, p. 54 – 62).

9.6 The breach at the Nieuwerkerkepolder in 1807 (project no. 40)

During a storm surge on 30 Sept – 1 Oct. the new sluice under construction is pushed out of the dike of the Nieuwerkerkepolder (island of Walcheren). This has probably been caused by overtopping of the bund around the building pit by the high tide during the storm surge. As a consequence the polder is flooded and a flow gap is created having a length of 28 m and an average depth of MLT – 0.95 m (max. depth at MLT – 1.25 m).

As the waterboard of this polder does not know how this breach should be repaired they ask the *inspecteurs van 's Lands Waterstaat* in Zeeland A. Schraver and A. Dingemans for advice. In fact, Schraver takes the lead and all considerations, as discussed below, about design, cost estimate and construction method of the closure works, are his.

Because of the storm surge the waterboard of the polder Walcheren cannot provide materials for closure as they need these themselves and the *inspecteurs* send a message to Werkendam asking for 25 boats (having each a capacity of 2.2 m^3)[23] for transport of earth fill (so-called *vletschuiten*). The crews on the boats should be skilled *rijswerkers* (artisans skilled in fascine works). The *Landdrost* of the Kingdom Holland in the department Zeeland is asked to commandeer three ships transporting fascines and the like (when they pass the fairway Zijpe) and sufficient ships loaded with stones from Vilvoorde, when they pass the fort at Bath.

Schraver starts his plan for closure[24] noting that the polder does not have a topsoil layer of firm clay or peat (like other polders in Zeeland) but sand instead, interspersed by thin layers of shells. And one finds these sands not only at the level of MLT[25] but also higher up (2 m or more). In this situation it is better not to act in a hurry. The best time for closure is when a certain equilibrium has developed between dimensions of the breach and ebb and flood flows. In fact the polder will then run dry during Low tide.

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[22] Report Commissie… p. 194

[23] In fact a capacity of half a *Schacht* implies 72 cubic 'Rijnland' feet = 72 x $(0.314)^3$ = 2.23 m^3.

[24] *'Consideratien van den Geadmiteerden Landmeter A. Schraver, hoedanig volgens zijn gedachten, de Doorbraak in den Dijk der Nieuwerkerker Polder kunstmatig, spoedig, solide en minst kostbaar zoude kunnen worden gesloten en de Geinundeerde Landen ververscht.'*

[25] The tidal range is not given but, according to Storm Buysing (1854, 578) it is 3.4 m at Arnemuiden. This town is situated opposite the Nieuwerkerkepolder at the other side of the Arnemuider Kanaal (Fig. 9.17).

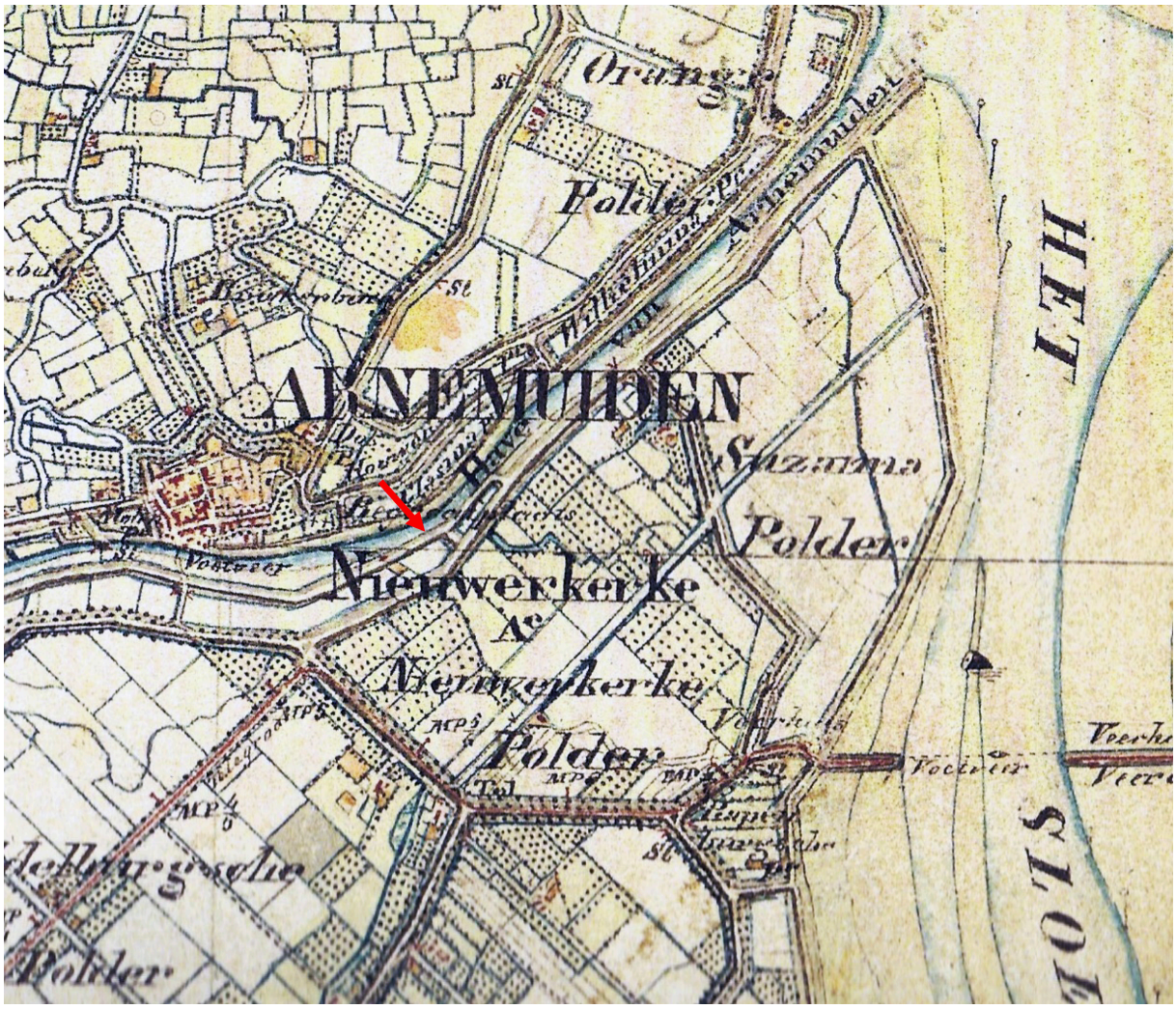


Fig. 9.17 Arnemuiden and the Nieuwerkerkepolder in ± 1857; the Arnemuider Kanaal is here called *Have van* (port of) Arnemuiden; the breach has been marked by a red arrow; source: Grote Historische Provincie Atlas, Zeeland 1856 - 1858

First of all the dike heads must be flattened to slopes of 1 in 5 à 6 (in any case to 1:3) and protected above MLT by fascines on a layer of straw, Below MLT these slopes must be protected by placing *baardwerk*[26]. The *baardwerk* must connect to the first fascine mattress (*grondstuk*) to be sunk in the breach (Fig. 9.18). Also *baardwerk* will be placed on dike heads of breached secondary dikes.

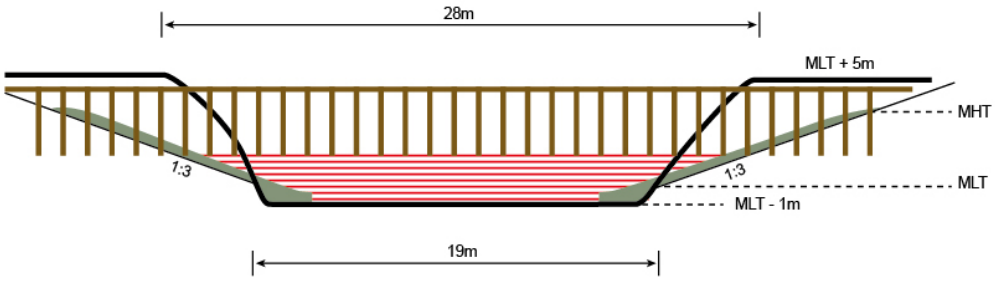


Fig. 9.18 Length profile over breach; fascine mattresses in red, *baardwerk* in green; side view of cofferdam; information based on Schraver's Notes on first plan for closure as filed in NL-MdbZA_361.1_134

[26] Assuming there is no good translation, this word will be used throughout. *Baardwerk* is an 'under water slope protection by fascines ballasted by stones or clay clods'; see Chapter 4, Section 4.6 or Van Breen (1920, 195-197).

The first fascine mattress to be sunk should be not thicker than 45 to 60 cm as it should be flexible enough to follow the uneven bed level of the breach. Then a fascine dam is made up to a level of MLT + 1.25 m. The top level of the mattresses should be lower in the middle of the breach than near the dike heads. Also a layer of clay clods covered by bricks is required at the canal side of breach.

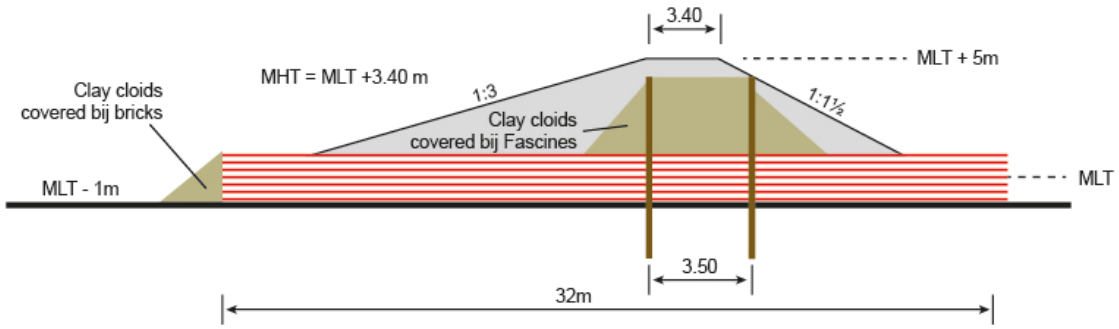


Fig. 9.19 Cross-section over bed protection and cofferdam; source: as given for Fig. 9.18

The breach is now narrowed by constructing a cofferdam (Fig. 9.19) as follows:

- Two or three rows of stakes are driven, stakes at distances of 80 -95 cm. Distance between the rows to be 3 to 4 m. Hurdles to be placed against the rows of piles and cofferdam to be filled by clay, which has to be compacted;
- each time the pile driving is interrupted, the rows have to end by making a semi-circular head.

During Low tide the completed part of the cofferdam must be protected by placing a slope of clay clods against it, covered by a layer of fascines.

The last opening will be closed during a neap tide and good weather. Three or four rows of stakes are driven, hurdles are placed and fill (stored nearby) dumped by a large number of men. Subsequently, the dike is constructed to its full profile (Fig. 9.19). The crest gets a level which is 95 cm higher than the existing dike (higher crest not drawn in Fig. 9.19). Crest level of existing dike is MLT + 16 ft. (= MLT + 5.02 m). Two contractors (from Werkendam and Giessendam) are willing to carry out the work in association with each other and a contract, dated 16 Oct. 1807, for the sum of f 13 000, is agreed.

Table 9.5 Unit rates in contract for repair of breach in dike Nieuwerkerkepolder

description	unit	rate (f)	rate (in € of 2020)[27]
stone from Vilvoorde	last (= 1 926 kg)	7.00	56.35
fascines from Gelderland (*Gelders rijs*)	100 bundles	7.10	57.15
stakes	100 pieces	1.16	9.34
hurdles	,, ,,	40.-	321.98
stone baskets	piece	5.00	40.25

Schraver has to leave for Schouwen-Duiveland[28] on 19 October. But prior to his departure he tells the Contractor to construct the works strictly in accordance with the specifications and written instructions given by him. However, the Contractor ignores that instruction and starts by placing fascine mattresses and *baardwerk* up to a level of MLT + 1.60 m without having flattened and protected the dike heads against scour by the currents. As a consequence the breach starts to widen and on 28 Oct. part of the *baardwerk* in the centre of the breach is washed out.

[27] For the transfer of guilders of a certain year into euros of the year 2020 the website: https://iisg.amsterdam/en/research/projects/hpw/calculate.php has been used.

[28] Because a serious flow slide had occurred in the sea dike at Flaauwers.

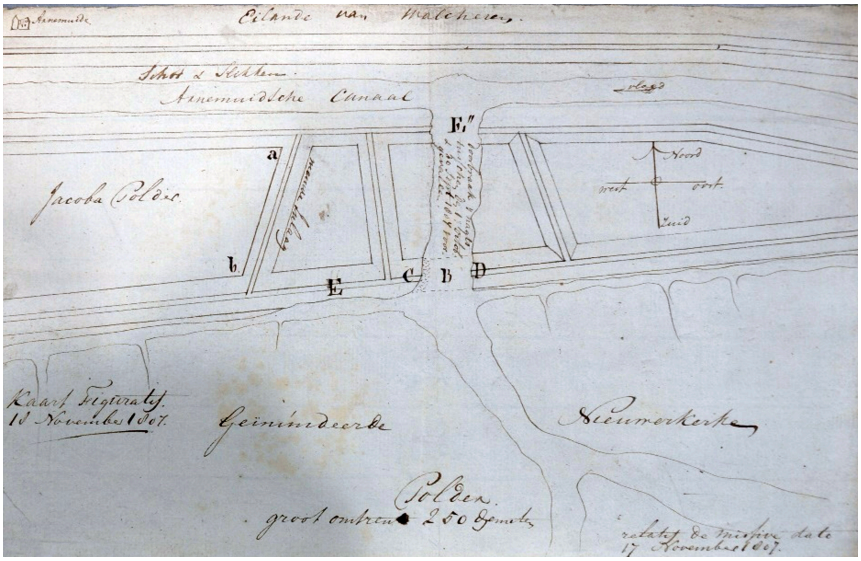


Fig. 9.20
Figurative Map of the breach in the dike of the Nieuwerkerkepolder bordering the Arnemuider Kanaal; source: NL-MdbZA_361.1_134

Subsequently, a storm surge on 29/30 October (springtide) destroys most of the fascine dam already constructed. The breach now has a length of 43 m (was 28 m) and its depth varies between MLT – 1.5 to – 3.5 m (average MLT – 2.5 m, was MLT – 1 m on 19 October). At the level of MLT the breach has a length of 23 m (was 19 m).
Schraver returns from Schouwen-Duiveland on 7 November and makes a new plan on the next day:

- first the dike heads to be protected against scour by heavily ballasted *baardwerk;*
- then fascine mattresses to be immersed from both dike heads sloping down to the centre of the breach; these mattresses to be heightened by *baardwerk*, but not higher than MLT – 1 m; Schraver expects siltation at both sides of this sill;
- sandy dike heads must now be flattened to a gradient of 1: 3 and provided with a good clay berm 60 cm deep and 1 m wide;
- subsequently, *baardwerk* from both sides to the centre of the breach sloping down until 4 m is left of the sill at MLT – 1 m;
- the next operation is the construction of the cofferdam, earlier mentioned, until a closure gap is left, long 19 m (5 rods);
- final closure can now be carried out, provided four conditions are met:
  - a large stockpile of clay clods in the direct neighbourhood of the closure gap;
  - a day of neap tide;
  - good weather;
  - east wind.
- in order to expedite the closure works, the contractor is permitted to construct a seaward slope of 1: 1½ instead of 1:3, provided the slope last mentioned is constructed in the spring of 1808. He is instructed to adhere fully to the design of closure works as now has been made and not to listen to anybody who has different ideas.

The last arrangements with the contractor are made on 13 November. The final closure is carried out on 23 November. At that day the sill made of fascine works (mattresses and *baardwerk*) has reached a level of MLT + 1.5 m. To close the last gap (19m wide), up to a level of MLT + 4.0 m, a volume of 320 m^3 of dike fill has to brought in and placed within one tide (NB: it is regretted that nowhere is mentioned how many labourers were involved).

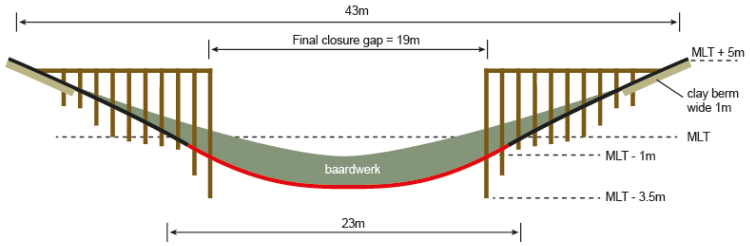


Fig. 9.21 Length profile over breach after storm surge; fascine mattress in red, *baardwerk* in green; side view of cofferdam and closure gap; information based on Schraver's Notes on the second plan for closure as filed in NL-MdbZA_361.1_134

On the morning of 3 December at 4.30 a.m. the flood reaches a level nearly equal to that of 1 October but it has no negative consequences for the closure works which, at that time, are nearly at full height and profile. On 5 December the seaward slope protection of fascines can be marked on site.

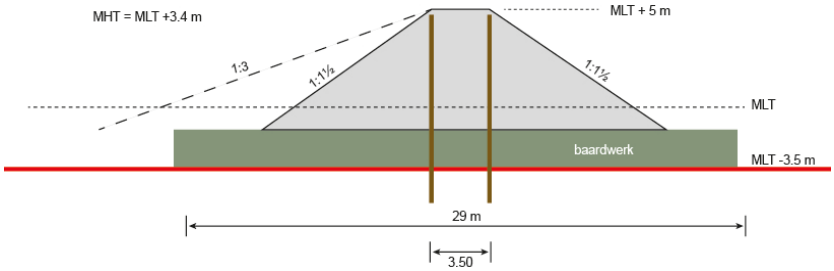


Fig. 9.22 Cross-section over bed protection in centre of breach; Note the temporary seaward slope of 1: 1½; source: as given for Fig. 9.21

The last action of Schraver on site is his recommendation to make the final inspection of the Works when they have passed successfully a springtide at full moon[29].
This successful closure merits a few comments.
First of all, it is surprizing that the fascine mattresses and *baardwerk* are not interrupted by a clay core. The necessity of a clay core is already mentioned by Vierlingh in the 16th century and also Caland (1833) mentions it. The clay core is in any case not mentioned in the closure plan or contract document. But Schraver stipulates that after closure no leakage was observed ,neither through the fascine works nor through the dike fill.
A second comment concerns the tidal range. The tidal range is not mentioned by Schraver and it has been assumed that it is 3.40 m as stated by Storm Buysing (1854, 578). Probably, in 1807 it is less than 3.40 m because this value results, according to calculations made using the Coastal and River Engineering Support System CRESS, in very high current velocities: 3.5 to 4.4 m/s during various stages of the closure operations.
The *Kroniek van* (Chronicle of) Arnemuiden[30] reports for the year 1807:

> "Already during a number of years the town has a great problem, i.e. the silting up of the fairway to the Sloe….At present the situation is such that loaded ships may only reach Arnemuiden during very high tides."

[29] Most of the information in this paragraph has been derived from a letter dated 6 December written by Schraver to the Director General '*over den Algemeenen Waterstaat van het Koninkrijk* Holland' (copy of letter in manuscript filed in NL-MdbZA_361.1_134).
[30] http://www.kroniek.arnehistorie.com/ (accessed on 1 Febr. 2023).

This silting up may well have resulted in lower tidal ranges than those mentioned by Storm Buysing and, consequently, to lower current velocities during the closure operations than, so far, have been calculated.

9.7 Breach in dike Kruininger polder, east of Hansweert, in 1808[31] (project no. 41)

During a storm surge in the night of 14 /15 January 1808 the sea dike, as well as a secondary dike behind it, breach and, as a consequence, the Kruininger polder is flooded (Fig.9.23).

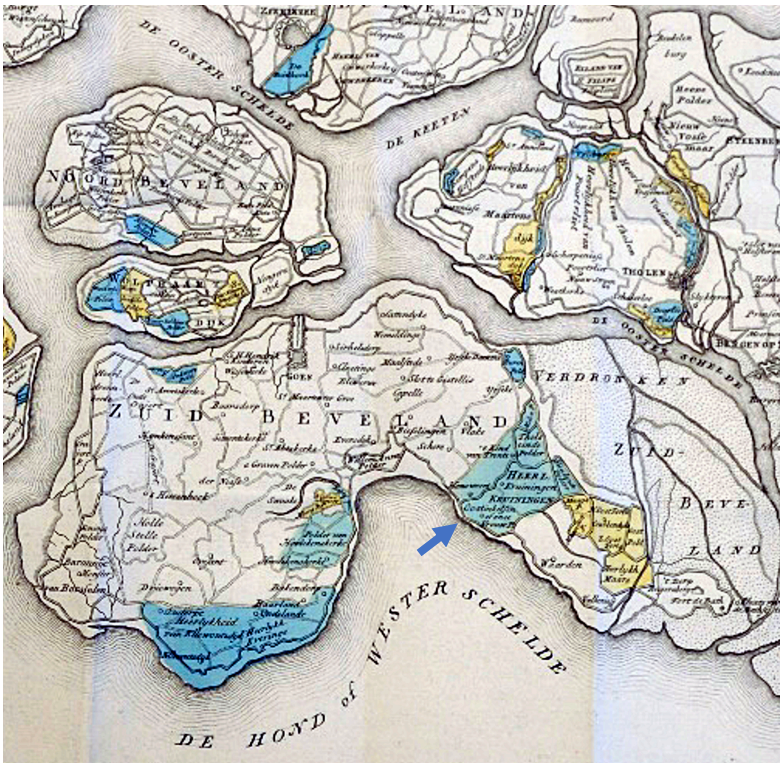


Fig. 9.23
Map of the eastern part of the '*Departement*' Zeeland showing the polders flooded in January 1808, either by breached sea dikes (blue) or by overtopping of dikes (yellow); source: Van Hoek (1808)

Location of breach in dike Kruininger polder shown by blue arrow

The Director- General of Waterstaat visits the breach on 27 February and makes sure that A. Schraver, *inspecteur* in the 11th District of Waterstaat, and at that time busy on the island of Tholen, is present.[32] Schraver is asked to make a design for closure of the breach and he presents it the following day. He is of the opinion that closure of the breach in the sea dike (meanwhile having a length of 250 m) or closure of the breach in the secondary dike (200 m long) is impossible. The tidal range is on average 3.8 m and the polder comprises an area of 1 323 ha, which, at that time, is considered rather large. Schraver notes that, after widening of the breach in March 1808 to 70 rods (265 m), 1.5 ft of water was left in the flooded area during Low tide. This implies a tidal volume for mean tides in the order of 19 million m^3. Jansen (1953, A 399) estimated the flood volume at 20 million m^3 and added that the closure must have been relatively easy because of the high ground level of the polder (MLT+ 1.90 m). Schraver proposes to construct a fascine dam located approximately 125 rods (470 m) behind the breach. The fascine dam would have a length of approximately 300 rods (1 130 m). It is shown in Fig. 9.24.

[31] The information for this Section has been derived from Van Hoek (1808, 214 – 220), De Bruin (1953a, 278 – 280) but especially from a manuscript titled *Waterstaatkundige Onderwerpen in Zeeland* (ca. 1800) and filed in the Zeeland Archives as part of '*KZGW Handschriftenverzameling Koninklijk Zeeuwsch Genootschap der Wetenschappen, 1400-1999*' under number 6785L. The manuscript comprises 11 different documents, two of which originate from Schraver. These documents have, respectively, been written in 1808 and 1812.

[32] Schraver is at that time responsible in his District for the repair of a total of 56 breaches, all due to the storm surge of 14-15 January 1808. By the end of February all breaches had been closed. The breach at Kruiningen was actually outside his District (De Bruin, 1953a, 278).

The fascine dam would comprise a layer of four fascine mattresses combined with *baardwerk* and ballasted using stone and clay clods (Fig. 9. 25). The crest of this fascine dam would be at MLT + 5 m. On top of it the bund would be constructed comprising clay clods (crest at MLT + 5.65 m).

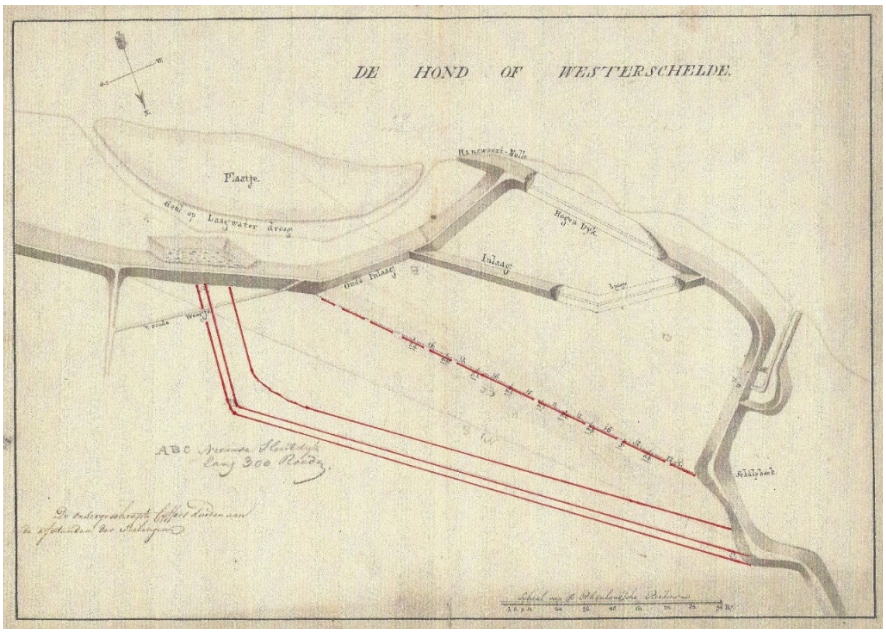


Fig. 9.24 Figurative map of the breaches and the alignment of the projected new dike (in red) in the Kruininger polder in 1808; the dotted line in red indicates the alignment of the rear dike *(vorskade)* protecting the borrow pits for dike fill against flooding during the construction period; source: NL-MdbZA_KZGW_6785L_024

This bund would be high enough to stem tidal levels during the summer when the actual dike would be constructed (crest at MLT + 8.15 m). In this respect the following water levels are given:

- MLT = 0; LWS = MLT - 0.65 m; MHT = MLT + 3.75 m; HWS = MLT + 4.40 m;
- during the storm surge of 14 /15 Jan. 1808 the maximum water level was MLT + 6.30 m;
- ,, ,, ,, ,, 9 October 1808 ,, ,, ,, ,, ,,MLT + 5. 00 m; at that occasion the rear dike breached; the new dike at that time had a level of MLT + 6.30 m;
- during the storm surge of 7 December 1808 a max. water level of MLT + 5.55 m was recorded.

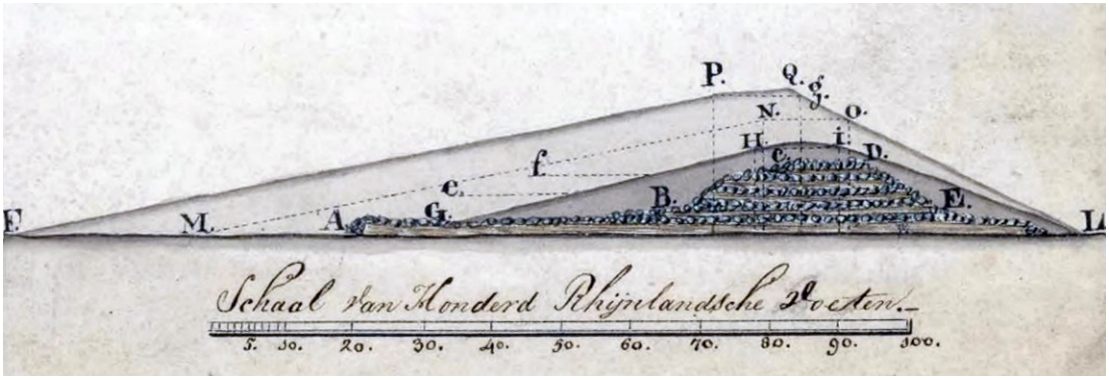


Fig. 9.25 Cross-section of fascine dam and final dike profile as designed by Schraver; source: NL-MdbZA_ KZGW_6785L_021

The four fascine mattresses are of different widths (vide Fig. 9.25) but all cover apparently the full length of 1 130 m. This implies an overall area of fascine mattresses of 102 000 m^2. This is

comparable to the area of bed protection (excluding the fascine works in the two main tidal channels) at Schelphoek in 1953[33] (one layer only over a total length of 1 800 m): 100 452 m^2.

The closure works are tendered in The Hague on 24 February and a combination of four contractors is awarded the contract for f 520 000. Ultimately, the Contractor receives a bonus payment of f 4 000 for early closure and f 18 000 is deducted for a reduction in quantities (bricks). Final construction cost is then f 506 000[34]. This represents € 3 937 420 in euros of 2020 which is € 3 485 per lin. metre.

The works start by the middle of March and the fascine works are progressing such that, 2½ month later, on 1 June, two layers of mattresses have been placed in the alignment of the dam over its full length, i.e. 70 000 m^2. It is a day of a very low neap tide. The lowest point of the dam is now at MLT + 3.00 m and the water level at the sea side does not reach that level. During the following days the dam gets steadily higher and the tide does not enter the polder anymore. On 12 July the level of MLT + 5.65 m is reached and it is considered feasible to open the sluice in order to start reclaiming the flooded area.

Fig. 9.26
The new dike at Kruiningen under construction in the spring of the year 1808; left in the background the breach in the sea dike;
source: Atlas van Stolk, Rotterdam

In the texts written by Schraver (referred to in footnote 31) two items are worthwhile to mention:

- Schraver estimates an ebb flow, having a height inside the polder of ± 1.25 m, at a distance of 190 m (50 rods) from the breach, to have a current velocity of 3.15 m/s. This is during MLT at sea and the fall in the breach is actually a little over 3 m. Near the fall the current velocity will be that over a clear weir:
  $u = (2.g.\ 1/3.\ h)^{½}$ in which u = current velocity, g = gravitational acceleration, h = water depth[35]; accordingly, u = 2.85 m/s. But at 190 m distance it will be less because of a larger flow profile.
- In his document of 1812, Schraver adds the cross-section presented in Fig. 9.27. In the text on the drawing he explains, inter alia, the measures taken and to be taken to prevent, or in any case minimize, the seepage through the dike. Also here, a clay core in the centre of the fascine dam is missing (as was the case in his closure design for the breach in the Nieuwerkerkepolder, referred to in Section 9.6). The triangle BMN marks a *beklamping* (berm of clay) on the heel of the dike to

[33] Van de Velde & Gerritsen (1955, 40/41).
[34] National Archives, Toegang 2.16.06, Archives of Inspectors and Commissions of Waterstaat in the Netherlands, 1797 – 1849; Inventory 449, Documents concerning sea defences and other works at Kruiningen 1808; '*Pro memorie*' document , dated 14 July 1808, signed by J. Blanken Janszn.
[35] As much as possible the symbols are utilized , listed in Notation in CIRIA, CUR & CETMEF (2007, xxvii – xxxv).

lessen the seepage but is was to no avail. Then, in 1811, a trench OPQR is dug at the seaward side[36]. The excavation cuts through the fascine mattress and the trench is backfilled with good clay but the seepage did not stop. In 1812 it is planned to construct berm WXYZ on the landward slope to check the seepage. It is not stated whether this was successful or not.

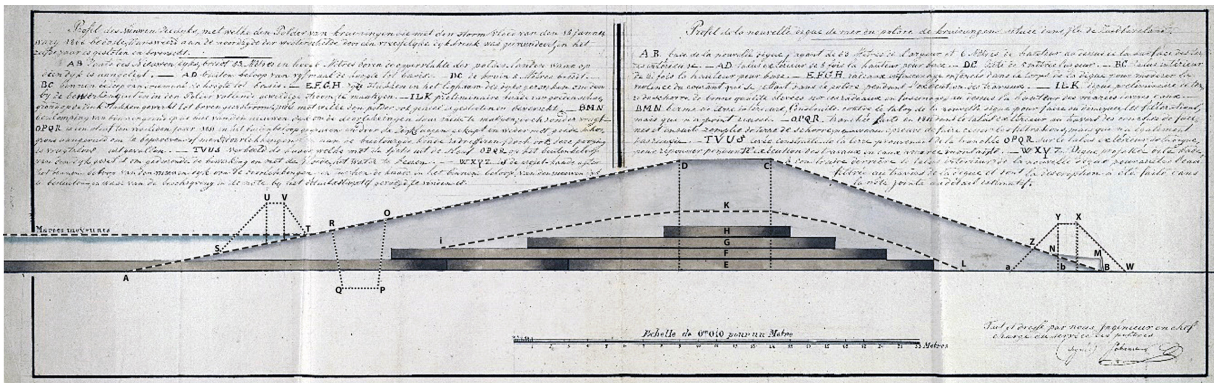


Fig. 9.27 Cross-section of new dike with texts (in Dutch and French) written by Schraver about various measures taken or to be taken against seepage; source: NL-MdbZA_KZGW_6785L_022

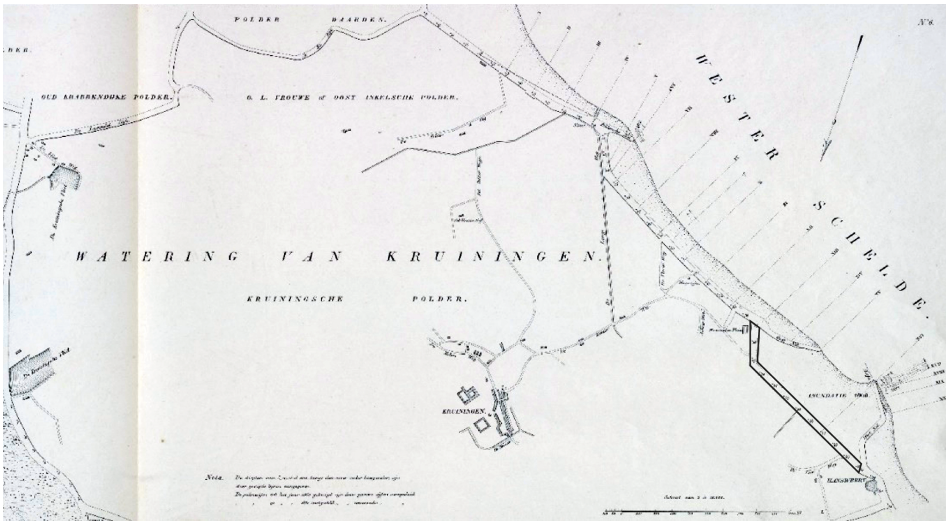


Fig. 9.28 Map of Kruininger polder (1860); the new dike is shown below right; source: NL-MdbZA _294_1110

9.8 The failure of dikes of the 'English' polders in the period 1853 – 1863 (project no. 42)

In the foregoing sections a number of breaches in dikes were discussed as well as the various methods used to repair these breaches. But the failure of the 'English' polders[37] is a different case. It has been described in great detail by Chamuleau (2014, 56 – 68) and he has also, in so far as the available information permitted, mentioned the reasons for the failure. For this information he could make good use of the report made by Ortt[38] (1853). Below, focus will be on the reasons for failure.

[36] The excavated earth serves as temporary bund TVUS keeping the tide out of the trench.

[37] The 'English' polders are now called the Bathpolders and are situated in the north-east part of the island Zuid-Beveland.

[38] Jhr. J.R.T. Ortt (1817 – 1887) was a very competent engineer of Rijkswaterstaat. He worked for this organisation during the period 1840 – 1881 but he got leave of absence in the year 1853 as he was appointed director of the 'Netherlands Land Enclosure Company' on first of May of that year. The Company was in the process of reclaiming polders in the Oosterschelde near Bath. Ortt turned out to be very critical about the manner in which this Company carried out these Works. (continued on page 201)

As such this section could also have been part of Chapter 7 as it concerns (failed) reclamation schemes. It is specifically discussed because of the many mistakes made. The information presented below has mainly been derived from Chamuleau (2014) and Ortt (1853) & (1854).

As already stated in Section 8.3 of Chapter 8, in 1846, a contractor in Zeeland, Dirk Dronkers, obtains a charter to construct a railway Vlissingen-Middelburg-Maastricht. This would imply dams and/or bridges to cross the Sloe and Oosterschelde (Kreekrak). In 1849 the charter is revoked and a new one given for the damming of the Oosterschelde (Kreekrak), but now a navigation canal through the island Zuid-Beveland has to be constructed before the damming of the Kreekrak may start. As compensation for the additional cost, Dronkers retains the right to reclaim 14 000 ha salt marsh/mud flats, situated in the flooded part of Zuid-Beveland and elsewhere in the Oosterschelde, as well as a long lease (99 years) for these reclaimed lands. Dronkers then forms an English- Belgian-Dutch company named the Netherlands Land Enclosure Company[39] (deed executed on 11 May 1852), here after called the Company.
The works will be constructed by an English joint venture of contractors under English – Belgian supervision. As 2/3 of the funds will come from England, the English take the lead. The Company has to reclaim 14 000 ha, to construct a navigation canal and to dam the Oosterschelde but its priority is the said reclamation because that is where the profit has to come from. On 31 July 1852 the permission to start the Works is reluctantly given.

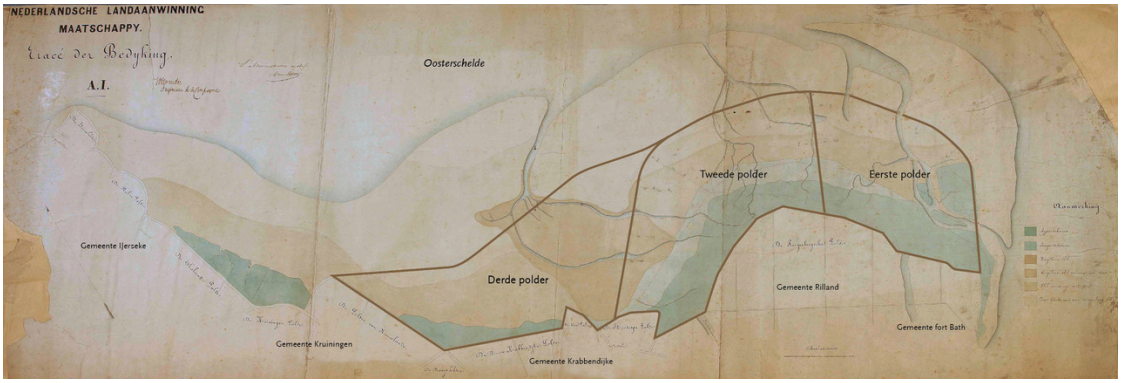


Fig. 9.29 Map of the projected three 'English' polders; source: Ortt (1853)

In fact at that time the Works had already started by constructing a dike for two polders named first and second polder[40] (Fig. 9.29), which, together, would have an area of 1 400 ha. The works start in the summer which is the first mistake made because it is too late to be ready before the storm season starts. Apparently, only the first (most eastern) polder (580 ha) is completed in 1852 but during a storm surge in February 1853 it is completely destroyed. There is no mention at that time about the second polder. That same year repair works start in July (again: three months too late) by approximately 2 000 labourers. During the first autumn storm surge all dikes of the first polder are partly destroyed and three breaches (having lengths of 35, 130 and 300 m respectively) develop (Fig. 9.30). Two of these breaches are at the sites of tidal channels. For the closure of these tidal channels, fascine mattresses had been sunk without a proper clay cut-off in between (such as shown

Note 38 continued: ….He therefore left the Company early 1855 and requested Rijkswaterstaat to be employed again. During his employment by Rijkswaterstaat he worked on many important projects and when he early retired for health reasons, he got the rank of Chief Inspector, which at that time was a very high rank in that organisation. N.B.: The information presented here has been derived from Blok & Molhuysen (1912, 1031 – 1033).
[39] Its name in Dutch: *Nederlandse Maatschappij tot indijken van Op- en Aanwassen in de* Oosterschelde.
[40] There was also a third polder planned (see Fig.9.29) but it has never been reclaimed.

in Ch. 5, Sub-section 5.6.2, Fig. 5.4) which resulted in seepage and piping followed by subsidence of the dike body which in turn led to overtopping during the storm surge.

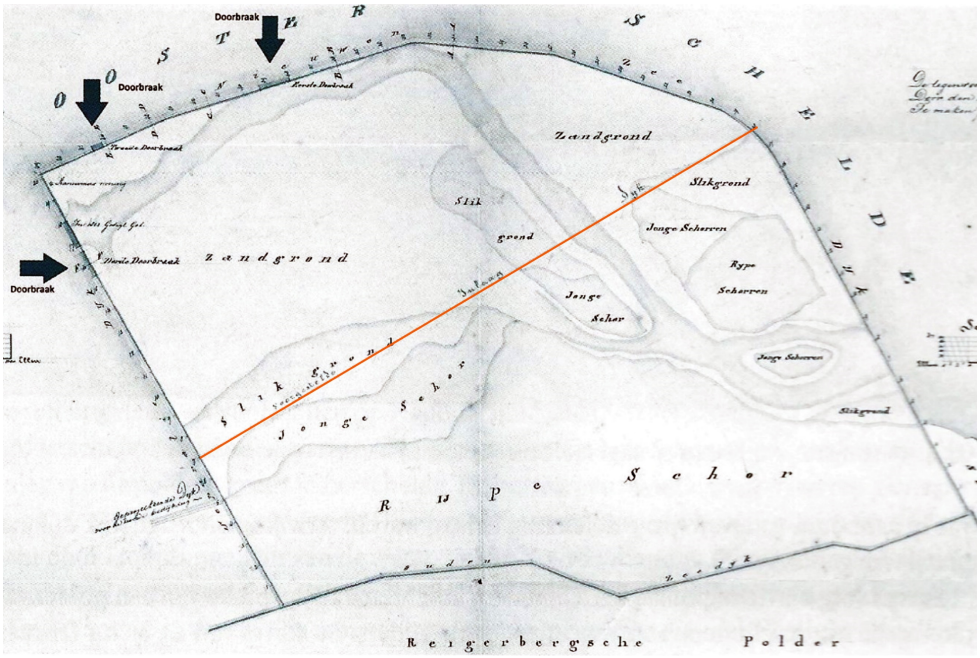


Fig. 9.30 The breaches in the dike of the first 'English' polder in the autumn of 1853; source: Chamuleau (2014, 62)

Ortt, who joined the Company in May 1853, writes a very critical report (dated 7 October 1853). He notes, inter alia, that the landward slope of the dikes did not have everywhere its clay cover while the dike body predominantly consists of sand. Moreover, the crest of the dike was not yet everywhere as high as designed (Fig. 9.31). At the end of the *separatiedijk* (dike between the first and future second polder) the water level at the hour of High tide was 1.8 to 2 m above MHT, Though the crest of this dike is everywhere at its design level (MHT+ 4.00 m) it has everywhere been overtopped by the waves. Ortt is afraid that there are more sections in the northern dike which have a fascine base without cut-off. Moreover, in the present lay-out the polder encloses a large sandy area. He recommends to rebuild the northern dike more to the south, near the edge of the fertile salt marsh (*schor*) (red line marked '*voorgestelde inlaag dijk'* in Fig. 9.30). That would still result in a polder of 300 ha fertile soils. He concludes his report by stating: "the calamities which occurred are due to works not having been made according to sound technical principles and the construction of dikes on low lying sandy soils."

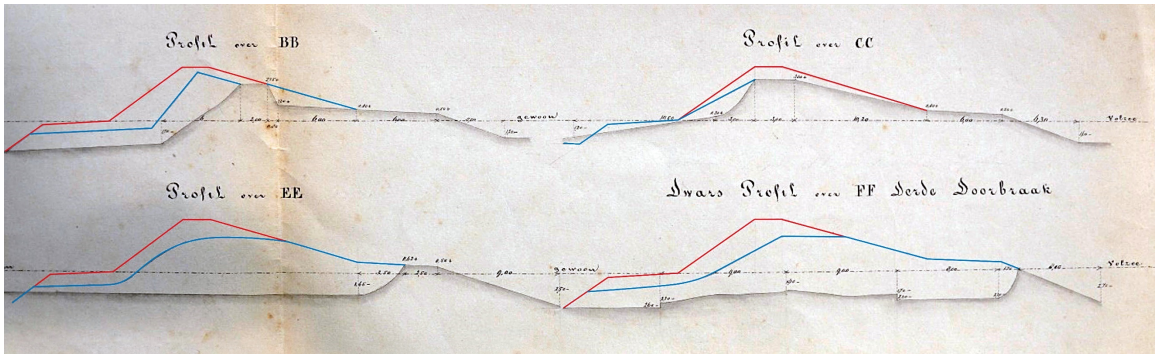


Fig. 9.31 Cross-sections of the dikes of the 1st English polder after the storm surge of September 1853 showing the design profile (red line), progress in construction prior to storm surge (blue line) and the profile after the storm surge (black line); source: Ortt (1853)

In 1854 Ortt writes his second report. He makes an evaluation of costs and benefits which the Company will incur during its actual and future activities: excavation of the navigation canal, damming the Kreekrak, reclamation of 14 000 ha and exploitation of the reclaimed polders. He comes to the conclusion that the Company after 70 years will have suffered a loss of f 1.3 million. Moreover, he is of the opinion that after 70 years not all 14 000 ha will have been reclaimed. Also the estimated cost of reclamation per ha is only f 200 and this is very low. He concludes that the enterprise will be unprofitable rather than profitable. Accordingly, he has decided that he cannot advise to continue with the work.
The works for the reconstruction of the first polder (453 ha) start in September 1855 and, apparently, have to come to a stop in December but, by the end of May 1856, the dike around the polder is closed (Fig.9.32).
If one compares Fig. 9.32 with Fig 9.30 (red line) it appears that the Company did not completely follow Ortt's suggestion but shifted the alignment at the west side some 200 m to the north.

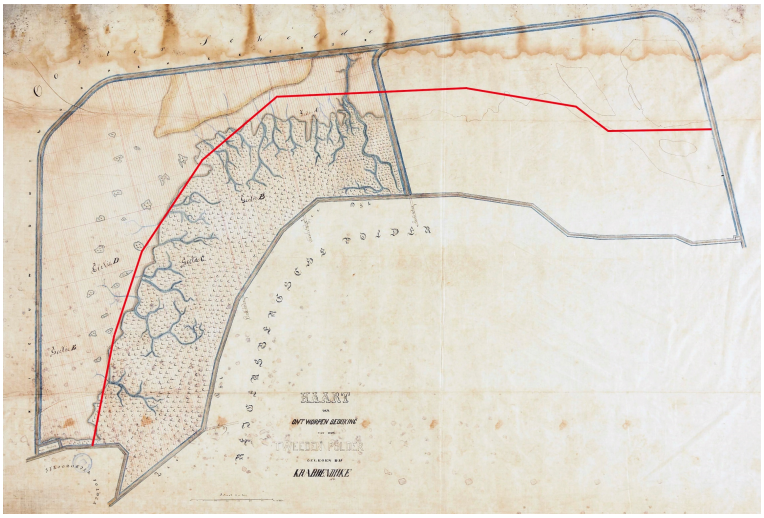


Fig. 9.32
Map showing alignments of dikes of 1st and 2nd 'English' polders as constructed respectively in 1856 and 1857; the red line marks the edge of the salt marsh (*schor*); source: NL- MdbZA_361.3_2050

In March 1857 works start for the second polder. The alignment is also shown in Fig. 9.32. From this drawing it is clear that the mistakes are repeated. Instead of taking as a start the point suggested at the separatiedijk by Ortt (Fig. 9.30, left side), the Company takes a more northern alignment in order to reclaim a polder of 700 ha having dikes crossing low lying sandy soils. The dike around this second polder is complete in June 1857.
Chamuleau notes the following subsequent events (Table 9.6):

Table 9.6 Damages and shortcomings to the dikes of the Bathpolders between 1858 and 1906

date	damage	shortcomings
1 Febr. 1858	breaches in dikes 2nd polder, damage to dikes 1st polder	dikes not constructed as specified in contract documents
8 Dec. 1862	25 % of dikes 2nd polder destroyed	dike condemned by RWS: inadequate profile, berms too narrow
24 Dec. 1863	dike 2nd polder destroyed over 20 m	connection of new dike section to old dike inadequate

Source: Chamuleau (2014, 68, Tabel 4)

In the process of successive repair works, dikes are realigned and the area of the second polder goes down from 700 to 285 ha. The dike section destroyed in 1863 is repaired in April 1864. But It turns out that the Company has difficulties in making the exploitation of the polders profitable.
Meanwhile a law has been passed in August 1861 which comprises a settlement between the Dutch Government and the Company. The latter does not have the obligation any more to excavate a navigation canal through Zuid-Beveland and to dam the Kreekrak. But it loses its right to proceed

with reclamation works in the Oosterschelde to a maximum of 14 000 ha. What remains is the 99 years long lease of both polders already reclaimed (now called the 1st and 2nd Bathpolders) and possible accretion in front of it to a width of 300 m.
During the storm surge of 1906 , both Bathpolders are flooded through a great number of breaches in its dikes.[41] The 1st Bathpolder gets five breaches having an overall length of 635 m. Four of the scour holes created by the storm surge are visible on the map (Fig. 9.34). In one breach (270 m long) the dike body is destroyed completely to below ground level. Also the railway embankment crossing this polder is seriously damaged. There are also numerous breaches in the dikes of the 2nd Bathpolder. The dikes have been damaged everywhere. At various locations the whole dike profile has disappeared. Most of the buildings in both polders are seriously damaged or collapsed.
The Company cannot finance repair works[42] and asks assistance from the Dutch Government. In view of the general importance of the first Bathpolder and in order to prevent a collapse of the railway embankment in this polder, a subsidy is granted of f 250 000 to repair the dikes. The Company decides to sell all its properties to a group of local citizens for the sum of f 175 000. The new owners finance partly (f 25 000) the repair of the dikes in the 1st Bathpolder (in so far as these repairs are not covered by the subsidy) and fully finance the repairs in the 2nd Bathpolder (f 290 000). The breaches in the 2nd Bathpolder are all closed by the middle of June 1906, those in the 1st Bathpolder two months later. On 7 February 1907 the N.V. Landbouwmaatschappij 'de Bathpolders' is founded. Its starting capital is f 614 000 incl. the amounts (f 490 000) already mentioned.

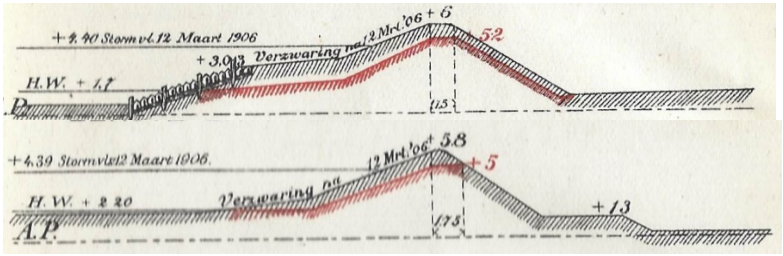


Fig. 9.33 Dike repairs at the 1st Bath polder after the storm surge of 1906;
top: dike along Oosterschelde
below: dike along Kreekrak;
source: Beekman (1911)

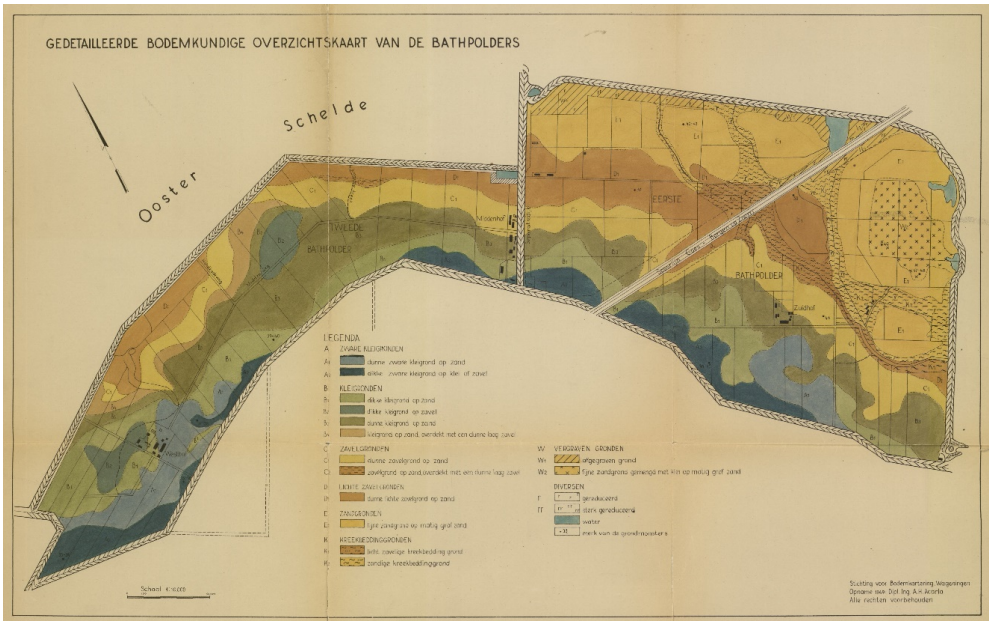


Fig. 9.34 Soil map of the Bathpolders; the sandy soil in the First Bathpolder is clearly visible; source: *Stichting voor Bodemkartering*, Wageningen

[41] Information derived from Rijkswaterstaat (1906, 39 – 40).
[42] Chamuleau states that the Company during the 55 years of its activities has lost f 8.2 million.

9.9 The floodings in the province of Noord-Holland, south of Edam, in 1916[43] [44] (project no. 43)

9.9.1 Introduction

In 1916, during a storm surge of long duration (two tidal cycles, 13/14 January) much water flows into the Zuiderzee while the wind setup aggravates the situation. This results in unusual high water levels along the western and southern coasts of the Zuiderzee and overtopping of sea defences. Many dikes are severely damaged and some of them breach. As a consequence a number of polders are flooded.

Below, only some aspects of the flooding will be discussed for the polders south of Edam: Zuidpolder, Katwouderpolder, polder Purmerland, polder Waterland and the polder Oostzaan.

Initially, breaches in the sea dikes of the Katwouderpolder and Waterland only cause flooding of these two polders. But, due to the modest height of the bunds between the polders and their connections by canals, soon all five polders mentioned are flooded. The overall area flooded, including a few small lakes, comes to some 14 000 ha (Fig. 9.35).

In the next sub-sections the emergency flood embankment, the dike profiles in relation to the flooding phenomenon, the tidal movement and, finally, the repair of the breaches will be discussed.

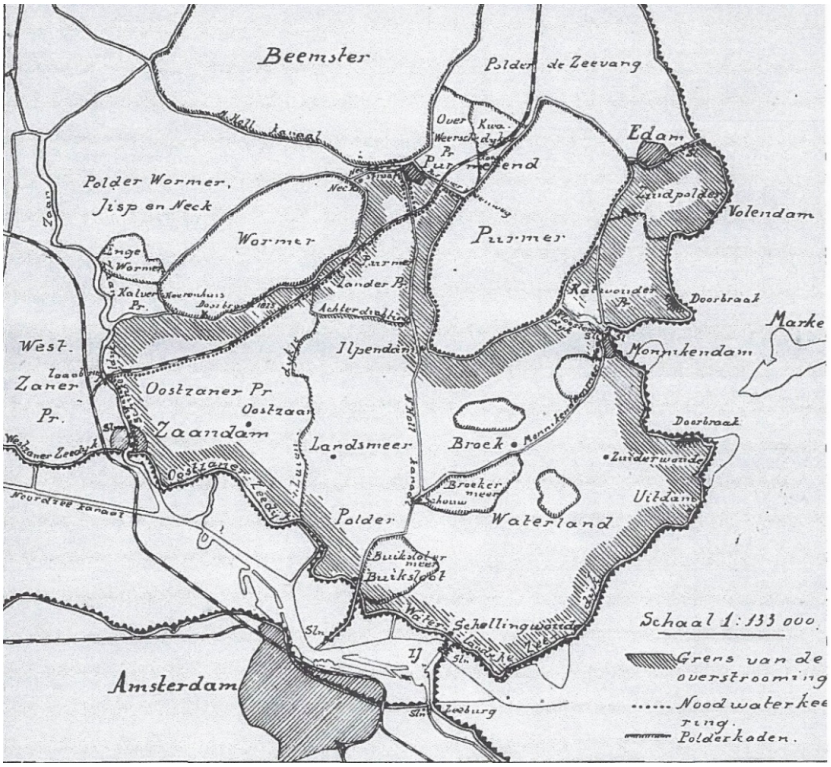


Fig 9.35 View of the flooding of Waterland c.a. on 14 January 1916; the EFE (*Noodwaterkering*) has been marked (by a dotted line) as well as the two breaches (‘*Doorbraak’*); source: Bos (1916, 36)

[43] The last breaching of dikes, discussed in this chapter up till now, was that of the dikes of the English polders in 1853 and 1856 (Section 9.8). It is considered useful to discuss the repair of breaches after the storm surge of 1916 because it shows a change in approach as compared to repair of breaches during the past centuries. Moreover, in 1916 the repair works for the greater part are still done by simple means.

[44] Most of the information discussed in this section has been derived from Bos (1916) and Reigersman (1917).

9.9.2 The provincial emergency flood embankment[45]

After the flooding, following the breaching of the dikes mentioned in Sub-section 9.9.1, the first action required is to prevent a further enlargement of the flooded area. The bunds between the flooded polders and those not yet flooded are in general not much higher than NAP while navigation locks and sluices in connecting canals and drains do not always have gates stemming the water in the right direction. As the tides at Monnikendam are MHT is NAP + 0.22 m and MLT is NAP – 0.28 m, the average level in the flooded area is expected to be around NAP and the emergency flood embankment (EFE) should have its crest at NAP + 1 m or a few dm higher. In all, the EFE gets a length of ± 45 km of which ± 30 km consist of cofferdams. The cross-sections of the cofferdams have to be adapted to the local situation and this implies that they are sometimes too fragile and breach during a storm surge, such as the one of 18 February. Also the dams are not always up to standard as much labour is unexperienced.

The Figures 9.36 and 9.37 show some of the cross-sections of the cofferdams. In Fig. 9.36 (*sub-Fig. 6*) the 'light' construction is a sand-filled cofferdam on an existing dike. Its completion enables the reclamation of the crest of the dike and then the heavy (in black) clay core can be placed, supported by cofferdam and stacked sandbags. But during the storm surge of 18 February the highest water level of the flooding period is reached[46] and the light cofferdam is breached over a length of 20 m at places where the clay core has not yet been placed which in turn results in a second flooding of the town of Purmerend as also, in two other sections, the EFE breaches.

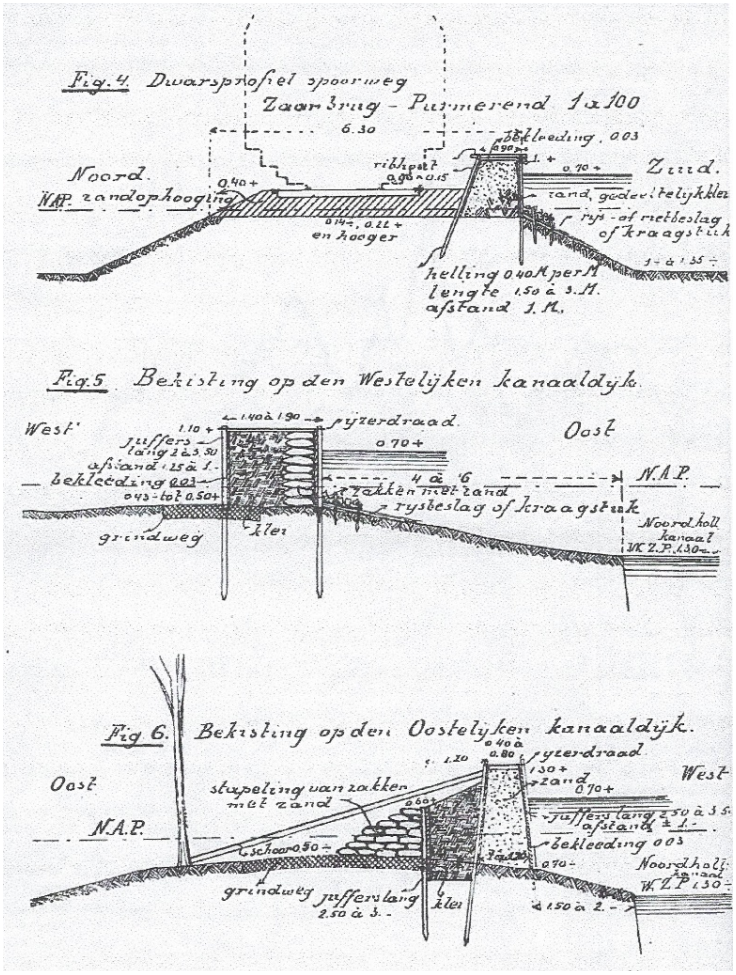


Fig. 9.36
Cross-sections of the emergency flood embankment; the cofferdam in *sub-Fig. 4* has been placed alongside the railway line Zaanbrug – Purmerend; cofferdams in *sub-Fig. 5 and 6* are at both sides of the Noordhollandsch kanaal near Purmerend;
source: Bos (1916, 38)

[45] Known in Dutch as the *Provinciale Noodwaterkering.*

[46] i.e. NAP + 0.70 m at Purmerend and NAP + 0.83 m at Oostzaan.

East of the Wormer, a cofferdam is placed on the Neckerstraat over a length of nearly two kilometres. It is constructed as shown in Fig. 9.37 (*sub-Fig. 12*) (cofferdam filled with sand, i.e. the profile at the right in light colour). It is seriously damaged during the aforementioned storm surge because of its meagre dimensions and a bad provision at the heel against piping. The permeable rubble layer had not been removed prior to construction. The piping results in two breaches, 25 and 50 m in length while the sand fill inside the cofferdam disappears over a length of 350 m. Subsequently, the breaches are repaired as shown in *sub-Fig. 13*. The water flowing across the street is kept at bay by a temporary bund of stone. This considerably lowers the current velocity and allows the placing of sandbags. Within two days after the storm surge a bund of sandbags, having its crest at NAP + 0.50 m, is complete over a distance of 350 m and the danger that new breaches will occur is small. A clay body (dark profile in *sub-Fig. 12 and 13*) is added to the still existing cofferdam.
The whole EFE is complete or repaired by 15 March. The overall construction cost is in the order of one million guilders (equal to € 8.9 million in the year 2020).

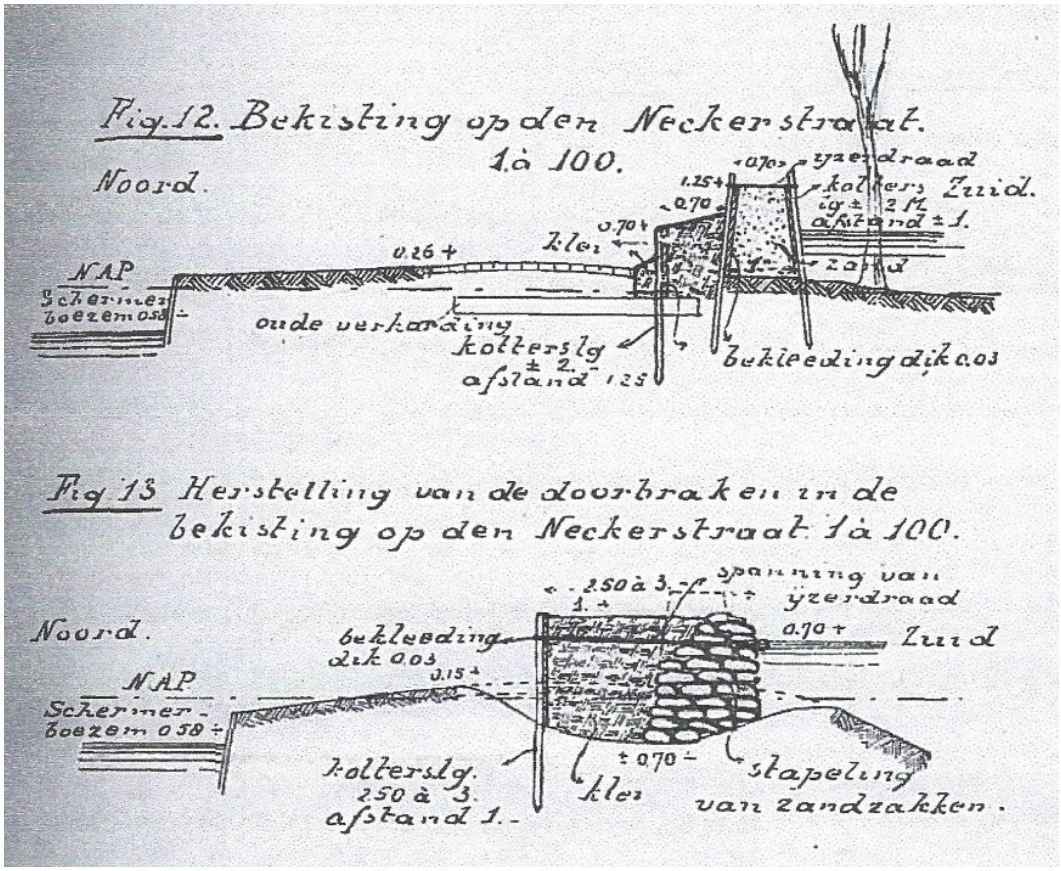


Fig. 9.37 Cross-sections of the emergency flood embankment on the Neckerstraat in Purmerend; after the storm surge of 18 February the 'light' profile in *sub-Fig. 12* at the right is strengthened at the back by a clay body (in black); *sub-Fig. 13* shows the bund constructed at the breached sections; source: Bos (1916, 47)

9.9.3 Dike profiles in relation to the flooding phenomenon

After the storm surge of 12/13 January 1916, an effort is made to establish the causes for the many breaches in dikes along the west coast of the Zuiderzee. Reigersman (1917, 808) admits that one can only answer that question qualitatively: for each breach several causes are possible and nobody knows what cause has been dominant.

He lists the following causes which, individually or in combination can result in a breach:

- the dike is too low and during a storm surge water flows over the dike and weakens the landward slope which starts to slide down;
- the road and its foundation on the dike crest, comprising a one metre (or more) thick layer of rubble and gravel, is permeable and water flows through it to the landward side of the dike;
- the presence of longitudinal vertical cracks in the dike between the rubble/gravel foundation and the clay verges at both sides (as has been observed, vide Fig. 9.41); during overtopping the cracks fill with water and cause saturation of the dike body as well as internal water pressure; these cracks also serve as vertical sliding planes;
- lack of support at the landward side as the slope is steep and a berm of sufficient height is missing; also without such berm, ditches are too near to the heel of the dike.

The dikes built on a weak foundation (for instance a peat layer) tend to settle and this lowers the height of the dike. As the (gravel) road on the dike requires a good foundation, a layer of rubble will be added each time the dike is heightened. In this respect Reigersman mentions the case of the slide in the landward slope of the dike at the Kinsel lake: it happened because the rubble/gravel core went down to NAP + 0.25 m and was connected to the rubble layer under the pitched slope at the sea side.

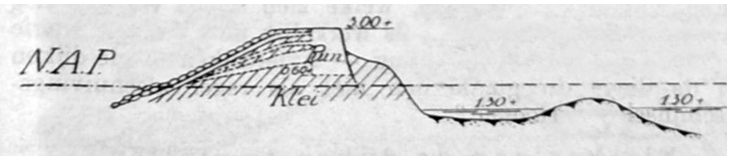


Fig. 9.38
Slide in landward slope of dike in Waterland at Kinsel lake;
source: Reigersman (1917, 807)

Reigersman (1917, 809), in his long article, presents new profiles for many dikes in the province of Noord-Holland. Here, only the new profiles of the sea dikes of the Katwouderpolder and that of the polder Waterland are shown.

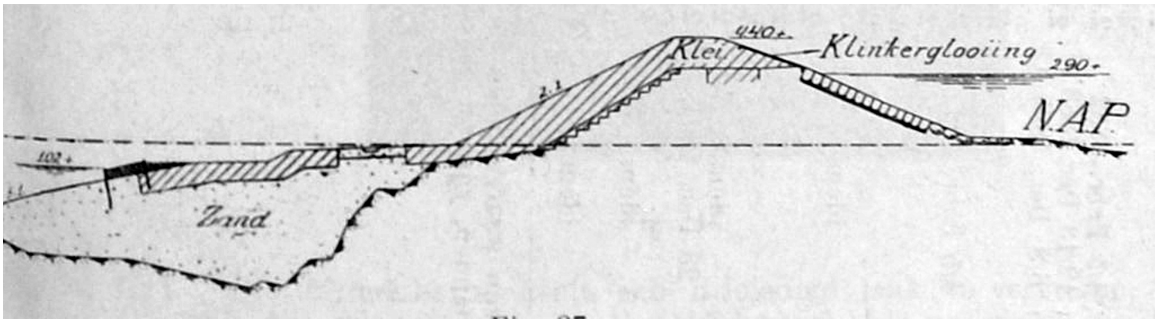


Fig. 9.39 Proposed cross-section of the sea dike of the Katwouderpolder;
source: Reigersman (1917, 809, sub-Fig. 37)

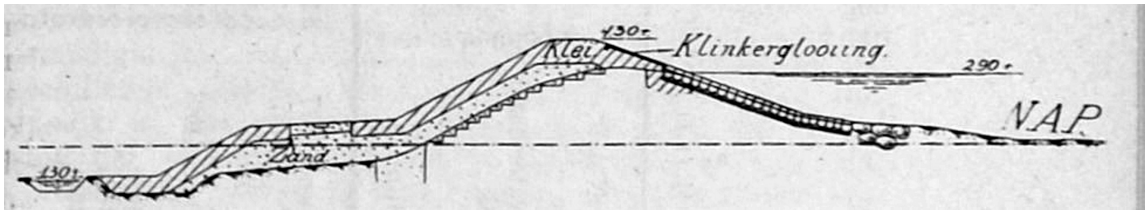


Fig. 9.40 Proposed cross-section of the sea dike of polder Waterland;
source: Reigersman (1917, 809, sub-Fig. 38)

9.9.4 Breaches in the sea dikes and tidal movement during the flooding period

According to Reigersman (1917, 783), during the storm surge the maximum water levels along the coast from North to South vary from NAP + 2.20 to + 2.90 m. The breach in the Katwouder sea dike

has a length of 120 m (excluding two islets left, vide Fig. 9.42). In the sea dike of Waterland over a distance of 2 300 m, eleven breaches develop, having an overall length of 350 m but only one develops as a flow gap, long 47 m.
As the polder Waterland on average has a ground level of NAP – 1 m, during the flooding the head difference over the gap must have been in the order of 3 to 4 m. But because of the quality of the clay in the dike body and thereunder, the gap does not widen or deepen after the initial flooding during the storm surge. Therefore, It is not necessary to protect the dike ends and bed in the breach against scour. The same applies to the breach in the Katwouder sea dike. From Figures 9.43 and 9.44, it can be concluded that depths in the dike axis are 11 (Katwoude) and 7.5 m (Waterland). Length profiles are not given and the depths as shown could be average or maximum depths in the breaches.
By using the Coastal and River Engineering Support System CRESS[47], a few approximate calculations are made. Based on information given already above one can assume:

A = area of flooded area = 14 000 ha = 140 km^2
B = length of gap(s) = 120 m (Katwoude) and 47 m (Waterland)
H_z = tidal difference at sea (Monnikendam) = 0.50 m
Depth of sill h (crest level of clay ridge) in relation to MSL (assume NAP) = 0.50 m (Katwoude)
,, h (bed level of scour hole) ,, ,, ,, =7.00 m (Waterland)
discharge coefficient for a short free overflow weir c = 0.90
period of the tide T = 12.5 h

Fig. 9.41 Cracks in the crest of the sea dike of Waterland polder;
source: Rijkswaterstaat (1916, 52)

[47] For equations used in CRESS , reference is made to Appendix N.

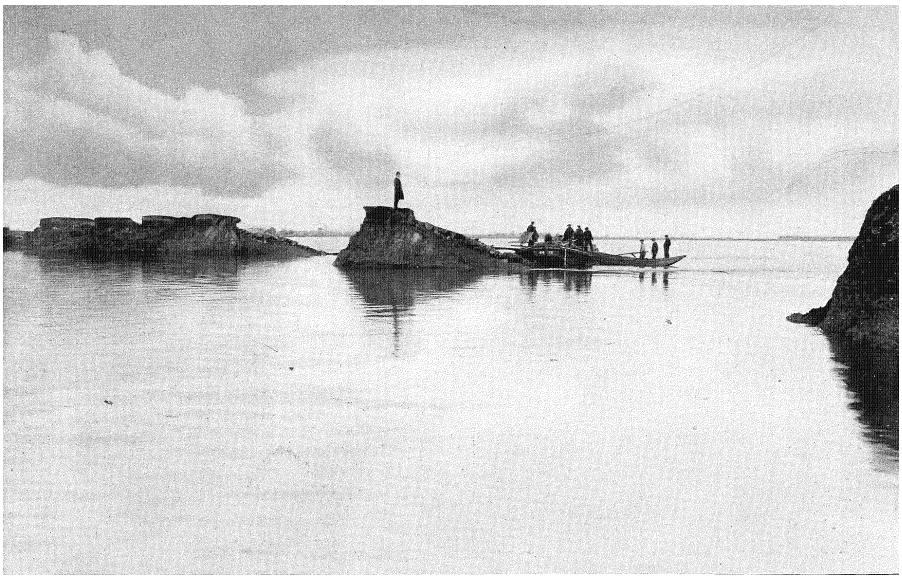


Fig. 9.42 Breaches in the sea dike of the Katwouder polder; source: Rijkswaterstaat (1916, 56)

Now the area A has to be distributed over the two gaps. This is done on the basis of the ratio of the area CS of the cross-sections below MHT.
At the breach in the Katwouder polder h_k = 0.50 m and B_k = 120 m, so CS_k = 60 m^2;
At the flow gap in the polder Waterland h_w = 7 m and B_w = 47 m, CS_w = 329 m^2.
The ratio CS_w / CS_k = 5.48, then Ak = 21.6 km^2 and Aw = 118.4 km^2.
Now the parameters:

- water level difference in the basin H_B (m)
- maximum current over sill U_{max} (m/s)
- max head over sill Δh_{max} (m)

can be calculated.

Table 9.7 Result of calculations made by using CRESS

	INPUT						OUTPUT		
Breach in sea dike:	**H_z (m)**	**c**	**T (h)**	**A (km^2)**	**B (m)**	**h (m)**	**H_B (m)**	**U_{max} (m/s)**	**Δh_{max} (m)**
Katwouder polder	0.50	0.9	12.5	21.6	120	0.50 1.20	0.19	1.91	0.23
Waterland				118.4	47	7	0.09	1.98	0.25

CRESS gives an output for Waterland (as shown) but not for the Katwouder polder. By varying h_k between 0.50 and 2 m it is found that h_k has to be ≥ 1.2 m to obtain an output (in red). Assuming a horizontal water level in the flooded area the tidal volume would be:

$$A \times H_B = (21.6 \times 0.19 + 118.4 \times 0.09) \times 10^6 = 14.8 \times 10^6 \ m^3.$$

The water level variation in the polder would be approximately 10 cm which shows that the effect of the breach in the Katwouder sea dike is much less than that in the sea dike of Waterland. But, just after closure of the breaches, the water level in the polders is at NAP – 0.16 m (Reigersman, p. 794). This is not in line with a water level variation of only 10 cm just prior to start of closure works because in the former case this variation in water level would be in the order of 30 to 40 cm. The last figure is more or less in line with the maximum and minimum water levels in the flooded area near Purmerend and Zaandam as presented in the Tables on pages 66 – 71 of the '*Verslag over den stormvloed van 13/14 Januari 1916*' [48] (Rijkswaterstaat, 1916). In the same Report (p. 56) it is stated

[48] Report about the storm surge of 13/14 January 1916.

that 'In the dike of the Katwouderpolder …three breaches took place having a joint length of 120 m. These breaches had no mud flat in front; the greatest depth was 11 – 12 m.'
From this text It would appear that the cross-section shown in Fig. 9.39 is not representative for the depth in the breach.
Accordingly, a new calculation is made varying the average joint depth in the breaches at Katwoude and in Waterland. The input for Hz, c and T is as earlier established. A = 140 km^2, B = 120 + 47 = 167 m.

Table 9.8 Result of calculations made for various values of h by using CRESS

INPUT	OUTPUT		
h (m)	H_B (m)	U_{max} (m/s)	Δh_{max} (m)
7	no output		
7.5	0.26	1.85	0.21
8	0.27	1.83	0.21
9	0.30	1.79	0.20
10	0.32	1.74	0.19

Given the average tidal difference at sea (Hz = 0.50 m), the water level variation in the flooded area will be in the order of 30 to 35 cm.
Apparently, a current velocity of 1.7 to 1.9 m/s is not enough to cause erosion of the clay in the breach. But also Hoffmans & Verhey (1997, 21 – 22) must be quoted who, in the Scour Manual, state: 'the erosion characteristics of cohesive sediments are not yet fully understood'.

9.9.5 Repair of the breaches

In Figures 9.43 and 9.44, cross-sections are presented showing the repair works. The method of placing a fascine dam comprising fascine mattresses (vertical closure) followed by a pakwerk[49] placed in sections (horizontal closure) on top, is a classic one (see Ch. 5, Sub-section 5.6.2, Fig. 5.4). It is only noted that, due to the First World War, ballast stone is not available and rubble as well as concrete thresholds and lintels are used for ballasting the mattresses. There is also a lack of manpower because of the mobilization.

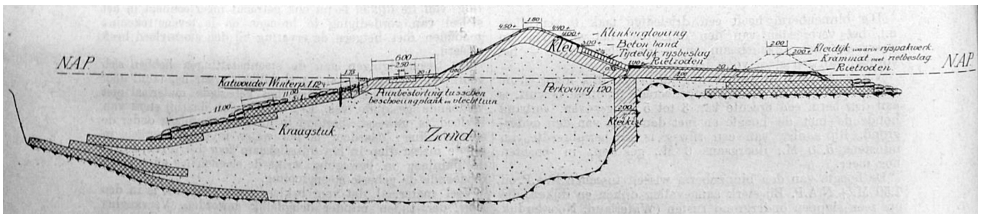


Fig. 9.43 Cross-section of rehabilitated Katwouder sea dike at the location of the 1916-breach; source: Reigersman (1917, 812, Fig. 46)

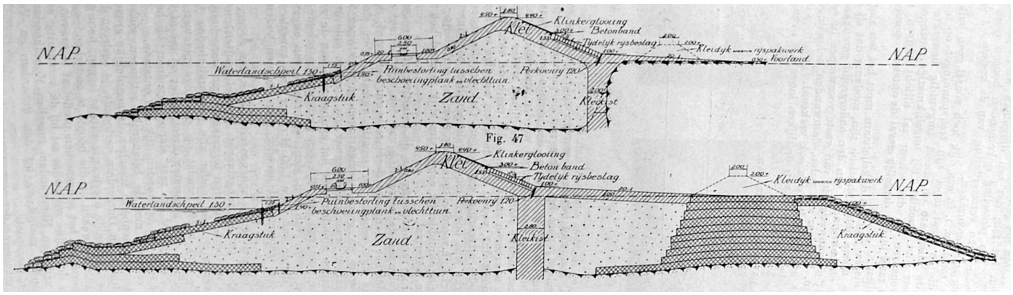


Fig. 9.44 Cross-sections of the rehabilitated sea dike of the polder Waterland (two different magnitudes of breaching); source: Reigersman (1917, 813, Fig. 47 & 48)

[49] In fact the text in the cross-sections reads: dike of clay with a fascine pakwerk inside (*Kleidijk waarin rijspakwerk).*

Apparent in the cross-sections are the abundant use of sand and the clay cut-off (*kleikist*). These are (in 1916) new features in dike design! According to Reigersman (p. 813) the following considerations have led to the use of sand:

- sand easily penetrates, thus improving the subsoil;
- it does not lead to voids during placing and subsidence in the subsoil and it links up very well with other materials;
- it is readily available and can be placed quickly.

But also the local lack of suitable clay may have been a reason for introducing sand. In this respect Reigersman (p. 813) quotes Blanken[50] .

The permeability of sand for water can easily be counteracted by using a thick skin of clay (0.8 to 1 m) on the dike body (Reigersman , p. 813).

The clay cut-offs have been introduced in line with the ideas of ir. V.J.P de Blocq van Kuffeler, who, according to Reigersman (p. 813) suggested these cut-offs in his report about the reclamation of the Wieringer Meer[51].

9.10 Summary and Conclusions

1 Dikes in most cases breach in winter during the storm season and it means that they have to be repaired during short days, in cold and, sometimes, bad weather. Moreover, lack of time results in limited planning and much improvisation which in turn leads to many failed closures.

2 Frequently in the past it was impossible to close a flow gap in a breached dike because a good closure technique had not yet been developed and/or the financing of the work could not be arranged.

3 The three early (1510 – 1675) closures described in this chapter were successful either because of a small tidal range or because of construction of a perimeter bund.

4 The two closures of breaches in areas having a moderate tidal range could be achieved by driving a row of piles or driving piles as part of a cofferdam.

5 Cofferdams have been applied for horizontal type closure works at Scharwoude, the Lekdijk Bovendams at Wyck and the Nieuwerkerkepolder.

6 Because of lack of data, no case histories could be presented about breach repair in the tidal zone during the 17th and 18th century. It can, however, safely be assumed that the suspended fascine mattress has been an important tool for closing the final closure gap.

7 The use of floating fascine mattresses for (a) protection of the bottom of the breach against scour, and (b) the build-up of a fascine dam by stacking fascine mattresses on one another, is clearly demonstrated in the five case histories presented for the period 1807 – 1916.

8 The type of closure for repair of breached dikes, during the 19th and first half of the 20th century, has been the combined vertical and horizontal method. The horizontal part of the

[50] The Government Counsel , Inspector-General of *'s Rijkswaterstaat*, J. Blanken, who, on 15 February 1825 (in one of his '*Berigten*' (Notes) states "a site where not a shovel with good clay is to be found".

[51] But these cut-offs do not appear in cross-sections of the Wieringer Meer dikes as built. De Blocq van Kuffeler must have got his idea from the earth fill dams (*stuwdammen*) built in other countries on alluvial soils. See in this respect for instance Van Duivendijk (2008, 102, Fig. 10.5). But these fill dams are being built using dry earth moving equipment (large trucks, scrapers). Hydraulic fill placing is less suitable for the introduction of cut-offs.

closure was normally made in sections as a gradual closure. The individual sections were built up as clay bunds, cofferdams or pakwerks.

9 The need to introduce a clay cut-off to avoid a thru-going seepage path in a fascine mattress is very well shown in the case histories about the closures in the Kruininger, 'English' and Katwouder polders and in the polder Waterland.

CHAPTER 10 Repair of dikes breached during the storm surge of 1 February 1953

10.1 Introduction

In Chapter 6, Section 6.5, 21 flow gaps, have been pre-selected for a longlist. Subsequently, out of these 21 flow gaps 14 nos. have been selected for further analysis. The main particulars of these 14 flow gaps as well as an outline of the closure methods used are presented below in Table 10.1. Most of the information in this Table could be found in Rijkswaterstaat (1961) and other publications mentioned in Chapter 6. But the information about tidal volumes in column 7 had to be calculated (that is to say: in so far as possible). This will be elaborated upon for each of the 14 case histories. Two flow gaps are situated in the province Zuid-Holland, the others in Zeeland. The locations of the 14 flow gaps are shown on the map (Fig 6.1) in Chapter 6.

Table 10.1 Selected projects concerning repair of breaches in dikes and damming of tidal channels in the year 1953 in the Netherlands by applying simple means

no. of pro-ject	name of Project, its location (province, island or town) and RWS ref. no. [1]	date of final closure ground level in polder relative to NAP[2]	tide: HW & LW in relation to NAP (m); and tidal range		area of tidal basin (km^2)		tidal prism during spring tide (10^6 m^3)	length[3] of actual flow gap (m) and max. depth (m)	length (m) & bed level of final closure bund	source[4]	closure method and /or other typical features
			spring tide	neap tide	spring tide	neap tide					
	1	2	3	4	5	6	7	8	9	10	11
44 (B-2)[5]	breach in Noorddijk Papendrecht, Alblasserwaard 275, no.9	5 Febr. - 1.1	+ 1.44 – 0.54 = 1.98	+ 1.21 – 0.54 = 1.75	5.4	5.4	not calculated, but maximum must be < 2.4	100 incl. 2 gullies of 20 m each at NAP – 2 m	110 NAP – 0.40 & -1.0 m	RWS (423)	bed protection, then hor. closure using rock, sandbags and clay from both ends
45 (B-3)	breach in dike Nieuwe Stadspolder, island of Dordrecht; 285, no 6	25 Febr. NAP	+ 1.44 – 0.54 = 1.98	+1.21 –0.54 =1.75	3.1	3.1	appr. 1	28 MLT – 6 m	28 NAP (assumed)	RWS (458)	fascine dam and closure bund of sandbags
46 (B-4)	breach in dike at Jillesweg, Polder Oudeland, Goeree 294, no. 274	27 Febr. + 0.2	+ 1.45 – 1.17 =2.62	+0.95 –1.02 =1.97	appr. 8	8	3.6	50 NAP – 2.25 m	50 MLT + 0.50 m	RWS (463, 466)	fascine dam up to MLT + 0.50 m; subsequently dam of rock and sandbags
47 (B-5)	two breaches in dike Kleine Zuiderpolder, but only one is a flow gap; South of Goedereede; 293, no. 259	first closure of flow gap on 11 April which failed next day, then closure on 21 April + 0.6	+ 1.45 – 1.17 =2.62	+0.95 – .02 =0.97	0.12	0.12	0.1	30 NAP – 3.5 m	30 NAP – 1 m	RWS (467, 468)	first two closures in secondary dike; followed by closure of flow gap: fascine dam up to NAP – 2 m, then bund of sandbags high 5 m
48 (B-8)	eastern breach in dike Everingepolder; SW side Zuid-Beveland 309, no. 20	8 March + 1.2	from NAP – 0.9 to + 2.20 = 3.1 m on day of closure		1.1	1.1	1.5	90; NAP – 6.4 m	perimeter bund = 250 ; final closure gap = 23; NAP – 0.50 m (sill)	RWS (574, 575) [6]	core of pressed straw packs in closure bund of sandbags
49 (B-9)	breach in dike Zuidvlietpolder; North side western Zuid-Beveland at Zand-kreek; 309, no. 5	7 Febr. + 0.5	+ 1.81 – 1.67 = 3.48	+1.27 – .36 =2.63	0.49	0,49	0,65	25 NAP – 1.50 m	25; NAP – 1.50 m	RWS (576, 577)	bund of sandbags
50 (B-10)	western breach in Reigersbergse polder, South of Rilland, Zuid-Beveland; 310, no.61	19 Febr. + 1.1	+ 2.79 – 2.28 = 5.07	+1.94 –1.79 =3.73	10,9	10,9	not calculated but max. less than 3	100	appr. 300; NAP + 2 m or higher	RWS (578 – 580)	perimeter bund of sandbags at sea side of breach on salt marsh

[1] For RWS read: *Verslag van de Stormvloed 1953* (Rijkswaterstaat, 1961); the relevant page in Chapter 3.3. in the *Verslag* is given as well as the reference number(s) of the breache(s) shown at that particular page; the latter are also shown in Figures 3.2 or 3.3 of the *Verslag*.

[2] In most cases these levels have been taken from Hage (2015).

[3] Length in columns 8 and 9 is always measured in the direction of the dike axis and/or perpendicular to the direction of the tidal flow.

[4] Rijkswaterstaat (1961); for each project the relevant page numbers are given. In a few cases additional information was found in Polytechnisch Tijdschrift, 1*953 Ramp en herstel*; Uitgave B, nos 31-34, 14 August 1954 (Polytechnisch Tijdschrift, 1954).

[5] The numbers between brackets refer to the numbers in Table 6.1 in Chapter 6.

[6] Additional information in Polytechnisch Tijdschrift (1954, 678b & 679b).

no. of pro-ject	name of Project, its location (province, island or town) and RWS ref. no. [1]	date of final closure ground level in polder relative to NAP[2]	tide: HW & LW in relation to NAP (m); and tidal range		area of tidal basin ($km^{2)}$		tidal prism during spring tide (10^6 m^3)	length[3] of actual flow gap (m) and max. depth (m)	length (m) & bed level of final closure bund	source[4]	closure method and /or other typical features
			spring tide	neap tide	spring tide	neap tide					
	1	2	3	4	5	6	7	8	9	10	11
51 (B-11)	compartmentalization bund '4de weg'; inside Reigersbergse polder; 310, nos 64 & 65	11 March + 1.1	0 to 1.5 m on road base		7.7	7.7	not calculated	n.a.	1 800, apart from local deep gullies, mostly road base	RWS (249, 578 – 583)	construction of bund by various means incl. 1.2 million sandbags; gradual hor. closure on road base at NAP + 1.20 to 1.50 m
52 (B-13)	breaches in Kadijk (secondary dike); dike between polders Kruiningen and Waarde 310, no.44	23 April - 0.6 to +1.1	1.80 m on sill	appr. 1 m on sill	8	8	not calculated but in the order of 8 through the 13 breaches in these dikes (RWS, p. 593)	No detailed inf. and also not relevant	260 sill at NAP + 0.8 m	RWS (592 – 599)	perimeter closure bund on existing road made by dumping clay filled bags from jetty
53 (B-14)	breach no. 13 in Lavendeldijk (secondary dike); dike between polders Kruiningen and Waarde 310, no. 45	closure failed first time on 1 March and again 10 March; final closure of 7 m wide flow gap 2 days later by dumping rock -0.6 to + 1.1	+ 2.52 – 2.36 = 4.88	+1.73 –1.76 =3.49				100 NAP – 2.8 to + 0.8 m	40 m NAP – 2 m (average.)	RWS (594 – 595)	Over 60 m (shallow part) cofferdam formed by 2 rows of piles, remaining 40 m closed by means of a jetty transferred into cofferdam; sandbags were dumped from jetty into cofferdam
54 (B-15)	breaches in dike KLaas van Steeland-polder at bunker and sluice; near Poortvliet, island of Tholen 307, nos 53 & 54	5 Febr. 7 Febr. - 0.50 to -2.10	+ 209 – 1.97 = 4.06	+1.45 –1.56 =3.01	21.5	21.5	10 (estimate) (thru 2 breaches) 6 to 7 (on 6th Febr.) (through 1 breach) (RWS, p. 618)	64 (but above low tide) 16 NAP – 3.2 m	64 above low tide 16 NAP – 5.7 m	RWS (618 – 620)	first closure: anchored dam comprising clay bags; second closure dumping rock and clay bags; also by driving wooden beams in the sub-soil as support for the rock
55 (B-17)	breach in dike Anna Vosdijkpolder; island of Tholen bordering Mastgat 306, no. 5	19 Febr. (failed) 20 Febr. + 0.50 to + 0.70	+ 1.74 – 1.63 = 3.37	+1.24 –1.36 =2.60	4.6	4.6	3	45 NAP – 3.50 m	60 NAP – 2.50 m 20 NAP – 4.5 m (approx)	RWS (620 – 622)	170 m long perimeter bund , inside polder, made up of clay bags wooden beams, stakes, wire and beet nets
56 (B-18)	two breaches in dike Willempolder; former island St. Philipsland 305, nos. 3 & 4	26 Febr. (western gap) 7 March (eastern gap) + 1.2	+ 1.75 – 1.60 = 3.35	+1.15 –1.42 =2.57	7.5	7.5	2.3 (for two breaches)	46.5, below low tide 42.5, below low tide	46.5, below low tide 42.5, below low tide	RWS (627 – 628)	all four flow gaps on the island were closed by driving two rows of wooden beams to form a coffer-dam, wide 4 m; the cofferdams were filled with sandbags or clay bags
57 (B-19)	breach in dike Suzannapolder; island of Walcheren 307, no, 14	7 Febr. + 1.2	+ 1.81 – 1.67 = 3.48	+1.27 –1.36 =2.63	0.84	0	0.5	60 m NAP – 1.90 m	60 NAP – 0.5 m (average)	RWS (638 – 640)	perimeter bund long appr. 200 m built up of clay bags, the central gully was coffer-dammed by driving six rows of piles

10.2 Breach in Noorddijk Papendrecht, Alblasserwaard (project no. 44)

Gullies (each 20 m wide) were protected, by placing fascine mattresses. Ground level in Polder Papendrecht is NAP – 1.1 m (Hage, p. 150).Works carried out by manual labour and by a crane. No further details.

Project has been inserted on shortlist to show the combination of fascine mattresses, bunds of rock and sandbags.

The early closure of this flow gap is no doubt achieved because of the presence of experienced contractors in the direct neighbourhood of the breach. The tidal volume is small though the ground

level in the Alblasserwaard is low and the area of the tidal basin is rather large. This low tidal volume is due to the toe of the dike not being destroyed during breaching. In fact the nappe over this ridge (NAP + 1 m) during springtide will be 44 cm only. This results in a maximum tidal volume of 0.44 x 5.4 x 10^6 = 2.4 x 10^6 m^3. Maximum current velocity in the breach is in the order of 1.7 m/s. The closure, using sandbags, rock and clay, placed from both ends, is facilitated by the 'high' ground level (i.e. above Low tide) in the centre of the breach (Fig. 10.1). As no quantities are given, and also in view of the important role for a crane (placing clay), no analysis on labour and their output is possible.

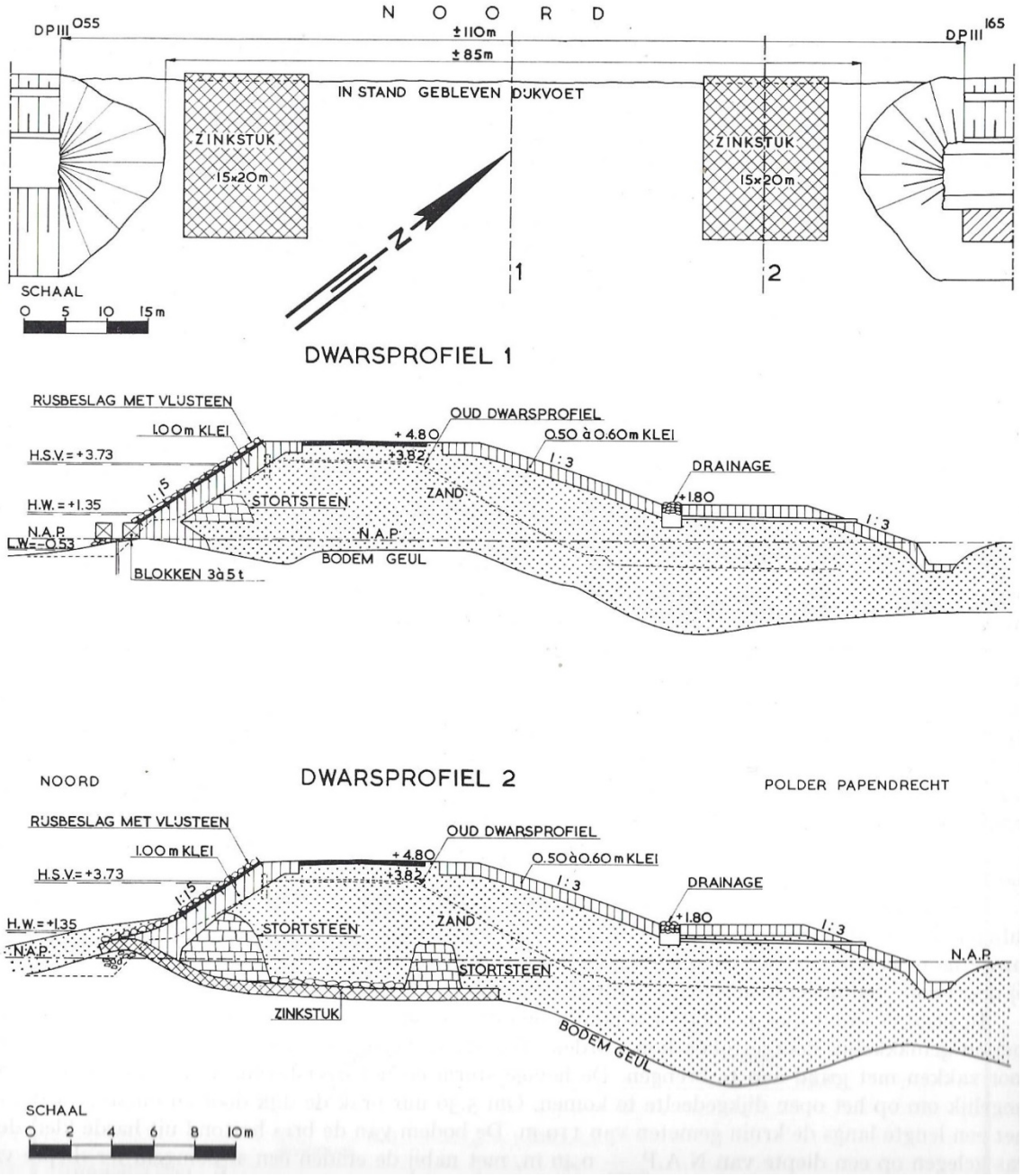


Fig. 10.1 Closure of flow gap in Dike along the river Noord near Papendrecht; source: RWS (p. 424)

10.3 Breach in dike Nieuwe Stadspolder, island of Dordrecht (project no. 45)

No cross-section is available of closure bund of sandbags made on a fascine dam. Crest level of fascine dam not given but is assumed to be the same as the ground level in the polder: NAP. By using the Coastal and River Engineering Support System CRESS some calculations were made regarding maximum current velocity and tidal volume. During a springtide the reduced tidal curve has a duration of 1.44/1.98 x 12.5 = 9.09 hours. During these 9.09 hours the level in the polder goes up (and down again) by 26 cm. But this implies that after this period the water level in the polder is at MSL – 0.13 m = NAP + 0.59 m and, during the next 3.41 hours water will flow out of the polder. Initially, it has a current velocity of $(2 \times 9.81 \times 1/3 \times 0.59)^{½}$ = 2 m/s but later on less. Accordingly, the water level in the polder goes down by some 6 cm. The tidal volume is therefore estimated at 1 x 10^6 m^3. The maximum current velocity during spring tide turns out to be 3.35 m/s. During a neap tide (NAP + 1.21 m) the max. current velocity will be 3.1 m/s and the water level difference in the basin will be 20 cm (between NAP + 0.50 and + 0.70 m). The head over the fascine dam then varies between + 60 cm (flood) and – 60 cm (ebb). These figures suggest that a closure only using sandbags placed by manual labour might have been difficult because sandbags placed in a closure bund are only quite stable up to a current velocity of 2.6 m/s (as follows from model testing reported about in Appendix H). Still, the closure bund only has to be 1.3 m high (neap tide) and the opening to be closed has a length of 28 m only. It is regretted that information given on quantities of materials used and labour input is lacking.

10.4 Breach in dike at Jillesweg, Polder Oudeland, Goeree (project no. 46)

Cross-sections are available. Some calculations regarding quantities of materials used can be made and also regarding fluid mechanics . Ground level in Polder Oudeland is NAP + 0.2 m (Hage, p. 130). Also this flow gap is closed by placing a fascine dam, comprising three fascine mattresses on top of each other, followed by, first, a bund of rock and sandbags and then a dam solely made up of sandbags. The fascine dam has its crest level at NAP – 0.60 m but the tidal volume during the closure operation is determined by the ground level in the polder (NAP + 0.2 m). The reduced tidal range during springtide is 1.45 – 0.2 = 1.25 m and the tidal period 1.25 /2.62 x 12.5 = 5.96 hours. At the end of this tidal period the water level in the polder is MSL – 0.20 m = NAP + 0.62 m. This implies a water depth of 42 cm in the polder. The water then starts to flow out of the polder during 6.04 hours. During the first hour the water level is lowered by 1 cm. Ultimately, within six hours the water level will go down by 5 cm. The tidal volume then comes to (0.40 + 0.05) x 8 x 10^6 = 3.6 x 10^6 m^3.

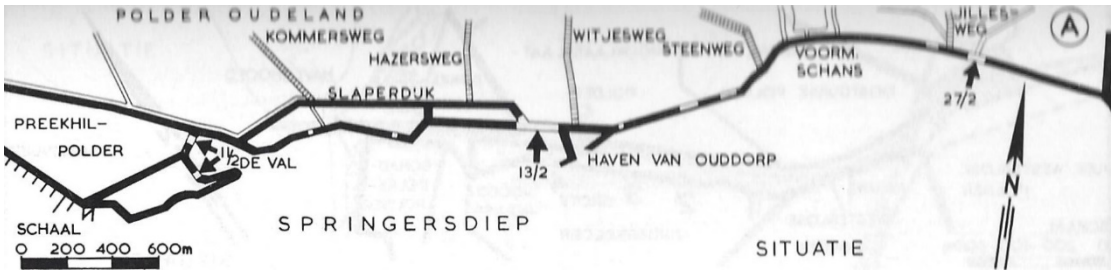


Fig. 10.2 Map showing flow gap at Jillesweg, polder Oudeland, Goeree-Overflakkee; source: RWS (p. 466)

For calculations, possibly to be made later, the following quantities have been determined:

- volume bund of sandbags and rock 185 m^3
- volume bund of sandbags sea side 200 ,,
- volume bund of sandbags polder side 270 ,,

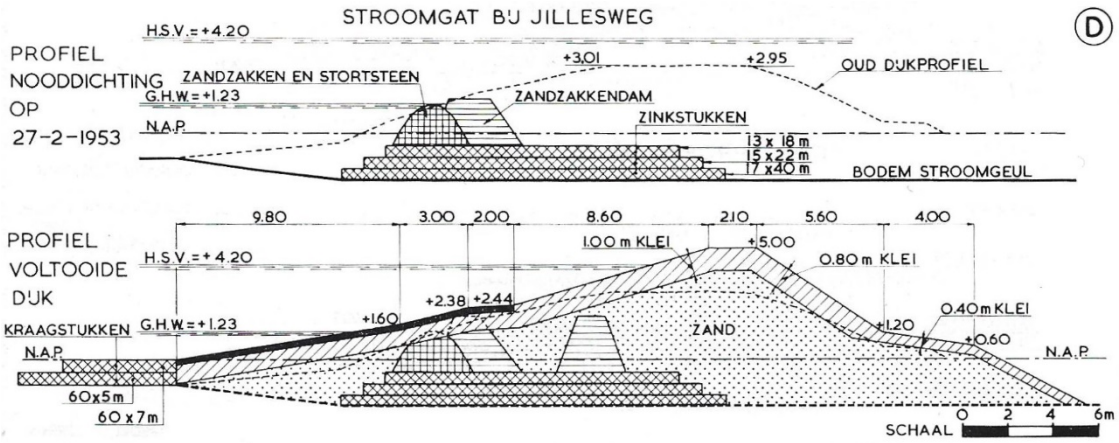


Fig. 10.3 Cross sections over closure works through flow gap at Jillesweg, polder Oudeland, Goeree-Overflakkee; source: RWS (p. 466)

10.5 Two breaches in dike Kleine Zuiderpolder (project no. 47)

This is a good example of compartmentalization and failure of first closure bund. There are two breaches but only one (cross-section 3) is a flow gap. It follows from Fig. 10.4 that the highest part of the fascine dam is at NAP – 1 m but ground level in polder is at NAP + 0.6 m. Therefore tidal range during spring tide is reduced to 85 cm and duration to 4.06 hours. This is a small polder and, consequently, it runs dry by the end of the reduced tidal period of 4.06 hours. Thus, the tidal volume is 0.85 x 0.12 x 10^6 = 0.1 x 10^6 m^3. The volume of the bund of sandbags is 780 m^3.

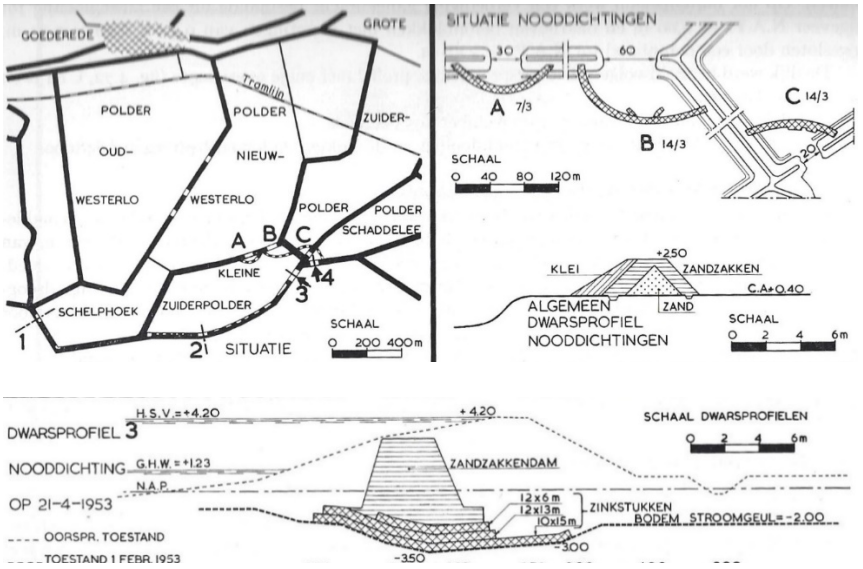


Fig. 10.4
Closure of flow gap in Kleine Zuiderpolder at site 3, South of Goedereede; source: RWS (p. 467)

10.6 Eastern breach in dike Everingepolder; SW side Zuid-Beveland (project no. 48)

The tidal volume is calculated for this major breach (long 90 m, deep NAP -5 to -6 m). As ground level in the polder is at NAP + 1.2 m (Hage, p.80), the tidal range at spring is reduced to 2.6 – 1.2 = 1.4 m. The tidal period is {1.4/(2.6 + 2.4)} x 12.5 = 3.5 h. By using the Coastal and River Engineering Support System CRESS one finds H_B = 1,40 m. So the polder will run dry. Accordingly, the tidal volume can be calculated as 1.40 x 1.1 x 10^6 = 1.5 x 10^6 m^3.

The polder is closed off from the sea by constructing a perimeter bund (Fig. 10.5), long 250 m, at the polder side of the breach. The perimeter bund, 2.5 m high over a length of 225 m, is constructed of earth borrowed nearby (apparently by utilizing draglines). The bund is protected by placing fascines, reed, sandbags and rock. Finally, a 23 m wide gully is left, having a length of 23 m between abutments

consisting of 3 m long piles. In fact during the period 18 Febr – 4 March the bottom of the creek had already been protected against scour by placing some 1 000 m^2 of fascine mattresses and dumping rock. As it is felt that the abutments might be destroyed by the strong currents, it is decided on 7 March to close the flow gap already on the next day though the available quantity of sandbags is still too small. In the early morning of 8 March rock is dumped in the flow gap and, by means of fascines and sandbags, a sill, wide 6 m, is formed at NAP – 0.50 m, just above low tide (NAP m – 0.90 m). Then the shortage of sandbags is solved by placing a core of pressed straw packs inside the bund of sandbags. At night the crest of the bund is just above high tide (NAP + 2.20 m). To enlarge the stability of the bund during the night, side tip wagons, each loaded with 1 m^3 of clay are placed on the crest. The volume of sandbags placed is 196 m^3. The volume of straw packs placed comes to 69 m^3.

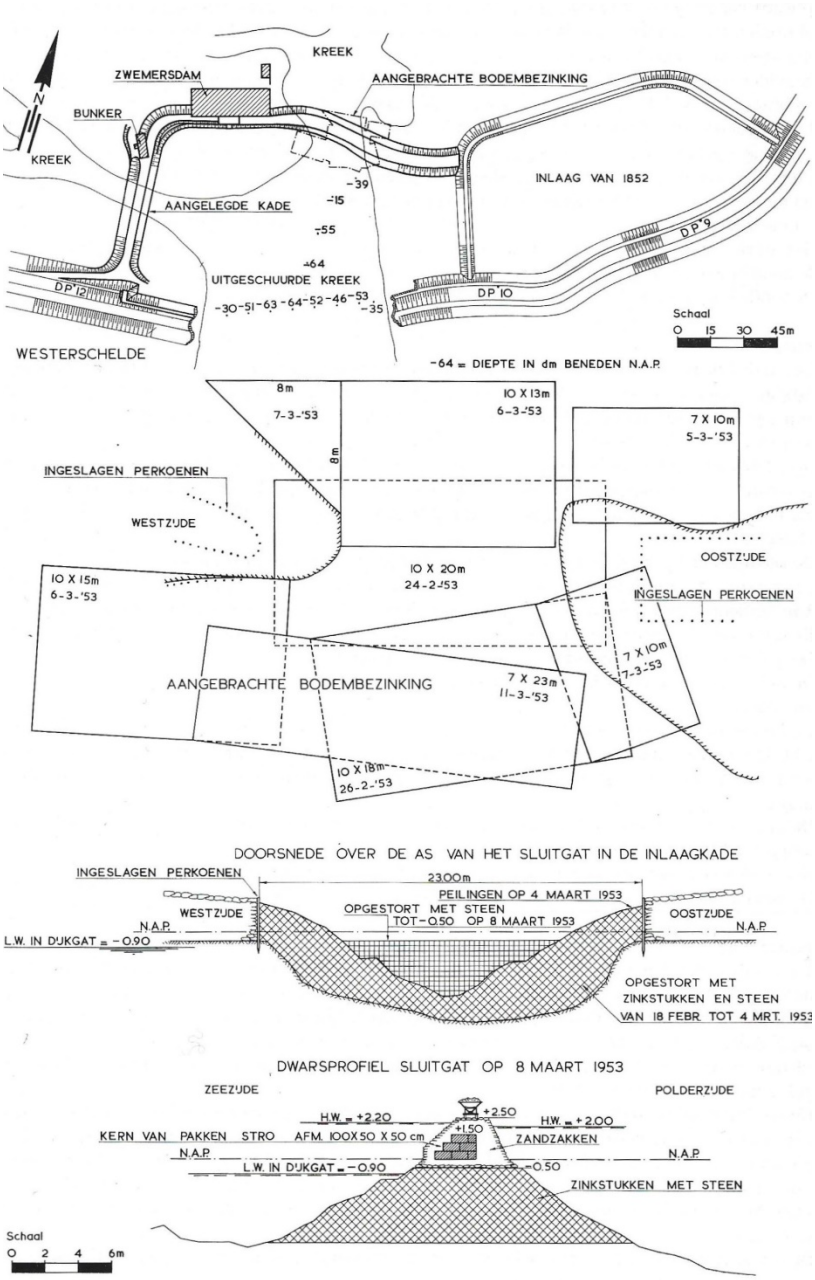


Fig. 10.5
Perimeter bund around breach in dike Everingepolder, south side western Zuid-Beveland; source: RWS (p. 575)

10.7 Breach in dike of the Zuidvlietpolder, Zuid-Beveland, at Zandkreek (project no. 49)

The size of the breach (length 25 m, depth NAP - 1.50 m) together with the ground level in the polder (NAP + 0,50 m) and its modest size (0.49 km^2) result in a complete filling and emptying of the polder during a spring tide. Accordingly, the tidal volume is (1.81 – 0.50) x 0.49 x 10^6 = 0.65 x 10^6 m^3. The max. current velocity in the breach is 1.6 m/s. No cross- sections or other details are available.

10.8 Western breach in Reigersbergse polder, Zuid-Beveland (project no. 50)

The sea dikes of this polder are breached at two locations. In Fig. 10.6 the dates of closure of these breaches have been marked: 19/2 and 21/4. The western breach has a high salt marsh in front. Because of the high level (estimated at NAP + 2 m or higher), it is possible to transport sandbags and, subsequently, clay over a narrow gauge railway to the site of the, approximately 300 m long, perimeter bund constructed on the salt marsh (Fig. 10.7). The bund of sandbags has a height of 1 m (crest at NAP + 3 m or higher) and is subsequently heightened to NAP + 5 m. The volume of the bund of sandbags must have been in the order of 350 to 400 m^3. It has been constructed during a period of 11 days.

The Reigersbergse polder is a relatively large polder and the reduced tidal period (1.95 hours because of the high salt marsh) is such that calculations using CRESS are not possible. Moreover, the distribution of tidal inflow over the two breaches is not known. The tidal volume for the western breach during a springtide is provisionally estimated at 3 x 10^6 m^3.

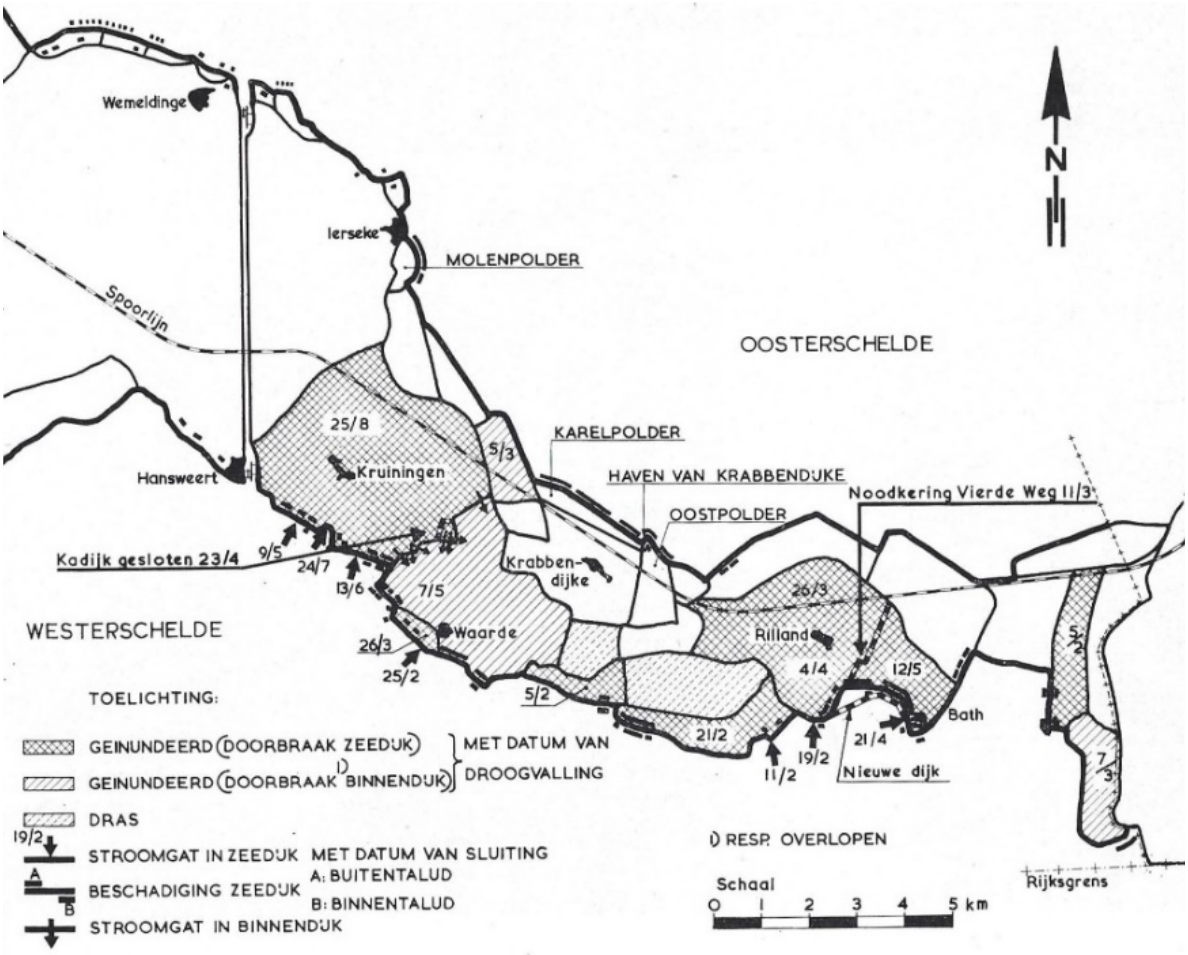


Fig. 10.6 Location of breaches and repair works eastern Zuid-Beveland; source: RWS (p. 579)

Fig. 10.7 Closure of breach in sea dike Reigersbergse polder by means of a perimeter bund on the salt marsh; source: RWS (p. 580)

10.9 Bund '4de weg', inside Reigersbergse polder, Zuid-Beveland (project no. 51)

This compartmentalization bund is built to reduce the area flooded through the eastern breach (marked by closure on 21/4 in Fig. 10.6) in the sea dike of the Reigersbergse polder by 8 km^2 to 3 km^2. The bund will be built for the major part on a paved road base. But It turns out to be difficult to close various tidal gullies already formed in this road. Some 1 800 soldiers are involved during a period of 21 days for filling of sandbags, transport of bags (by GMC trucks, pontoons and small motorboats) and placing of sandbags. Moreover, draglines place beet nets filled with sandbags in flow gaps. The volume of the bund of sandbags, excluding the sandbags required to close (and reclose after failures) the tidal gullies, is 13 200 m^3. Assuming 50 sandbags in 1 m^3 (see Section 10.10) in any case 660 000 sandbags are placed. When there are 70 sandbags (see Sub-section 10.16.4) in 1 m^3, the number of bags is 924 000. This would imply 24 sandbags placed per man day but again: this excludes the sandbags placed in gullies and failed closures. According to RWS (p. 582) 1.2 million sandbags have been used.

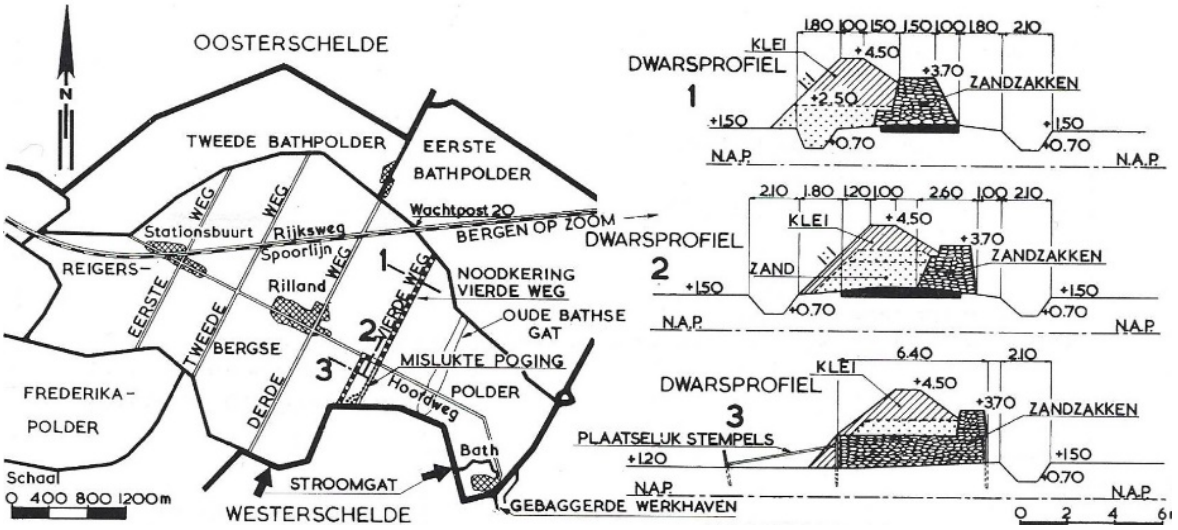


Fig. 10.8 Location and cross-sections temporary bund on 'Vierde weg', Reigersbergse polder; source: RWS (p. 583)

10.10 Breaches nos. 6 to 11 in Kadijk (secondary dike), Zuid-Beveland (project no. 52)

De Kruiningenpolder and the Oost-Inkelse Polder flooded through three breaches in the dike along the Westerschelde. Subsequently, the secondary dikes, Kadijk and Lavendeldijk, between the Kruiningenpolder and polder Waarde, breach at 13 locations and the polder Waarde (8 km^2) floods as well. Consequently, the tidal basin increases from 14 km^2 to 22 km^2. Ground level in both polders varies between NAP – 0.6 and + 1.1 m (Hage, p. 84-86).

In order to decrease the size of this tidal basin, it is then decided to repair these two secondary dikes as soon as possible.

In order to close breaches nos. 6 to 11 a perimeter bund C-D-E is constructed. Part of it is the construction of a 350 long bund of sandbags D-E on the (concrete) pavement of the highway (*Rijksweg*) (Fig. 10.10). As this highway does not run dry during the low water period a 350 m long jetty (Fig. 10.9) is constructed from which the sandbags will be dumped. The sandbags are stored in 14 stockpiles with thirteen 20 m long openings in between (Fig. 10.11). On the day of closure, during a neap tide, the thirteen openings are closed by, first, dumping the sandbags from the jetty (Fig. 10.12), followed by placing the sandbags in the bund. When the work starts, the water flowing over the highway is at its lowest level, i.e. 40 cm above the pavement. The bund to be constructed is 2.8 m high and will be ready before High tide.

In the stockpiles 14 x 6 500 = 91 000 sandbags are stored. Apparently, this excludes the sandbags already being part of the bund. The bund in the openings has overall a volume of 7 x 13 x 20 = 1 820 m^3. This implies that there are 50 sandbags[7] in 1 m^3. In that case the bund comprises 350 x 7 x 50 = 122 500 bags. According to RWS (p. 599) the work is carried out by 1 000 labourers[8]. This implies that two gangs of 38 labourers each have to close a 20 m wide opening. From the length of the loops shown in Fig. 10.10, it follows that these labourers are distributed over 28 m. As a gang has to close a 10 m long bund section it has to transport 7 x 10 x 50 = 3 500 sandbags or 92 bags per labourer. Regrettably, none of the publications referred to, mentions the duration of the closure works. But if it is assumed that five hours were available, each man would have to carry 18 bags in one hour or one bag every three minutes. In order to boost 'job satisfaction' among the labour force, the contractor had installed loudspeakers playing appropriate music[9].

Fig. 10.9
Tidal flow over highway near Kadijk prior to closure; stockpile of sandbags in background;
source: RWS (p. 598)

[7] This figure excludes losses due to the flow over the highway because measures had been taken to avoid such losses (cylinder of wire mesh fixed to the highway pavement, see Fig. 10.11 and 10.12).
[8] Also Polytechnisch Tijdschrift (1954, 650b) mentions this number but Stuip ,ed. (1984, 30) suggests that the number was 3 000. However, 3 000 is unlikely because of the space on the jetty.
[9] Verbal information received in the past from Prof Dr. Ir. J.F. Agema.

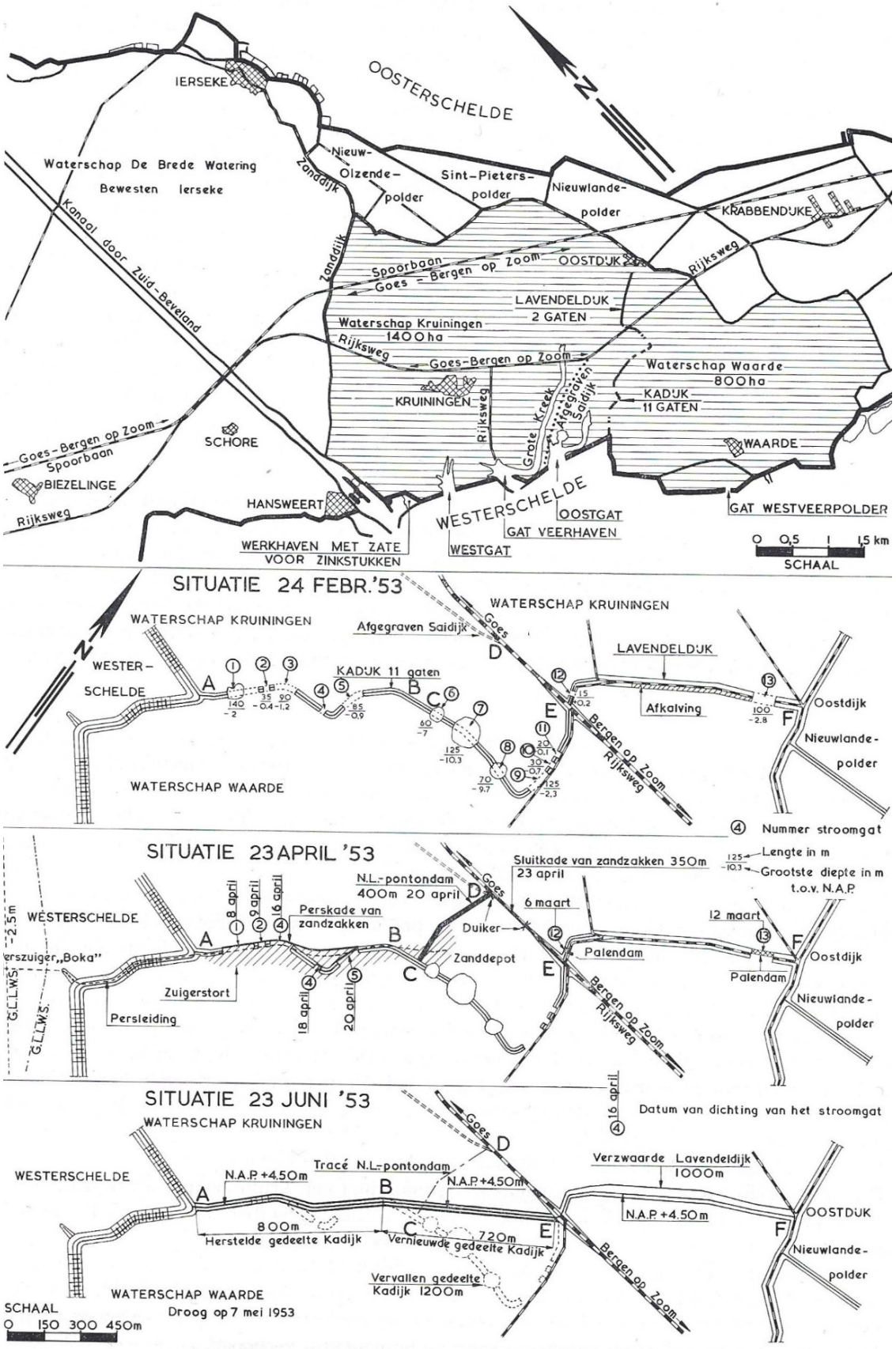


Fig. 10.10 Situation in 1953 in the polders Kruiningen and Waarde including the breaches in the secondary dikes Kadijk and Lavendeldijk; source: RWS (p. 592)

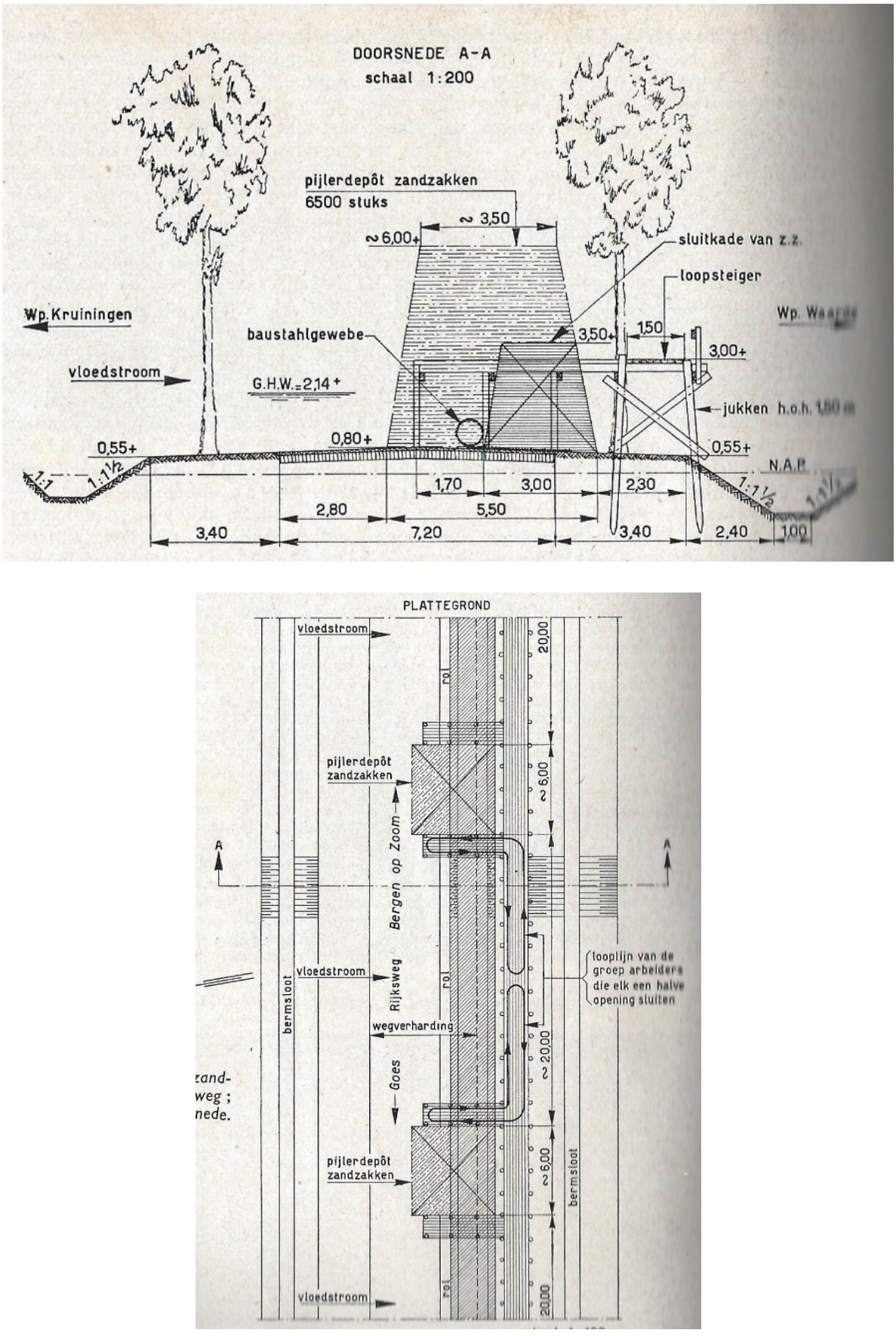


Fig. 10.11 Cross-section and plan of temporary bund on highway near Kadijk, Kruiningenpolder, Z-Beveland; source: Polytechnisch Tijdschrift (1954, 650b)

Fig. 10.12
Dumping of sandbags from the jetty starts; note cylinder of wire mesh at downstream side to prevent bags from being removed from bund;
source: Stuip, ed. (1984, 30)

10.11 Breach no. 13 in Lavendeldijk (secondary dike), Zuid- Beveland (project no. 53)

The method used for the construction of the 350 m long perimeter bund described in Section 10.10 is not suitable for closure of breach no. 13 in the Lavendeldijk, near Oostdijk (Fig.10.9 at F). This breach has a length of 100 m and a maximum depth of NAP - 2.8 m. Ground level in both polders varies between NAP – 0.6 and + 1.1 m (Hage, p. 84-86).
It is decided to construct a 'closed' cofferdam[10] (*palendam)* in the shallow part (long 60 m) of the breach. The width between the rows of the cofferdam is 1.5 m and 4 piles are driven in each metre of a row. The piles in a row are connected to each other at the top by a horizontal beam. The rows are connected to each other by means of steel wire and swivels. First the cofferdam is filled by dumping sandbags. Subsequently, the cofferdam is strengthened by stacked sandbags at the west side and a sand body placed by dump trucks at the east side (Fig. 10.13, top).

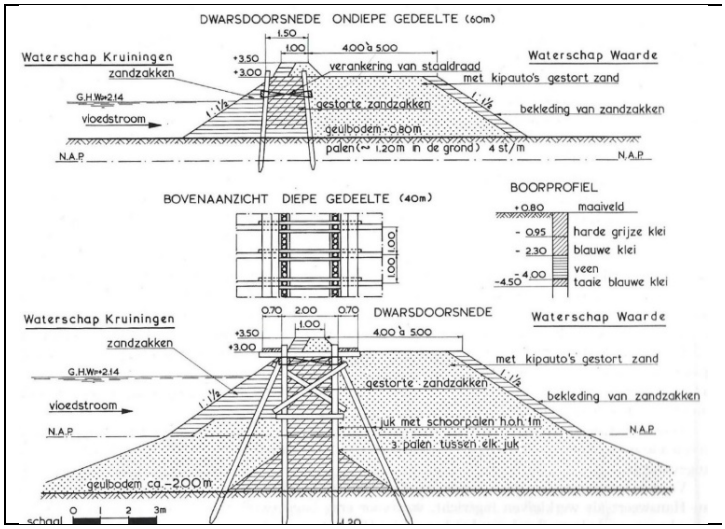


Fig. 10.13
Cross-sections and plan of cofferdam through breach no. 13, Lavendeldijk near Oostdijk, polder Kruiningen Zuid-Beveland;
source: RWS (p. 594)

[10] The cofferdam is 'closed' in the sense that the piles in each of the two rows are driven as close as possible to each other, thus avoiding the need for boarding up or placing of hurdles prior to filling the cofferdam.

In the remaining deep section, long 40 m, a heavier type of cofferdam is constructed, as is shown in Fig. 10.13 (below). Note the walkways at both sides of the cofferdam for labourers carrying sandbags. This cofferdam is ready to be filled on Sunday 1 March, but the majority of the labourers does not turn up and the cofferdam collapses in the current. On 10 March at 17.00 hours the newly built cofferdam had been filled with sandbags and immediately construction of a sand body at the east side starts. But the next day, at 21.00 hours the dam collapses at the connection to the south abutment over a length of 7 m due to heavy wave attack caused by a strong northern wind. The newly created breach is then closed by dumping rock from wheelbarrows, driven on girders laid over the breached section. Also sugar beet nets and baskets made of wire mesh, all filled with sandbags, are being dumped. On 12 March at 19.00 hours the rock bund has its crest above high tide. At that time 200 tons of rock had been dumped. The closure is then completed by placing sandbags at the west side and a sand body at the east side. In all some 55 to 60 thousand sandbags must have been placed. According to the cross-sections the number is 53 000 (assuming 50 bags in 1 m^3).
Remarkable is the ratio width /height of the cofferdam in the deep section. Also the limited driving depth (1.2 m) of the piles is noted. May be, the narrow width is to save on sandbags and the limited driving depth might be due to the lack of longer piles.

10.12 Breaches in dike Klaas van Steelandpolder, island of Tholen (project no. 54)

It is noted that both breaches took place at structures (bunker[11] and sluice) present in the dike body. In Fig. 10.14 also two breaches (marked C and D) are shown in a secondary dike. Breach D is already closed in the beginning of February. Breach C is left open because it is felt that an early closure of the breaches in the sea defence should get priority as otherwise the sluice might get lost in the quickly deepening breach (Fig. 10.14, dwg. 2).
The breach at the bunker is shallow and runs dry at Low tide. Ground level in polder is NAP + 0.3 m (topographic map, 1950) but probably lower in the adjacent, much larger, polder which is also flooded[12]. The breach at the bunker is closed first to enable access from both sides to the major breach at the sluice. On 5 February the breach at the bunker is closed, as well as the narrow gully between sluice and eastern dike head, Fig. 10.14, dwgs. 3 (below) and 5. The closures are carried out by 700 labourers[13]. Preparations are made to work day and night at the closure of the last breach. This breach deepens by 2.5 m during the night of 5-6 February. The flood volume is then 6 to 7 million m^3. The ebb volume is less because the polders are still being filled. The closure starts in the early morning of 6 February. At that time the current velocity is in the order of 3.5 to 4 m/s. Overall, 1 500 labourers are involved. As can be seen in Fig. 10.14, dwg. 5 (top), the cross-section is a mixture of rock, clay bags, vertically driven wooden beams and swivel connections. In this manner, first of all, abutments are formed, which leave at Low tide (13.20 h) an opening of 10 m only, while the difference between inside and outside water level is still 1 m.
At 15.00 h, when this difference has gone down to 0.8 m, closure starts. Stones in bags are dumped to form a ridge, behind which clay bags are dumped (initially floating away in the current). Beams, 6 to 8 metres long, are driven, using hand-pile-drivers and rock is dumped at both sides of these beams. At 17.00 hours a level of NAP is reached and the night shift continues the closure works. It is able to stay ahead of the rising tide and at 20.00 hours the High tide is stemmed. During the night

[11] A bunker is a fortification, partly inside a dike or dune having thick concrete walls and roof for protection against shelling by the enemy.
[12] Polytechnisch Tijdschrift (1954, 632b): NAP – 0.50 to – 2.10 m.
[13] From the text (RWS, p. 618) it cannot be established whether the 700 labourers were only involved in carrying and dumping clay bags or also in the filling of these bags and miscellaneous works (like dumping rock or driving vertical beams using hand-pile-drivers).

the dam is heightened and widened and it is at NAP + 2 m in the early morning of the 7th (dotted lines in dwg. 5, top). Work on this closure continues until 14 February when its crest is at NAP + 3.50 m.
One can see in the drawing that tarpaulins have been fixed at the sea side to the aforementioned beams.
On the basis of the cross-sections shown in the drawings in Fig. 10.14, a calculation has been made of the volume of clay bags placed:

- closure bund, long 64 m, made on 5 Feb. In front of bunker	1 440	m^3
- this closure bund enlarged in period 8 – 14 February	1 200	,,
- closure bund through gully, east of sluice on 5 Feb. (estimate)	100	,,
- closure bund, long 16 m, made on 6 -7 February	924	,,
- this closure bund enlarged in period 8 – 14 ,,	1 492	,,
total theoretical volume of closure bund (excluding losses)	5 156	m^3

On the basis of 50 bags in 1 m^3, this comes to roughly 260 000 bags. But RWS (p. 620) mentions 700 000 bags. This would imply a loss factor of nearly 3. But here the lack of length profiles, and only one cross-section being available for each closure, may also lead to under-estimation in volumes. Moreover, bags may have been used for protection of nearby damaged dike sections while also the number of bags in 1 m^3 may be higher: 70 is a more realistic figure (see Sub-section 10.16.4). In the last case the number of bags is roughly 360 000.
For the closures on 5 February, 700 labourers are available. They will have worked on filling, transporting and placing bags. Also beams had to be driven and rocks dumped. Assume therefore that only 600 labourers are actually working on the bag operations. This comes to (1440 + 100) x 70 /600 = 180 bags per labourer. It is assumed that losses during these two closures (which are relatively easy) are negligible.
The initial closure in front of the sluice is done during two days (6 and 7 Feb.). Assume that two shifts of each 750 labourers[14] work on this closure. The calculated volume of clay bags placed is only 924 m^3 but the losses are high (assume a loss factor of 2[15]). Assume also that only 600 labourers are working on 'bagging works'. Then the output in 1 200 man-days is 2 x 924 m^3 x 70 = 130 000 bags or 108 bags per man-day.

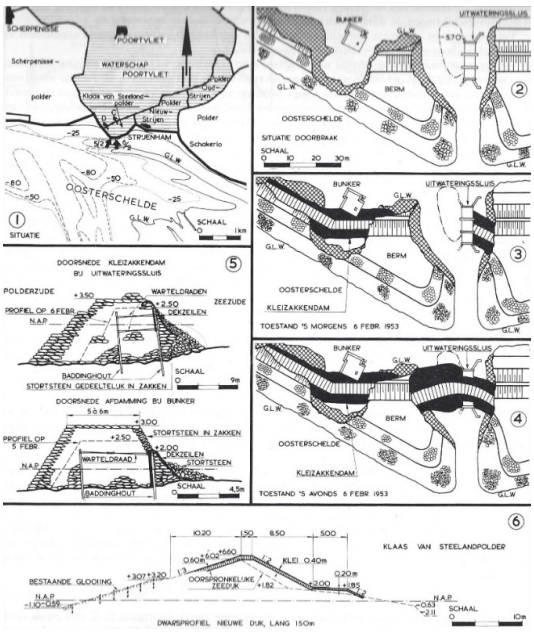


Fig. 10.14
Repair of breaches Klaas van Steelandpolder; source: RWS (p. 619)

[14] RWS (p. 619) mentions 1 500 labourers, but as there are two shifts it is assumed that only 750 labourers are at work at any time.
[15] This includes the losses on 4 February when an effort to close the gap in front of the sluice was abortive.

10.13 Breach in dike Anna Vosdijkpolder, Island of Tholen, side of Mastgat (project no. 55)

This breach is closed by more or less the same method and materials as used for closing the flow gap at the sluice of the Klaas van Steelandpolder. But here a perimeter bund will be constructed inside the polder.

The subsoil in the breach consists of sandy clay and peat. Luckily, peat has a certain resistance against scour because apparently a bed protection comprising fascine mattresses ballasted by stone cannot be made[16]. This situation was the same at the Klaas van Steelandpolder.

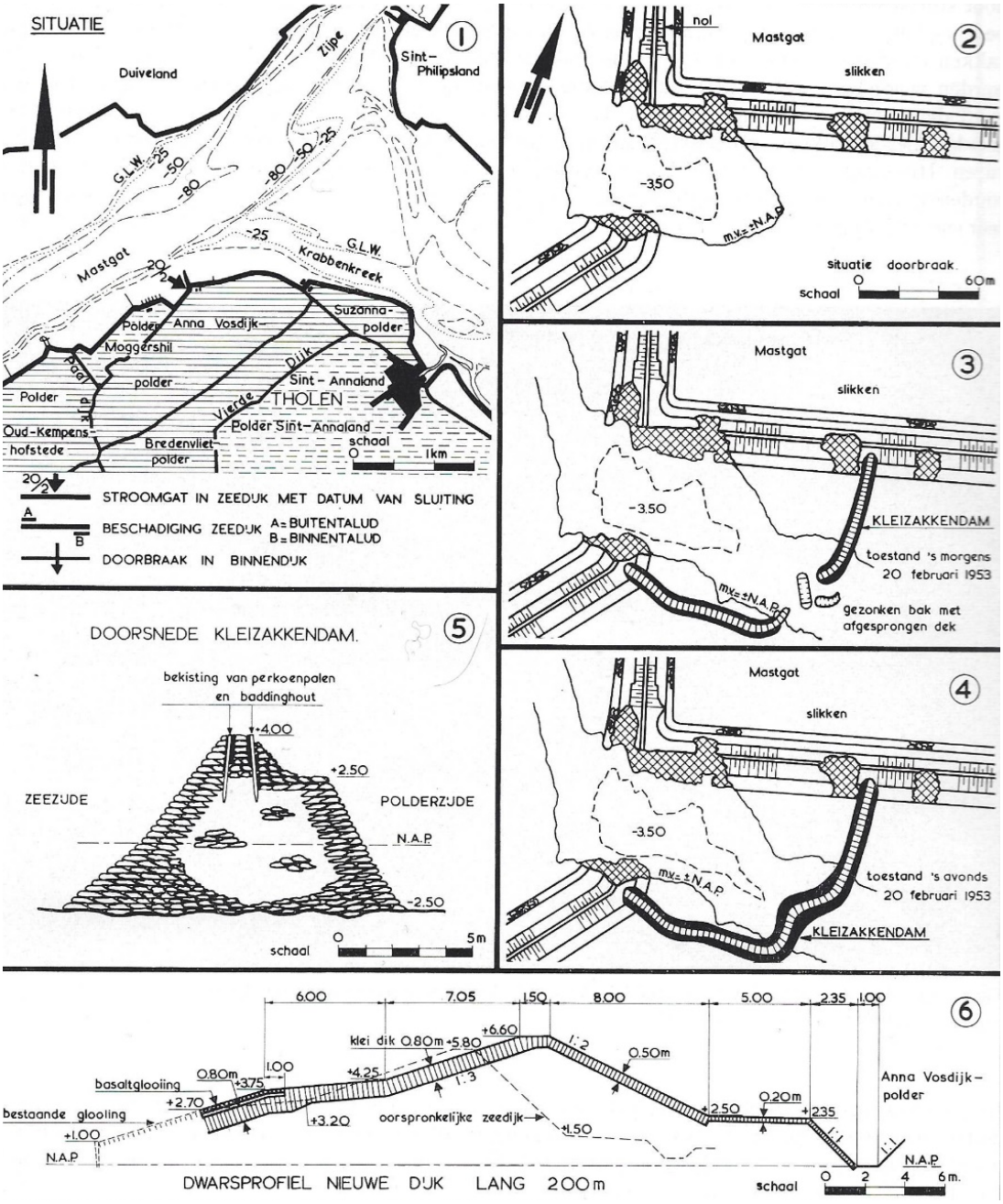


Fig. 10.15 Closure by means of a perimeter bund, Anna Vosdijkpolder, island of Tholen; source: RWS (p. 621)

[16] It stands to reason that, after the storm surge of February 1953, the firms in the Netherlands carrying out fascine works were directed to the major dike breaches like the ones at Schelphoek, Ouwerkerk and Kruiningen ferry harbour.

Ground level in the polder is at NAP + 0.50 to + 0.70 m (Topographic map, 1950).
The plan is to start building the perimeter bund from both ends. On the day of final closure some 150 000 bags filled with clay will be stockpiled to fill the last gap. But it turns out to be all a little bit more difficult than foreseen. On 17 and 18 February[17] the dam sections are built following the alignment as shown in Fig. 10.15. By the end of the second day the distance between both bund sections is still 80 metres. In the early morning of 19 February 1 200 labourers are available to carry and dump clay bags and the idea is to have the perimeter bund closed by 17.00 h[18].
When the opening has been narrowed to 60 m, one starts to drive rows of stakes in the ground while clay bags are placed in between to prevent scour of the soil. This is successful until the remaining opening has been narrowed to 35 m. The completed bund sections are then heightened to NAP + 2 m and the method of closure applied from then on is the one already described for the closure of the breach at the Klaas van Steelandpolder: Wooden beams, 6 to 8 metres long, are driven, using hand-pile-drivers and rock and clay bags are dumped at both sides of these beams. This system works until the opening left is 25 metres and the head over this opening is 80 cm (i.e. current velocity in the order of 4 m/s). The closure elements now have to be bigger. This is solved by dumping stone and clay bags in beet nets. It brings about another 5 m of narrowing. It is the intention to close the remaining, 20 m wide, gap at the turn of the tide. It will be done by dumping clay bags from two flat deck barges. At the same time beams will be driven alongside the barges and bags and stone dumped at both sides of these beams. But the period, at the turn of the tide, during which current velocities are zero or small, is very short, and the incoming flood starts to rise quickly. The two flat deck barges are pushed by the current through the closure gap and one of them runs aground near the gap (see Fig. 10.15, dwg. 3). The other one floats into the polder. The next morning, 20 Feb., a new attempt is made and a flat deck barge is sunk more or less in the alignment of the closure gap. During the day 1 200 labourers are able to close the gap by dumping clay bags in beet nets, fixed to each other by iron wire. The remaining leaks are closed by dumping clay bags and stones against beams, driven into the soil. During the following days the 170 m long closure bund is strengthened and heightened to NAP + 4.00 m (Fig. 10.15, dwg. 5 and Figures 10.16 & 10.17).
As only one cross-section of the bund, mainly built up of clay bags, is available, assumptions have to be made regarding the ground level under the bund. Length of bund is 170 m. Assume 110 m is built on a ground level of NAP (Fig. 10.15, dwg. 3) and 60 m on a scoured ground level of NAP – 2.50 m (dwg. 5).

A calculation of the volume of the closure bund is then as follows:

- bund sections, having total length of 110 m, up to NAP + 2.50 m	1 856 m^3
- bund section , ,, ,, ,, ,, 60 m, ,, ,, ,,	2 700 ,,
- upper part of bund between NAP + 2.50 and + 4.00	765 ,,
total theoretical volume of closure bund (excluding losses)	5 321 m^3

On the basis of 70 bags in 1 m^3, this comes to roughly 372 000 bags. But RWS (p. 620) mentions 550 000 bags[19]. This would imply a loss factor of roughly 1.35.
For the bund sections built on 17 and 18 February, no information about available number of labourers is given. It is assumed that losses during the construction of these two bund sections (which are easy) are negligible.

[17] Note that springtide was on 16 February.
[18] At 17.00 hours the flood is already rising and the turn of the tide in the remaining opening is expected at that time.
[19] This figure includes the clay bags used for repair of a shallow gap (*bres*) in the secondary dike Paaldijk, but no information is available about the extent of this repair work. It is estimated roughly at 50 000 bags.

For the construction of the last 80 metres 150 000 bags are in stock. Ultimately, it takes two days to close this gap. Assume 1 000 labourers out of 1 200 labourers are working on 'bagging works'. This implies an output of 150 bags per labourer in two days.

Fig. 10.16 The breach at the Anna Vosdijk polder closed by a perimeter bund of clay bags; source: FOTO AVIODROME, Lelystad

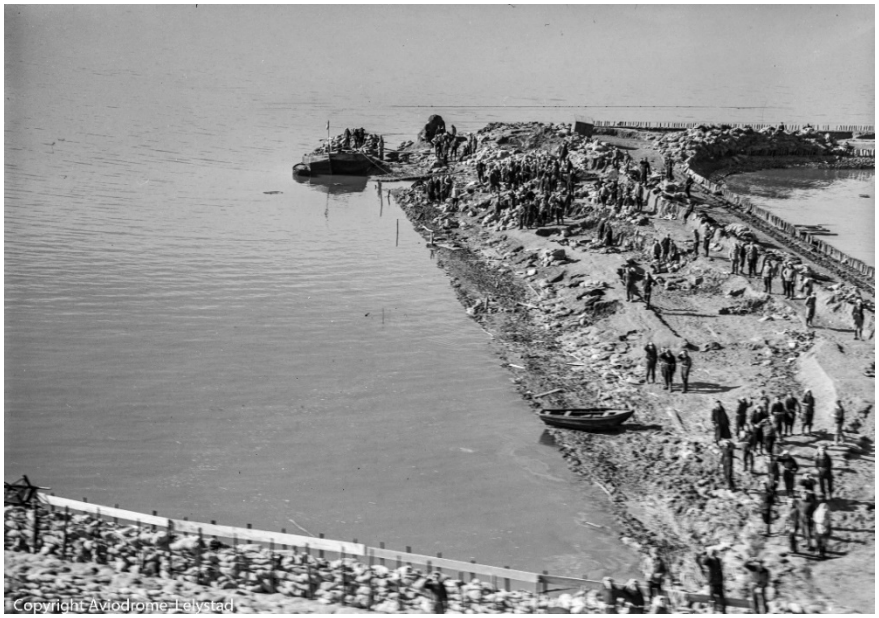


Fig. 10.17 Sand bagging operations on the sea dike of the Anna Vosdijkpolder; part of closure bund is visible in the foreground; source: FOTO AVIODROME, Lelystad

10.14 Two breaches in dike Willempolder, Island St. Philipsland (project no. 56)

On the island St. Philipsland the dikes have breached and scoured to dephs below Low tide at four locations. Two of these locations are at the Willempolder. This polder is small (30 ha) Its ground level is at NAP + 1.1 m[20]. The secondary dike between this polder and the much larger Anna-Jacobapolder (719 ha, ground level at NAP + 1.2 m) breaches as well and deep scoured channels start to develop rapidly (Fig. 10.18).

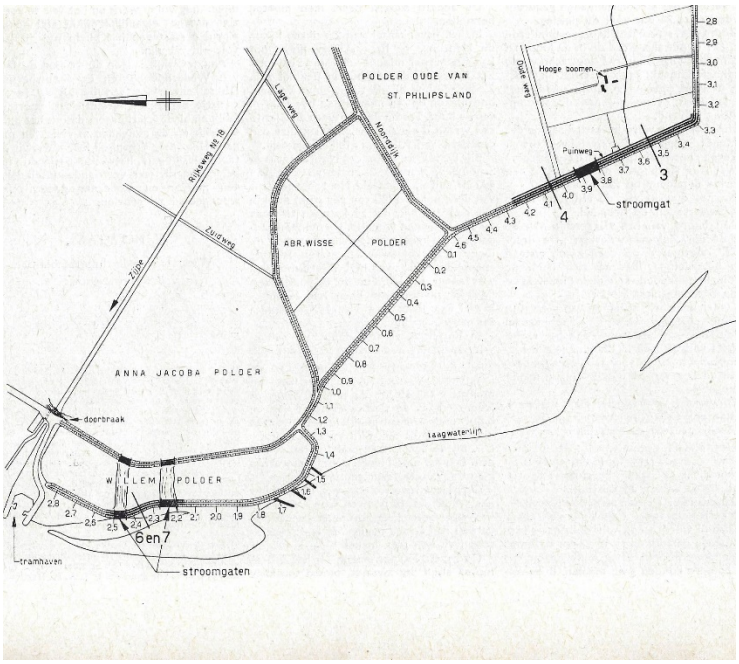


Fig. 10.18
Breaches in sea dike and secondary dike Willempolder, Sint-Philipsland; breaches in the sea dike have a length of 42.5 m (no.7) and 46.5 m (no. 6) respectively;
source: Polytechnisch Tijdschrift (1954, 628b)

Because of this scour development it is decided to close the two breaches in the sea dike of the Willempolder first. It is noted that, also here, structures (bunkers) are present in the dike body at the location of the breaches[21].
For all four breaches the same closure method is applied: a cofferdam (Fig. 10.19) is constructed, using wooden beams (*baddings*). The cofferdams have a width of 4 m and the beams are driven applying hand-pile-drivers at centre to centre distances of 50 cm. The beams are connected to each other applying swivels, while shoring timbers serve as support at the outside. To prevent bags from being pushed out of the cofferdam by the current, chain link wire mesh is placed. As soon as possible after beams have been driven, inside the cofferdam a layer of one metre of sandbags is dumped as a protection against scour.
Meanwhile stockpiles of sandbags and clay bags are made at the abutments of the cofferdam in order to close the flow gap within one tide. Immediately after this final closure to a level of MHT, sand will be placed by a suction dredger at both sides of the cofferdam.
In this manner all four breaches are closed successfully.
For the four closures by means of cofferdams an overall quantity of 323 500 bags are available. The overall length of the four cofferdams is 284 m. If the breaches have a depth of one metre below Low tide (which is at NAP – 1.60 m at springtide) and the cofferdams have their crest at MHT (NAP + 1.46 m), the area of the cross-section of bags placed is 4.06 x 4 = 16.24 m^2. The overall volume of bags in the four

[20] Topographic map (1950).
[21] A bunker is a fortification, partly inside a dike or dune having thick concrete walls and roof for protection against shelling by the enemy. Prior to the start of the closure works the bunkers were blown up and debris removed by soldiers of the French army.

cofferdams is then 4 612 m^3. The number of bags is 323 000, assuming 70 bags in 1 m^3. But this figure excludes losses while also bags are used for closing a breach in the main road (Fig. 10.18, left) and, later on, for a provisional slope protection of the closed breaches at the sea side (sandbags on a layer of fascines).
It is stated that the closure of a flow gap is done during one tide. At MLT (NAP – 1.5 m), only 10 cm of water flows over the low, one metre high, sill.
For the western closure up to MHT (NAP + 1.46 m) 46.5 x 4 x (1.6 + 1.46) x 70 = 40 000 bags have to be dumped during six hours. If one man carrying a bag walks at a speed of 1 m/s (excluding loading and dumping a bag), he will transport a bag every 1.5 minute. This is 40 bags in one hour or 240 bags during six hours.

Fig. 10.19 Cofferdam being constructed for closure of northern flow gap in dike of Willempolder; southern breach in secondary dike in background; source: RWS (p. 627)

The author of the article in Polytechnisch Tijdschrift (1954, 630b), J.M. Faillie, notes rightly that one was lucky about the scour in the flow gaps not penetrating the clay layer in the base of the dikes. He also admits that the closure of the western gap on 26 February nearly became impossible because of possible earlier failure of the cofferdam still under construction. Only by working day and night this failure could be prevented.

10.15 Breach in dike Suzannapolder, Island of Walcheren (project no. 57)

The Suzannapolder is a small polder (84 ha) having its ground level at NAP + 1.2 m (Hage, p. 78). Accordingly, tidal volume at springtide (NAP + 1.81 m) is limited to $0.84 \times (1.81 - 1.2) \times 10^6 = 0.5 \times 10^6$ m^3. A calculation by means of the Coastal and River Engineering support system CRESS confirms the last figure.

During the storm surge flood of 1 Feb. the sea dike is breached at two locations (one flow gap) and the railway embankment at one location The last mentioned breach, as well as a closable opening in this embankment (*coupure*) enables tidal flow towards the southern part of the Suzannapolder. Prior to closure of the breach in the sea dike, the breach in the railway embankment is repaired and the *coupure* is temporarily closed by clay bags. This decreases the area of the tidal basin to approximately 50 ha.

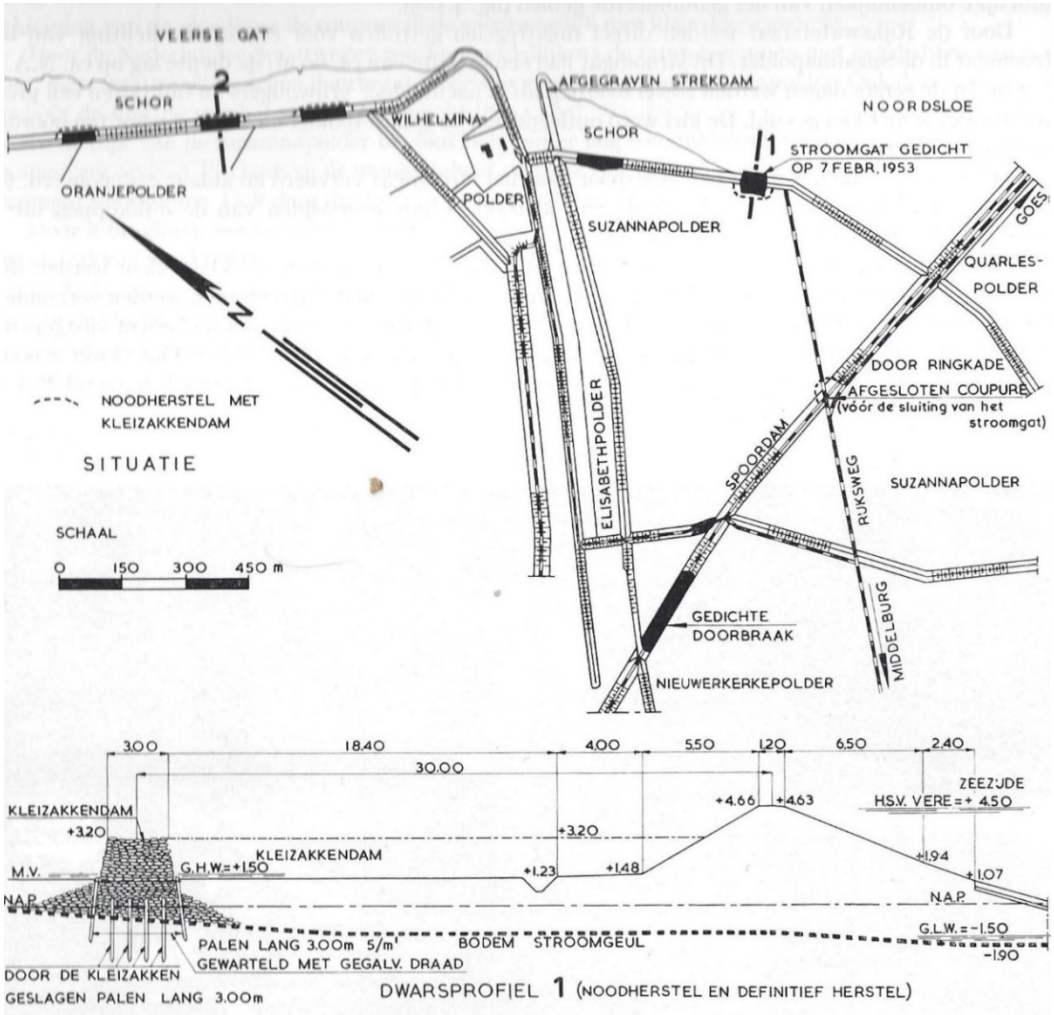


Fig. 10.20 Repair of breach in sea dike Suzannapolder, island of Walcheren; source: RWS (p. 639)

The tidal flow is stopped by means of a 200 m long perimeter bund comprising clay bags.
A calculation of the volume of the perimeter bund is as follows:

- bund sections on the undisturbed ground level in the polder, having total length of 140 m, up to NAP + 3.20 m — 980 m^3
- bund section in the scoured gully, long 60 m, up to NAP + 1.50 m — 840 ,,
- upper part of this bund between NAP + 1.50 and + 3.20 — 357 ,,

total theoretical volume of perimeter bund (excluding losses) — 2 177 m^3

On the basis of 70 bags in 1 m^3, this comes to roughly 152 000 bags.
The bund sections on the undisturbed ground level in the polder are constructed on 5 and 6 Feb. The closure works for the actual gully start in the morning of 6 Feb. by driving six rows of 3 m long stakes (Fig. 10.20) and by placing clay bags in between. The clay bags are pegged down. In this manner, by working day and night, the 60 m long closure is already effected in the morning of 7 Feb. At 12.00 a.m. the crest of the bund is at MHT. This means that 840 x 70 = 59 000 bags are placed in a period of 28 hours,

i.e. 2 100 bags per hour[22]. By 8.00 a.m. on the following day, the bund has reached its final crest level of NAP + 3.3 m.
No doubt this speedy closure of the tidal gully has been successful because of (a) the small tidal basin (50 ha), (b) the limited local depth of the gully (NAP – 0.50 m), (c) the high ground level of the polder (NAP + 1.2 m) and (d) the relatively small tidal range (because it is 2-3 days prior to neap tide): NAP + 1.4 to - 1.45 m.
A calculation, using the Coastal and River Engineering support system CRESS, turned out not to be possible for this situation.

10.16 Some remarks regarding the 14 presented projects and the works carried out

10.16.1 General

Below an effort is made to find what particulars the breaches[23] and the closure methods have in common. This should give an indication of the technical feasibility of simple means for the repair of dike breaches.
It is recalled that the storm surge flood of 1 February 1953 causes 89 breaches which can be considered as a flow gap. Ultimately, 14 projects, related to the repair of these breaches, have been discussed in Sections 10.1 to 10.15 (incl.) of this chapter. In the next sub-sections the following aspects will be discussed:

- physical particulars of these breaches and of the tidal area flooded;
- fluid mechanics;
- closure methods and materials applied for closure, deployment of manual labour;
- duration and timing of closures performed.

10.16.2 Physical particulars of the selected breaches and of the tidal area flooded

Physical particulars of the selected breaches are presented in Table 10.1.
From column 8 in that Table it follows that eight breaches have a length of 50 m or less while six breaches have a length between 50 and 100 m. The overall and average length are 797 and 57 m respectively. The 89 breaches mentioned by Rijkswaterstaat (RWS, p. 159) have an overall length of 5 683 m, which implies an average length of 64 m. This is not surprizing because even the major breaches, initially, have a rather moderate length[24] (RWS, p. 273 – 312).
The maximum depth in these breaches is for 11 nos. between NAP -1.5 and – 3.5 m, which means that they are, more or less 0 to 2 m deep below MLT. Only two breaches (nos. 45 and 48) have a maximum depth of NAP – 6 m. The rather shallow depth in 11 breaches is probably due to the presence of clay and/or compressed peat in the base (*zate)* of the dike section where the breach occurs. The depth in breach no. 45 must be due to the relatively small length in relation to the size of the tidal basin which results in maximum current velocities during a tidal cycle of more than 3 m/s as mentioned in Section 10.3. The depth in breach no. 48 has most probably been caused by scour of the sand layer under the dike and beyond. At 90 m from the breach the depth is still NAP- 4 to - 6 m. It explains also the large area of bed protection in the closure gap (1 000 m^2).
An important feature for closure of a breach is the ground level in the tidal basin. These ground levels are given in column 2 of Table 10.1. For most projects the ground levels have been taken as given in Hage (2015). These have been checked, in so far as possible, against ground levels shown on the related cross-sections in Rijkswaterstaat (1961) and/or Polytechnisch Tijdschrift (1954).

[22] But see also foot note 35 on page 237.
[23] Not in all cases the projects presented concern the direct repair of breaches: the compartmentalization bund '4de weg' (project no. 49), as such, is not a repair of a breach. Also projects may concern more than one breach (nos. 54 and 56)
[24] Like the initial length of the breach at Schelphoek/Serooskerke: 180 m, but it developed to a length of 525 m.

Discrepancies found have been solved by using ground levels shown on digitalized topographical maps of 1950[25].
It is important to note that, with the exception of project no. 44[26], the tidal basins have a ground level which is partly or completely above NAP. This reduces the period of tidal in- and outflow considerably. And, this in turn, renders the closure operation more easy.
Last but not least, the great variety in area of the tidal basins is noted: from 0.12 to 21.5 km^2. But this parameter has less impact on the magnitude of the closure operation than the ground levels in the tidal basins.

10.16.3 Fluid Mechanics

By using the Coastal and River Engineering Support System CRESS an impression can be obtained regarding the difficulties one can expect during closure operations. Preferably, the situation on the day of closure should be calculated but available information does not always render this possible. In Table 10.3 maximum current velocities and tidal volumes are presented. For projects nos. 50 to 53 (incl.) this is not possible. For projects nos. 44 to 47, 49 and 57 it is done for a springtide. Finally, For projects nos. 48, 54, 55 and 56 it is done for the days of closure.

Table 10.3 Area of tidal basins, tidal volumes, lengths of final closure gaps and max. current velocities as calculated for some projects

no. of project	breach in dike of:	area tidal basin (km^2) neap tide/springtide	tidal volume in million m^3	remarks	length of final closure gap(s) (m)	max. current velocity (m/s)
44	Noorddijk Papendrecht	5.4	2.4	springtide	110	1.7
45	Nieuwe Stadspolder	3.1	1	springtide	28	3.35
46	Jillesweg, Polder Oudeland	8	3.4	springtide	50	3.1
47	Kleine Zuiderpolder	0.12	0.1	springtide	30	0.2
48	Everingepolder	1.1	0.65	day of closure	23	2.5
49	Zuidvlietpolder	0.49	0,65	springtide	25	1.4
52	Perimeter bund Kadijk	not calculated	not calculated	neap tide	13x20 = 260	no info.
54	Klaas van Steeland polder	21.5	7.1	day of closure	64 & 16	4.95
55	Anna Vosdijkpolder	4.6	1.3	day of closure	60 & 20	2.6
56	Willempolder, 2 breaches[27]	7.5	1.1	day of closure	46.5 & 42.5	1.6
	east breach only		0.7	day of closure	42.5	1.6
57	Suzannapolder	0 /0.84	0.5	springtide	60	0.91

The max. current velocity as well as the tidal volume are an indication for the difficulty of closure. But, apparently, even in the case of high current velocities it is possible to close dike breaches by using simple means.

10.16.4 Closure methods and materials applied for closure, deployment of manual labour

In order to check and compare different closure methods as well as materials used, the information presented in column 11 of Table 10.1 is summarized in Table 10.4.
The Table merits a few comments. First of all about the bed protection of fascine mattresses and about the fascine dams (*rijzen dammen*). These are only used in seven[28] out of 14 projects. Apparently, it is quite possible to do without. In fact, in the 16th century, Vierlingh notes already that a bed protection and a fascine dam are not always needed[29] and … the use of fascine constructions is expensive! Also Matelé[30] (18th century) prefers to close dike breaches or tidal creeks without such constructions.

[25] Information received by courtesy of Water Board Scheldestromen, Middelburg.
[26] But the breach in this project has a ridge at NAP + 1 m, so it has the same advantage as the others.
[27] West and east breach.
[28] This figure includes the road base (twice) as it also functions as a kind of bed protection.
[29] Vierlingh (1920, 338).
[30] Matelé (1731, 2)

Table 10.4 Materials used for the closures in the 14 selected projects

no. of project	area flooded through breach in dike of:	bed protection (x)	fascine dam (x) or as ind.	materials used in closure: bags (b), rock (r), stakes (s), beams (bs), straw packs (sp), beet nets (bt)						cofferdam filled with bags	vertical (V) or hor. (H) closure
				b	r	s	bs	sp	bt		
44	Noorddijk Papendrecht	x		x	x						H
45	Nieuwe Stadspolder	x	x	x							V/H
46	Jillesweg, Polder Oudeland	x	x	x	x						V/H
47	Kleine Zuiderpolder	x	x	x							V/H
48	Everingepolder	x	x	x	x			x			V/H
49	Zuidvlietpolder			x			x				H
50	Reigersbergse polder			x							H
51	Compartmentalization bund '4de weg'	(road base)	pon-toons[31]	x					x		V/H
52	Kadijk (secondary dike)	(road base)		x							V
53	Lavendeldijk (second. dike)			x	x					x	V
54	Klaas van Steeland polder			x	x		x				H
55	Anna Vosdijkpolder			x	x	x	x		x		V/H
56	Willempolder			x			x			x	V
57	Suzannapolder			x						x	V/H

The item used in all 14 projects is the bag, i.e. the jute bag filled with sand or clay and, sometimes, rock. RWS (p. 364-365) is rather vague about the volume of a bag. Also the weight (actually mass) is not given. But it can safely be assumed that in the Netherlands such mass (to be carried on a shoulder or on the back) will not have been more than 20 kg. If the mass of 1 m^3 of dry sand is assumed to be 1 500 kg, one m^3 of a bund of sandbags will then contain a maximum of 1 500/20 = 75 bags. But this ignores the voids left between dumped or stacked bags. If the percentage of voids is estimated at 5 %, the number of bags in one m^3 goes down to approximately 70 bags. This assumption of 70 bags in 1 m^3 is based on the various calculations and information presented for the 14 projects in Sections 10.2 to 10.15. It is known that, especially at the start of the closure works, available bags had all kind of sizes. The figures in red (based on 100 bags in 1 m^3 as calculated in Ch. 3, Sub-section 3.3.2.1) in Table 10.5 are undoubtedly impossible as they exceed those given in Rijkswaterstaat (1961). The various materials (apart from the bags), listed in Table 10.4, as well as the cofferdams, all have as purpose to render the placing of bags possible in moderate to high current velocities. In Table 10.5 an effort has been made to compare the number of bags placed with the labour input. This has not been very successful as for most projects essential data are lacking. No doubt the local circumstances to a large extent determine logistics and these have a high impact on production figures.

Table 10.5 Bags placed in projects and labour input

no. of project	area flooded through breach in dike of:	vol. of bags in closure bund (m^3)	theoretical number of bags in bund (x 1000)			no. of bags placed according to RWS (x 1000)	labour input		remarks
			50 bags/m^3	70 bags/m^3	100 bags/m^3		no.	bags placed per man-day[32]	
	1	2	3	4	5	6	7	8	9
48	Everingepolder	196		14					bags placed in 12 h
50	Reigersbergse polder	400		28					bags placed in 11 days
51	Compartmentalization bund '4de weg'	13 200	660	924	1 320	1 200	1 800	32	figs. In col. 2-5 exclude failures and gullies
52	Kadijk (secondary dike)	2 450	123			91[33]	1 000	92	92 bags are placed in appr. 5 h. by each lab.
53	Lavendeldijk (second. dike)	1 063	55-60						
54	Klaas van Steeland polder	1440 + 100 924	80 45	110 65	155 90	700 (overall)	600 600	183 & 54[34]	figs. In col. 2-5 exclude losses
55	Anna Vosdijkpolder	5 321		372		550	1000	?	
56	Willempolder, west breach	570		40			166	?	
56a	All four breaches on the island St Philipsland	4 612		323	461	324			
57	Suzannapolder	2 177		152					59 000 bags placed in 28 h, i.e. 2 100 /h[35]

[31] Pontoons have only been used to a limited extent for this project.

[32] Calculated on the basis of column 6.

[33] This is the number of bags placed on day of closure.

[34] Calculated on the basis of column 4.

[35] This figure is based on the assumption that the cross- section of the bund in the gully below MHT (Fig. 10.19) is directly being built to its full profile (width at base 10 m). This is probably not correct. If it is assumed that in first instance a bund has been built having a base of 5 m and a crest width of 1 m, then the number of bags comes to (5 +1) x 2/2 x 60 x 70 = 25 200 bags or 900 bags/hour, which is still a high figure.

10.16.5 Length, duration and timing of closures performed

In Table 10.1, column 9, the lengths of 17 final closure bunds are given for the 14 closure projects discussed in this chapter. Three closure bunds (nos. 44, 50 and 51) are so-called gradual horizontal closures having lengths of 110, 300 and 1 800 m respectively. The other 14 closures can be considered as speedy closures and the lengths of the closure bunds across the final closure gap is:

- 16 to 50 m for 10 closure bunds: the projects 45 to 49 (incl.), 53, 54 (7 Feb.), 55 (20 Feb.) and 56 (two gaps);
- 60 to 64 m for 3 closure bunds: the projects 54 (5 Feb.), 55 (19 Feb.) and 57;
- 260 m (nett) for closure project no. 52.

The closure operation can be considered to be a mixture of a horizontal and vertical closure except for the project last mentioned. Here a vertical closure has been applied. This project no. 52 can, like the projects nos. 50 and 51 also be called a '*maaiveld sluiting*' which can be translated as OGL – closure[36], i.e. the OGL is used as base for the closure bund.

The basic principle of emergency closures as the ones described in the foregoing sections is '*time is the essence'*. You have to prevent the breach from 'deteriorating', i.e. getting longer and deeper. So the more quickly you close a breach, the better it is. But this is not always possible as can be expected because of difficult logistics, lack of materials and lack of manpower[37]. The sites of the breaches are difficult to reach, local inhabitants have been evacuated. Flooded lands limit working space. Required materials are also on demand for many other closure projects.

But, as soon as these problems have been solved, the actual closure works can be carried out and these, in general, do not take much time: from a few days to a few weeks.

It is remarkable that, notwithstanding the aforementioned difficulties, many successful closures take place at an early date:

- five closures already in the first week after breaching;
- another five during the month of February;
- four during March, and,
- the last two in April.

But one should add: No time has been lost due to 'red tape', long tender procedures, site surveys, scale modelling or time-taking considerations regarding design alternatives.

Not only time was the essence but also the readiness to improvise and willingness to take the risk of failure of a closure project.[38]

[36] OGL stands for original ground level which is the most accurate translation of *maaiveld.*

[37] The lack of equipment is not mentioned here because it concerns closures carried out by simple means and an overpowering role for tugboats, draglines, floating cranes, flat deck barges, suction dredgers and the like is not foreseen in these projects.

[38] Projects nos. 47, 54 and 55 failed the first time. Project 53 failed twice. In the long (1 800 m) project no. 51 four failures occurred but it was still closed over its full length only 3½ weeks after start of the works.

CHAPTER 11 Closures in Ghana and Bangladesh during the period 1969 – 1985

11.1 Introduction

The closure of tidal channels by using simple means, outside the Netherlands in the 20th century , including abundant use of manual labour, starts in Ghana in 1969. Due to this successful closure, Van Duivendijk obtains the assignment for similar work in Bangladesh "because of his knowledge of that country and his experience of closing tidal channels by means of labour-intensive methods without the application of modern large equipment" [1]. This in turn leads to the closure of the Feni river in 1985 by 10 000 men.

11.2 Closure of a flow gap in the sand ridge of Keta lagoon, Ghana, in 1969[2] (project no. 58)

11.2.1 General

Keta (Ghana) is a town on a sand ridge between the Keta lagoon (area 300 km^2) and the Gulf of Guinea. The lagoon is normally practically completely separated from the sea (Fig. 11.1). The water level in the lagoon goes up and down with the seasons and in most of the years this fluctuation causes only limited hardship: a few fields are inundated, a few dm of water in houses at the outskirts of the town. But in some years the rainfall in the catchment (4 000 km^2) is heavy and the whole town of Keta is flooded (Fig. 11.2) because now the water level has not gone up by one metre as usual but by two metres. The Government is then asked to help and in 1963 as well as in 1968 an opening is made in the said sand ridge in order to discharge excess water from the lagoon into the sea. A few days later the water level in the lagoon has gone down by one metre and the flooding problem has been solved.

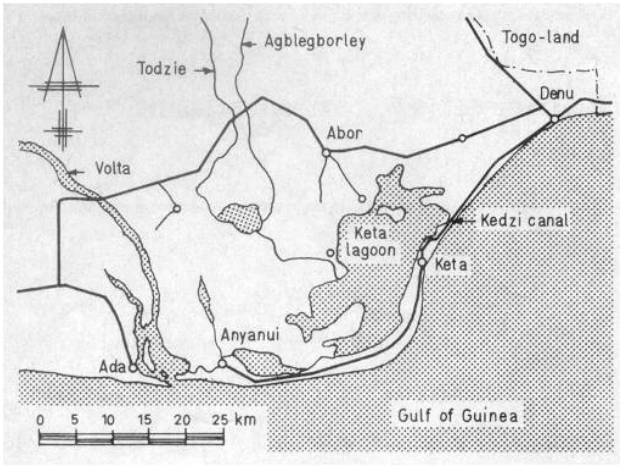


Fig. 11.1 Location Keta lagoon in Ghana

Fig. 11.2 Flooding of Keta in 1963

The idea behind the cut in the sand ridge is that it will close itself within a number of months. This happens indeed in 1963/64: after ten months the flow gap (tidal gully) closes itself. In 1968/1969 the

[1] The Ministry of Development Cooperation in: Van Duivendijk (1977).

[2] The source of information for this closure is the article written by Van Duivendijk (1970, B 61 et seq.) as well as the contract document for construction (NEDECO, 1969) and the report (Write Up) 'Keta Lagoon, Closure of tidal inlet (called Kedzi Canal) in October 1969' (NEDECO, 1969a). To avoid misunderstandings: the contract document as well the Write Up have also been written by Van Duivendijk.

situation is different. Also then a cut is made in the sand ridge which, after five days, is 250 m wide and 4.5 m deep. But after 11 months it is still open and it is shifting in eastern direction.

This is due to littoral sand transport along the coast in easterly direction. In 1963/64 the gap shifts 200 m but in 1968/69 the shift is 360 metres; 200 houses disappear in the gap (Fig. 11.3), 1 000 persons become homeless. In fact the whole population suffers because vehicular traffic along the coast is interrupted (Fig. 11.4) and waves cause damage to the sand ridge. In 1969 the narrow sand ridge loses 100 m of its width over a distance of 1 to 2 km and parts of the road along the sand ridge disappear as well.

Fig. 11.3 House destroyed by the current

Fig. 11.4 Ferry across the Kedzi Canal

11.2.2 Plan for closure of the flow gap

In July-August a plan is made[3] to close the flow gap[4]. Its width is now 170 m at mean sea level (MSL) and the maximum depth is 4.5 m. Tidal differences are 0.30 (neap tide) to 1.50 m (springtide). As current velocities appear to be 0.5 to 1.5 m/s, the tidal volume must be in the order of 4 to 5 million m^3.
For Dutch engineers, then working on the Delta Project, closure of this tidal channel must seem to be easy but in Ghana there are some constraints.
First of all instrumentation and data: during the field visit there is no way of measuring the depth of the gap other than by tying some range poles (*jalons*) together, no boat with outboard engine, no tidal gauge near the gap, no detailed map of the (shifting) gap. The tide table for Takoradi (350 km west of Keta) is hopefully also applicable for Keta .
Then equipment: no suction dredgers, tugs or flat deck barges. Dry earthmoving equipment is in the country but it might be too large to board the ferry across the Volta river.
Finally, closure materials: coarse sand is abundantly available at the west side of the gap. But there is no clay. Maybe, rock can be obtained from an unused quarry at 70 km distance.
There are small stock piles of concrete blocks near the houses but, later, it turns out that the owners do not want to sell these.
Locally, mats are made of cocos palm leaves, reed and straw. Maybe they can be used for bed protection. Last but not least: there are jute bags in the country but they are rather big for carrying.
Based on these constraints the plan for closure (Fig. 11.5 and 11.6) is as follows:

- In order to stop further shifting of the gap a shore protection is made at the east side of the gap using mats ballasted with concrete blocks or sandbags;
- keep the axis of closure as far as possible away from the sea (to prevent/reduce wave attack);
- try to make a bed protection (a sill, comprising mats loaded with sandbags) on the eastern foreshore, from the bank down to a level of 1 m below Low tide;

[3] NEDECO (1969).
[4] Called locally the Kedzi Canal as it is near the village Kedzi.

- make a sand stockpile on the west bank and then start pushing sand into the tidal channel;
- close the last opening (closure gap) during a neap tide by dumping concrete blocks or sand bags on the sill.

Obviously, many uncertainties are included in the plan:

- Nobody knows how large current velocities will become during this horizontal closure;
- if only bulldozers type D4 can cross the river Volta by ferry the earth moving (dozing) capacity may turn out to be too small in view of the erosion by the tidal currents;
- closure is foreseen for 7 September, only four weeks after the plan is submitted (9 August); but this is done on purpose because it keeps everybody on the alert; in fact the closure is made on 3 October thanks to this, too optimistic, time schedule;
- nobody knows whether the bed and shore protection will remain stable in the tidal in- and outflow of the lagoon;
- work is only possible during the day; it implies that by the end of the day the works have to be protected against erosion during the night.

But it is assumed that all difficulties can be solved in the course of time!

During August a further shifting of the tidal channel occurs and there is a danger that the sea will break through the sand ridge.

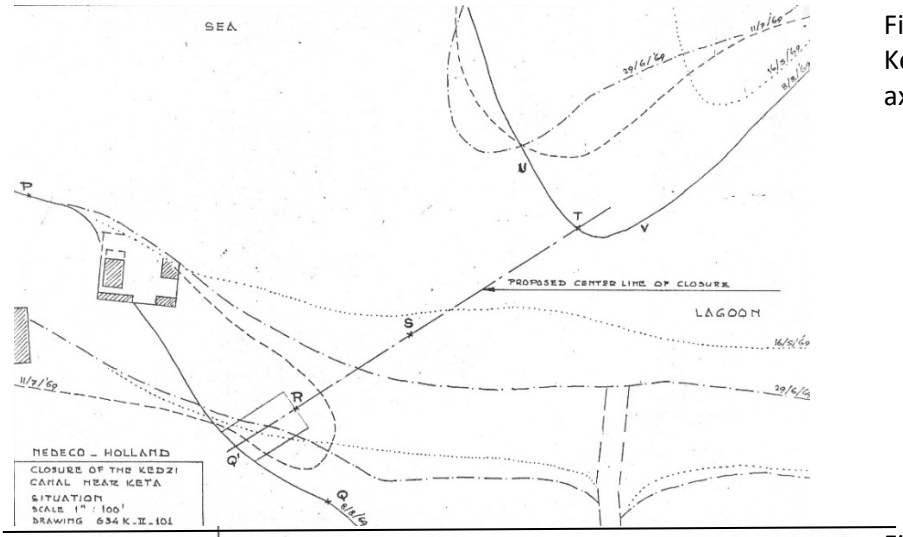


Fig. 11.5
Kedzi canal, proposed axis of closure shown

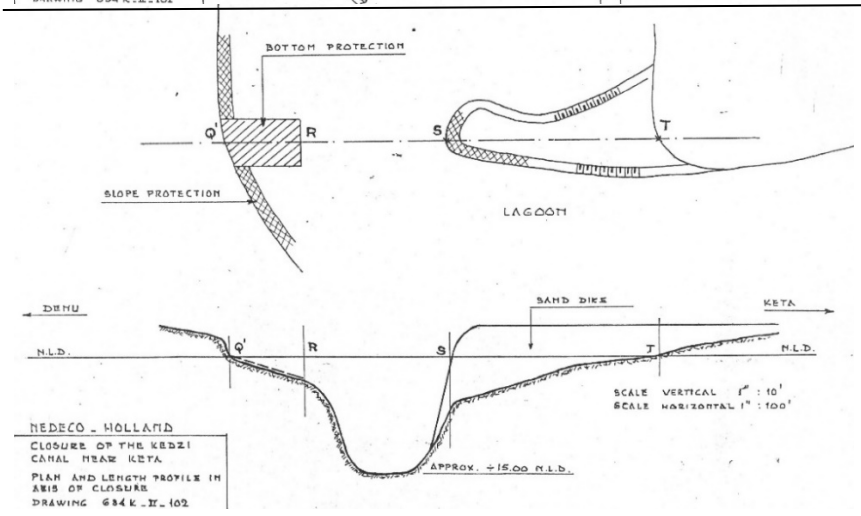


Fig. 11.6
Kedzi canal, slope and bed protection (Q-R) shown, construction of sand dam in progress (S-T)
Note: on various drawings levels are given in relation to the Ghana survey datum NLD; at the site of the works MSL = NLD + 0.40 m

The work will be constructed in day works by a state contractor. He arrives at the site on 4 September and the work can start in earnest. He is successful in transporting two bulldozers type D6, one bulldozer type D7E and two scrapers type TS 14 across the river Volta to the site. As the concrete blocks are not available, the operation is now a sand-moving, sandbag-filling and sandbag-transporting/placing operation.
A few comments are now be made and characteristics given of the various closure operations.

11.2.3 Protection of slopes etc.

This operation is very successful for three reasons:

1. the crew of 20 fishermen, employed by the contractor through the good offices of the Chief of Kedzi village, after a few days, knows exactly what to do and carries out its work mostly in the tidal region in currents, sometimes up to 2.5 m/s, to everybody's full satisfaction,
2. the straw mats chosen for the operation are strong, thick (ca. 3 cm) and dense (no 'leakage' of sand),
3. the sandbags, if filled completely, have a dry weight of 200 kg and dimensions of 25 x 60 x 90 cm, volume is 0.135 m^3.

The work starts with the protection of slopes along the east bank. A strip having a width of 3 m is protected over a length of 200 m (Fig. 11.6 and 11.7). Bags are only filled for one third to acquire a continuous layer of bags without any space in between.

Fig. 11.7 Placing mats along east bank

Fig. 11.8 Placing mats under water as part of the sill; sand stockpile in background

The bed protection across the tidal channel is placed in the same manner, initially over a width of 27 m, length 27 m. In view of the reduced depth of the tidal channel it is possible to place another 5 mats (one after the other, width 3 m, overall length 28 m) in front of this protection. The fishermen are able to place and ballast these mats neatly at a depth of nearly 2 m below Low tide (Fig.11.8).
During the construction of the sand dam it becomes necessary to protect the head of the pushed out bund each afternoon by placing and ballasting mats (Fig.11.9) This is done in order to avoid serious scour during the night. These protection works become more and more sophisticated as the gap narrows and velocities increase (Fig. 11.11). Each afternoon some 200 to 250 heavy bags are placed over an area of 80 to 120 m^2, most of it under water; 30 to 40 men and women are involved in filling (Fig. 11.10), transporting and placing the bags.
It turns out that in general such protection remains stable up to a current velocity of 2 to 2.5 m/s. Overall, some 2 500 m^2 of mats have been used during the operations described above.

Fig. 11.9 Placing slope protection; stockpiles of sandbags for final closure in the background on east bank

Fig. 11.10 Filling the large sandbags

11.2.4 Sand placing

The sand dam is built forward each day with the aforementioned protection as a starting point. This protection is always left in place to have a defence to fall back on, in case of failure (machine breakdown or lack of fuel). During 10 days, sand is pushed and daily progress gradually decreases from 22 to 15 metres/day. During the last days 50-60 % of the sand pushed forward by bulldozer is lost, the actual percentage depending on the tidal amplitude and the time of pushing in relation to the tidal curve.

11.2.5 Placing of sandbags during final closure

On the afternoon of 2 October the edge of the bed protection (sill), placed from the east bank on the foreshore to a depth of 2 m below Low tide, (Fig. 11.11) is reached and final closure is planned for the next day. For this purpose, during three weeks prior to day of closure, 9 000 bags have been filled on the east bank half with sand (dry weight of bag is appr. 100 kg[5]).

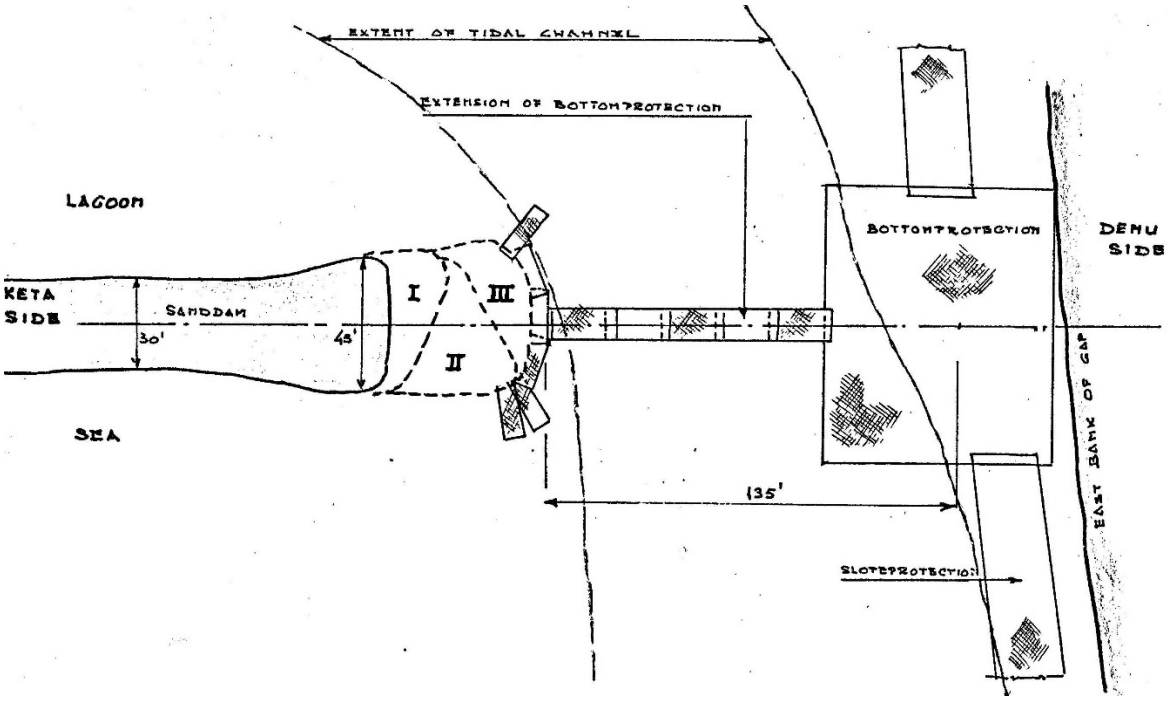


Fig. 11.11 Progress of sand dam on 2 October 1969 (last day before final closure)

At 8.15 a.m. operations start with 40 labourers. Between 10.00 and 10.30 a.m. 300 students arrive from secondary schools and soon the narrow bund made of sandbags starts to grow. The whole gap,

[5] It was the intention to fill all the bags up to a dry weight (mass) of 100 kg but a visiting Government official disagreed, “as these bags are expensive”! So a number of the bags had a dry mass of 125 kg.

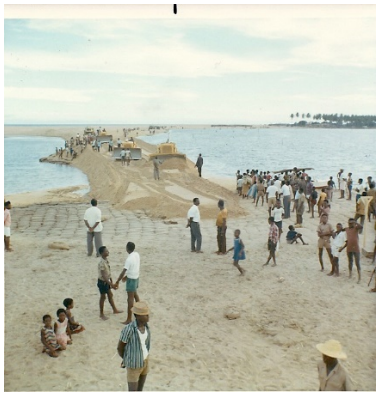
measuring 45 m (length) by 2 m (depth), is actually closed in four hours by, on average, 250 men having dumped 6 000 sandbags. Dimensions of this closure bund are: length 45 m, height 3 m, width of 3.5 m(base) to 2 m (crest). Thus, volume of closure bund is 405 m^3. Accordingly, 16 bags in 1 m^3. The production per manhour looks small (6 bags) but one should realize that a heavy bag has to be carried by four (young) students (boys and girls). So transport and placing of a bag requires four students on land, two men for transport on dam and one man for actual placing. The actual production is 42 bags per hour per crew. Production during four hours: 250/7 x 42 x 4 = 6 000 bags
During this horizontal speedy closure the turning of the tide, lasting 1½ hours, is fully used but, before and after, velocities are high (2.5 m/s).
The problem turns out to be the dumping of bags in flowing water. Solely dumping the bags is not enough. They only have been filled half with sand (otherwise the weight would have been too much for transport) and they tend to float away in the current.
The system is now to grab the bag (dumped or handed to the small crew of people standing in the water), stand on it till all air (free in the bag and in the sand) has escaped and the sand is soaked by water and, subsequently, put the next bag on top of it or at its side. An additional problem is how to remain standing in the fast flowing water. People hold to each other in so far as possible. As, however, each of the crew placing the bags in the water can swim, this is a relatively small nuisance. The problem could have been solved easily by introducing an anchor with rope to which safety belts are attached.

Fig. 11.12 Morning of closure, weather is bad; stockpiles of sandbags in background

Fig. 11.13 9 a.m. closure has started from east bank (in foreground)

12.00 a.m. fight against the tide (ebb)

1.00 p.m. bund closed



afternoon: dam made safe for the night[6]

Fig. 11.14 Closure operation

[6] Safe in this case has a double meaning: safe against wave attack by the sea and/or overtopping by a high tide, but moreover: safe against possible sabotage: during the night the ferrymen ,who now had lost their job, tried to breach the closure bund, but they were not successful.

11.2.6 Some additional remarks

11.2.6.1 Introduction

In the foregoing sections a straightforward description has been given of the closure works. But certain considerations made during construction and difficulties encountered (and how they were solved) are lacking. These will be discussed in this sub-section.

As already stated in Sub-section 11.2.2 there were not much data available which could give an indication of the hydraulic features to be expected during the closure operation. The supervisory staff, during the critical period of the work (say the last two weeks) was guided by certain principles which in general are always applicable to closures of tidal gaps. These principles do not specifically relate to any circumstances prevailing for the closure of the Kedzi Canal but they are repeated here because of the unusual circumstances.

The principles are, inter alia:

- try to close the gap during the period of neap tide,
- as scour is a function of time etc. try in any case to effect final closure as soon as possible after the start of the works in the tidal gap,
- always try to advance with the sand dam during periods of or in places having low currents,
- never leave any embankment or slope subject to scour, without protection during a longer period, and, last but not least:
- because of lack of knowledge the attack must be careful but use every opportunity to see how far one can go without serious loss of progress achieved earlier.

Having noted these principles the story of the successful closure from a hydraulic point view can be described as follows:

11.2.6.2 Inflow into the lagoon and maximum depth in the gap

As stated in Sub-section 11.2.2 during the survey in July-August depths of 4.5 m are measured in the gap. On 17 of August however the max. depth had been reduced to 3 m. It was suggested that this depth in fact is in line (in equilibrium one would say) with the tide and bed material in the gap. The greater depth being caused by scour due to heavy discharges of Todzie and Agblegboley rivers into the lagoon during July, which inflow has to be evacuated into the sea through the Kedzi Canal. During August and September there is no inflow into the Lagoon from these rivers and also only limited rainfall. Consequently the max. depth measured decreases to 2 – 2.5 m by the end of August.

11.2.6.3 Scour in front of the sand dam

Progress during the last week prior to final closure is determined inter alia by the scour - time relationship. It was found that during the night in general the unprotected bed of the tidal gully in front of the protected end of the sand dam is being scoured 0.5 to 1 m. This means that continuous progress is essential. More scour would have meant steeper slopes for the protecting straw mats and sliding off of the heavy sandbags. Also more embankment material would have been needed.

11.2.6.4 Day to day planning

Actual day to day planning is based on experience of the previous day, expected time of turn of tide and expected labour output. The strategy of the last 4 days prior to final closure is the most interesting and is given below.

11.2.6.5 Monday 29 September

Pushing of sand along sand dam into gap starts at 8 a.m. and continues until 2.15 p.m. This is 6¼ hours. on the 28th there were 4½ hours and then also 25 m were gained. This 'decrease' in production must be due to: (a) dam is longer, so hauling distance increased, (b) ebb currents high, estimated at 1 to 1.5 m/s.

Pushing of sand continues until 2.15 p.m. Turn of tide is at 2.50 p m. (3 hours after Low tide Takoradi). Protection work starts immediately after bulldozers have left the narrow (Fig. 11.15) sandspit. Protection works actually started too late because were only ready at seaside when the eroding flood currents were already quite strong. The protection works always start at the seaside prior to turn of tide and it is the intention to be far enough advanced at the time that flood currents exceed 1 m/s and at that time protection at lagoon side can be taken in hand. For protection works see Figs. 11.15 and 11.16.

1.2.6.6 Tuesday 30 September

Ebb currents have become now so high (2.5 m/s) that pushing during ebb (ebb is now during the morning and this is the only time one can push) is no longer favoured. In fact one seems to lose more than one has gained. As a rainstorm (squall line, see foot note 7) stops work for one hour, 8 m of (unprotected) dam are lost again and the balance of today is only ± 10 m.

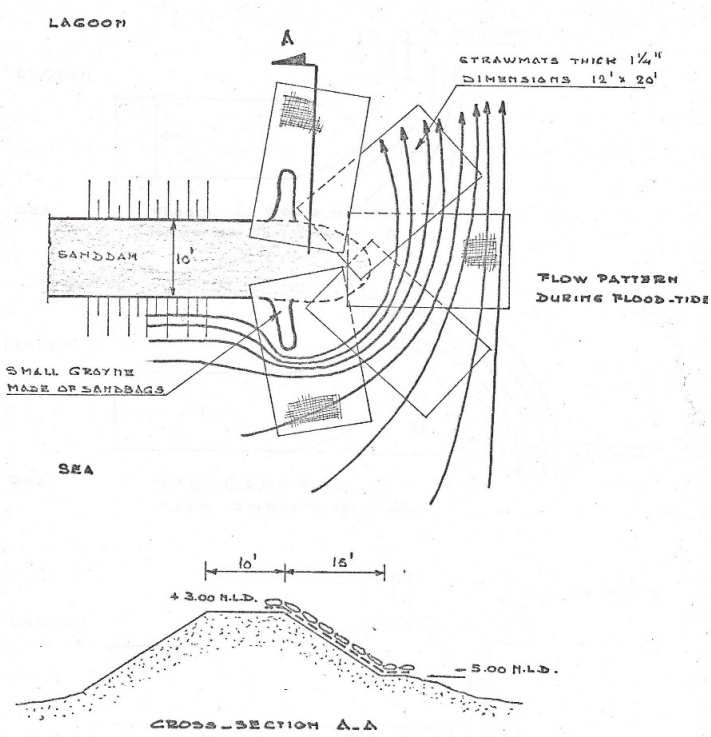


Fig. 11.15
Protection at end of sand dam made by placing straw mats ballasted by 5 cu. ft. bags

Today the protection works start at 1 p. m. and this is early enough to obtain an optimum result. The sandspit is also wider now: 7 m against 3 m yesterday.
The ebb current decreases to 0.3 m/s at 2.15 p.m. but after 20 minutes increases again to 1 m/sec.
The turn of the tide is at 3.10 p.m. (again 3 hours after Low tide Takoradi).
Flood starts at 3.20 p.m. Maybe the rainstorm has caused a wind set-up on the lagoon which could be the reason for the strange ebb flow pattern.

1.2.6.7 Wednesday 1 October

Today the strategy was changed successfully. The story can best be illustrated by sketches (Fig. 11.16) and a table (Table 11.1).

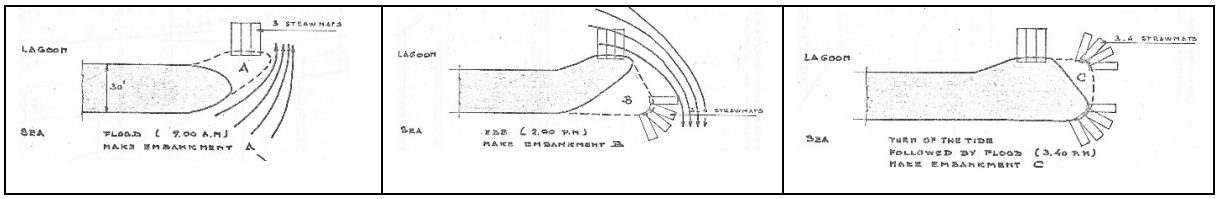


Fig. 11.16 Progress of sand dam in relation to the tidal current (1 October 1969)

Table 11.1 Diary of activities on 1 October

time	activity
8 – 8.40 a.m.	continue making stockpile of sand
8.40 – 9.15 a.m.	make embankment 'A' in shadow of flood current
9.15 – 10.25 a.m.	make slope protection lagoon side (3 mats of 22 m^2 each)
10.15 a.m.	turn of the tide (3½ hour after Takoradi)
10.20 a.m.	already considerable ebb currents
1.30 p.m.	new stockpile of sand made
1.30 – 2 p.m.	embankment 'B' made
2 – 5.25 p.m.	make slope protection (3+ 4 = 7 mats of 22 m^2 each)
3.40 p.m.	Turn of the tide, make embankment 'C'

Because of this new method the end of the dam has become square, width 15 yards at the level of High tide. Distance gained today is 20 m.

1.2.6.8 Thursday 2 October

The method used yesterday is nearly not feasible today. Influence of eddies is so strong that only the continuous pushing (and loss) of enormous masses of sand results again in a progress of 20 m. The aim was to reach the bed protection (sill) laid earlier from the other bank and this is achieved. Today's progress and difficulties can again best be shown by looking at sketches (Fig. 11.11) and quoting an extract from the diary (Table 11.2).

Table 11.2 Diary of activities on 2 October

time	activity
8 – 8.25 a.m.	high stockpile of sand completed
8.25 – 10 a.m.	push sand in area I
8.20 – 10.35 a.m.	limited protection works in zone I
10.35 a.m.	turn of the tide (3 hours after Low Tide at Takoradi)
after 10.35 a.m.	pushing of sand in area II
11 a.m. – 2 p.m.	new stockpile made, heavy ebb-current (eddy) erodes and removes area II completely, protection works partly destroyed; ebb-current seemed to be 3 to 3.5 m/s, therefore waited until:
2.45 p.m.	start for area II to be filled up
3.15 p.m.	protection works for zone 2 started, though ebb currents still very strong, but in view of coming darkness (6 p.m.) a start must be made; reason for heavy ebb currents must be the wind over the lagoon in direction Kedzi Canal which causes wind set-up
2.45 – 5.00 p.m.	continuously pushing of sand to hold area II, to protect the protection works being made in zone 2 and, finally, to fill up area III.
5.20 – 6.00 p.m.	turn of tide (3.25 hours after Takoradi); it is still turn of tide but sand dozing stops, as it is dark now. Protection works are ready though not as extensive as one would wish; width of dam now 20 m; gained today 20 m

1.2.6.9 Friday, 3 October: Day of closure

A neap tide is forecasted for today but between 7.00 and 8.00 a.m. a squall line[7] develops. This thunderstorm has a most peculiar effect. Between 8.00 and 9. 00 a.m. the water level at sea goes up and down: 3-4 times the water level is so high (MSL + 1.3 m) that flood currents must have reached 4 m/s. In this respect reference is made to Fig. 11.12.

It could be seen that bed protection formed in fact a sill in the gap. The slope protection made on Thursday starts already to disintegrate (bags sliding off mats). But the flood currents are back to normal (2 – 2.5 m/s) at 9. 00 a.m. During closure, ebb currents become as high as 2.5 m/sec. (30 cm difference in water level between lagoon and sea, vide Fig. 11.14 and 11.17). Obviously, the waters pushed into the lagoon during the squall line have to flow out again during the ebb.

After the closure an effort is being made to determine the scour at both sides of the protected sill: depths are found of 2.1 to 2.7 m below MSL, the sill itself being at a level of MSL – 1.5 m.

[7] A squall line is a fast-moving system of thunderstorms in a long, narrow line formation. They can stretch for hundreds of miles (typically arranged north-south) but are usually less than 32 km (20 miles) wide. As they sweep across the landscape, they deliver a short, violent burst of weather that can include dangerous straight-line winds, lightning, hail, and torrential rain. Squall lines tend to form in the mid-latitudes, where they move from west to east, or the tropics where they move east to west.
Source: https://www.sciencefocus.com/planet-earth/what-is-a-squall-line/ (accessed on 20 Nov. 2023)

11.2.7 Quantities and Cost of construction

The original cost estimate[8] for the closure works in Ghanese cedi (GHS) was ¢ 74 000 which, (in 1969) is equivalent to approximately f 250 000. This estimate is so high because of the relatively expensive concrete blocks (representing 46 % of total cost) and the depth of the tidal gully. Ultimately, no concrete blocks have been used and, at the time of closure, the gully, meanwhile had lost 50 % of its original depth.

Fig. 11.17 Increase of the current during narrowing of the closure gap

The contractor chose to use heavy earthmoving equipment and a quick calculation showed that even with a (not expected) loss of 2/3 of the sand pushed into the Kedzi canal, the construction cost would be much less than originally foreseen. This is because purchase and placing by hand of concrete blocks is always much more expensive than hauling and placing of sand by earth- moving plant. On the 3 October 1969 (day of closure) the estimate of the cost of the works done so far is ¢ 32 200, which is equivalent to approximately f 110 000 (exchange rate ¢ 1.00 = f 3.40).Table 11.3 provides details. A purchasing power of f 110 000 in 1969 would have a purchasing power of € 240 000 in the year 2020.

[8] Called 'guestimate' because of the many uncertainties.

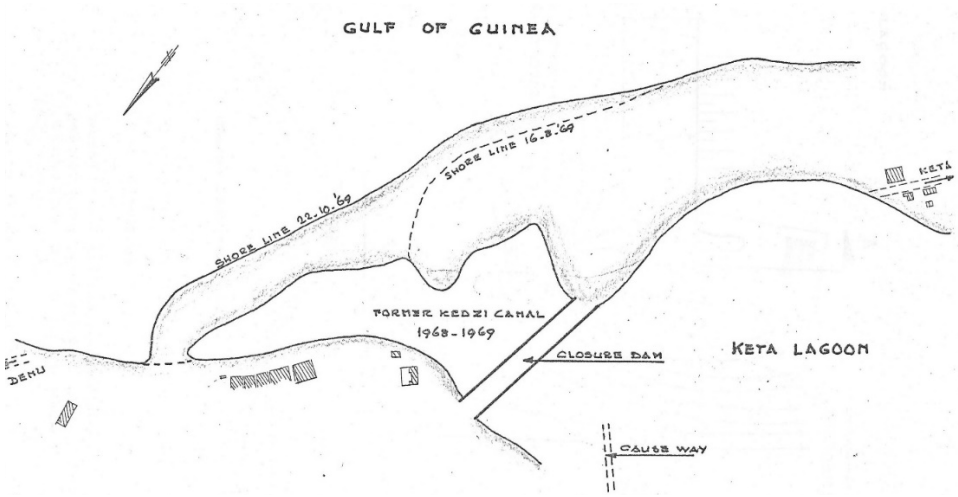


Fig. 11.18 Re-establishment of the sand ridge along the sea at the former Kedzi canal within three weeks after closure

Fig. 11.19
The sand ridge along the Keta lagoon at the former Kedzi canal in January 1976;
source: NEDECO (ir. R. Glazener)

Table 11.3 Estimate of construction cost up to and including 3 October 1969

description of plant	unit	quantity	rate (¢)	total (¢)
labourers	day	2 000	1.-	2 000
sandbags	no.	20 000	0.58	11 600
straw mats	square yard	3 000	0.15	450
dozer, type D 6	hour	415	11	4 565
dozer, type D 7	hour	70	15	1 050
dozer, type D 8	hour	20	15	300
scraper Euclid	hour	265	20	5 300
transport of plant and materials	Lump Sum		(estimate)	4 000
total				29 265
overheads and profit 10 %				2 926
overall total				32 200

The exchange rate between the cedi and the guilder of 1969 cannot be compared with the exchange rate between the cedi and the euro of 2020:
- in 1969 one cedi is equivalent to f 3.40 which is € 1.54,
- in Jan. 2020 one cedi is equivalent to € 0.15.

11.3 Dutch labour-intensive closure methods in Bangladesh during the period 1978 – 1982 (project nos. 59, 60 and 61)

11.3.1 Introduction

In May 1976, the International Technical Assistance Department (DTH) of the Ministry of Foreign Affairs of the Netherlands commissioned ir. J. van Duivendijk of Haskoning, Consulting Engineers and Architects, to carry out a study into the methods being used in Bangladesh for closing tidal channels. This commission was connected with the execution of so-called 'early implementation projects' in which The Netherlands (Ministry of International Development Cooperation) was involved. These are drainage and flood control projects which, at that time, were carried out in Bangladesh.
Bangladesh is a delta formed by three large rivers and the conditions regarding morphology, hydrology and tidal movement are more or less similar to those prevailing in the south-western part of the Netherlands.
In the sixties of the 20th century in East-Pakistan[9] a number of tidal channels had to be closed as part of the first phase of the Coastal Embankment Project. For these tidal closures a local method was used, known as the mata method[10]. Van Duivendijk and his colleagues studied the mata method and concluded that, though it is a technically feasible method, it is risky and only suitable for closing small size final closure gaps (say 12 to 15 m long) and tidal volumes not exceeding 2 million m^3 during the closure period. It was therefore proposed to introduce a vertical[11] closure based on labour-intensive methods, as used by the Dutch for the last time in 1953, and by applying locally available materials. While the mata method is basically a land-based operation (horizontal closure), the vertical closure is a combined land-based and water-borne operation.
After studying the application of the mata method in 1976 and 1977, 'fascine' [12] mattresses were successfully constructed and applied in 1978 as bed protection for a mata closure.
In the period 1979 – 1982 three vertical closures, designed and supervised by Dutch experts[13] have been realized of which one failed, four days after closure. In the following sub-sections the four closures in which Dutch methods have been introduced are discussed in greater detail[14].

11.3.2 Outline of the vertical closure concept

In principle the vertical closure concept comprises the following activities:

- a design based on physical site conditions (topography, bathymetry, tidal movement, soil data and available type of fill in potential borrow pits;
- preparation of tender documents, tender procedures and award to a contractor;
- construction of a bed protection;
- construction of a horizontal sill across the tidal channel;
- construction of a jetty to enable the dumping of closure materials on the sill;
- completion of closure dam[15] by borrowing, transporting (in head pans carried by labourers) and dumping of fill in profile of dam body.

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[9] At that time the region, known in colonial times as East- Bengal, was still part of Pakistan. The region declared itself independent from Pakistan in March 1971 under the name Bangladesh.
[10] This mata method will be described in detail in Chapter 14.
[11] The word 'vertical' is slightly misleading as in practice it turned out to be a vertical/horizontal closure (ref. Chapter 5).
[12] The word 'fascine' is used here to indicate the type of mattress but in its composition no fascines have been applied.
[13] The 'Dutch experts' in this case were employees of the engineering consultancy firm Haskoning (now Royal HaskoningDHV), Rijkswaterstaat and ACZ (a firm specialized in fascine works, now part of Van Oord).
[14] The text in the present and following sub-sections has been derived from the reports/articles listed in the References under the headings 'Bangladesh Water Development Board', Smith and Van Duivendijk (1977, 1980 and 1986).
[15] Locally called cross dam.

Basically, this concept is the same for all closures. It was therefore decided not to discuss each closure separately but to discuss each of aforementioned activities and highlight the particular characteristics of the activity as well as the possible modifications/ adaptations for an individual closure.

11.3.3 Physical site conditions and basic design of closure bund

In Table 11.4 the available physical site conditions have been summarized. However, these are not always consistent and some are rather vague (for instance, soil conditions inside the creek at bed level for Chakamaya are stated to be 'relatively hard'). The data about the (failed) mata closure in 1978 at Madargong creek are also given in the Table as here the experiment with bed protection mattresses started.

On the basis of these physical conditions, for the damming of the Chakamaya and Madargong creeks, design reports have been prepared[16]. In the report about Madargong also specifications for construction have been included.

Already in the first design report (BWDB, 1978a) the Dutch closure method to be adapted to Bangladesh' circumstances, as shown in Fig. 11.21, is presented in successive steps:

- protect the bed of the channel to be closed against scour over a width (perpendicular to axis of closure) of 90 m, by placing bed protection mattresses;
- construct a sill, having a width of 40 m by dumping clay filled gunny bags[17] in the axis of closure in such manner that the horizontal crest (wide 27 m) has a level of 30 cm below lowest Low tide occurring at springtide;
- protect the sill against scour by covering it by sill mattresses;
- construct 30 m wide abutments by dumping and placing gunny bags;
- construct a jetty on the sill by driving four rows of piles; distance between rows 2.40 m, crest level of jetty 30 cm above crest of future closure bund;
- dump gunny bags on sill simultaneously from barges and from jetty over a height of 2.7 m; width of crest to be appr. 15 m; its level is PWD (in this Design report for Chakamaya).

Table 11.4 Physical conditions at and near three closure sites in tidal creeks in Bangladesh

project no.	activity undertaken & name of creek or river to be dammed; location (province, town, district, island) & country	date of final closure	tidal range (m)		area of tidal basin (km^2)	tidal prism during spring tide (10^6 m^3)	overall width of creek (m) in axis of closure & max. depth relative to PWD	soil condition in axis of closure	level of bank or crest level flood embank-ment rel. to PWD	Sources
			spring tide	neap tide						
	1	2	3	4	5	6	7	8	9	10
-	Construction of bed protection in closure gap for mata closure at Madargong creek; Khulna District, Bangladesh	(mata closure failed on 19 April 1978)	4.55 (April)	2.6 (April)	2	5.8	166 & -7.9	layer of soft clay in bed of creek	+ 3.0	BWDB (1978)
59	Damming of Chakamaya creek, Patuakhali District, Bangladesh	22 April 1979	3.3 (April)	1.5 (April)	3	10	213 & - 6.4	soil in bed of creek 'relatively hard'	+ 2.15	BWDB (1978a) BWDB (1979a); Smith (1979)
60	Damming of Madargong creek; Khulna District, Bangladesh	26 March 1980 (failed 4 days later)	5.1	1.8	2	7	160 & -6.0 over full width	layer of soft clay in bed of creek	+ 3.0	BWDB (1979); BWDB (1980a)
61	Damming of Amtali creek; Patuakhali District, Bangladesh	5 March 1982	2.4	1.5	2.3	6	150 & -7.8	fine sand with silt to silt with fine sand	+ 2.0	BWDB (1982)

[16] BWDB (1978a) & (1979).

[17] Gunny bag is the local word for jute bag. Other words, mentioned in this Section and/or Appendix L are golpata leaves (golpata is a certain dwarf palm), bullah piles, hetal pegs, etc. As these Bengali words are related to the names of indigenous plants, no translation is given.

Fig. 11.20 Map of southern Bangladesh, with locations indicated of closure sites; source: Google maps

The day of final closure, preferably, is the day of a neap tide; High tide will be about PWD + 1.2 m, so at High tide at least a level of PWD + 1.8 m has to be reached by dumping bags from jetty, while the bund during the remainder of the day must be strengthened and heightened to PWD + 2.4 m.[18]

The closure method as outlined above for Chakamaya, could be decided upon because of the successful introduction of bed protection mattresses the year before (1978) at Madargong.
A very important item for the closure works is the construction programme for the various operations listed above, because only a period of three to five months is available for the closure works:

- harvesting goes on up to the end of November, while the fields have to be prepared as soon as the first rains arrive and that can be in the second half of March but also by the end of April; so the availability of a sufficient number of labourers is only guaranteed during the period December - mid-March;
- the tidal ranges are lowest during the period December – March.

Still, it turned out to be practically impossible to start physical works on site prior to January. Either the contract was awarded one month later than foreseen or supplies of closure materials (like reed) and availability of plant (barges) were late. Only a contractor having a tight grip on his labourers may be able to recover the time lost. In Table 11.4, column 2, the dates when the tidal flow was stopped are either in March or in April. It is sheer luck that the rains in the years concerned did not come in March or April.[19] As an example of a construction programme, showing planned activities and realization, Fig. 11.23 has been inserted.

[18] During construction in 1979 the bund profile and levels have been slightly changed as shown in Fig. 11.22.
[19] If it had happened labourers would have started to leave the site and borrow pits used for filling the bags with clay would have become flooded.

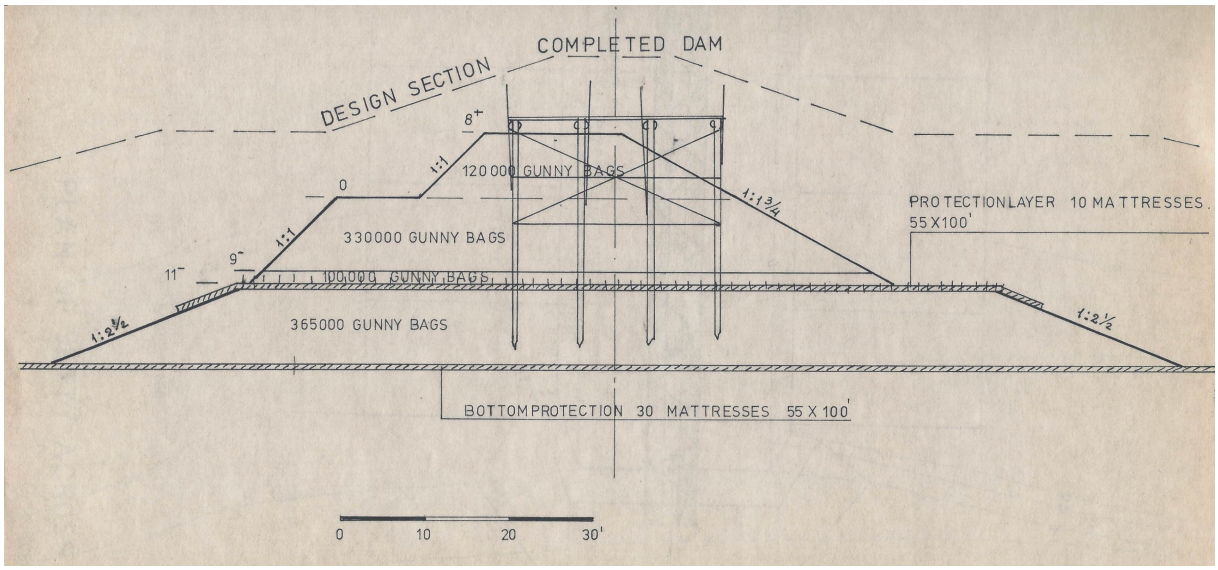

Fig. 11.21 Cross-section of closure bund as designed for Chakamaya; source: BWDB (1978a)

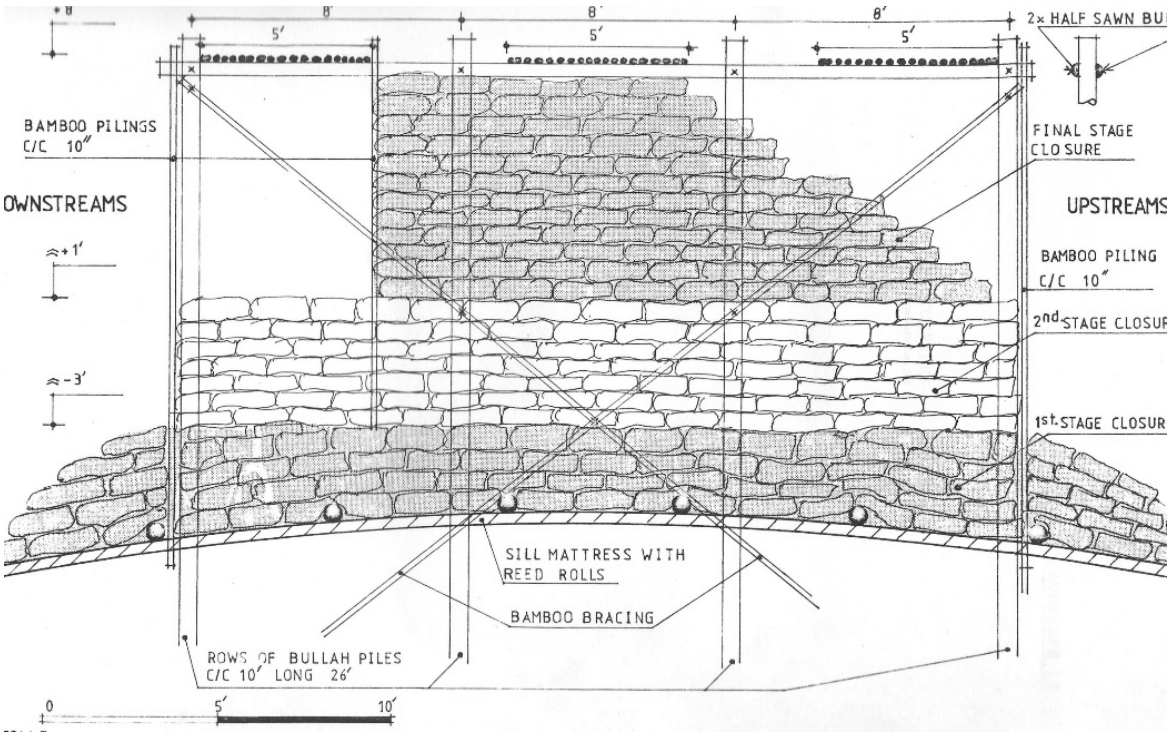

Fig. 11.22 Cross-section of upper part of closure bund at Chakamaya as-built; source: BWDB (1979a)

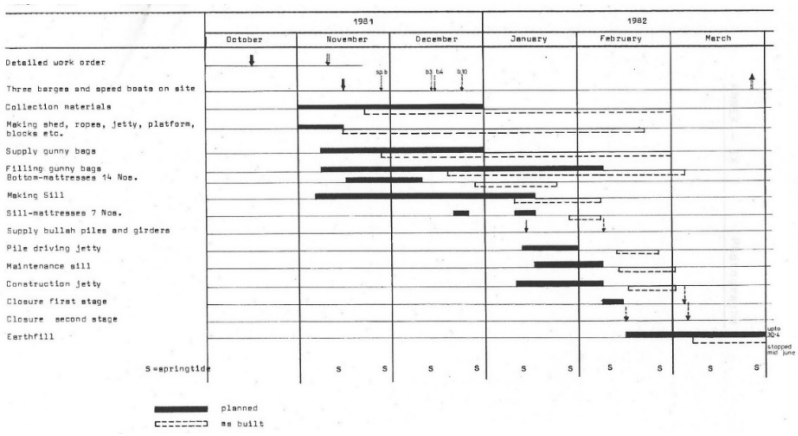

Fig. 11.23
Construction programme for Amtali closure as planned and realized;
source:
BWDB (1982, Annex 8)

11.3.4 Preparation of tender documents, tender procedures and award to a contractor

Preparation of tender documents for the works is the responsibility of the Bangladesh Water Development Board (BWDB) but the specification of the proposed closure method was made by the Dutch experts. The tender documents are partly drawn up as a measured contract (for construction of the actual closure bund) and partly as a Lump Sum contract (for the earth works of the dam incorporating the closure bund). The idea behind the measured contract is that you cannot hold a contractor in all respects responsible for the proper construction of a closure bund which in fact is an experiment, i.e. by introducing a closure method which is new to Bangladeshi. Also, the correct organization of the closure works like sequence of operations, timing, quality and quantity of closure materials (like gunny bags filled with clay) cannot be left entirely to the contractor.
Closure materials like gunny bags and bullah piles in most cases are separately tendered by BWDB and, subsequently, made available to the contractor against agreed unit rates. But sometimes it became clear that direct purchase by the contractor was more efficient.
The tender procedures and the award to a contractor are the responsibility of BWDB. It turned out in all three closures that the contract had to be re-tendered and, ultimately, the contract awarded to the lowest (re-) tenderer always had a higher contract sum than originally estimated[20].

11.3.5 Remarks about the various steps of the closure method listed in Sub-section 11.3.3

In Appendix G the proposed closure method for Amtali creek is described in detail. Also some drawings as well as photographs have been attached to this Appendix[21]. The paragraphs in the Appendix have been numbered for easy reference.
Below, some remarks will be made about the successive operations to be carried out for this closure. Moreover, the adaptations in time[22] to the closure method will be highlighted.

11.3.5.1 The bed protection mattress

Those who are acquainted with the Dutch floating fascine mattress as described by, for instance, Van Breen (1920, 178 et seq.), will agree that the construction and ballasting of the bed protection mattress defined in par. 4 – 7 of Appendix G, is very similar to that described by Van Breen[23]. But in Bangladesh different materials have to be used. In Bangladesh brushwood (i.e. *rijshout*) does not float and can therefore not be used. Instead of fascine strings, bamboo is used for the gratings. The first bed protection mattresses (at Madargong in 1978) had a filling of three layers of fresh (green) goalpata[24]. The buoyancy of fresh green goalpata is low but it can be improved by mixing it with partly dried goalpata. However, the buoyancy of the mattress for the greater part comes from the bamboo in the gratings. One year later, In 1979, reed was found to be a very good replacement for goalpata because the harvesting of the latter is, in most cases, too late for use in the bed protection mattresses.

[20] Exceedance of cost estimate by 60 % (Chakamaya), 22 % (Madargong) and 29 % (Amtali).
[21] Text, drawings and photographs of the Appendix have been copied from BWDB (1982), but some changes have been made in the text as well as additions for a better understanding by the reader.
[22] i.e. during the successive closures in the period 1978 – 1982.
[23] Van Breen, however, describes mainly the instantaneous immersion of the complete mattress at the turn of the tide while the mattresses in Bangladesh were immersed gradually (i.e. in the current, *op stroom*). The latter was possible because depths were relatively shallow. The same method was used in 1953 at Schelphoek for the mattresses forming the sill for the unit-caissons outside the main tidal channels (so-called *maaiveldsluiting*).
[24] Leaves of the Nippa palm growing in the mangrove forests of the Sunderbans.

11.3.5.2 Use of bags filled with clay for ballasting the mattresses and as fill for the jetty / cofferdam

Because of the high current velocities which were expected, dumping of clay[25] as ballast on the mattresses and as fill in the cofferdam could not be considered.

Rocks (in fact boulders, *rolstenen*) can be collected in the North of Bangladesh but application was considered to be too expensive. The same applied to bricks in bags. This left, as ballast material and fill for sill and closure bund, bags filled with clay[26]. Small gunny bags[27] were selected, of 45 by 60 cm having a volume of 0.28 m^3 (1 cu ft.). When partly filled the weight (mass) is about 45 kg.

As these clay-filled bags in fact are the most important and voluminous[28] item of the three closures a few questions to be answered must be highlighted. First of all the quantity of clay to be put in the bags. Should the bags be full or should some room be left in the bag for its adjustment (deformation) after placing? (in fact the latter). How much leakage of clay from the bags had to be expected (no firm conclusions could be drawn) and, consequently, what would be the impact on settlement of the closure bund and on the stability of the bags in heavy currents? In this respect it is noted that the clay did not always have the desired quality: it could be sandy clay, or even silty clay. What would be the stability of a clay bag in the current? In other words: what would be the highest permissible current velocity across the sill being constructed by dumping of bags and, after its completion, over the sill mattress ballasted with bags? The answers to all these questions had to be found in an empirical way.

Bags are in principle attacked by the current in four different positions:

- as ballast on the bed protection mattresses; but by placing this bed protection outside the spring tide period high currents can be avoided; moreover, the bags in the sill will cover these mattresses in time;
- bags dumped for the sill; it requires a speedy placing of the sill mattresses to minimize damage but it cannot always be prevented that a part of the bags washes away and then additional bags have to be dumped as part of 'maintenance of the sill';
- as ballast on the sill mattresses; high currents over this sill cannot be avoided; the conclusion after three closures was that the height of the crest of the sill should be such that maximum current velocities should not exceed 2.5 m/s while for short periods a current velocity of 3 m/s would be acceptable; the 45 cm high reed rolls have an important function in this respect (Fig. 11.24);
- bags dumped from barges and jetty during construction of the final closure bund; it was soon realized that the jetty should be transferred into a cage /cofferdam by bamboo piling[29]; this not only prevented the bags dumped from the jetty from washing away, but also, to a certain extent, the bags dumped outside the jetty from barges (by energy dissipation caused by the bamboos, see Fig. 11.26).

Above it was stated that there is a relationship between the height of the sill and the maximum current velocity. It is illustrated in Fig. 11.25 (BWDB, 1982, Fig. 13). A tide having a range of 1.75 m was assumed for the days after sill completion. The max. current velocity will stay below 2.5 m/s if the crest of the sill is at MWL (mean water level) – 1.4 m, which is PWD - 1.02 m (= - 3.4 ft) at Amtali. For practical reasons the crest was fixed at PWD – 4 ft. (Fig.11.24).

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[25] Clay clods, which were used as ballast material in the Netherlands up to the end of the 19th century, were not suitable for use in Bangladesh. The clay is too soft and clods cannot be stockpiled.

[26] Sand in bags is not feasible because the fine sand leaks out of the bags, moreover, it is not everywhere available.

[27] In fact, mostly second-hand cement bags were used.

[28] In the three closures an overall number of 2.5 million bags filled with clay, were filled, carried, dumped and placed in the closure works.

[29] Compare in this respect Figures 11.21 and 11.22.

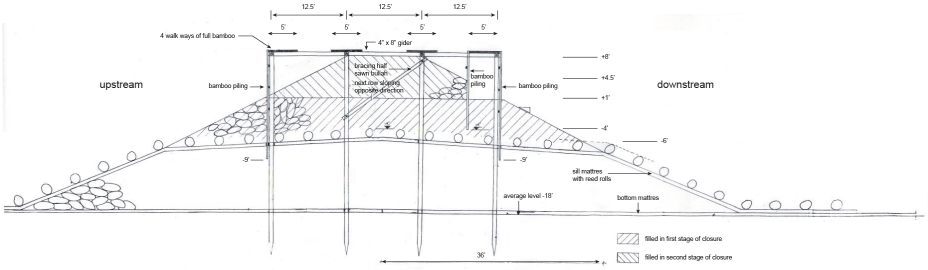

Fig. 11.24 Cross-section of jetty /cofferdam and dumped clay bags at Amtali; source BWDB (1982)

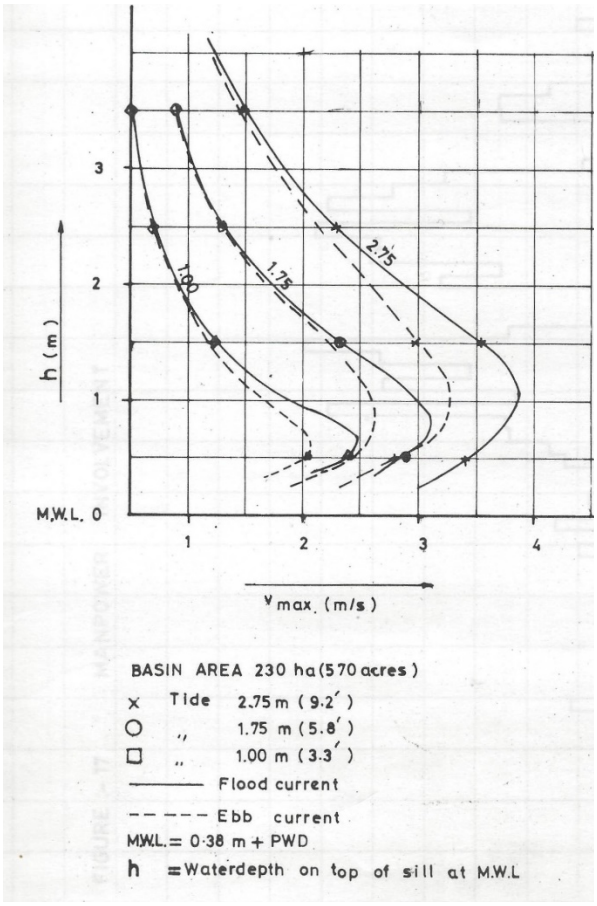

Fig. 11.25
Computed maximum current velocities for different sill levels and three tidal ranges at Amtali; source: BWDB (1982, Fig. 13)

Fig. 11.26

Energy dissipation downstream of jetty /cofferdam during closure of Chakamaya creek; source: BWDB (1979a)

Three years after the publication of the report about the Amtali closure (BWDB, 1982) tests at a scale 1 : 1 were done by Delft Hydraulics Laboratory[30] to check the stability of gunny bags, of similar dimensions and composition as used in Bangladesh. Quotations from this report are presented in Appendix H. The tests showed that the current velocity over the bags in a dam (like an unprotected sill or closure bund without bamboo piling) should not exceed 2.6 m/s and over bags as ballast on a mattress 3 m/s. Moreover, a check applying the Isbash formula showed a critical velocity for a single bag of 2.3 m/s. An important reason for movement/washing away of the bag is its loss of mass due to the leaking out of fill particles from the bag.

11.3.5.3 Jetty

In order to construct the closure bund on the sill a jetty with walkways was required. The first design shows two walkways of 2.75 m width each while the jetty has a width of 7.32 m.

[30] Delft Hydraulics Laboratory (1985)

But already at Chakamaya the jetty, still being 7.32 m wide, shows three walkways each 1.52 m wide (Fig. 11.22). Also at Chakamaya the bamboo pilings were introduced which changed the jetty into a cofferdam. Finally, at Madargong and Amtali, the jetty was widened to 11 - 11.4 m and the number of walkways increased to four (Fig. 11.24).
A solid construction of the jetty is a must because of the hundreds of labourers loaded with bags, walking on it during final closure (Fig. 11.27). The trestles of the jetty are supposed to be placed at equal distances of 3.05 m, but at Madargong these distances varied between 2.1 and 5.5 m (on average 3.4 m). Construction of the jetty in the three closures took 3 to 4 weeks while, usually two pile driving rigs were applied (Fig 11.28). At Chakamaya the jetty had a length of 195 m and a total of 1275 labour days were spent for its construction which implies 6.5 labour days per metre of jetty. At Madargong this figure was 5.4 labour days per metre of jetty; but the average distance between trestles is 11.1 % more than at Chakamaya.

Fig 11.27 1 600 labourers are busy closing Amtali creek; source: BWDB (1982)

Fig. 11.28 Piling rigs at Amtali creek; at left side the jetty is under construction

11.3.6 Quantities and cost

In Table 11.5 some quantities and costs are given for the three closures discussed in this section. For easy reference, similar figures are inserted for the closure of the Feni River in Bangladesh. That closure will be discussed in Section 11.4.

Table 11.5 Particulars of four tidal closures effected in Bangladesh during the period 1978 – 1985 through application of Dutch methods; source: Van Duivendijk (1986)

river/creek & season of closure	unit	closure projects			
		Chakamaya 1978 – 1979	**Madargong 1979 – 1980**	**Amtali 1981 – 1982**	**Feni 1984 – 1985**
project no. →		**59**	**60**	**61**	**62**
cofferdam/cross dam:	m (PWD)				
- elevation of sill		- 1.80	- 1.80	- 1.20	+ 0.70
- elevation of cofferdam/neap tide dam		+ 2.40	+ 2.40	+ 2.40	+ 3.30
- elevation of winter springtide dam					+ 5.50
- elevation of cross dam		+ 4.80	+ 4.20	+ 4.20	+ 10.50
area of:	m^2				
- bed protection mattresses		15 350	11 700	10 550	183 000
- sill mattresses		6 060	4 440	5 580	n.a.
number of clay bags:	nos. x 10^3				
- in bed protection		110	87	90	-
- in sill		475	375	390	1 000
- In cofferdam / neap tide dam		395	312	300	1 300
volume of earth works (excl. cofferdam and flank embankments)	m^3 x 10^6	0.11	0.07	0.07	0.77
contract sum incl. technical assistance	DFL x 10^6	1.5	1.4	1.6	65
ditto in euros of 2020	€ x 10^6	1.7	1.5	1.6	55
bed material		clay (medium hard)	soft clay	fine sand	silt, fine sand

To get an idea about the magnitude of dumping production during final closure (during two days) the following is quoted from the Works Diary of Amtali (BWDB, 1982, 27):

- On 4 March 1 600 labourers started final closure at 07.15 a.m.; by 12.15 p.m. 75 000 bags had been dumped from the jetty and sill level had been raised from PWD – 1.5 m up to PWD.
- On 5 March final closure continues at 07.40 a.m.; flow was definitely stopped at 11.35; crest level of bund reached PWD + 3 m; total dumped today 115 000 bags.

From the first entry it can be concluded that 4 x 200 labourers from each bank carried bags over de four walkways. On average each labourer carried and dumped 47 bags during the period of 5 hours.

A similar calculation can be made for the closure of Chakamaya creek. According to the Works Diary (BWDB, 1979a, Appendix 2, p. 4) in the period 17 – 22 April an overall total of 327 500 bags have been dumped from the jetty. According to Appendix 10 of the same document 5 620 labour days were involved which implies an average production of 58 bags carried and dumped per labour day. At Chakamaya there were three walkways from each bank.

The records for Madargong are not so straight forward as here the figures for labour days are jointly for bag filling, carrying and dumping (BWDB, 1980a, Appendices 8-8 and 10.2).

11.3.7. Why the closure at Madargong failed

The closure of Madargong creek failed four days after its successful closure in March 1980. This was due to several reasons. First of all the settlement of the closure bund must be mentioned. The bund settled 30 to 60 cm over its entire length during the three days after closure. This was compensated by placing 40 000 bags. At the location of the later breach the piles in the middle of the trestle settled 30 to 60 cm while the outside piles had little or no settlement. The breach occurred in the night of 29 – 30 March, one hour after High tide. At High tide the freeboard between crest of bund and high tide was approximately 50 cm. In the early morning of 30 March a 25 m wide breach had developed (Fig. 11.34). The tide was outgoing. The piles in the middle of the aforementioned trestle had settled more (now 1.3 m). This is approximately 80 cm more if compared to the settlement measured a day earlier. It was not clear whether the breach occurred due to quick settlement, followed subsequently by overtopping or by the loss of horizontal stability. The latter might have been caused by the piles in the trestle being too short (actually 9.8 m long instead of the specified 11.6 m). If the Contractor, immediately after closure, had employed 1 200 to 2 000 labourers (as required by the Contract) to place the earth fill in the dam body, the cofferdam probably would not have collapsed. But after the day of closure only 250 labourers stayed, the others stopped working or left the site because the rates were lowered!

Fig. 11.29
Bed protection mattress being ballasted at Chakamaya creek

Fig 11.30
Sill mattress being towed to its destination at Madargong creek

walkways on jetty being made ready for closure

stockpile of clay bags

four rows of labourers carrying each a 45 kg clay bag to the dumping site

Fig. 11.31 Preparations and start of closure at Amtali creek

view below the walk-ways of Madargong cofferdam; dumped clay bags in background.

horizontal closure at Amtali in progress

a few hours later at same spot

Fig 11.32 Closure activities at Madargong and Amtali

walkway gets drenched when bags are dumped

closure bund in cofferdam nearing completion

Fig. 11.33 Closure at Amtali nearing completion

Fig. 11.34
Flow through breached cofferdam at Madargong

11.4 The Feni River Closure dam in 1985 [31] (project no. 62)

11.4.1 Introduction

The Feni River Closure Dam was the last major component of the Muhuri Irrigation Project (25 000 ha of rice-land) which was under construction during the eighties of the 20th century. The project area is located in the South East of Bangladesh.

The Feni River Closure Dam separates the Bay of Bengal and the former Feni Estuary, which was intended to become an irrigation reservoir[32] of 1 200 hectares.

In the wet season there is no need for irrigation; the water discharged by the Feni and Muhuri rivers into the reservoir would soon flood the project area unless the flow is released into the Bay of Bengal. For this purpose a bypass-canal and a regulator[33] has been constructed. The regulator was also indispensable for the closure operations as will be explained below. This regulator was designed by others and was at its final construction stage when HASKONING, in 1983, took up its assignment for design and supervision of construction of the Feni River Closure dam.

As the estuary of the Feni River had an open connection with the Bay of Bengal the more or less regular tidal surges (caused by cyclonic storms) are felt in full force at the project area.

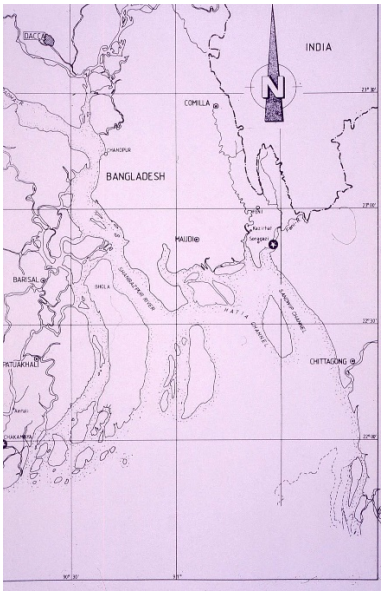

Fig. 11.35
Map of Bangladesh, location of Feni river closure dam marked by black dot

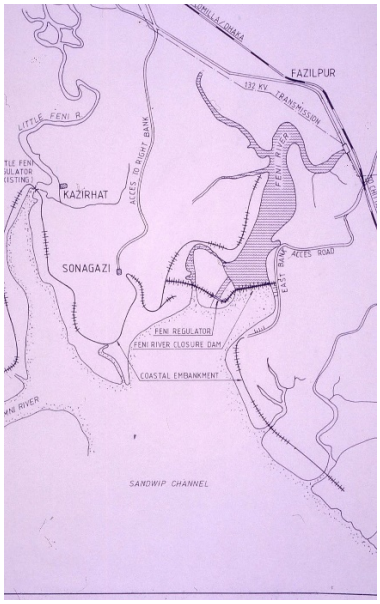

Fig. 11.36
Feni river closure dam, regulator and irrigation reservoir

11.4.2 Site conditions

11.4.2.1 Hydraulic conditions

Tides with large amplitudes occur at the closure dam site. The high water level during springtides reaches an average of SOB[34] + 3.9 m. During the wet season higher levels are reached, and in the dry season lower levels prevail. High spring tide levels reach to S0B + 4.5 m, while tidal surges can result in a further increase of the high water level of 3 to 4.5 m. The tidal ranges are lowest around February when they vary between 3.6 m (springtide) and 2 m (neap tide) (Fig. 11.37). The tidal prism during spring tide in February is 25 million m^3.

[31] Most of the text and figures have been copied from Haskoning's Project Description, written by Van Duivendijk in 1985 and the article in Water Power & Dam Construction by Van Duivendijk & Te Slaa (1987).
[32] Meanwhile, 35 years after its impounding, the Feni reservoir has silted up and it is now an agricultural area.
[33] A regulator is in fact a sluice which can be used for drainage; inland it will also be used as intake of water for irrigation.
[34] SOB is an abbreviation for Survey of Bangladesh and came in use as a datum instead of PWD.

A remarkable aspect of the tide in the Feni Estuary is the uneven distribution between flood and ebb. On the average the flood lasts less than three hours, while the ebb lasts approx. 8 hrs. 30 min. The current reversal at the end of ebb time is almost instantaneous and is often accompanied by a tidal bore (a propagating hydraulic jump).
The low water level at the closure dam site is more or less constant during both spring and neap tides, and is essentially governed by the bathymetry at the site and the quantity of upland discharge. In the natural situation (before the construction of any closure works) the maximum current velocities occur during flood and reach values of approx. 2 m/s.

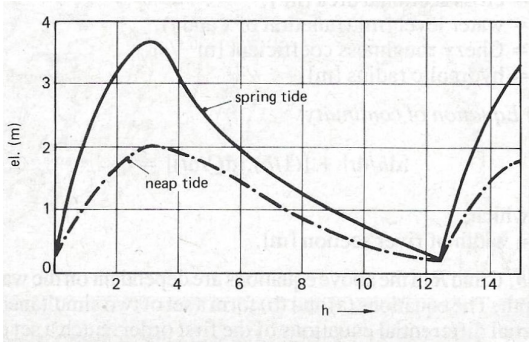

Fig. 11.37 Curves of spring tide and neap tide in February

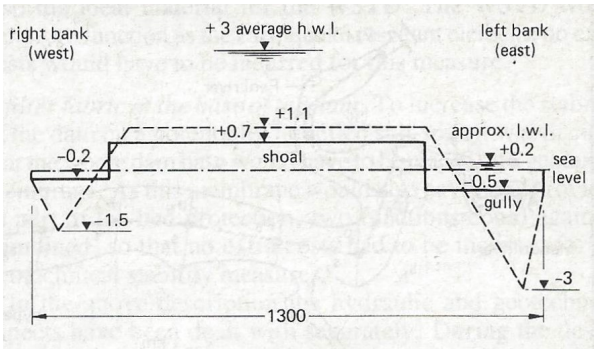

Fig 11.38 Schematic profile of the river and dam axis

Source of both figures: Van Duivendijk & Te Slaa (1987).

11.4.2.2 Geotechnical conditions

The subsoil at the closure dam site consists mainly of sandy silt and silty sand, with occasional clay layers. At several locations the sand/silt is very loosely packed and has densities below the critical level. This implies that the soil can be subject to liquefaction if unsuitable loadings occur. 'Unsuitable' loadings may result from earthquakes or from scour of the river bed prior to the closure operations; after closure no further scour can take place.

To increase the stability of the dam on a potentially liquefied sub-soil, it was decided that the whole dam base would have to be placed on a geotextile membrane. As this membrane would also have to be provided as part of the bed protection, two functions could be combined, so that no extra costs had to be incurred for the technical stability measures.
The loose packing of the subsoil also implies high erodibility. Substantial changes of the river bed are to be expected once the current patterns at the closure site would be disturbed by the closure works.

11.4.2.3 Availability of labour

Though labour does not strictly form part of the 'site conditions', the availability and use of labour, as practised in the closure works described earlier, dictates certain boundary conditions for the closure and working methods to be adopted.
As also noted in Section 11.3 of this Chapter, work forces in excess of several thousand labourers are quite normal in Bangladesh. They can reach, when properly managed (and paid!), production levels which are, to say the least, not normally associated with the words 'manual labour'. Though at Feni the wages were forced upwards by some strikes, settlements were reached quickly and no appreciable delays occurred.

11.4.3 Design of closure works

Hydraulic and practical considerations led to the conclusion that the closure method as used for the tidal creeks (Section 11.3) was not feasible for the Feni River Closure Dam.

After due consideration a closure method was adopted whereby no substantial narrowing of the river would take place. As most of the river bed runs dry during low tides, it was considered feasible to close the river over its full width[35] in a very short time (say 6 to 7 hours during Low tide, a so-called speedy closure) by labourers carrying and placing clay bags. A pre-condition is then that the materials necessary to achieve the closure could be stockpiled in the river, without substantially disturbing the flow patterns in the river.

For this closure method it is further necessary to create uniform flow conditions over the full width of the river. Accordingly, construction of a sill in the gullies, to raise these to match the level of the shoals, becomes an essential feature of this scheme. A flow profile at the closure site prior to start of the works is shown in Fig. 11.38.

Prior to the start of any construction work the shoal fell dry for a considerable period during each tide. The construction of a sill in the gullies (after the bed protection, mentioned below, was placed over the entire river width) would change this situation.

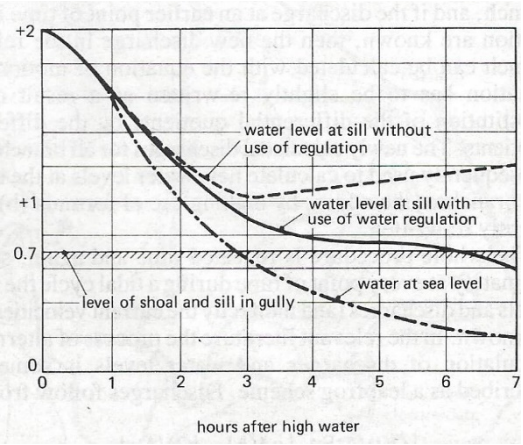

Fig. 11.39 Water levels during closure operation

Fig 11.40 Silll mattress being ballasted

The speedy closure method envisaged (after completion of the sill in the gullies), required a dry (or almost dry) shoal. This could be achieved by opening the gates of the regulator, so that the estuary could be emptied through the regulator, rather than over the mattresses and the sill. The relative contribution of the regulator to the total discharge would become substantial when the water depth above the mattresses would be small. The effect of the discharge through the regulator on the inside water level is reflected in Fig. 11.39: with the regulator the effective working time on the (almost) dry mattresses is approximately seven hours; without the regulator the sill and the shoal would never have become dry.

Notwithstanding the small degree of narrowing (by the stockpiles of clay bags), fixation of the river bed becomes necessary in view of its erodibility. This can be achieved by placing bed protection mattresses (Fig. 11.40), comprising a grid of bamboos and reed-rolls fixed to a polypropylene filter-cloth and ballasted by river boulders. A filter-cloth is also necessary for geotechnical reasons, namely to prevent (or at least to limit) local subsoil failure during construction. Obviously, it has been considered to use bed protection mattresses as described in Section 11.3 but the said geotechnical reasons as well as the difficulties expected in timely supply of 'fill' materials (reed, goalpata) for these mattresses resulted in the decision not to go ahead with this type of bed protection mattresses.

[35] Ultimately, the total length of openings between stock piles and river banks to be closed during one tide was reduced to 775 m (Fig. 11.46). The width of the river between its banks was 1 330 m.

The possibility to stockpile clay bags in the river (without affecting the current velocity criteria) is limited. Only clay bags to construct a bund which is high enough to withstand the high water level during a neap tide can be stockpiled. In order to cope with the springtides, which are to be expected approx. five days after neap tide, it is necessary to raise the level of the closure bund (hereafter called Neap Tide Dam or NTD) 'sufficiently' to withstand these springtides.
The snag lies in the word 'sufficiently' since vast quantities of clay are required: approx. 120 000 m^3 to reinforce the NTD (volume of the latter is 'only' 15 000 m^3) to withstand the springtide. Because this springtide occurs in the winter the relevant section of the closure dam is called Winter Spring Tide dam or WSTD. The construction period available for the WSTD is only five to six days and the clay, stockpiled on both river banks, has to be transported over distances of 800 to 1 000 m.
The WSTD, consisting of clay, forms at the same time a 'stable' element in the whole dam construction, since it would, by itself, not be affected by liquefaction and subsequent loss of stability. This measure against earthquakes is no guarantee for 'nil' damage; but it is sufficient to prevent a breach of the dam and consequent disastrous results.
While the WSTD is an impressive structure by itself, it is insufficient to withstand the springtide levels during the monsoon and post- monsoon period, let alone the surge levels generated by the periodic cyclonic storms in the Bay of Bengal.

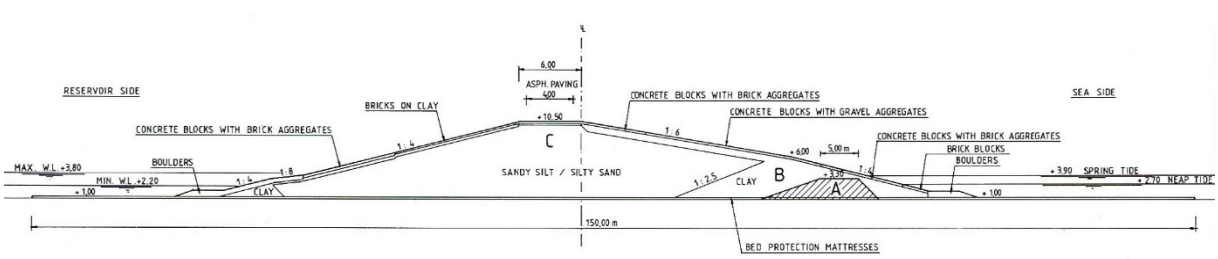

Fig. 11.41 Cross-section over Feni closure dam. A = neap tide dam, B = winter springtide dam, C = final dam profile.

To give an impression of some levels:
the WSTD should reach a level of SOB + 5.5 m. The completed dam should be able to withstand water levels of SOB +8 to 9 m, while wave attack could occur at the same time.
For the completed dam a level of SOB + 10.5 m has been fixed.
A cross section of the final dam is given in Fig. 11.41. Note the almost negligible dimensions of the NTD, when compared with those of the main dam. The NTD is however the 'king pin', without which the construction of the entire dam would not be possible.

11.4.4 Construction of Feni river Closure dam

11.4.4.1 Award of contract for construction

The contract for the closure dam and ancillary works, such as dikes connecting the dam to the existing coastal embankments, was awarded in April 1984. The Contractor was SHIMIZU from Japan. The water-borne operations were sub-contracted by SHIMIZU to the Dutch firm ZINKCON.
The construction period for the whole of the works was only 480 days. The closure itself was to be realized within 10 to 11 months after the contract award in order to utilize the most favourable natura! conditions. Below the major components of the work will be discussed.

11.4.4.2 Bed protection works

The entire river width at the closure location (approx. 1300 m) has been protected by mattresses. The required length of the mattresses (measured parallel to the current direction) was 150 m. For

the gullies the mattresses were pre-fabricated on the river bank and floated to the correct location around the time of high water. They were subsequently ballasted with boulders and sank to the river bed. Dimensions of these mattresses were approx. 60 by 30 m^2.
On the shoals the mattresses were assembled in-situ during Low tide. Dimensions of the mattresses varied. The maximum dimensions were 80 by 50 m^2..

sill mattress pulled from shore (*zate*)

shoal mattress being made in -situ

mattress on shoal; note wire mesh, installed to prevent smaller boulders from being removed by the current

Fig. 11.42 Some details of bed protection mattresses

11.4.4.3 Construction of the sill

The construction of the sill in the gullies was a kind of closure by itself and proved to be the most difficult part of the work. One of the problems encountered was the continuous shifting and scour of the gullies. As soon as a gully mattress had been placed, a scour hole developed at the edge of the mattress in the loosely packed subsoil and the gully started to shift and to deepen. Fig. 11.43 shows the variations in the elevation of the river bed in a period of seven weeks after the start of bed protection works.
The lower part of the sill was built up of clay bags, while the upper part consisted of river boulders (partly in gabion nets) and clay bags. The slopes of the lower portion of the sill were covered by bed protection mattresses.

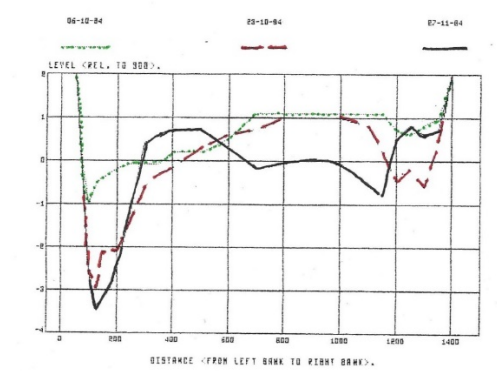

Fig. 11.43
After start of placing the first bed protection mattresses in the gullies along left and right bank, the gullies started to shift and deepen;
see also Fig. 11.38

trying to make the sill in the ebb current, note labourers carrying boulders on the head

two ridges of boulders, as part of sill in gully, nearly ready, inside clay bags will be placed

incoming flood flows over sill one day before closure between stockpiles S-10 and S-11 (see Fig. 11.46)

Fig. 11.44 Construction of sill in the gullies; flow over sill by incoming flood, current vel. 3 m/s

11.4.4.4 Stockpiling work

After completion of the bed protection a start could be made with the construction of the stockpiles of clay bags. A total of 950 000 bags had to be head carried to the appropriate locations in the river during Low tide. The stockpiling operations started in the middle of January 1985 and were completed a few days before the closure date.

clay bags being carried to stockpiles | some labourers carry two bags (2 x 35 kg) | stockpiles; note water mark along bags

Fig. 11.45 Transport of bags to stockpiles; stockpiles will have on average a freeboard of 1 m during high water level of a neap tide

During this period the labour force gradually increased from a few thousand to approx. 15 000 labourers. The labour force was managed extremely well by several Bangladeshi sub-contractors. During a day, on average, 24 000 bags were carried from the river banks to the stockpile locations. The day's maximum was 60 000 bags. During a few days this figure was 50 000.

To reduce the transport distances on the closure day and to provide sufficient working space, it was necessary to create a total of 12 stockpiles at 100 m intervals in the river.
A few days before the actual closure, when the high water levels became gradually lower after the springtide of 22 February, small sections (so-called middle islands) of the neap tide dam were already constructed halfway between the stockpiles of clay bags, in order to reduce the workload on the closure day. On the day of closure still 775 metres of the closure bund (NTD) had to be constructed (Fig 11.46).

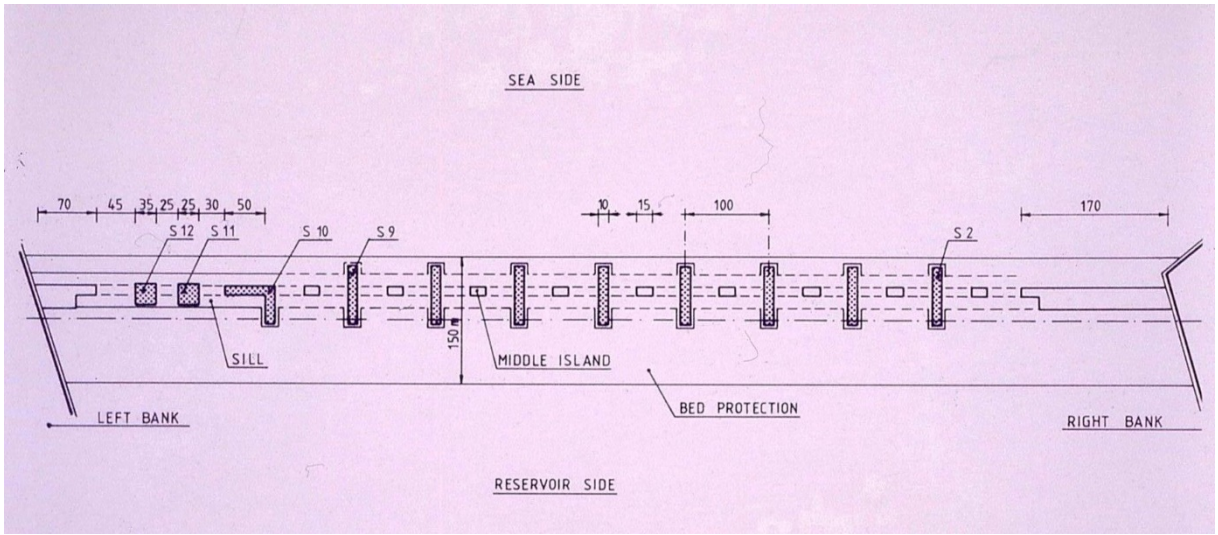

Fig. 11.46 Top view of abutments at river banks, stockpiles and 'middle islands' one day before closure; overall length of openings to be closed on the next day (28 February) is 775 m

11.4.4.5 Closure operation

8 a.m. 12 000 labourers assemble

regulator discharging part of the ebb flow from the reservoir

9 a.m. group of labourers walking to their stockpile

10.30 a.m. first bags being carried to base of closure bund

10.45 a.m. first bags placed in the still flowing water

1.00 p.m. closure works in full swing

Fig.11.47 closure bund (NTD) under construction on 28 February 1985

The closure operation was planned for 28 February 1985. By selecting a closure date one day ahead of a neap tide, extra time could be gained for the construction of the winter springtide dam (WSTD). While persistent Southerly winds caused a substantial increase of the water level (in excess of the predicted high water levels) it was decided to go ahead with the closure operations, as some safety margin had been built in the design of the neap tide dam.
The labour force for the closure operation was assembled around 8 a.m. (Fig. 11.47). Overall, 10 000 labourers were assigned to the twelve stockpiles while 2 000 labourers were kept on stand-by. At 9 a.m., the first group of labourers departed for their stockpile. As the water level was not yet sufficiently low, the labourers had to walk through fast flowing water, which was about 0.5 m deep. At around 10.30 am, by which time all the teams of labourers were assembled on their stockpiles, the actual closure operation could start.
By 6 p.m. all the clay bags had been placed in the gaps and the NTD was completed, just before the arrival of the incoming tide (0.9 m in excess of the predicted water level).

11.4.4.6 Winter springtide dam

The construction of the WSTD was the most critical activity. If the WSTD would not be ready in time (six or seven days after the closure) the springtides of around 7 March could easily breach the dam, and all efforts of the previous ten months would have been in vain, with no chance to make any timely repairs.
The WSTD was constructed with heavy earthmoving equipment On the average the work progressed sufficiently fast to dispel a disaster.

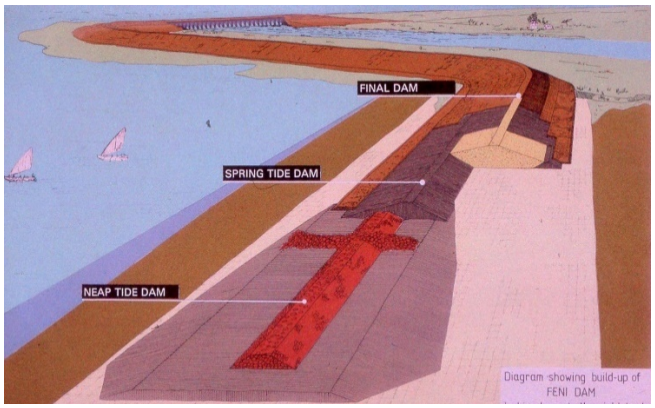

Fig. 11.48 Artist view showing build-up of Feni Dam looking towards the right bank; regulator in the background

Fig. 11.49 Winter spring tide dam under construction

Fig 11.50 Piping through sill in gully after construction of NTD

11.4.4.7 Remainder of the works

Large quantities of work are hidden in the word 'remainder', such as:

- 830 000 m^3 of earthworks (incl. flank embankments outside the river)
- 500 000 nos. of slope protection blocks (concrete and brick blocks).

lt was essential that these works progressed fast so that a sufficiently strong profile would be reached before the onset of the monsoon and the pre-monsoon cyclones.
This was indeed achieved and the Feni River Closure Dam, while not yet completed, easily withstood the disastrous cyclone and tidal surge on 25 May 1985 that killed 10 000 people and caused enormous damage close to the work site. The highest water level during this event, however, fell within the design criteria.
The Feni River Closure Dam was substantially complete late August 1985. The total construction cost (excluding the regulator) was approximately US$ 20 million.

Fig. 11.51 The Feni Dam in July 1991; the foreshore at the right side has meanwhile silted up to the level of HWS

Fig. 11.52 Feni Reservoir in 1993

The closure operation. with its many thousand labourers, was undoubtedly the most spectacular component of the project. Without the experience on the closure projects described in Section 11.3 (though these were at a smaller scale). It would not have been possible to apply this method successfully. For future closure projects it would be possible to apply this method again, especially where the large scale employment of labourers would be possible or desirable.

11.4.5 Quantity of bags placed in the Neap Tide Dam on day of closure

In Fig 11.53 a cross section is shown of the NTD. The cross-section has an area of 19.35 m^2. As the nett length of NTD to be built on day of closure was 775 m, the volume is 15 000 m^3. For 1 m^3 of bund are required 35 (dry) or 50 (wet) bags. As the bags in the lower part of the stockpiles are indeed wet it is assumed that 1 m^3 contains 45 bags. Then the number of bags placed on the day of closure is 675 000[36]. The bags are placed in 7.5 hours by 10 000 men which implies an average production of 9 bags per manhour. Obviously, the distance from stockpile to the location where the bag had to be placed in the cross-section of the bund plays a role in the actual production[37]. Moreover, building up a neat cross section of clay bags takes more time than straightforward dumping of a bag. Compare in this respect the Figures 11.54 and 11.33.

[36] The quantity is 1.3 million bags for the overall length of the NTD (see Table 11.5).
[37] In this respect the remark made by ir C. Biemond (on behalf of Haskoning supervising the organization of the works on the day of closure) is interesting. When during the day asked whether progress was satisfactory he commented: ' I had calculated that the labourers had to walk 1 m/s with a 45 kg bag on their head but it turns out that they have a speed of 1.5 m/s, so progress is indeed satisfactory' (source: De Gelderlander of 16 March 1985).

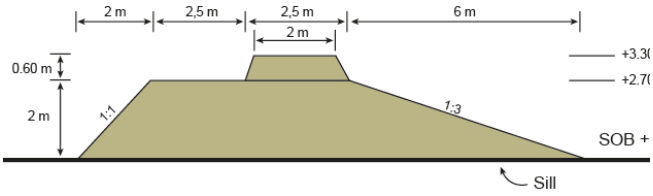

Fig. 11.53
Cross-section over neap tide dam; during High tide on day of closure predicted water level was SOB + 1.80 m, wind setup was 0.5 m and 'resonating tidal wave' (*opslingering*) 0.40 m; remaining freeboard for wave action 0.5 to 0.6 m

Fig. 11.54
Completed neap tide dam; sea side at right

11.5 Summary and Conclusions

11.5.1 General

1 In the 20th century five closures have been constructed outside the Netherlands (one in Ghana, four in Bangladesh) by using simple means and Dutch methods.

2 The required adaptation to circumstances different from those prevailing in the Netherlands (in respect of local culture and physical environment) was successful.

3 By making full use of local skills, locally available materials and much local labour the input of foreign expertise and equipment could be kept to a minimum.

4 In all five closures the introduction of a bed protection and the application of sand or clay filled jute bags, as a replacement for stony ballast materials, had a great share in the realization of the five projects.

11.5.2 The closure of a flow gap in the sand ridge of a lagoon in Ghana

5 In 1968, in order to end the flooding of the town of Keta, a cut was made in the sand ridge between the Keta lagoon and the Gulf of Guinea.

6 Initially, the thus created flow gap had a length of 250 m, but after 11 months and a shift in easterly direction of 360 m, its length was 170 m by a maximum depth of 4.5 m below MSL.

7 Locally, tidal differences are 0.3 m (neap tide) to 1.5 m (springtide). The tidal volume of the flow gap was in the order of 4 to 5 million m^3.

8 The sand ridge in which the cut had been made consists completely of very coarse sand; no clay, bricks or rocks were available for closure of the flow gap.

9 Part of the bed of the flow gap and its eastern shore have been protected, by placing straw mats ballasted with sandbags, against erosion by tidal currents.

10 The gap has been narrowed from the western side by bulldozers pushing sand into the tidal channel. During a period of 10 days the gap was narrowed from 170 to 45 m.

The bed of this 45 m long closure gap had been protected earlier (see above) by straw mats ballasted with heavy (125 kg) sandbags and thus a narrow sill had been formed on which the closure bund of sandbags has been built.

11 Closure of the last gap started on 3 October 1969 around 8.30 a.m. by 40 labourers, supported from 10.00 a.m. onward by 300 young students. During four hours 6 000 sandbags were placed and a closure bund formed having a crest width of 2 m and a height of 3 m. Its volume was 405 m^3.

12 This closure can be characterized as a speedy horizontal closure.

13 Up to the day of closure the estimate of the cost of the work done so far (expressed in euros of the year 2020) was € 240 000.

11.5.3 Closure of three tidal creeks in Bangladesh during the period 1978 – 1982

14 In the sixties of the 20th century in East-Pakistan (now named Bangladesh) a Coastal Embankment Project was being carried out. In the seventies still a number of tidal creeks in that Project had to be closed.

15 These creeks could not be closed by using the local (mainly horizontal) closure method as the tidal volumes were too large (> 2 million m^3) and it was decided to introduce Dutch labour-intensive methods instead.

16 The creeks to be closed were 150 to 213 m wide and had maximum depths in the axis of closure of 6 to 8 m.

17 A bed protection mattress was developed by using bamboos for the gratings and palm leaves (later on reed) for the 'fill' layers. These bed protection mattresses were immersed by using jute bags filled with clay as ballast.

18 After protecting the bed of the creek against scour by a layer of these mattresses, a horizontal sill was formed over the full width of the creek by dumping clay bags in the gullies. To prevent these bags from being removed by the tidal current, they were covered by a so-called sill mattress which had an upper grating comprising bamboos wrapped in reed. The crest of the sill was at 30 cm below Low tide at spring (LWS).

19 Subsequently, a jetty was constructed on the sill across the creek. The jetty functioned as a walkway for the labourers dumping bags on the sill. By placing vertical bamboos the jetty had been transferred into a cofferdam filled by the dumped bags.

20 During the final closure 950 (at Chakamaya) and 1 600 (at Amtali) labourers were carrying and dumping 327 500 and 190 000 bags respectively. The production per labourer was 58 and 47 bags per labour day of 5 hours, respectively.

21 The final closure could take a few days and can be characterized as a gradual vertical closure.

22 The overall cost of the three closures (including technical assistance), expressed in euros of the year 2020, has been € 1.7 million (Chakamaya), 1.5 million (Madargong) and 1.6 million (Amtali).

11.5.4 The construction of the Feni river closure dam in Bangladesh in 1985

23 In order to create an irrigation reservoir of 1 200 ha, the estuary of the Feni river would be separated from the Bay of Bengal by a 1 330 m long dam. A sluice had already been constructed to discharge river flow not needed for irrigation.

24 Apart from a few shallow gullies most of the estuary became dry during Low tide. Tidal ranges in winter time varied between 2 (neap tide) and 3.6 m (springtide). The tidal prism during springtide in February was 25 million m^3.

25 The subsoil at the closure dam site consists mainly of sandy silt and silty sand. At several locations the sand/silt was very loosely packed and had densities below the critical level. This implied that the soil can be subject to liquefaction.

26 To increase the stability of the dam on a potentially liquefied sub-soil, the whole dam base was placed on a 150 m wide geotextile membrane. As this membrane would also have to be provided as part of the bed protection, two functions could be combined.

27 Also because of the loosely packed subsoil it was decided to close the river over its full width in a very short time (say 6 to 7 hours during Low tide, a so-called speedy closure) by labourers carrying and placing clay bags. This required the construction of 12 stockpiles in the river. Ultimately, the total length of openings between stockpiles and river banks to be closed during one tide was reduced to 775 m.

28 For this closure method it was further necessary to create uniform flow conditions over the full width of the river. This necessitated construction of a sill in the gullies which turned out to be a difficult operation.

29 The use of the sluice during the closure period was essential as otherwise, because of the sill in the gullies, the ebb would have continued to flow through the openings between the stockpiles , thus preventing the final closure to proceed.

30 Closure of the remaining openings between the stockpiles was effected on 28 February 1985 by 10 000 labourers during the 7.5 hours of low tide by carrying 675 000 bags from the stockpiles into the cross section of the 2.6 m high closure bund, called Neap tide dam.

31 The closure can be characterized as a vertical speedy closure.

32 The neap tide dam was transferred into a much larger dam (called Winter spring tide dam) by heavy earthmoving equipment transporting clay, in time to counter the spring tide, seven days later.

33 The Feni River Closure Dam was substantially complete late August 1985. The total construction cost (excluding the sluice), expressed in euros of the year 2020, was approximately € 56 million.

Part III Closures in other countries without or with very limited input from Dutch side

CHAPTER 12 Introduction

Straight from the start of the studies leading to this dissertation, it was the intention to look for case histories of closures in the Netherlands as well as in other countries. Preferably, the closures in other countries would have been done without any input from Dutch side. The closures, finally selected, and discussed in the chapters 13 to 18 (incl.), have been realized in the period 1720 – 2020, i.e. during three centuries.

The countries involved are China, East-Pakistan/Bangladesh, England and Japan. In China and Japan it concerns only river closures outside the region of tidal impact. In the other countries it are all tidal closures along or near to tidal rivers.

In principle there are five aspects which determine the method of closing a breach in an embankment or a tidal channel:

- the nature of the subsoil in the closure gap in relation to geotechnical stability and erodibility by the current;
- the maximum current velocity through the final closure gap either due to difference in head between river side and land/creek side or depth (and length) of the gap;
- the available closure materials;
- the availability of skilled manpower to prepare desired constructions to be used for closure (like fascine mattresses, sink-fascines, jetties);
- the tidal differences (if applicable).

It will be noted when reading about the closures in the following chapters that lack of information about some or most of the aforementioned aspects is quite common. It is therefore difficult in most cases to make a quantitative analysis of these closures. Also the construction cost is usually unknown.

However, the Nalyan Closure (Chapter 15) is an exception. This recent closure (2020) has been fairly well documented and it is interesting to compare its characteristics in terms of closure method, labour force, labour output and materials with the other closures described in Parts II and III. Also, it is the only recent closure by using simple means about which information is available. One should also bear in mind that the closure of the Feni River (Chapter 11, Section 11.4) took place in 1985, i.e. at the time of writing already 38 years ago.

But what have these closures in common with each other and with the closures discussed in Part II of this dissertation?

First of all the abundant use of unskilled manpower: in all projects hundreds, and sometimes thousands of men (in some cases also a limited number of women) were employed.

Secondly, the use of stone and/or bags filled with clay or sand, which materials, in most cases, are head carried.

Last but not least, in the past in many countries for closures the kind of sausage-shaped constructions, as described in Chapter 4, Section 4.2, were used. These constructions were in use for closures in East-Pakistan/ Bangladesh, China, Germany and the Netherlands and they will be called sink-fascines[1]. These are long, flexible, more or less cylindrical 'sausages' having a 'skin' of fascines, millet stalks or palm leaves and a core of earth or stones. They were placed in the direction of the current[2].

In China and Japan in the past also 'stone baskets' were in use having a 'skin' of bamboo netting, but, in so far as known, these were not used very much in closure works as their mass (15 000 kg)[3] renders handling by manual labour rather difficult.

[1] Other names are *saucissons, sinkwürste, salsiccia, dodemans, mata,* stone sausages.

[2] Contrary to the current use of geo-tubes which are placed perpendicular to the current.

[3] De Rijke (1900).

CHAPTER 13 River closures in China

13.1 Introduction to the Yellow River[1]

13.1.1 General

The Yellow River, Chinese (Pinyin) Huang He, is the principal river of northern China, east-central and eastern Asia. With a length of 5 464 km, it is the country's second longest river—surpassed only by the Yangtze River (Chang Jiang)—and its drainage basin is the third largest in China, with an area of some 750 000 km^2.

The river rises in southern Qinghai province on the Plateau of Tibet and crosses six other provinces and two autonomous regions in its course to the Bo Hai (Gulf of Chihli), an embayment of the Yellow Sea of the North Pacific Ocean (Fig. 13.1). In its lower reaches it is a shifting, turbulent, silt-laden stream that often overflows its banks and sends floodwaters across the North China Plain. For that reason, it has been given such names as "China's Sorrow" and "The Ungovernable." The Mandarin Chinese word *huang* ("yellow") is a reference to the fine loess sediments that the river carries to the sea. The Yellow River basin has an enormous population, exceeded by only a small number of countries.

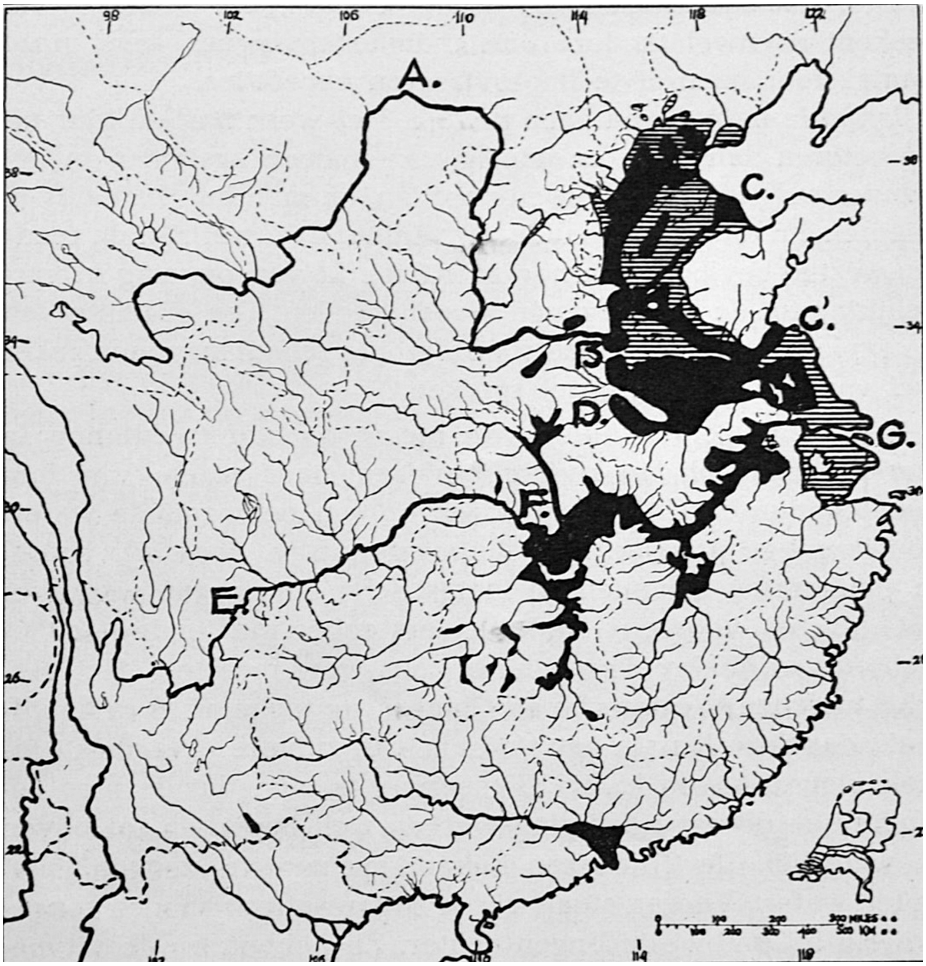

Fig. 13.1
Main rivers and flooding (black) areas in China;
A-B Yellow River middle course;
B-C is lower course;
B-C' is lower course before 1853;
E-F-G is Yangtze Kiang;
source: Tellegen (1942)

[1] In view of the unusual nature of the closures to be discussed it would appear necessary, for the benefit of the reader, to present some features of the Yellow River on which, in the past, many dike sections breached and had to be repaired. The text in this Section is an abbreviated and slightly modified version of the lemma Yellow River in the Encyclopaedia Britannica (editors: Igor Vladimirovich Popov & Charles E. Greer); source: https://www.britannica.com/place/Yellow-River (accessed on 20 Nov. 2023).

13.1.2 Physical features

The Yellow River is divided into three distinct parts: the mountainous upper course, the middle course across a plateau, and the lower course across a low plain. Below, only the middle and lower course will be discussed.

13.1.3 The middle course

Most of the middle course is cut through the Loess Plateau, which extends eastward from the Plateau of Tibet to the North China Plain at elevations ranging between 900 and 2 100 metres. The river has cut at least six terraces across the plateau, which rise to more than 500 metres above the present river level. The underlying rock systems are covered with thick layers of loose soils, consisting mainly of wind-deposited sand and loess. The loess strata reach thicknesses of 50 to 60 metres and in some places as much as 150 metres. Through those loose deposits the river has cut deep valleys, carrying away with it huge quantities of surface material. The easily eroded loess soil accounts for the instability of the riverbed both in the middle basin, where the erosion is considerable, and on the plain, where deposition builds up the channel bed.

13.1.4 The lower course

Downstream from Zhengzhou the Yellow River broadens out to flow through Henan and Shandong provinces across the North China Plain. The plain is a great, nearly featureless alluvial fan broken only by the low hills of central Shandong; it was formed over some 25 million years as the Yellow River and other rivers deposited enormous quantities of silt, sand, and gravel into the shallow sea that once covered the region. The plain has been densely inhabited for millennia and long has been one of China's principal agricultural regions. The river has changed its course across the plain several times, and the region's inhabitants have built extensive systems of levees and irrigation works in an attempt to control the river's flow. The area illustrates perhaps better than any other place on Earth how human activity has combined with natural forces to shape the landscape.
The lower Yellow River is about 700 km long with an average fall of about 5 cm per km. Along the river are found occasional areas of sand dunes 5 to 9 metres high. In general, however, the plain is an area of great floods because the riverbed, built up gradually by sediment deposits, lies above the surrounding land in many places. In the section north of the city of Kaifeng in northern Henan, the low-water level is some 5 metres above the surrounding countryside, the mid-water level between 6 and 7 metres, and the high-water level sometimes as much as 30 to 35 feet (9 to 11 metres) above the land. From Kaifeng to the Grand Canal (Da Yunhe), the levees are lower than farther upstream, rarely exceeding 1 to 2 metres in height. Marshes are common. Below the Grand Canal the height of the levees increases to between 4 and 5 metres and in some places to 8 metres.

Until the late 20th century the Yellow River delta was one of the most actively growing deltas in the world, as the North China Plain continued to extend farther into the Bo Hai (the remnant of the ancient sea now covered by the plain). In the century from 1870 to 1970 the delta grew an average of more than 19 km. Some outlying parts expanded even more rapidly: one area grew 10 km during the period 1949–51, and another grew more than 24 km in 1949–52. However, beginning in the 1950's, dam construction upstream—notably the Sanmen Gorge installation in Henan province—began to reduce the silt load that the river could carry to its mouth.

13.1.5 Hydrology of the Yellow River

The lower Yellow River has changed course radically throughout its geologic history. The river's decreased gradient and velocity on the plain cause its suspended load of silt to settle. As the riverbed builds up, the stream shifts course to occupy a lower level. In the past four millennia the river has entered the Yellow Sea at points as much as 500 miles (800 km) apart.

As the Chinese developed agriculture on the plain, they became more adept at building levees to stabilize the channel and thereby protect the inhabitants against the floods brought by shifts in the channel. Tens of thousands of km's of levees have been constructed through the centuries. The overall effect of those structures has been to delay flooding, but, because the riverbed has been elevated and confined artificially, levee breaching and channel shifts have become more dramatic and destructive than they otherwise would have been. The few hydraulic engineers who succeeded in decreasing rather than increasing the flood hazard have gained legendary status in Chinese history.

Breaks in the levees have been more frequent than course changes throughout history. Such events have triggered cataclysmic floods, notably during the 18th and 20th centuries. Between 960 and 1048 there were 38 major breaks, and 29 more were recorded from 1048 to 1194. In later years such breaches were less frequent as a result of systematic improvements to the levee system. In 1887 the Yellow River burst the levees near Kaifeng and began to flow into the Huai River, but engineering efforts succeeded in returning it to its former course in 1889. The flood of 1887 covered thousands of square miles, completely burying many villages under silt. In 1889 another flood destroyed 1 500 villages. The next major flood, in 1921, wiped out hundreds of populated places, mainly near the river's mouth. In the flood of 1933 more than 3 000 populated places were submerged and 18 000 people killed. Other floods occurred in 1938—when the levees were purposely broken near Zhengzhou to delay the advance of Japanese troops—and in 1949.

The Yellow River carries an average annual volume of about 56 km^3 of water down to the sea, an average discharge of 1 770 m^3/s. The average discharge can be as much as 2 200 m^3/s in high-volume years and as little as 600–800 m^3/s in low-volume years. There also is considerable seasonal variation in its volume. The river has a low discharge —eight other Chinese rivers exceed that of the Yellow River—because its basin encompasses large areas of arid or semiarid land, where considerable quantities of water evaporate or are diverted for irrigation. Seasonal maximum flows can be considerable: 5 350 to 6 120 m^3/s near Lanzhou, 10 000 m^3/s near Longmen, and 36 000 m^3/s (recorded in 1943) in the lower parts of the river.

The Yellow River carries along the highest concentration of sediment load of any river in the world, amounting to about 34 kg of silt per m^3 of water, as compared with 1 kg for the Nile River and 13 kg for the Colorado River. Floodwaters may contain up to 710 kg of silt per m^3 of water (70 percent by volume). The river, unimpeded, carried down to the sea about 1.52 billion tons of silt per year, a large part of it loess, which was loose and easily washed away. Other factors contributing to the high volume of silt included the steepness of the slopes, the rapidity of the current, and a lack of forested areas to check erosion. The reservoirs created by dams have allowed increasing quantities of silt to settle out.
On the North China Plain near Kaifeng there are 15 to 20 icebound days per year, but farther downstream there are none at all. Ice jams are broken up with the help of aerial bombardment or sometimes by artillery shelling.

13.1.6 Economic development by means of multipurpose dams

Water resources in the Yellow River basin have been managed by irrigation and flood-control works of significant size since the 3rd century BCE. Modern hydraulic engineering techniques have been applied since the 1920s, while basin-wide multipurpose development efforts have been under way since the mid-1950s. The major accomplishments of that program have included giant hydro-electric dams at the Liujia Gorge and other gorges near Lanzhou and major irrigation projects—some with smaller hydroelectric stations—at several locations farther downstream. On the plain, levees have been strengthened and the flood-control system rationalized and integrated with reservoirs and with the Grand Canal (which crosses the Yellow River in western Shandong province). Erosion-control measures on the Loess Plateau have reduced the silt load carried downstream. The key project has

been the huge dam at the Sanmen Gorge upstream of Luoyang and the reservoir impounded behind it. The project has augmented flood control on the plain and has also provided water for irrigation and hydro-electric power generation, although silt deposition in the reservoir has reduced its functional capabilities.
As economic development has increased throughout the basin, however, pollution from industrial sources and agricultural runoff has also increased. In addition, the greater demand for limited water resources has caused shortages, which have been severe at times and have included instances when river water did not flow to the Bo Hai.

13.2 Sources of information about river closures in China

The accessible information about closures in China concerns only river closures and nearly all described closures are at the Yellow River. The consulted information covers the period 1888[2] up to 2005. It concerns four books, thirteen articles, two films and one poster presentation (ex TU Delft, made around 1947). Two books have been published by Chinese authors, i.e. Kung (1936) and CNCID (2005). All other publications are from American (two authors), British (one author), Dutch (nine authors) and Norwegian (one author) origin. The three articles by Dutch authors published in the weekly '*De Ingenieur*' in 1888 are not directly relevant to the subject of this dissertation and will not be referenced[3]. This leaves six Dutch authors. In most of the articles the emphasis is on hydrology, river morphology and river training (as summarized in Section 13.1). But that is not surprizing as these articles analysed the results of surveys, hydrometric measurements etc. which were done for the first time on the Yellow River. The article of Freeman (1922) is 55 pages long and that of Todd & Eliassen (1938) counts 71 pages. Freeman did his surveys in 1919 and, during that period, had daily contact with Van der Veen[4]. The latter made a report in 1921 (Van der Veen, 1934) comprising recommendations on improvements of the Yellow River, partly on the basis of Freeman's surveys and Eliassen's hydrometric measurements. In his article of 1939, Van der Veen summarizes the aforementioned article of Todd & Eliassen and notes with satisfaction that the hypotheses which he had to make in his report of 1921 turned out to be basically correct.
The publications of these four gentlemen about these studies, measurements and surveys of the Yellow River are to a certain extent comparable to the report 'River Studies and recommendations on improvement of Niger and Benue' made by NEDECO during the fifties of the 20th century. Because also in Nigeria it concerned a river about which, until the outcome of the studies etc. had been published, much basic information was lacking.
The functions of the four experts in China are given as 'engineer in charge survey department , Yellow River Commission' (Eliassen), 'Director of the Grand Canal[5] Board' (Freeman), 'Consulting engineer to the Yellow River Commission and Shantung Government' (Todd) and 'Advisor to the Chinese Government' (Van der Veen).

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[2] This does not apply to CNCID (2005). This book describes the history of irrigation and flood control in China from the 22nd century B.C until the year2000.
[3] The information in the following sections has mainly been derived from the following publications: CNCID (2005), Fijnje van Salverda et al (1891), Freeman (1922), Kung (1936), Tellegen (1942), Todd (1933) and (1937), Todd & Eliassen (1938), Tyler (1906), Van Breen (1946) and (1966), Van der Veen (1934), (1937) and (1939). The films about, inter alia, aspects of river closures in China, have been made by ir F. J.M. Bourdrez (https://www.francoisbourdrez.org).
[4] Van der Veen was actually already in Sjanghai in 1903 where he was member of a team supervising improvements to the Whangpoo River. Head of team was J. de Rijke (Van Gasteren et al, 2000, 368).
[5] The 1 776 km long Grand Canal starts in Beijjing and ends in the city of Hangzhou, linking the Yellow River and the Yangtze River; source: https://en.wikipedia.org/wiki/Grand_Canal_(China) .

The book published by Fijnje van Salverda et al (1891), comprises a desk study (called Memorandum) of the Yellow River by Fijnje van Salverda and a Report about a field visit made by Van Schermbeek and Visser. This was an initiative of the 'Society for the Promotion of Dutch Engineering Works abroad', founded in 1889 and, though its purpose was obviously commercial, the book contained already recommendations for improvements on the Yellow River.

Captain W.F. Tyler (1906), based in Shanghai, was a Coast Inspector on the Yellow River. He made a reconnaissance along the Yellow River by order of the Inspector General of Customs in Peking. The purpose of this reconnaissance was related to the navigational possibilities of a tributary of the Yellow River. An important factor in this respect was the danger of floods from the Yellow River and, consequently, Tyler discusses this aspect (materials used, measures taken, their effectiveness, maintenance, etc.) in some detail.

Bourdrez was a Dutch civil engineer, sent in 1931 to China by the League of Nations (*Volkenbond*). He visited the closure site at Tung Chuang in 1936 and made some photographs and film footage. He had a fatal accident in 1939 during an expedition on the Upper Yangtze. More details can be found in the 'Introduction to the films made by Bourdrez' presented by Tellegen (1942) and on the website https://www.francoisbourdrez.org .

Finally, Van Breen (1945) and (1966) merely summarizes (1945) the article by Van der Veen (1934) and copies an article (1966) about river closures in China which appeared in the weekly '*De Ingenieur*' of 2 April 1965.

13.3 Materials used for river closures in China

Tyler (1906) describes the various materials used for bank protection and river closures at the Yellow river. A material used abundantly is the Kaoliang stalk (Latin: Sorghum vulgare) which is a millet stalk, a kind of reed, but it is especially its matted bunch of fairly hard and strong roots which render the face of a Chinese pakwerk[6], made by placing alternatively layers of kaoliang and earth, nearly watertight. A Chinese pakwerk can be used as slope protection or, as a dike section in a gradual river closure. Reference is made to Box. 13.1 for a better understanding of its construction. The disadvantage of the kaoliang stalks is their lightness and their limited life: 2½ years, while the stalks without the roots rot away within one year. It implies that the kaoliang in bank protection works requires continuous maintenance. The stalk is 1.5 to 3 m long and 2 cm in diameter. The bunch of roots is about 7 to 12 cm in diameter.

[6] Tyler uses the Dutch word *pakwerk* on purpose because he has no good English translation and considers a Dutch *pakwerk* to be very similar to the Chinese equivalent. Tyler must have learned the word from J. de Rijke who, like Tyler, resided in Sjanghai at that time. In this respect the following is quoted from a letter dated 26 April 1908 from J. de Rijke in Sjanghai to G.A. Escher in Arnhem:

> "Sir R. Hart (Ambassador of Great-Britain), on his trip to England, arrived here yesterday evening…. The second man in Peking (on the Embassy), Sir Robert Bredon (member of the Conservancy Board), is now temporarily the *chef de poste* and possibly he may remain *chef de poste*, first man here in Sjanghai is Commissioner Hubson (Commissioner of Customs), who comprises , together with the Sjanghai Taotai (Governor), the Whangpoo Conservancy Board. An important person here in Sjanghai is captain Tyler (member of the Conservancy Board) who is already for some years the Coast Inspector…..Tyler is a special confidant of Bredon, which is already for a dozen years my friend. It was Tyler who, in the past did everything to make the governor call me for the Whangpoo."

The letter is quoted in Kamibayashi (1999, 235).

Also Van Schermbeek and Visser (in Fijnje van Salverda, 1891, 101) report about the kaoliang. As there are slight differences between their report and that of Tyler, their text is quoted in full:

> "In North China these works (e.g. closure works) are made chiefly of millet-stalks Chin. *Kaoliang,* (Lat. *Sorghum vulgare),* either alone, or mixed with earth. The stalks of the millet- plant, which is very extensively cultivated, are from 3 to 5 metres[7] long and correspond closely in appearance with our reeds. The bark however is less brittle, and the stalks are 1½ or twice as thick, and not hollow, but filled with pith resembling that of the elder-tree. Although there is abundance of excellent willow-wood in North China, we saw nothing else but *Kaoliang* used for river-works. We believe the osier would be more durable, both under and above water; but bearing in mind what prodigious quantities of millet-stalks remain over after the harvest, which could otherwise only serve for fuel, and how cleverly the Chinese know how to use them, we must admit them to be a very cheap and serviceable material."

Tyler notes that:

> "The kaoliang pakwerks used in slope protection have, apparently, of late years, been strengthened by four methods: (a) by willow basket work on their faces, (b) by dumping rubble stone to form an apron[8] to prevent undercutting, (c) by facing the pakwerks with stone, and (d) by placing stone on top of the pakwerk to increase its weight."

The pakwerks are fixed to the slope of the dikes by means of ropes. In fact ropes (up to 20 cm diam.) and hawsers (up to 40 cm in diam.) comprise the second important material in bank protection and river closures as also pointed out by Van Schermbeek and Visser (p. 101):

> "The chief peculiarity of Chinese river-works is the frequent use of heavy cables, made of hemp, bamboo or other plant fibres by which the newly made work of millet-stalks, f.i. a jetty or a new dike in a breach, is brought into connection with the bank, the old dike, or the completed work."

The willow baskets, filled with stones, apparently, were new to China. Van Gasteren et al (2000, 368) note that these *gabbioni*[9], are very similar to the yakago, in use in Japan (Ch. 18, Fig.18.1). They were used for the improvements of the Whangpoo River (as well as fascine mattresses) during the period 1903 – 1910 and most probably have been introduced in China by De Rijke. In this dissertation the word gabion will not be used as the latter nowadays is known as a rectangular steel wire box filled with stones. In China the aforementioned willow baskets are used as sink-fascines in a closure in 1936 (Sub-section 13.6.3.1, but there the sink-fascines are called 'willow-stone sausages').
Branches and twigs of willows are also used in China in river training but Tyler remarks that this type of willow does not have the quality of the osier (like the very flexible type of willow, *katwilg*, Latijn Salix viminalis). The twigs of the Chinese willow are shorter, thicker and less flexible[10].

13.4 Why did river dikes along the Yellow River breach?

It follows from the text in Sections 13.1 and 13.3 that dikes along the Yellow River may breach for the following reasons:

- a high flood during spring or summer overtops a low[11] dike section;
- a shifting low water channel in winter time undercuts the non-protected under water slope[12] of a dike section, resulting in a breach;

[7] Most probably is meant 3 to 5 feet.
[8] The apron behaves like a 'falling apron' and ends up as a slope protection under 45°.
[9] The English name is gabion (*schanskorf).*
[10] Note that this contradicts the statement made by Van Schermbeek and Vissser.
[11] Low is a relative notion; it is the height of the crest of the dike above the river bed which has silted up and is much higher than the surrounding country.
[12] Tellegen (1942, 6) states: "It is for instance typical that the Chinese bank protection works are only applied above the water level." Van Duivendijk observed the same practice in Bangladesh.

- the kaoliang bank protection is in such bad state of maintenance that river flow along the bank cuts through the bank protection and scours the fill in the dike body;
- ice jamming in winter time which leads to dike breaches;
- piping.

13.5 Closure methods

Various authors describe the closure of dike breaches in China. In CNCID (2005, 162) it is stated that the 'blocking technology'(i.e. the closure of a breach in a flood embankment along a river) became mature during the Yuan, Ming and Qing dynasties. These successive dynasties cover the period 1270 – 1912.

In the text of CNCID (2005) a distinction is made between three types of constructions: 'rolled kids'(p. 85), 'wing kids' (p. 85) and 'wing enclosure kids' (p. 164 et seq.) in use for the closure of dike breaches. But, apparently, the original Chinese text has been translated by a non-technical translator and this results in indistinctness and also in some obvious mistakes. In order not to confuse the reader too much, these have been rectified (in as far as possible) in Appendix J. Figures from CNCID have been added for easy reference.

Rolled kids are in fact sink-fascines, having a diameter up to 1 – 2 m and a length of some 20 m, and they were already in use during the Song dynasty (960 – 1279). In Appendix J, Section J.2 the method of fabrication is presented. They are being used in the final stage of closure. One could describe their use as a speedy closure method.

Wing kids are used in the early stages of a gradual horizontal closure[13]. In Appendix J, Section J.3 a description is copied from CNCID (p. 85 – 86) and Kung (1936).

Finally, the wing enclosure kid was developed during the Qing dynasty (1644 – 1912). A wing enclosure kid is a modified wing kid. It is described in Appendix J, Section J.4 and, together with the presented figures, this description (from CNCID, p. 164 – 166) should be clear to the reader. It is also a gradual closure as the 'kid body' (the plug) is gradually built up and lowered in stages into the final closure gap.

The main problem when closing a flow gap in a flood embankment along a flowing river is the continuous high current velocity of the flow through the gap. In China this problem was solved by replacing the so-called single-dam blocking by a double-dam or triple-dam blocking (CNCID, p. 163- 164). In that case the head over the breach is distributed over two or three closures. This method is also sometimes used in the coffer damming of a river section for building a large dam (*stuwdam*)[14].

13.6 Closure works during the period 1889 – 1947

13.6.1 Closure of breaches in 1889 and 1903 (projects nos. 63 and 64)

In 1889 Van Schermbeek and Visser attend the closure of a dike along the Yellow River breached in 1887. Their description of this closure is cited in full in Box 13.1. It is however remarkable that the final closure is not described so it is not known whether final closure took place using sink-fascines or a plug. To rectify this omission the description made by Tyler (1906, 18-19) is presented:

> "I, unfortunately, just missed seeing the closing of the breach at Liu-wang-chuang, which occurred in September 1902, and was finally closed on the 16th March 1903. Some 1 500 yards of the dike were washed away and the greater part of the water flowed through the breach.

[13] And also in bank protection works.
[14] ICOLD-Bulletin no. 48a (1986, 79).

The closing was effected in the following manner: a dam was run out from either side of the breach by the successive building of pakwerks (Fig. 13.2). When these approached within 55 feet of each other a huge deflecting groyne of pakwerk was built on to the up-river dam, to minimise the rush of water through the aperture. The width of the channel abreast of the breach had been about 600 feet, but was reduced to less than 300 feet by the formation of a sandbank growing out from the opposite shore.

When the entrance had been constricted to 55 feet, ropes were stretched across and belayed to anchor-piles. Over 100 8-inch ropes were used, spaced close together. On these were then placed, in alternate layers, kaoliang and sacks of clay. When these materials were up to the level of the dam, the ropes were manned, and, at a given signal, were each lowered 1 foot on each side. The mattress thus formed was again added to with kaoliang and clay and again lowered, and so on. On the first attempt to close the breach, on the 2 January, the mattress, when it felt the full strength of the torrent, canted and capsized. It tore away a portion of one dam, and many of the men who were on it at the time drowned. The process was again repeated, but this time 15-inch hawsers were made fast to the mattress and led across the river channel and made fast to anchor-piles in the sandbank. Another failure occurred on 11 March, also with loss of life; but on 16 March it was finally closed.

The deflecting groyne was a remarkable structure, projecting 120 feet into the full strength of the current; it was tied up to the kaoliang dam by an immense number of 8-inch hawsers."

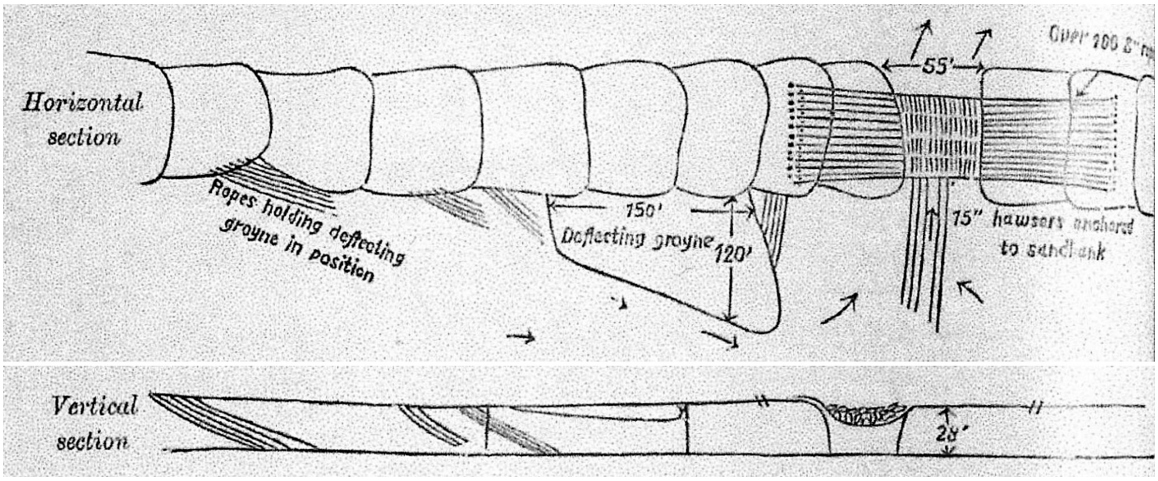


Fig. 13.2 Sketch of the closure works at Liu-wang-chuang in 1903 (project no. 64) showing, pakwerks, deflecting groyne and plug; source: Tyler (1906, 18)

13.6.2 Closure of breaches in 1923, 1925 and 1928 (projects nos. 65, 66 and 67)

In Todd & Eliassen (1938, 1929) it is reported that a serious dike 'breach occurred in Eastern Shantung near the coast when the entire flow of the river left the old bed.' The breach occurred in 1921. In this case, at the suggestion of Todd[15], this breach was closed in the spring of 1923 by means of a rockfill bund, constructed by dumping rocks from push cars crossing a 244 m long jetty (Fig. 13.3). The rockfill bund was made watertight by applying reed mats and burlap bags filled with clay to the face of the bund. In all 10 000 mats and nearly 300 000 bags were used.

The rock (31 000 m^3) had been quarried some 160 km upstream and transported to the site during several months by 300 junks. Subsequently, gangs of workmen, working day and night, had loaded the pushcarts.

[15] Todd (1933, 305 et seq.)

Simultaneously with the closure works, a cut-off channel had to be dug, having a length of 1 800 m and a width of 150 m, to lead the main flow back into the old river bed. The latter also had to be deepened over a length of 11 km. When the river was diverted to its old channel the discharge at that time was 425 m^3/s. At the peak the working force numbered 25 000 men.
Todd (1937, 6) points out that you can, however, not apply this dumping of rockfill in mid-winter as running ice and ice jams may completely raze the wooden piles of the jetty.

BOX 13.1 A closure of a breach in a flood embankment along the Yellow River in 1889[16] (project no. 63)

"Let us, for example, take a breach through which part of a dike bas been washed away. In China the new dike is generally made just where the old one lay; they begin at the same time at both sides and go on gradually approaching each other till they meet and the opening is thus closed. Let us now imagine one of the extremities of the old dike shortly after the closing works have begun, and we shall see it lengthened for some metres by a dam of millet-stalks and earth, with vertical inclines. Very firmly anchored at about two metres from the head of the dam, with her bows facing the rushing stream that flows from the river through the breach, lies a ship; and at the extremity of the old dike, both along the surface plane and the inclines, the men are busily engaged driving in wooden stakes to which are attached strong cables stretched tight along the dike in the direction in which the works are proceeding. If we follow these cables with the eye, we shall see them disappear in the water at the head of the dam and reappear again about two metres further on against the ship. Here they are wound round a windlass by which they can be tightened or loosened. They hang loosely in the water between the head of the dam and the ship, and thus form a kind of network, in which layers of millet-stalks are constantly placed right across the direction of the embankment, the surface plane being kept about half a metre above the water. The men who bring the bundles of millet, stamp the stalks firmly together; other workmen cover them with a layer of earth, which they bring up in wheel-barrows; and the men on board the ship keep loosening the ropes a little, so that the net filled with earth and stalks gradually sinks to the bottom and rests there. Then the ship is towed on a couple of metres; new cables are fastened, and the work proceeds in the same way. It is remarkable how orderly and rapidly the work is thus carried on. Considering that the depth of the water in the breach of 1887 was about 30[17] metres, while the current through it must have been extremely strong, it must, we think, be admitted, that no other known method would successfully have dammed the breach. The dam was 2200 metres long, had a maximum thickness of 120 and minimum thickness of 40 metres; its minimum height above the level of the river at the time of our visit was 5.20, its maximum height 10.30 metres. The old dike was at the upper end 6.50, and at the lower end 7.20 metres above that level. When such great dam works are being carried on, of course more than one ship lies at the head. As the works advance, the cables are no longer fixed in the old but gradually fastened on the new dike. Evidently these millet- stalks, especially at first, are compressed to a large extent and let a great deal of water through. The dike by *Lai-t'ung-chai,* three months after the closing of the breach, during our visit, was still being raised constantly and shook pretty strongly under our feet. As far as we could judge, the dike let very little water through. This will be easily understood when we consider the great mud alloy of the water which at first oozes through and must certainly contributes gradually to fill up the interstices between the millet-stalks."

[16] The text has been copied from the Report made by Van Schermbeek and Visser as presented in Fijnje van Salverda et al (1891, 102).
[17] It is assumed that something has gone wrong during translation from Dutch to English by a third party: 30 m should probably read 30 ft. The same applies to other figures in the text following.

Fig. 13.3 Completing a rockfill dam from a jetty across a breach in the dike along the Yellow River, April 1923 (project nos. 65); source Todd and Eliassen (1938, p. 1929)

The next major breach occurred in August 1925 (Todd & Eliassen, p. 1930) in Western Shantung. Also here Todd made a plan (surveys and cost estimates) for closure which was effected within six months. But here only half of the river flow had left its bed.
"In late February 1928, a heavy ice jam caused ponding and overtopping of the south main dike in two places east of Litsin, Shantung, near the coast. This breach was closed within a few months using old Chinese methods but considerable help was given by the river itself in that it partly silted up the breach openings." (Todd & Eliassen, p. 1930).

13.6.3 Closure of the major breach at Tung Chuang in 1935 – 1936 (project no. 68)

13.6.3.1 Construction of the closure works

H. Y. Kung was chairman of the Yellow River Commission and wrote a book[18] about the closure under his supervision of the breach at Tung Chuang (Shantung, China). He notes first of all that closure of a breach in a dike along the Yellow River has always been a difficult task not only from a technical point of view but also financially and logistically as large quantities of materials have to be purchased and transported to the site. Kung states that he has introduced a new method for closing the final opening: 'willow-stone sausages' (i.e. sink-fascines having a core of stone[19]). But for narrowing the initial opening the old method using kaoliang stalks, earth and rope has been used.

Breaches occurred because of (a) high discharges (14 000 m^3/s on 10 July 1935) and (b) the Shantung Yellow River Bureau not yet being fully prepared for the high river stages during summer. The south flood embankment breached at six places. Breaches had lengths of 113, 120 and 112 m while the 4th , 5th and 6th breach led to one large breach having a length of 2 400 m. The main river flow concentrated on the most eastern breach of 700 m length. The breaches were due to three reasons: (1) a sharp bend in the river, (2) no good maintenance since the last breaching in 1926, (3) since 1932 huge deposition of sediment downstream of Tun Chuang during the annual flood season.

[18] H.Y. Kung (April 1936); 'The Yellow River crevasse closure work at Tung Chuang'.
[19] The sink-fascines described in Appendix J had a core of earth and were much larger in diameter.

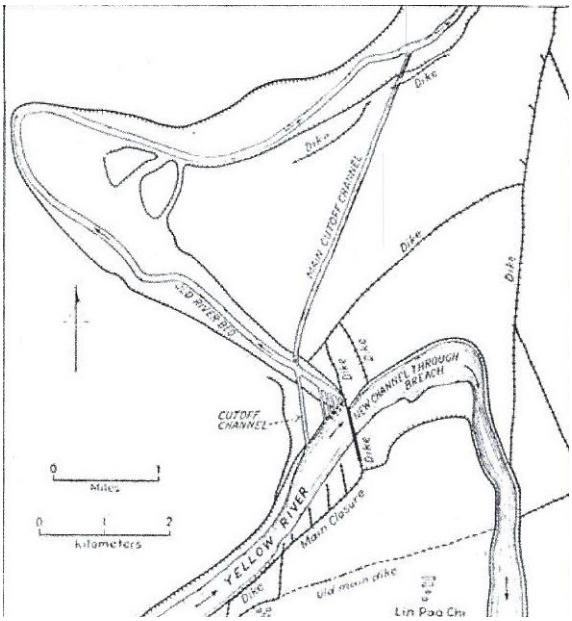


Fig. 13.4
Map of the situation at and near the breach at Tung Chuang after the repair works had been carried out; source: Todd (1937, 5)

The repair works started by repairs and lengthening of existing groynes in the area of the breaches. Four new groynes should direct the flow in northern direction. The breaches would be repaired by starting from both ends using kaoliang stalks. When a dike section had been repaired a fascine mattress ballasted with stone (*kraagstuk*) would immediately be placed on the river side slope[20]. Kung recalls that the initial plan of the 'Crevasse Closure Commission 'was based on a 100 m long final closure gap which would be closed by dumping sandbags, stone, bricks and 'what not' from a jetty.

He considers this method not to be suitable for closing the Yellow River breach, because:

- the method is difficult for water depths exceeding 6 m;
- many such closures failed in the past because the river bed was too sandy;
- it is difficult to build a jetty in the breach which will remain stable in the high current velocities.

Kung got the supervisory job on 16 December 1935. At that time repair works to dikes and groynes were already well underway. His first task was the purchase of materials: kaoliang stalks, willow brushwood, stone, galvanized wire (Φ 1.3 mm), rope, millet straw, jute bags and hemp.

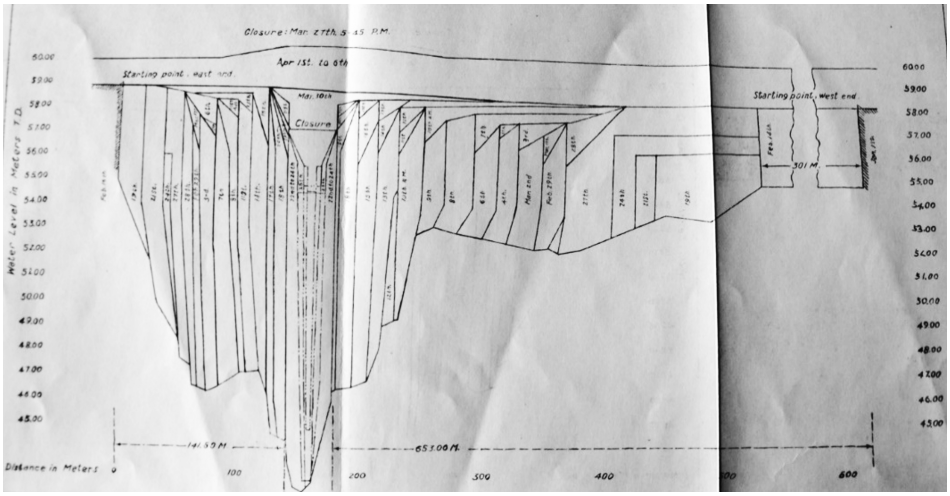


Fig. 13.5
Longitudinal profile of Tung Chuang breach closure; source: Kung (1936)

[20] Bourdrez states in his film that fascine mattresses had been introduced at the Yellow River 25 years earlier by ir. H. van der Veen.

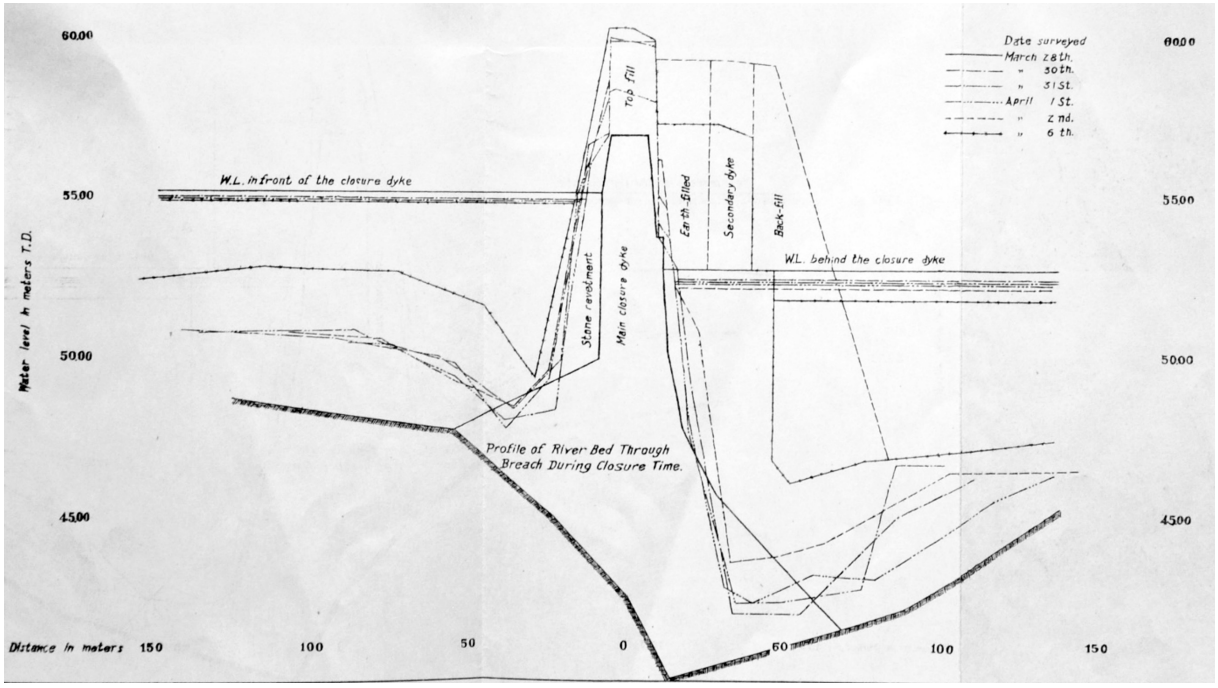


Fig. 13.6 Cross-section over finally closed breach at Tung Chuang; also the gradual silting up of the scour hole is shown; source: Kung (1936)

The stone was transported from a number of quarries by train, transported from various train stations to the banks of the Yellow River and then transported further by boat. Not less than 664 small sailing ships (junks) were operated.

The new groynes were constructed in the shallow part of the river by using earth. In the deep part kaoliang stalks were used. Groynes had a length of 108 to 300 m. After dike repairs a breach remained having a length of 834 m. The full river discharge was flowing through this breach. In order to bring the river back in its original bed six cut-off channels had to be dug through the silted up old river bed, varying in length between 165 and 5 383 m (Table 13.1).

The cut-off channels (overall length 7 443 m) were dug by 20 000 farmers which had fled the flooded area. They worked in groups of 100 men.

The groynes as well as the cut-off channels were required to enable closure of the breach.

This still left the most difficult task: closure of the breach. First the west dike (653 m) was built in 31 sections using kaoliang stalks, clay and 40 to 50 long ropes per section (Fig. 13.8) as described in Appendix J and Box 13.1. The sections had a width of 15 m and a length of 10 to 15 m. they went 9 to 12 m down settling in the mud. The clay was transported in baskets and in wheelbarrows. Compaction of the clay was achieved by means of a 35 kg weighting 'flapper' which is tossed up and down by 8 men (Fig. 13.7).

Table 13.1 Cut-off channels; no. IV cuts a meander ; source Kung (1936, 10)

cut-offs	length (m)	width (m)			depth (m)	volume (m^3)	opened on
		top	bottom	top & bottom			
no. I	166.6			16	1.5	3 996	Jan. 14
no. II	434.7	25	3		2	1 737	March 20
no. III	1 040	43	15		2.7	81 432	,, 20
no. IV	5 383	45	15		3	484 470	,, 20
no. V	254	16	8		2	6 096	,, 19
no. VI	265	11	5		1.5	3 180	,, 19
					TOTAL	580 911	

Fig. 13.7 Compaction by tossing a 'flapper' up and down;
source: https://www.francoisbourdrez.org/fotos-en-film (accessed on 20 Nov. 2023)

A start was made on 1 January during severe frost (the ice in the river was cut by hand) and work continued by day and night. At the river side of the dike, fascine mattresses ballasted by stone were placed on the slope as protection against ice-floes during the thaw period. At the land side slope a layer of clay was placed to counter seepage through the kaoliang dike body.

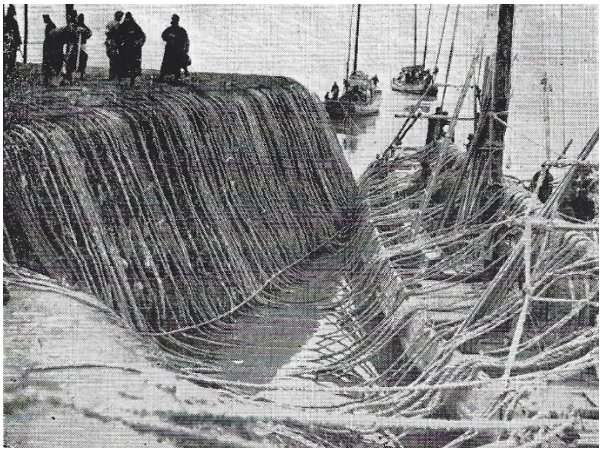


Fig. 13.8 Ropes being laid out for kaoliang dike section, note boat at right side;
source: Kung (1936, 13)

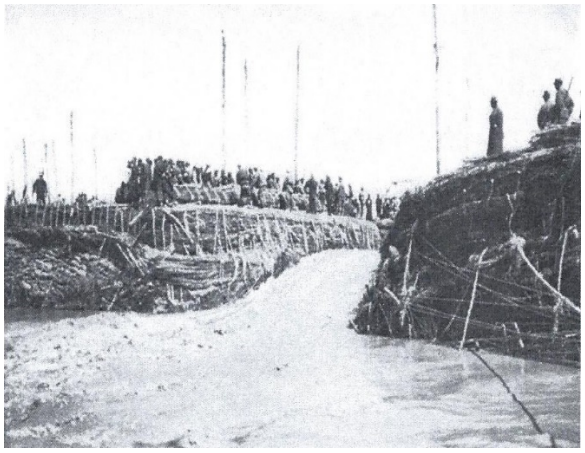


Fig. 13.9 Torrent through final closure gap
source: Todd (1937, 9)

On 16 February 1936 work on the east dike started (141 m). On 18 March an opening was left of 40 m. The dike ends were protected by fascine mattresses. This opening would be closed by rolling sink- fascines (diam. 1 m) in it. The encasement of the sink-fascines was composed of fascine strings (*wiepen*), long 10 to 18 m, having a diameter of 15 cm; the willow branches having been bundled using galvanized wire. A long heavy rope is woven through the centre of the core to act as an anchor to hold the sink-fascine in place when it is being rolled into the swiftly flowing stream (Todd, p. 8). The core of stones and fascine strings were tied together using hemp ropes. A sink-fascine would be rolled and pushed

into the gap by 50 to 60 labourers. The sink-fascines were constructed by six groups. Each group could make a sink-fascine in 10 to 20 minutes. This meant that each hour 20 sink- fascines became available. As soon as the gap was closed, stone was dumped on the sink-fascines and, subsequently, fascines, sandbags and a thick layer of clay. The final closure works started on 22 March and were complete on 27 March at 5.45 p.m. (Figures 13.10 and 13.11).

Fig. 13.10 Sink-fascines being dumped on day of closure; source: Kung (1936, 18)

Fig. 13.11 Last sink-fascine ready for dumping; source: Kung (1936, 19)

Meanwhile construction of a secondary dike, parallel at the land side to the first dike, had started. The profile and composition was the same as the east and west dike. As there was considerable leakage through the voids between the sink-fascines, the last opening (wide 13 m) in that dike was closed on 4 April by using the plug method. The net of ropes across the opening was fixed to piles at both sides of the opening. The ropes were slackened inch by inch following beats on a gong.

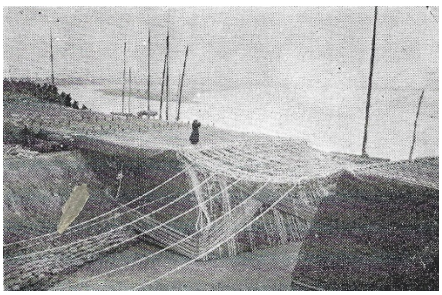


Fig. 13.12 The 'dragon's net' over de final opening in the secondary dike

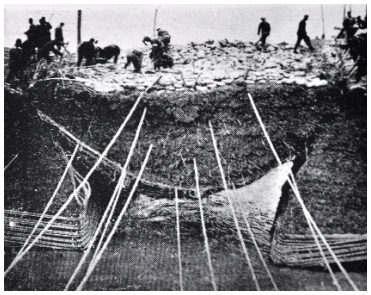


Fig. 13.13 The plug ready for lowering

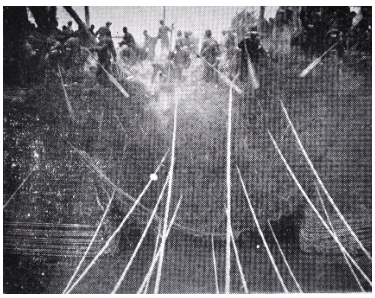


Fig. 13.14 Slipping the plug

source for all three photos: Kung (1936, 25 and26)

The space between both dikes (14 m) was filled by sandbags and clay. Finally a berm of clay, wide 14 m, was dumped behind the secondary dike to render the dike absolutely watertight.
In order to arrange this watertightness the water level at both sides of the dike had to be the same. This was achieved by constructing a 1 376 m long perimeter bund at the land side of the dike wide 10 m and having slopes 1: 2½.
The first spring flood arrived on 12 April. This was later than usual and this also contributed to the success of the closure.

13.6.3.2 The flow through the closure gap

In Appendix K hourly hydrometric measurements are presented for the period 25 March until closure, 2½ days later. On 25 March at 7 a.m. water level difference in the remaining opening is 1.44 m and the discharge through the opening is 530 m^3/s.

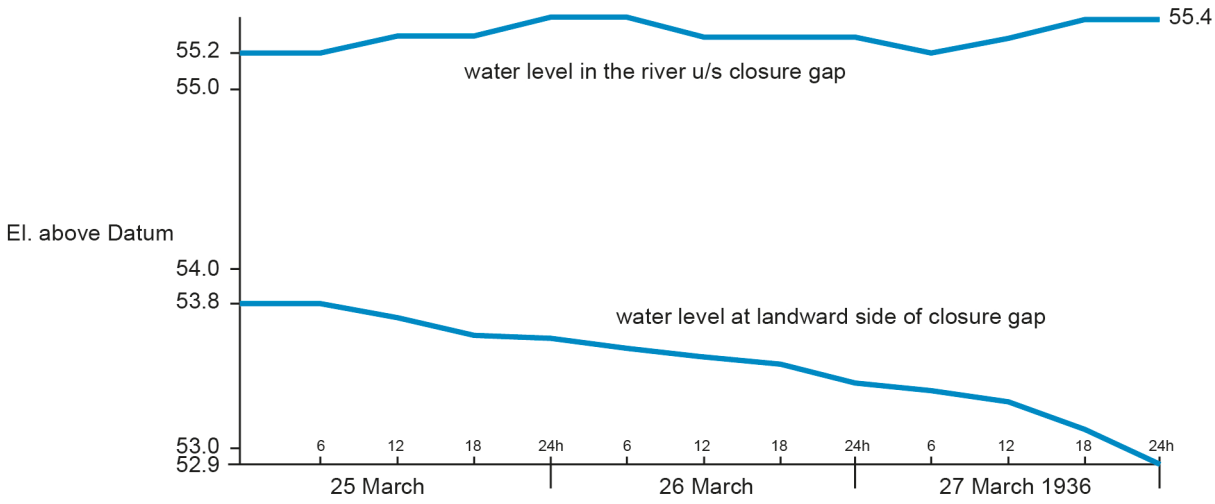


Fig. 13.15 Water levels during three last days of closure at Tung Chuang; source: Kung (1936)

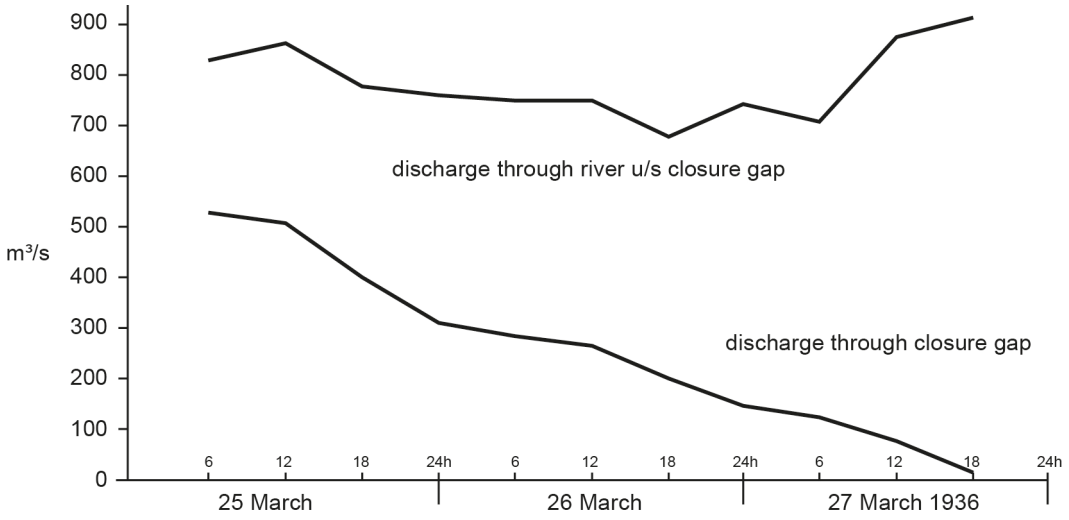


Fig. 13.16 Discharge through river upstream of closure gap at Tung Chuang and the part of this discharge flowing through closure gap; source: Kung (1936)

On 27 March, 1¾ hour prior to closure, at 4 p.m. the water level difference had gone up to 2.15 m and the discharge had gone down to 30 m^3/s.
An effort has been made to check these figures. On 25 March at 7 a.m. the final closure has been going on for 3 days and it will have been, until that morning, more or less a horizontal closure.
Most probably the flow is still sub-critical, so the formula[21] for the discharge is:

$$Q = \mu . (b_w + b_t)/2 . h_2 . \{2.g.(h_1 - h_3)\}^{½}$$

In which:
- Q = discharge = 530 m3/s (from table in Appendix K)
- μ = contraction coefficient = 0.8 (estimate)
- b_w = width of closure gap on waterline
- b_t = width of closure gap at bed level
- h_2 = water depth in closure gap, relative to bed level
- g = gravitational acceleration = 9.81 m/s^2
- h_1 = water depth upstream of dam, relative to bed level (or datum)
- h_3 = water depth downstream of dam, relative to bed level (or datum)

[21] As much as possible the symbols are used from Notation in CIRIA, CUR & CETMEF (2007).

As $h_1 – h_3$ = 1.44 m (from the Table in Appendix K) and h_2 is estimated to be 10 (from Fig. 13.6), as, moreover, the value of $(b_w + b_t)/2$ = 12.5 m (because a value for b_w = 20 m and b_t = 5 m, at that stage of closure, would appear to be quite realistic), then the current velocity is 5.3 m/s.
During the last day of closure the flow will be super-critical as the horizontal closure will have changed into a vertical closure:

$$Q = \mu . (b_w + b_t)/2 . {}^{2}/_{3} h_2 . \{2.g.({}^{1}/_{3}\, h_2)\}^{½}$$

This implies, as the closure proceeds, a decreasing value for h_2 and for $(b_w + b_t)/2$. and, consequently, for Q. This also follows from the Table in Appendix K.

13.6.3.3 Quantities, labour and cost of construction

For some quantities, please refer to Table 13.2.

Table 13.2 Quantities of construction materials incorporated in the closure works (project no. 68) source: Todd (1937, 8 – 9)

type	item	unit	quantity	comments
stone		m^3	40 000	transported from 70 miles u/s in 600 junks[22]
plants	kaoliang stalks	ton	15 000	arriving in hundreds of cars from other farming districts
	burlap bags	no.	300 000	to be filled with clay
	willow branches	ton	4 500	arriving in cars, for fascine mattresses and sink-fascines
	hemp	ton	1 000	for rope to be made on site; arrives in wheelbarrows
	willow stakes	no.	90 000	to anchor the ropes and wires
wire	Galvanized wire	ton	2 000	used in fascine strings and small cables
clay and silt	in dikes	m^3	710 000	
	in excavations	m^3	581 000	details in Table 13.1; earth was hand-carried in baskets

The number of labourers, at its peak working on the job, was 25 000. It is stated that the excavations for the cut-off channels were carried out by 20 000 men. Apart from a few pumps and some light railway push carts, the equipment most used were native picks, shovels and wheelbarrows.
The overall cost of construction, according to Todd (p. 9) was around US $ 800 000. From Kung (1936, 31) it can be concluded that overall $ 2 638 000 was spent. This amount must have been in Hongkong dollars because the so-called British Trade dollar was only in use up to 1935. The Hongkong dollar at that time had a fixed exchange rate to the British pound: 16 HKD = 1 GBP.
So HK $ 2 638 000 = GBP 164 875. In 1935 1 GBP = US $ 4.90, in 1936 1 GBP = US $ 4.97. If an average exchange rate is taken of 1 GBP = US $ 4.94, the construction cost has been US $ 814 482. This figure matches with the 'around US $ 800 000' of Todd. US $ 814 482 of 1936 comes to US $ 14 896 000 of the year 2020, which in turn is equal to € 12 953 000 (average exchange rate in 2020 US $ 1 = € 0.87).

13.6.4 Closure of the breach near Kai Feng Fu in 1947 (project no. 69)

According to a poster presentation[23], a breach in the southern flood embankment at Kai Feng Fu (Fig. 13.17) was repaired in 1947. Only a few figures are available and, accordingly, only a few remarks can be made. From Figures 13.18 and 13.20 it can be concluded that the base of the new dike is a 3 m thick layer of stone. At the riverside of that layer of stone the traditional kaoliang dike sections have been constructed until a closure gap is left wide 55 m. This is being closed from the left side by dumping the traditional sink-fascines and from the right side by dumping stones encased in steel nets. On the day of closure (15 March 1947) an opening is left, long 25 m. Remarkable is the jetty which was needed to dump the sill of stone (Fig. 13.19). One would expect it to be a nuisance for the dumping of the kaoliang and sink-fascines but apparently both have been kept outside the jetty (Fig 13.20).

[22] "Where the stone has been brought by ox carts and light railway push carts 10 km from a station to which it has been shipped 160 km by standard railway from the quarries" (after Todd, p.7).
[23] It was found in the archives of the Faculty of Civil Engineering and Geo Sciences of Delft University of Technology.

The final cross-section (Fig. 13.20) shows kaoliang bunds bordering the dike body at land and river side while also sink-fascines have been placed at the river side as protection against scour (and wave attack?).

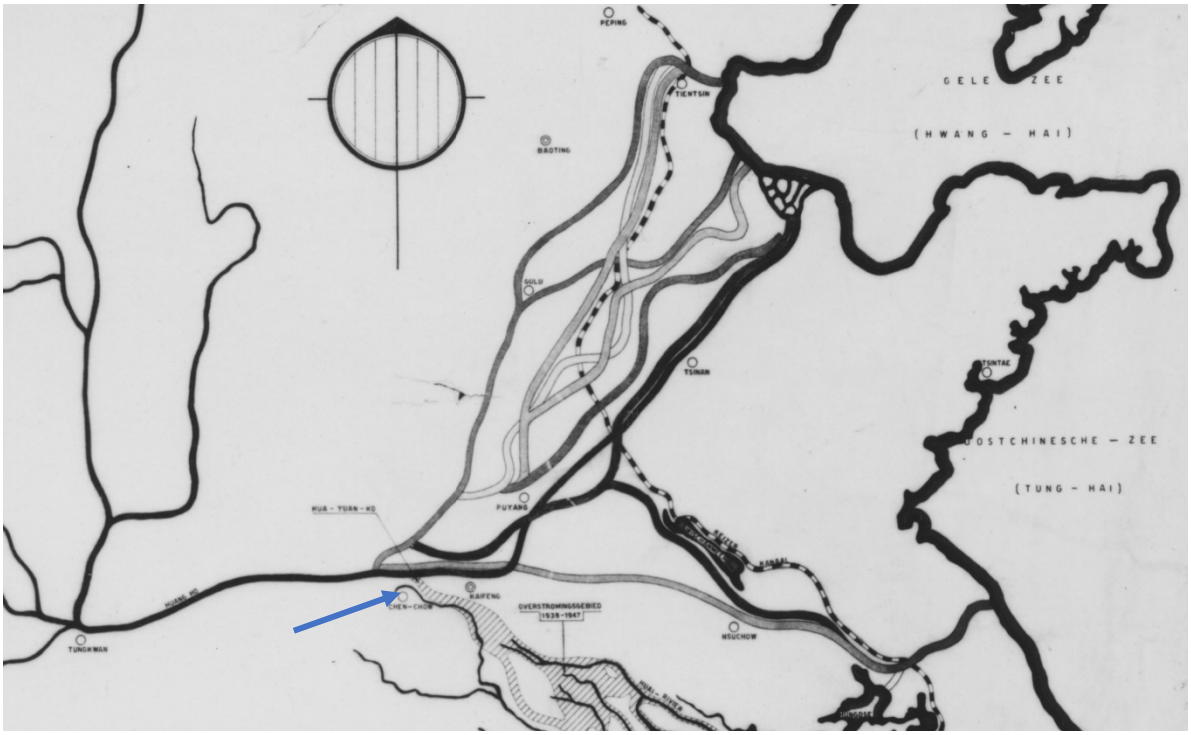
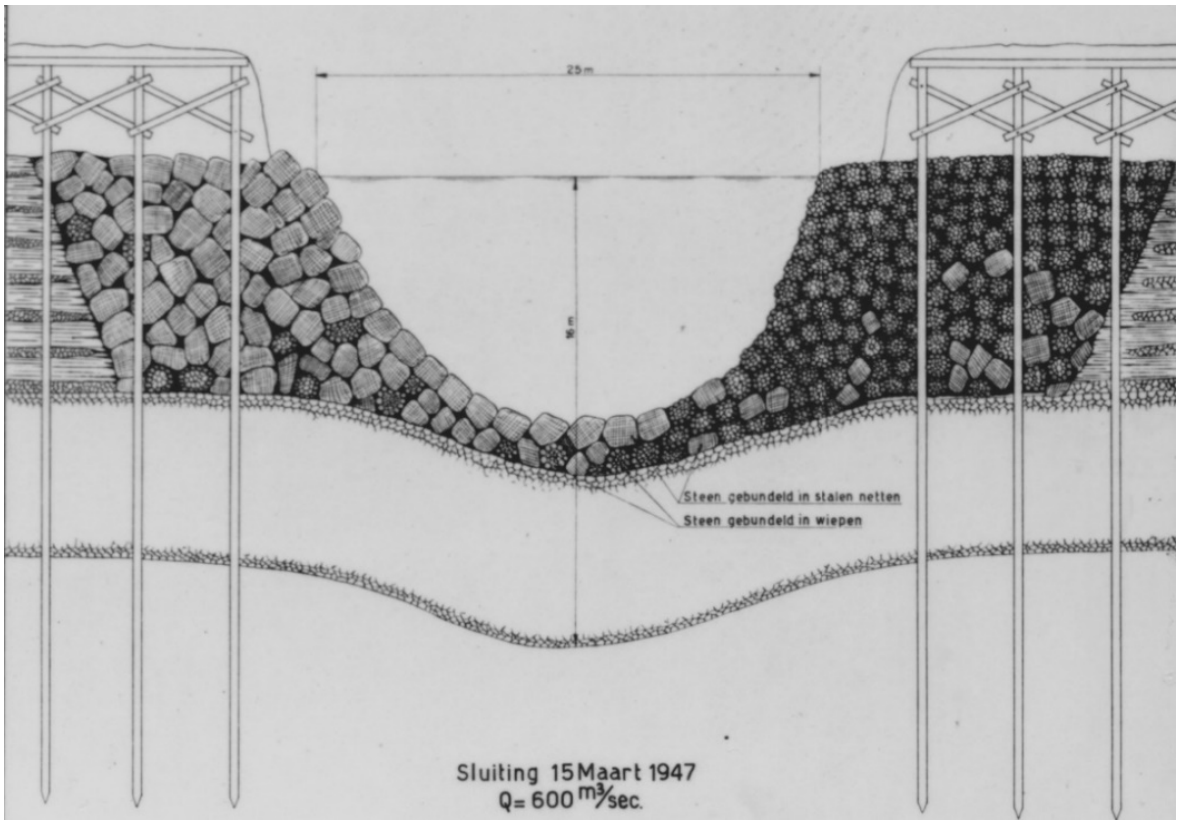


Fig. 13.17 Location of the breach in southern flood embankment of the Yellow River in 1947



Fig, 13.18 Cross-section of the closure gap on the day of closure (15 March 1947); discharge is 600m^3/s

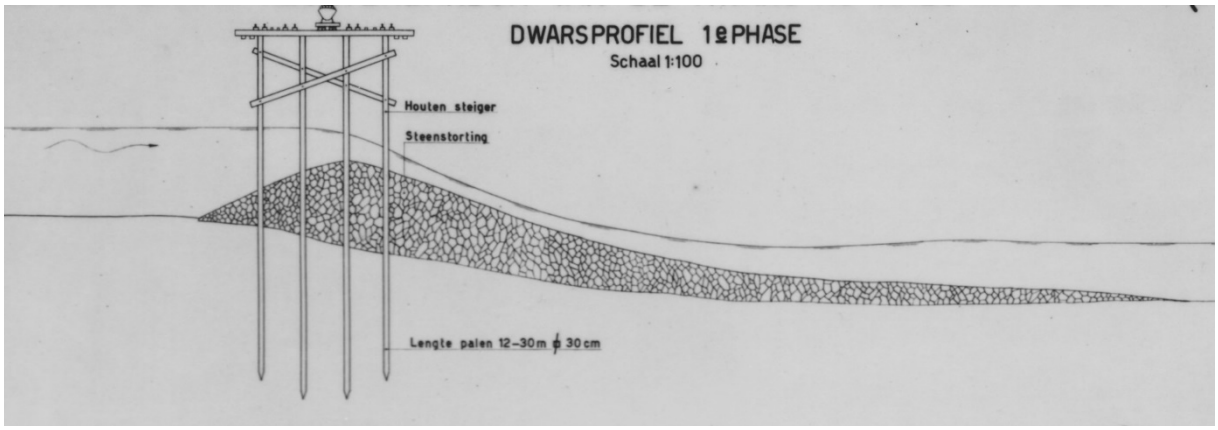


Fig. 13.19 Cross-section of jetty and dumped sill of stone

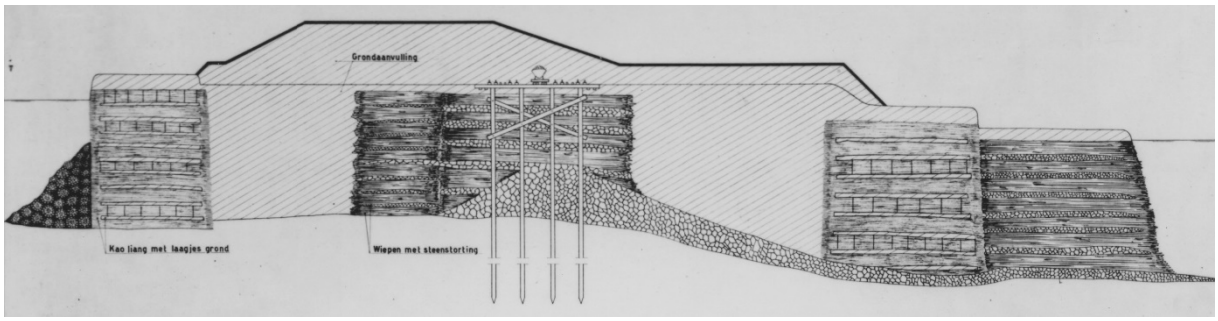


Fig. 13.20 Final profile of flood embankment

13.7 Some remarks about the closure methods used in China

During the past 2 100 years dikes along the Yellow River and its tributaries frequently breached. One of the phenomena causing these breaches was the huge quantity of silt transported by this River: the riverbed was continuously being heightened and as a result its level is now well above the surrounding country side.
Consequently, after a breach, the River tended to take a new course through these lower laying agricultural areas. This caused considerable hardship to the local population and through the centuries the authorities have tried to repair the breaches and bring the River back in its old course. But this was not always successful.

In the foregoing sections the emphasis has been on successful closures: the difficulties encountered and the closure methods used.
Below, the main technical features, characterizing a breach in a dike along the Yellow River, as well as its subsequent repair are listed:

- the head over the breach can be considerable and its bottom is easily erodible resulting in great depths;
- in order to make a closure technically feasible the head is sometimes distributed over two or even three closures (‘triple dam blocking’);
- after the river has taken its new course through the breach, the original river bed downstream of the breach will rapidly silt up, necessitating its re-excavation prior to closure of the breach;
- until the twenties of the 20th century the only materials available for closure of a breach were millet stalks, willow branches, bamboo, ropes and earth; though, apart from these materials, the closures were only possible by the well- organized employment of many thousands of labourers (sometimes up to 25 000);

- the use of thick ropes and hawsers was essential to prevent the (light) closure materials from being removed by the current;
- breaches were closed by, firstly, narrowing the gap through construction of successive pakwerk-sections, firmly tied to the existing dike head or to each other (horizontal closure);
- the final closure was vertical, either by gradually lowering a plug comprising millet stalks and earth, into the remaining gap, or by rolling large sink-fascines into the gap from both sides;
- in the twenties of the 20th century the closure methods described above were modified thanks to suggestions from American and Dutch side;
- the Americans introduced the use of rock in river closures, dumped from a temporary bridge across the gap; this rock dumping was, for instance applied during the closures in Eastern Shantung (1923) and at Kai Feng Fu (1947) (projects numbered 65 and 69);
- the Dutch, as part of their works on the improvement of the Whangpoo River at the beginning of the 20th century, had introduced fascine mattresses and sink-fascines in China; both fascine constructions were abundantly used during the closures at Tung Chuang (1935) and Kai Feng Fu (1947);
- the abundant application of millet stalks renders a closure bund rather permeable, but the high silt content of the river water seeping through the millet stalks helps to obtain watertightness.

In CNCID (2005, 84) it is suggested that "Ningxia Hui Autonomous Region still applies sunkun fascine works to block breaches …." . But from the text it is not immediately clear what exactly is meant by "sunkun fascine works", probably the whole range of constructions like pakwerks, plugs and sink-fascines. It would be interesting to know whether these closure methods are still applied at the time of writing (August 2023).

CHAPTER 14 Closure methods in East-Pakistan / Bangladesh prior to 1979

14.1 Introduction

Prior to the introduction of Dutch labour-intensive closure methods in Bangladesh during the period 1978 – 1982 (Ch. 11, Section 11.3), a local closure method was in use, known as the 'mata method'. A mata is in fact a sink-fascine (as discussed in Ch. 4, Section 4.2) , adapted to local circumstances.

In May 1976, the International Technical Assistance Department (DTH) of the Ministry of Foreign Affairs of the Netherlands commissioned ir. J. van Duivendijk of Haskoning, Consulting Engineers and Architects, to carry out a study into the methods being used in Bangladesh for closing tidal channels. This commission was connected with the execution of so-called 'early implementation projects' in which the Netherlands (Ministry of International Development Cooperation) was involved. These are small drainage and flood control projects which, at that time, were carried out in Bangladesh.
In the sixties of the 20th century in East-Pakistan[1] a number of tidal channels had to be closed as part of the first phase of the Coastal Embankment Project. For these tidal closures the said mata method was used. It is not known by whom this method was developed . The following possibilities have been considered:

- It is an indigenous method which was developed in Bengal in colonial or pre-colonial times;
- it has been introduced as sink-fascine (doodeman) by the Dutch, and adapted for the use of local materials, during the period the Dutch had a trading-post (1607 – 1825) at Hooghly in Bengal;
- it has been introduced by one of the American consultants (IECO and/or Leedshill-Delew) who were either directly or indirectly involved in the Coastal Embankment Project and who could have been aware of a similar method used in China through the publications by Kung (1936), Todd (1937) and Todd & Eliassen (1938).

Various sources have been contacted in the USA while the publication by Rashid & Rahman[2] has been studied. No reactions were received from the USA and there is no reference whatsoever in the said publication last mentioned to the mata method.
In 1976, some of the aforementioned tidal channels were still open and the aim of the study made by Van Duivendijk and his colleagues was to ascertain whether the mata method was in fact appropriate for closing these remaining open channels. In the affirmative, the construction of the closures would be funded by the Dutch Technical Assistance programme.
Below, first of all the mata method is described. Subsequently, the experience with the mata method will be discussed and analysed for one (failed) closure and conclusions drawn.

14.2 Description of the mata closure method[3]

14.2.1 General

The description of the methods used in South-West Bangladesh for closing off creeks, rivers and polders from the tidal influence has in first instance been made on the basis of discussions with officers of the Bangladesh Water Development Board (BWDB) and field visits to completed cross-dams or sites where closures failed.

[1] At that time the region, known in colonial times as East- Bengal, was still part of Pakistan. The region declared itself independent from Pakistan in March 1971 under the name Bangladesh.
[2] Rashid, Salim & Rahman, Rezaur, editors (2010); Water Resource Development in Bangladesh, Historical Documents; Dhaka, Bangladesh, The University Press Limited.
[3] Most of the text in the following sections has been copied (though slightly modified) from the reports BWDB (1976) and (1977). Both reports have been written by Van Duivendijk.

Some 25 closures have taken place as part of the Coastal Embankment Project, Phase I. After constructing embankments along tidal creeks and rivers, invariably a last gap has to be closed through which a creek is filled and, depending on the tide and local circumstances, part of mud flats are covered with salt water. The method used to close such gaps is unique in a sense that it combines local skills with the materials locally available.
Two factors govern the method used:

1 Everything is done by manual labour.
2 The only construction materials available are wood (bamboo, goalpata, i.e. palm leaves, brushwood, bullah piles), soft clay, gunny (= jute) bags, jute ropes and, sometimes, old country boats. In Appendix L, a detailed description of these materials has been presented.

14.2.2 Sites and available data

Gaps to be closed on average have a length[4] of 120 m and a depth of 10 m below R.L.[5]
For the various sites mentioned during the discussions the dimensions are given in Table 14.1.
As can be seen, the largest gap mentioned had a length of 245 m and a depth of only 8.5 m (because it was a shallow silted-up river).
It was noted that in the beginning (the early sixties) the openings to be closed were shallow in depth and narrow in width, as, however, the Coastal Embankment Project progressed, conditions became more severe.
In general, tidal differences are 2 to 3 m during spring tides and 1.5 to 2 m during neap tides.
For some sites more information was obtained (Table 14.2).

Table 14.1 Information about constructed closures

project no.	site	width of creek[6] (m)	original depth below R.L. (m)	construction period[7]	remarks
70	Gangrail	125	11	-	closure failed, depth became 15 m
-	(no name)	70	7.5	10 days	
-	,,	55		15 days	
-	Koyra River	90		90 days	
-	(no name)	245	8.5	80 days	
-	Minhaj	215			

Table 14.2 Information about tides for some sites

project no.	site	low low tide (m)	high high tide (m)	tidal differences	
				springtide (m)	neap tide (m)
70	Gangrail	R.L. - 0.60	R.L. + 2.40	2.7 – 3.0	2.1
-	Koyra River			3	2.1 – 2.4
-	(no name)			3	2.4[8]

It was stated that tidal gauges are being read during closure works. Preferably, existing gauges are used, attached to regulators (i.e. drainage sluices in flood embankments).

[4] In the reports, which are at the base of this Chapter, all dimensions are in ft. These have been transferred to metres but the measures on the figures are still in ft.
[5] R.L. (Reference Level) = zero PWD.
[6] In this dissertation a creek or river has a width while a gap has a length.
[7] Construction period is the time elapsed between start of so-called platform and primary closure.
[8] During a second discussion it was said that tidal differences were 1.5 m during neaps and 2.1 to 2.4 m during spring tide.

Daily hand soundings are made in the axis of closure which are plotted onto a graph in various colours. Other soundings are made at 30 m and 60 m distance from axis of closure (both sides) but these are normally not plotted.

14.2.3 Construction of closure bund

The construction of the closure bund (locally called 'primary dam') is as follows.
On both sides of the creek or river to be dammed a platform is constructed of earth, the width along the bank depends on the length of the mata's to be used but varies mostly between 30 and 45 m. The width perpendicular to the banks is 15 m. This platform forms the start of the cross-dam and is in general used for preparation of mata's, although at a later stage of the construction period it is sometimes also used for storage of brushwood and golpata. Throughout the construction period the level of this platform is maintained at R.L. + 2.7 to + 3 m. (Fig. 14.1).
The major part of the gap is dammed by constructing, filling and maintaining (to a safe level) compartments, one after the other, working from both banks. The compartments have dimensions of 30 to 36 m (perpendicular to axis of primary dam) by 3 to 6 m approximately.
When coming to the final gap, the length of 30 to 36 m is mostly reduced to approx. 15 m to speed up the final closing operation. The final closure gap has a length of 9 to 15 m.

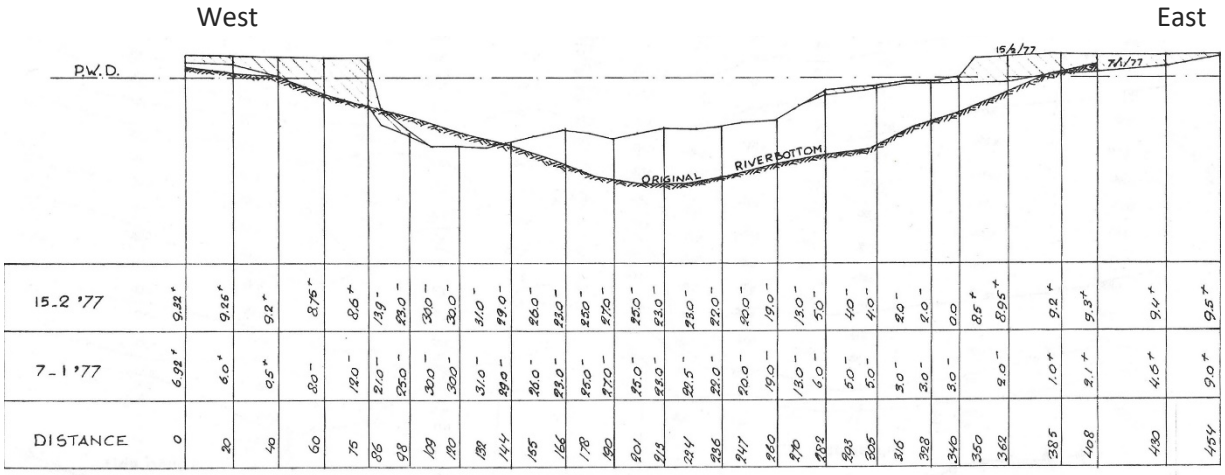


Fig. 14.1 Length profile over dam axis at Gangrail (7 Jan. and 15 Febr. 1977); source: BWDB (1977)

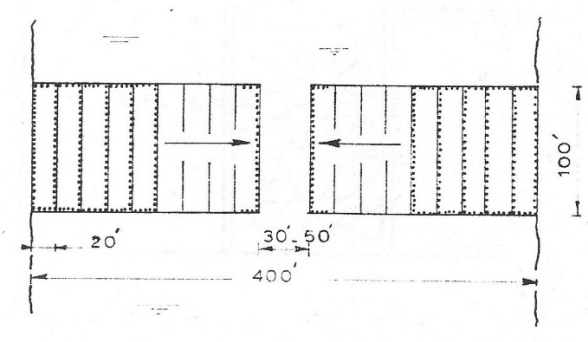


Fig. 14.2 Top view of constructed compartments Source: BWDB (1976)

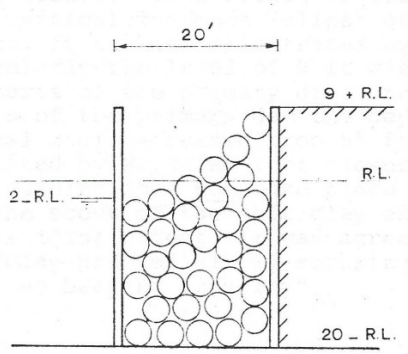


Fig. 14.3 Cross-section over mata's placed in the 1st compartment; Source: BWDB (1976)

In principle the compartments are constructed as follows:

- An enclosure is made by driving bullah piles with a floating piling rig made of bamboo and placed on a 'catamaran' arrangement of two local boats, some 10 m long. (Ch. 11, Fig. 11.28)

The block weight, which is some 180 kg, is handled by 10-12 labourers. A piledriving party consists at least of 12 labourers, one for guiding of the block, one for guiding the pile and the others for heaving and dropping the block.

Seven to ten blows are given one after the other before a short rest is taken.

The block is guided by the help of a steel rod going through the block and fitting in a predrilled hole in the top of the pile. In this manner sometimes piles are being driven down below the water level.

In the front face of the compartment, the distance between the piles is in general 1.5 to 3 m, but much depends upon local soil conditions, and distances up to 4.5 m have in fact been noted in the field. The head of the pile is driven to a level around Low tide as otherwise it will hamper the easy placing of the mata's.

Towards the deeper parts of the creek, the width of the compartments is reduced to 3 m and in a later stage even to 1.5 m. This is done to bring the level of the placed mata's easier to the surface of the water during one low tide. The bullah piles at the front face and at the sides are sometimes bolted together by lengthwise sawn-through bullah piles.

After the enclosure of a compartment is complete, it is filled up with mata's up to 1 m above Low tide.

A mata is a roll, about 30 m long, built up of different layers of golpata (palm leaves), straw and earth. Construction takes place as follows: straw ropes are laid every 25 cm (so 121 ropes), then golpata (length 1.5 to 3 m, width 0.6 to 0.9 m), straw at random and earth on top, total thickness of sandwich is 7.5 cm. 300 Labourers will roll the sandwich up and the final result is a kind of sausage called mata, which is 30 m long and 0.9 m thick. (Photographs 1, 2, 3, 4 in Fig. 14.6). The uplift of such mata will be 6.34 kN/m^3; the weight of a dry mata is said to be between 6.6 and 7.3 kN/m^3.

On top of the mata's comes the normal fill material, which in general consists of layers of clay and straw, with brushwood at the front end and at the sides. The level of this fill is at least 30 cm above High tide.

- When 20 mata's have been piled up at the side of a compartment, filling of this compartment may start one hour before low water slack, and after the current velocities have slackened sufficiently. The mata's are rolled and dumped, one after the other.

  The rolled-in mata will sink gradually and labourers on boats will pin the mata's to each other and to the bottom with sharpened bamboos.(Photographs 5, 6 in Fig. 14.6). When all available mata's have been placed in a compartment the situation is more or less as shown on the sketch (Fig. 14.3), albeit that this is an ideal situation. In practice the situation is less straight forward, with 30% of the mata's being wasted due to impact of the current on the mata during its immersion. The open space is filled up with brushwood, straw and earth. This is more difficult than placing mata's as it is essential to keep ahead of the rising tide. The brushwood and straw are pinned down by horizontally placed bamboos. The filled-in compartment is also rigidly being maintained at a level of R.L. + 2.7 m throughout the closure period.
- After the first two compartments have been completed, the next two will be constructed and so on until a gap of 9 to 15 m remains. In this gap the velocities are usually high, over 3 m/s. Once a head difference was measured over the gap of 1.8 m. Sometimes the completed parts of the dam are protected by groynes (Fig. 14.4), built of bullah and bamboo, to check the effect of horizontal eddies.

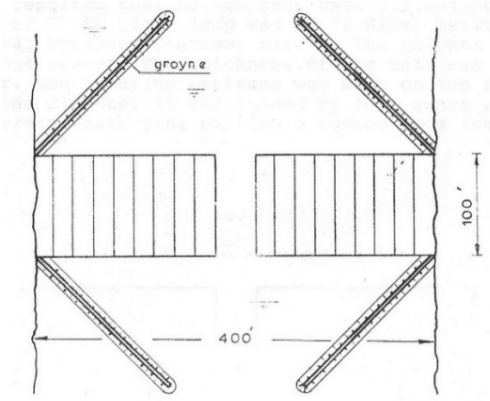


Fig. 14.4 Groynes to deflect eddies

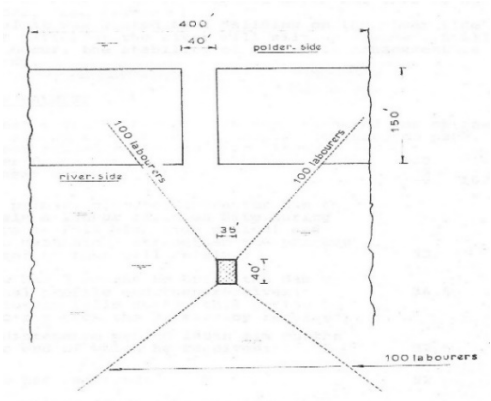


Fig. 14.5 Thick floating mattress

source both figures: BWDB (1976)

1 mata

2 closure gap and compartment in the forefront

3 dumped mata's & compartment being made

4 mata's ready for dumping

5 'ganji piling'

6 'side protection'

Fig. 14.6 Various details about mata's, compartments and ganji piling; source BWDB (1977)

Within the period of 15 days of the tidal cycle the dam is normally built out from the sixth up to and inclusive the tenth day after full moon.
Normally the soundings, made daily, show that the bottom in the centre of the creek will raise with the progress of the closure. This effect is clearly due to horizontal and vertical movement of the completed compartments. It is also illustrated by the need to restore regularly the level of R.L. + 2.7 m of the completed parts of the primary dam. Once during the construction of the primary dam the depth of the centre part (= final gap) decreased from 17 to 8 m. It would appear that one may consider this as the geotechnical phenomenon 'squeeze'. This phenomenon has been described very well in d'Angremond & Van Roode (2001, 86 et seq.).
It was admitted that closure becomes a real problem if sliding does not take place or, when the scour of the soft clay exceeds the raise due to the squeezing effect.
It was agreed by BWDB Officers that "(the soft) clay has definitely scouring resistance but practically no bearing capacity".

14.2.4 Closure of the last gap

For the final closure all materials must be prepared in advance and stored near the gap. The gap will only be closed over a width (perpendicular to axis of closure) of 12 to 15 m.
Closure will take place within 5-6 hours of a neap tide and starts before the water level is at its low water slack. A number of 4 000 to 5 000 of labourers is required to dump 200-300 mata's into the gap within a very short period and to close remaining holes with brushwood and straw. If closure fails, a second attempt may be made but this is not possible if deep scouring holes have formed on both sides of the dam in the centre line of the main current.
BWDB reported that it has once used a floating mattress of 10.5 m width (gap was 12 m long) built up of golpata, brushwood, straw, nara[9]. The golpata must be dry (not green). The thickness of the mattress was 1.5 to 1.8 m. The floating mattress was made on the river side of the closure. It was guided by four gangs of 100 labourers, each gang pulling a bamboo rope (Fig. 14.5). It was floated into position just before the turn of the tide (low water). Then sixty mata's were placed on top. Original depth prior to final closure was 11 m. After placing mattress and mata's the depth was 6 m. The sixty mata's were pinned down with bamboo. Then again mata's and brushwood were placed. The whole operation was carried out during one tide (seven hours).

14.2.5 Final profile of cross-dam

For the final profile of the dam soft clay is placed by hand including double handling using country boats. Dumping under water is necessarily without compaction and in this respect the under-water slopes 1 in 4 (river side) and 1 in 5 are considered to be a good measure against slips.
Clay is normally borrowed from the banks at the riverside of the closure. The scour holes near the centre of the dam should be filled up with gunny (= jute) bags filled with sand or clay to their original depth but this is not always done.
In general it was stated that "sliding on the river side does not matter as the river will silt up anyhow". Still, if slips occur, the stability of the whole embankment is in jeopardy.

14.2.6 Contractor's organization

Mr. M. Abul Hashem Biswas, one of the most successful closure contractors, was interviewed in order to learn something about the organization of contractors carrying out closure works.
Mr. Hashem Biswas, at the time of writing (1976), is an independent contractor since 1958 and belongs to the A-class contractors since 1965. He constructs buildings, earthworks, sluices, cross-dams and regulators. His permanent staff amounts to 200: agents, headmen, clerical staff, skilled labourers.

[9] Roots of rice plants.

Not less than 12 closures were completed by him of which eleven were immediately successful and one was closed after a second attempt. When closure works continue during the period of small tidal ranges (6h to 11th day after full moon) 4 000 labourers work during 7 days a week. Preparation of mata's takes place by three gangs each working 8 hours. Earth bearers work from 8 a.m. to 2 p.m. and from 4 p.m. to 10 p.m.
In his opinion a Lump Sum forms the best approach to closure works and is more in the interest of the work than an itemized contract. As the Contractor in that case is fully responsible he will make sure enough quantities of materials are ordered. For instance, the Engineer has given so-called ''provisional quantities" and the Contractor orders 50% more materials for mata's.
Mr. Biswas suggested to pay 50% of the Contract Sum after closure (instead of 33%) and the remainder when reaching final profile as per final design.

14.3 Gangrail closure, case history of the third, unsuccessful, attempt (project no. 70)

14.3.1 History

The tidal creek, having a width of approximately 120 m, and an original depth of 10 m below R.L., was first closed with a primary dam on 23 March 1971.
The primary dam however, was not maintained due to the critical political situation in Bangladesh, which caused the labour force to leave the site at a critical stage of construction. Subsequently, the dam was overtopped and washed away.
A second attempt was made, during the financial year 1974-75. Work started (too) late in the season. This fact and a large slip taking place at the west bank, which concerned the greater part of the dam, made this attempt unsuccessful and most of the work was lost. Subsequently, the Contractor abandoned site.
A third attempt was made, starting February 1977, which is described below.

14.3.2 Description of the site

The Gangrail closure site is located in south-west Bangladesh, polder 17/2, Khulna District, a 100 km from the sea. The tidal range during spring tide is 2.7 to 3.0 m and 2.1 m during neap tides.

14.3.3 Soil conditions

Visual observations of the banks of the creek showed a mixture of soft grey silty clay, with traces of peat at the west bank of the river at the closure site and a soft greyish clay at the east bank. At Low tide the banks look the same over great distances upstream and downstream of the closure site. Apparently, no borings or soil penetration tests had been made in the bed of the creek or along the banks.

14.3.4 Closure works in the year 1977

Works started early February and proceeded such, that the platforms were ready by 15 February. In Fig. 14.1 the results are shown of the surveys in the primary dam axis made on 7 Jan. and 15 Febr. As can be observed in Fig. 14.7, the construction of the primary dam started on top of a ridge of remaining material of the 1975 attempt, the bed of the creek outside the ridge being 10 m below R.L. Platforms were constructed, starting on both banks, by using clay, brushwood and straw as construction material. Construction level was R.L. + 3.35 m.
Then the bullah piles were driven for the first compartments. The size of these first compartments was 43 m by 6 m. Bullah piles in the front face of the compartment were placed at 4.5 to 6 m distance from each other. The top of the piles was at the level of Low tide in order to facilitate passing of the mata's over the top.

Compartments were subsequently filled with mata's up to MSL (= R.L. – 0.45 m), and on top of the mata's, clay, mixed with straw, was put. The side and the front face were protected with brushwood. The top level of the primary dam was maintained at R.L. + 3 m.
Between the bullah piles at the front and also at the sides of the compartments, a framework of bamboo was constructed. The vertical bamboo stakes were forced into the bottom by hand labour. The horizontals of the frame were tied to the verticals using jute ropes. The squares of the framework had sides of about 60 cm. When completed the whole compartment looked like a box of bamboo, at least above Low tide.
In addition to the framework, ganji-piling was driven at the sides of the compartments. Ganji-piling consists of long bamboos forced into the ground at an angle and two by two forming a cross. See photograph no. 5 in Fig. 14.6. The distance of the crosses is 15 cm. To the cross one or two bamboos are tied horizontally, thus connecting all the verticals together with jute ropes.
This piling is meant to reduce slipping of the mata's in a sideways direction. This side protection has been shown on Photograph no. 6 of Fig. 14.6.

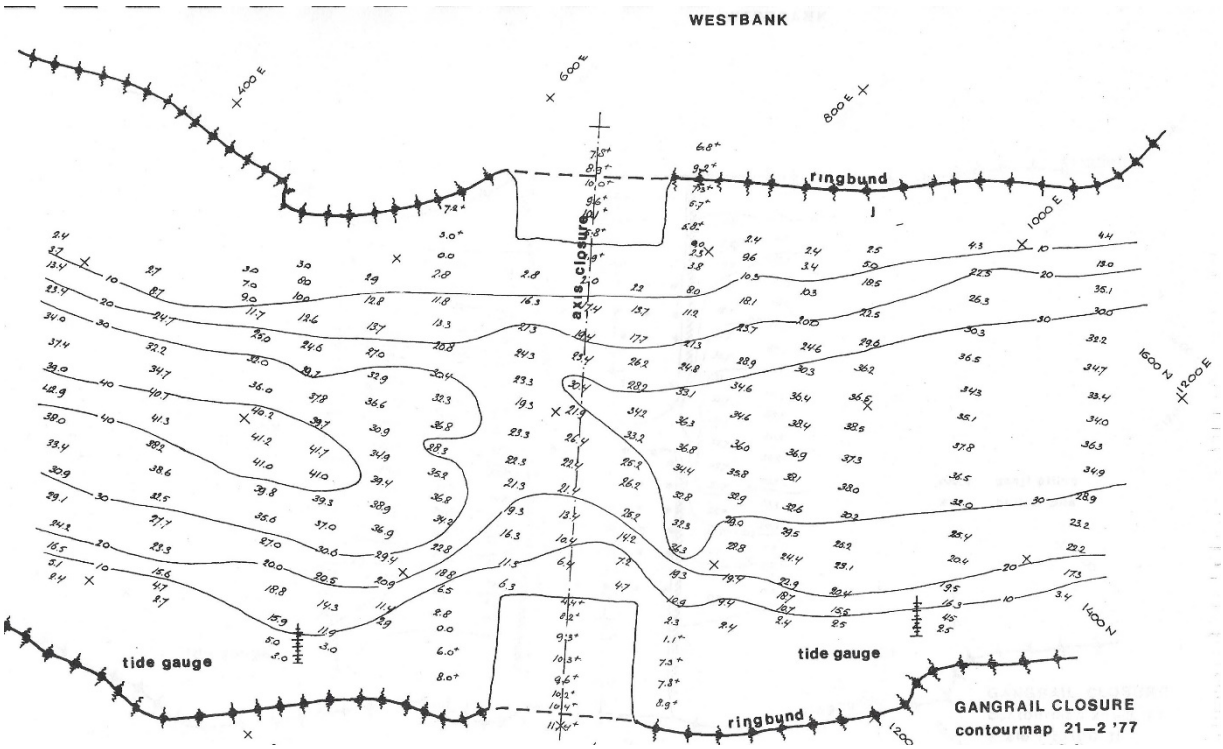


Fig. 14.7 Gangrail closure contour map 21 Febr. 1977; depth in feet below R.L.; scale (appr.) 1: 2200; squares of the grid measure 200' by 200'; source: BWDB (1977)

After one compartment had been filled to MSL, during the next tide the following compartment is first filled to the level of Low tide, prior to the first compartment being raised to the full level. By thus working in steps, slips are avoided as the bottom is only gradually loaded.
The mata's were being pinned to the lower level mata's by means of long bamboo crosses (ganji-piling) at distances of approximately 60 cm. When comparing the length of the available bamboo with the depth to cover, it can be noted that the mata's in most cases could only be pinned to old bottom material (mata's), the bamboos being too short to possibly reach the original riverbed.

The river had its deepest section at the west bank as can be seen in Fig. 14.1. Here, at almost regular intervals, slips could be observed. At each large slip (settlement) the top level of the dam went down by approximately 1.5 m. Slips took place mainly during Low tide. The first slip took place,

when dam construction had proceeded some 20 m into the creek. The bulk of the material stayed in the compartment and was not lost. These slips were not stopped by the bullah piles, but the piles went down with the slip, thus indicating that the movement was situated below the point of the piles. As a direct effect of these slips, a raise of the bottom could be observed at the toe of the dumped material.
At the east bank, the behaviour was quite different. Slides did not take place with the same frequency and were also much smaller in size than at the west bank. The bottom of the gap however, likely due to a combined effect of the slips and the loading charge from the constructed dam sections, was rising gradually from R.L. -7.5 to – 3.0 m in the central part of the creek. As this material was not the original riverbed, but the remainders of the 1975 closure attempt consisting of mata's, hardly any scour could be noticed. Not even when current velocities went up as high as 2.5 to 3 m/sec. Also very little or no scour could be observed at a distance of 30 m upstream and downstream of the dam axis. Please refer to Figures 14.7 and 14.8.
It was observed, however, that the west bank section of the primary dam in fact never stopped slipping, and that there was a tendency to slip over a longer distance at the north side of the dam, than at the south side.

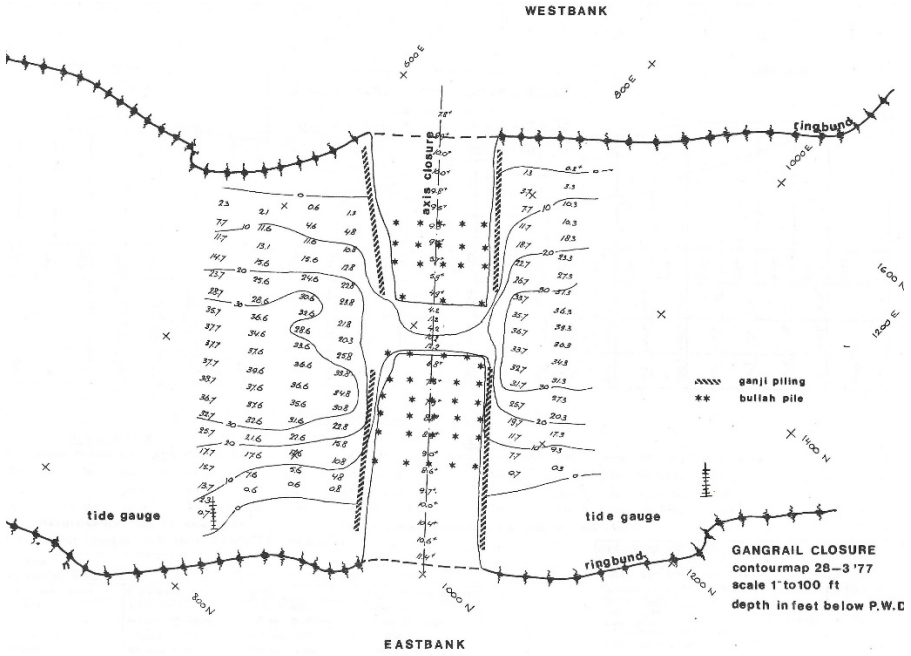


Fig. 14.8 Gangrail closure contour map, situation on 28 March 1977; scale (appr.) 1: 2800; squares of the grid measure 200' by 200'; source: BWDB (1977)

The full length of the mata's being 40 m, Contractor started dumping several shorter mata's, only 15 m long, to match the lines of dumped mata's. This is very important for a fast placing of mata's. The material slipping at the north side of the dam could not be traced in the surveyed cross-sections and must be considered lost. This was however a very small part of the material placed.
As soon as the continuous slipping took place at the west bank, Contractor did not construct compartments any more here, but just drove a bullah pile here and there trying to control the slips and avoid loss of material. The length of the longest available bullah piles being 14 m, he could not manage to reach the original riverbed, but could only pin the old and the newly placed mata's together.
Construction continued at the east bank by making compartments, size 40 m by 3 m, and subsequently filling these with mata's which were covered up to R.L. + 3 m with clay etc.

At the west bank it was not an easy job to reach the required level, as the placed mata's as well as the other construction materials kept slipping down and forward. Considerable time was lost in maintaining the top level of the dams.
As the ends of the dams were approaching each other, the level of the bed of the creek at the west side came up rapidly, creating a gap with a shallow part of approximately R.L. – 1 m at the west side, and a deeper section at the east side of approximately R.L. – 3 m. The progress of the Works is shown in Fig.14.9.

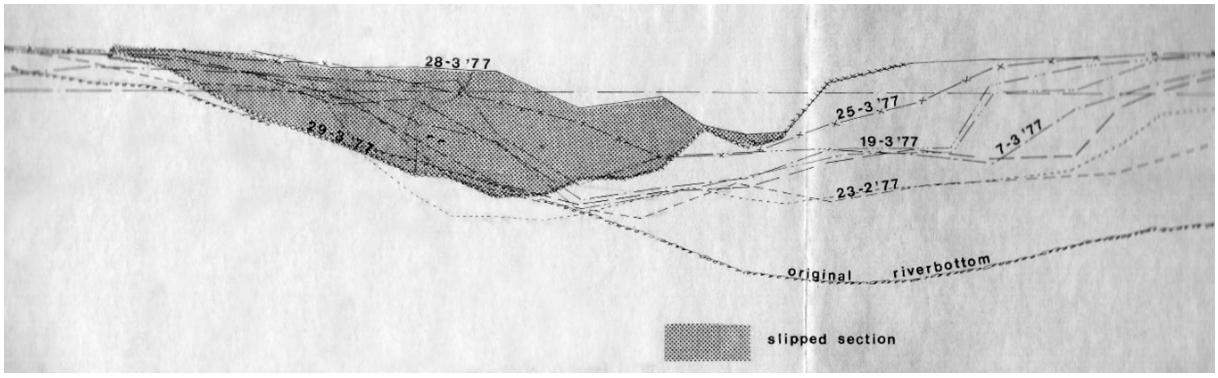


Fig.14.9 Longitudinal profile over closure bund with lines showing progress of works between 23 Feb. And 29 March 1977; the slipped section at the west side has also been shown; source: BWDB (1977)

On 25 March the Contractor installed on both banks a small diesel generator and a lighting system, from the borrow area for clay to the gap. On the same date the Contractor started working during two tides. The labour force, however, was not increased to the extent that work could continue with two different shifts. The final date for closure was now set at 29 March.

Current velocities in the gap increased rapidly and no slack in the flow of a workable length of time during the turn of the tide was available any more. While before, mata's had mainly been dumped, starting at Low tide, during one or two hours of the flood period, now mata dumping already started during the ebb period, as soon as velocities dropped below 0.6 m/sec. Pinning down of the mata's had to be done immediately after dumping to avoid loss in this swiftly flowing water.

On 28 March the gap at R.L. had a length of only 18 m, of which 9 m was not deeper than 1,2 m and the remaining 9 m was 3 m deep (Fig. 14.9).
Now mata dumping at the east bank was stopped prior to Low tide to avoid making the final closure gap too narrow. However, during the rising tide, with a head difference of 75 to 90 cm, the west bank section of the dam settled quickly over a length of 27 m and slipped further into the river at the north side of the dam, covering the bed of the creek over a distance of 30 to 60 m from the dam. The whole slip did not take more than 15 minutes. The depth in the closure, as measured 18 hours after the failure, was R.L. – 8.5 m (Fig. 14.10).
Visual observation of the bamboo sticking out above the water level and comparison of the survey with previous surveys, gives the impression that most of the material on the bed of the creek still consists of a packet of mata's held together by the bamboo pins.
Following this mishap, Contractor abandoned site in the morning of 29 March, having tried to close this creek three times without success, and not being able to claim any payment for his efforts.

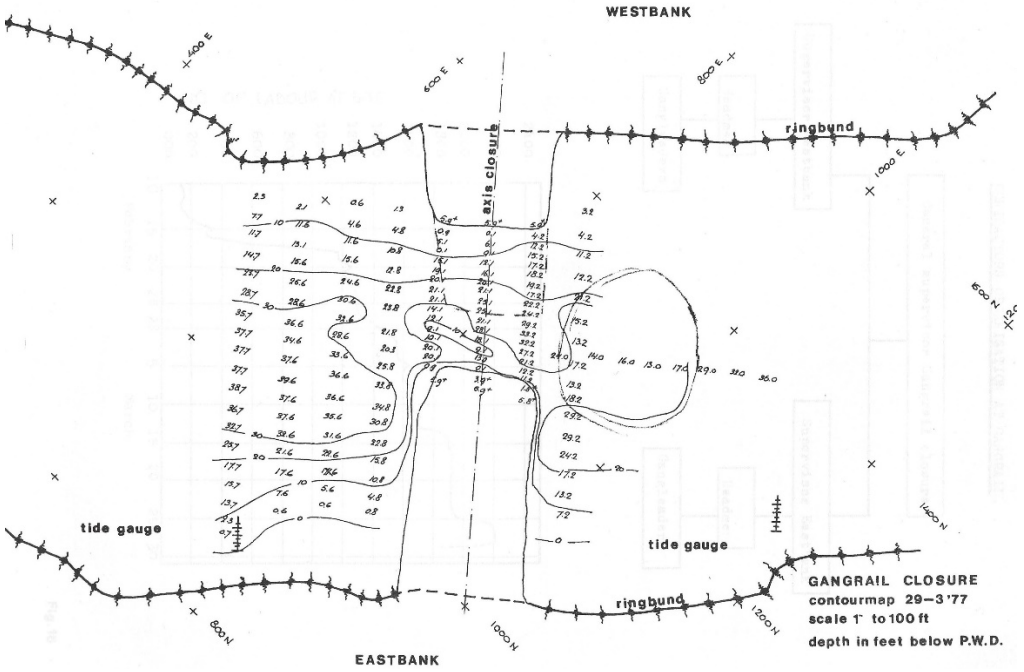


Fig. 14.10 Gangrail closure contour map, situation on 29 March 1977 after major slip in closure bund near west bank; scale (appr.) 1: 2500; squares of the grid measure 200' by 200'; source: BWDB (1977)

Fig. 14.11 Gangrail closure, different stages of collapse; source: BWDB (1977)

14.3.5 Cause of the failure

After having observed in the field the main part of the construction, it is thought that the main reason for the failure must be sought in unfavourable sub-soil conditions. A fact is also, that the bullah piles, meant to pin the primary dam to the bottom of the creek, over the greater part of

the creek could not even reach the bottom as the lengths required (18 to 21 m) were not available on the market. According to information obtained from a forest engineer these lengths are also hard to find in the Sunderbans. Moreover, the bullah were not bolted together as had been a departure point for the design.
Finally, the labour force was kept to an absolute minimum and so was the availability of the construction materials in stock.
All these shortcomings, however, cannot be the reason for a sudden large landslip. Even when the bullah piles would have been long enough to reach the bottom of the river, they would not have been able to stop a slip of this size as the forces are too large.

14.4 Review of the failed closure works at Gangrail

14.4.1 General

For the closure of the Gangrail creek in Bangladesh, the closure method as described in Section 14.2 was used.
The fundamentals of this method are to create, by means of vertical and horizontal narrowing of the river, a final gap that can be closed during one neap tide. The dimensions of the gap have to be based on the materials that can be produced and handled in this short period. Important is also, that construction must stay ahead of the rising tide. Most favourable is therefore a shallow gap.

14.4.2 Reduction in width of creek

Reduction in width of the creek was realized by constructing from both banks, using mata's, a primary dam until the desired length for the final gap was reached.
Mata's are of an excellent help to build fast (large volume in a short period of time), provided they are handled with care.
The mata also offers the possibility to construct a rather lightweight dam structure, which can be built on a soil having unfavourable bearing capacities, without large settlements due to the weight of the dam while so also regular progress is possible.
Furthermore, the mata is quite resistant against attack by the current. At Gangrail, current velocities up to 2.7 m/s caused hardly any noticeable damage.
When the subsoil has such characteristics that no sudden slips take place, the construction of a primary dam such as at Gangrail, does not have to create problems provided the depth of the river does not exceed the length of the available piles.
The question arises, however, whether such a dam which mainly consists of straw, golpata and very little clay, will not lose its stability as a result of soaking when being under a hydraulic head for a longer period of time.

14.4.3 Reduction in depth of creek

Reduction in depth was obtained by the uplift of the subsoil. It is assumed that, as a result of the increasing load on the bed of the creek during construction, the subsoil in front of the head of the dam will rise. During construction, the bed material is pushed forward, which results in shoaling of the gap. This is one of the uncertain factors encountered in the Bangladesh' method. By not having soil investigations carried out in advance, it is not certain how the subsoil will behave. Also it is not known to what extent shallowing will take place in the final gap.
As a consequence thereof the actual current velocities in the final gap cannot be predicted and nothing can be planned to control a possible erosion which will take the bed material away.
Unexpected uncontrolled settlements and slips can cause delay in the progress of construction of the primary dam (as was experienced at Gangrail, west bank).
In particular at an advanced stage of the closure works, when high current velocities occur, this may cause scour of the bed of the creek (also because of increased turbulence currents due to bullah piles

in the gap). But at Gangrail, no erosion was observed in the final closure gap. Moreover, upstream and downstream, the creek was deep enough to allow the current to spread vertically, reducing possibilities for erosion.

14.4.4 Mata placing

During the construction of Gangrail closure works, it was observed that the mata's are being placed fastest when they are being rolled directly down the slope to their final location. As soon as they have to be transported horizontally or over an uneven slope, the whole operation becomes time consuming and the output in a given time period decreases tremendously. Speeding up production by increasing the labour force helps only to a certain extent, due to the cramped working space and the people getting in each other's way.
The production of mata's is limited, and therefore the acceptable size of the final gap. The length of the final gap is under reasonable control. The depth, however, is an unknown factor. If the final gap becomes too deep, it will be difficult to close and other measures have to be considered such as filling it in with gunny bags. This is a time consuming operation and it can create scour holes upstream and downstream of the dam while the stability of the latter may be endangered. Notwithstanding the execution of soil investigations, the prediction as to what extent shallowing will take place, will be very difficult while the desired progress stays unknown.

14.5 A few design considerations for a successful closure of Gangrail

14.5.1 Planning

In order to make a closure successful it is necessary to take away in advance as many uncertainties as possible and thus reduce the risk of failure.
By carrying out soil investigations one may be able to predict whether the bed of the creek at the selected site has sufficient bearing capacities and is of such a nature that no irregular settlements will take place.
From hydraulic computations it can become known what the depth and the length of the final gap should be, and whether the creek can be closed at all by using the available construction materials. After carrying out these investigations, a detailed plan for closure can be made, in which for forthcoming difficulties a solution can be sought. In this manner the possibility of occurrence of unexpected events during construction can be reduced which in turn decreases the risk.
One of the major conditions to arrive at this situation will be the stability of the bed of the creek which can be increased by applying a bed protection.

14.5.2 Bed protection

Though bed protection measures have already been discussed in detail in other chapters[10], it is worthwhile to recall the functions of a bed protection:

- to prevent scour of the underlying bed of the creek or river at the site of closure as well as immediately upstream and downstream of it;
- to prevent slip circles from going through the bed protection;
- to provide a good foundation on a bad subsoil;
- to prevent, or at least hamper, horizontal movement of the subsoil.

Obviously, such bed protection would take away a number of risks which are inherent to the mata closure method as described above.

[10] See Chapter 4, Section 4.4 and Chapter 11, Section 11.3.

14.6 General considerations for a successful application of the mata method

In the foregoing sections the emphasis has been on a description of the mata method in general and on the failure to close Gangrail by applying this method in particular. In this respect it is regretted that Van Duivendijk and his colleagues have not been able to attend a successful mata closure. However, based on the observations during the failed mata closures at Gangrail (BWDB, 1977) and Madargong (BWDB, 1978) and some hydraulic calculations it is possible to formulate the conditions[11] which are required to apply the mata method successfully:

- prior to the tendering of the closure, investigations should be carried out regarding the bathymetry, geotechnical conditions in the area of closure and the tidal movement;
- the tidal volume during closure should not exceed 2 x 10^6 m^3;
- the bed of the creek to be closed should be not deeper than 10 m in the axis of closure, unless the bed rises during closure works due to squeeze;
- the bed of the creek should have a reasonable resistance against erosion by currents of, say, 3 to 4 m/s;
- the contractor should be able and willing to mobilize sufficient funds for carrying out the works;
- the contractor should play safe by investing adequate amounts of money in vital parts of the closure construction like compartments and in stocks of materials;
- the contract between BWDB and the Contractor should not be on a 'no cure no pay basis', as the latter results in the Contractor taking too many risks.

Because this is a horizontal closure the maximum current velocities will increase as the remaining opening gets smaller. The length of the final closure gap is determined by the maximum allowable current velocity[12] which in turn determines the maximum allowable tidal volume. But, in order that the speedy closure can be made within the five to six hours available in a neap tide, this length should not be more than 9 to 15 m.

[11] These conditions are a modified version of the conditions presented in BWDB (1982a).

[12] The expression 'maximum allowable' is a function of, on the one hand, the resistance of the bed of the creek against rapid scour and, on the other, of the need to place and pin down mata's in the closure gap without too much difficulty and mishaps.

CHAPTER 15 The Nalian closure in Bangladesh in 2020[1] (project no. 71)

15.1 Introduction

During the sixties of the 20th century the Nalian River was closed at its mouth, i.e. where it flowed into the Shibsa River. That closure was part of the first phase of the Coastal Embankment Project (CEP). The Nalian basin thus became a fresh water area and comprised a large part of polder-32 in Khulna District in south-east Bangladesh (Figures 15.1 and 15.2).

Drainage of the basin took place through the drainage sluice in this cross dam, comprising 8 nos. 0.9 m diameter concrete pipes. This sluice was built in 1972/1973.

In the period 1986 – 2015 the river Shibsa shifted some 100 m to the east, seriously threatening the flood embankment along that river. Finally, during cyclone Aila in May 2009, the Nalian cross dam, being part of that flood embankment, was breached and, from that date onward, during high tides, the Nalian basin was flooded by saline water.

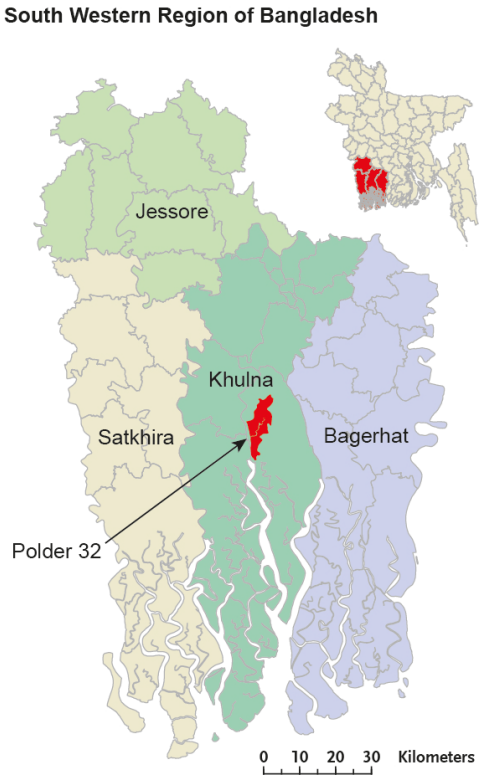


Fig. 15.1 Location of Khulna District; source of map: WARPO (2019)

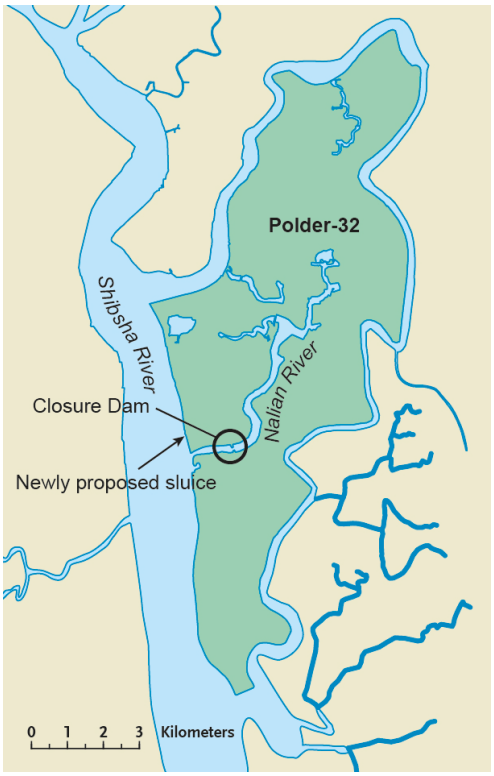


Fig. 15.2 Polder-32 in Khulna District and Nalian River; the site of the new Nalian closure dam has been clearly marked

[1] Most of the information in this chapter has been derived from the 'Report on Nalian Closure CEIP Phase 1 in Polder-32 (draft), May 2021, submitted by RHDHV & Partners to BWDB and received by the author with permission of BWBD to be used for this research. This document is herein after referred to as the draft Report. Useful information was also found in the Feasibility Report CEIP-1 (2012), BWDB (2013), Khan et al (2019) and Hale et al (2019).

In order to prevent this saline water intrusion BWDB[2] constructed low provisional flood embankments along both banks of the Nalian River. But this in turn created a drainage problem as no sluices were built in these embankments.
Subsequently, the studies for reconstruction of the Nalian cross dam were taken up as part of the Coastal Embankment Improvement Project, Phase-1 (CEIP-1). The works for rehabilitation of polder-32 included reconstruction of this cross dam at a new location and were going to be implemented under Contract Package-1 under CEIP-1 by a Chinese contractor, the First Engineering Bureau of Henan Water Conservancy (hereinafter called CHWE). Royal HaskoningDHV (RHDHV) & Partners were appointed as the Engineer for the said FIDIC based Contract.
In the Contract, construction of Nalian Closure is a lump sum item and thus CHWE had to design and construct the new Nalian cross dam and carry out the necessary site investigations. Also a bypass channel, to connect the Nalian River to the Shibsa River, and a drainage sluice DS-10, had to be designed and constructed. This would solve drainage problems in the basin.
The Contractor had to obtain endorsement for his design and method statement and underlying studies for construction of aforementioned works from the Engineer and BWDB well in advance of the date of closure.

15.2 Physical site conditions

15.2.1 Hydraulic conditions

The area of the tidal basin behind the new Nalian cross dam is 480 ha. The MSL value is not given in the draft Report[3] but it is stated to be at PWD + 0.46 m. PWD is the Datum in Bangladesh. The tidal ranges during the dry season have been derived from Hale et al (2019, 236): 2.3 m during neaps to 5.6 m during springs. However, the last figure would appear to be too high. A tidal range during springs of 4 m during the dry season seems to be more realistic. In the draft Report a tidal volume of around 7 million m^3 is mentioned which seems to be a realistic figure for a neap tide.

15.2.2 Geotechnical conditions

The length of the cross dam between the provisional flood embankments is about 500 m of which approximately 250 m is in deeper water. The deepest bed level is at PWD - 6 m. Around 2013 this level was at PWD – 15 m. Meanwhile, because of the construction of the aforementioned flood embankments and other upstream closure works, the tidal flow as well as the tidal prism decreased and the deep river channel silted up over a height of up to 10 metres.
In October 2017, the Contractor carried out geotechnical investigations in the selected alignment for the closure dam. In the said deposits, comprising very soft clayey silts, SPT's[4] had values of 1 to 2 in the upper layers but below PWD – 7.5 m the SPT values found (15 – 30) were considered satisfactory.
In view of the soft deposits, the following geotechnical risks, during construction as well as during the period after completion of the cross dam were identified:

- squeezing effect during construction due to lack of bearing capacity;
- slope stability during construction and after completion of the final profile of the dam;
- differential settlements after completion of the dam.

The Contractor, after considering other alternatives (like dredging or shifting the alignment of the cross dam to a more upstream location), decided to place a thick and wide blanket of sand-filled geo bags on the soft layers to establish a stable foundation for the cross dam (Fig. 15.3).

[2] Bangladesh Water Development Board.
[3] i.e. RHDHV & Partners (2021).
[4] SPT stands for Standard Penetration Test.

The draft Report stipulates that:

> "the effect is that the very soft top layer is displaced and "washed-out" … (due to) the impact of the dumped bags and that the remaining soft layers are consolidating under the weight of the geo bag blanket thus improving the bearing capacity of the subsoil."

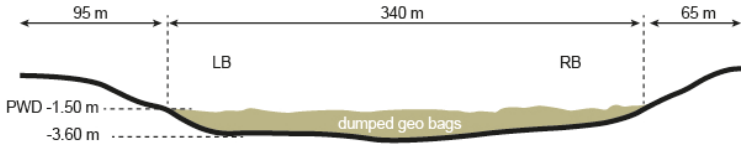


Fig. 15.3 Survey August 2019 of top levels of dumped geo bags in alignment of closure; Source: RHDHV & Partners (2021, 6/22)

15.3 Design of closure works

15.3.1 Hydraulic aspects

The basic principle for the design of the closure is outlined in the draft Report as follows:

> (Given the) "substantial large length of the closure of 500 m and a tidal volume of around 7.00 million m^3 in the Nalian river, the combined method (vertical and horizontal) was considered appropriate to close the channel. Prior to closing the channel, Drainage Sluice (DS#10) under CEIP-1 Project and the associated linking diversion channel were made operational to divert flow[5] from Nalian River basin."

The vertical part of the closure would comprise the dumping of geo bags which would, as stated above, establish a stable foundation for the cross dam as well as a horizontal sill for the final closure section.

Aspects to be taken into account for the hydraulic design of the closure works were:

- the progressive increase of current velocities due to the decrease of the wet profile (because of horizontal narrowing) in relation to the stability of the geo bags in the sill;
- the scour near the edges of the sill during the closure operations.

From the draft Report (figures 5.2 and 7.5) it can be concluded that the initial narrowing of the River would take place from both banks over lengths of 270 and 155 m respectively, which would leave a final closure gap of 75 m (at sill level of PWD -1.5 m). See Fig. 15.4 and 15.7 below.

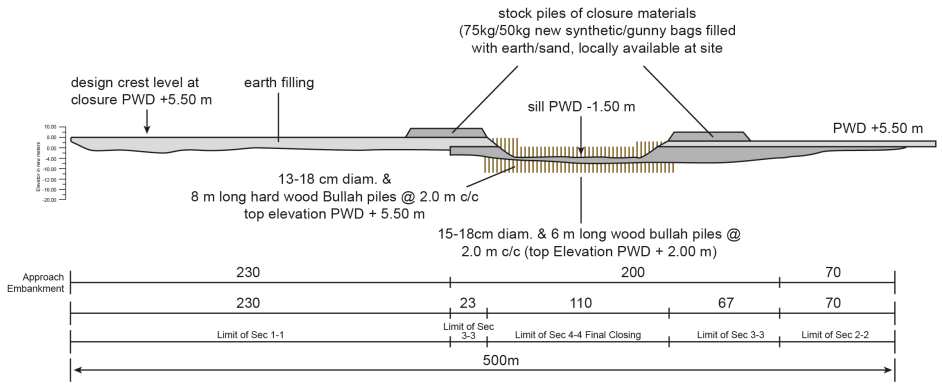


Fig. 15.4 Designed longitudinal cross-section of the cross dam; the length of the closure gap is given as 110 m. This is at the crest level of the dam (PWD + 5.50 m); given the slopes at the dike heads of 1: 2.5, the length of the closure gap at sill level (PWD – 1.5 m) is 75 m; source: RHDHV & Partners (2021, 9/22)

[5] Even under ideal circumstances (wet profile over full cross-section of openings, no loss of head in drainage channel) the share of the drainage sluice in the tidal flow into or from the basin would be no more than 3 to 4 %.

Hydraulic modelling studies were conducted but not included in the draft Report. To generate insight into the flow conditions, some tidal calculations are therefore given in Box 15.1. During a day of a neap tide (tidal range of 2.3 m), max. current velocity in the final closure gap will be 3.4 to 4.3 m/s, or, depending on the area of the tidal basin, smaller. During a springtide (tidal range assumed to be 4 m), having the sill at PWD – 1.5 m, the max. current velocity will be in the order of 4,9 m/s. The apparent stability of the geo bags in the current is also shortly discussed in Box 15.1.

BOX 15.1 Calculations of tidal flow versus stability of geo bags at Nalian closure[6]

1 Introduction

Below, an effort has been made to calculate the current velocities on the sill of the Nalian closure for tidal flow and to estimate the stability of 250 kg bags in this tidal flow.

By using the Coastal and River Engineering Support System CRESS some calculations can be made regarding the maximum current velocity in the cross-section of the closure for various situations.

2 River channel 500 m wide, sill at PWD – 1.5 m, A = 4.8 km^2, B = 500 m

2.1 Neap tide

In this case Hz = 2.3 m, T = 12.5 hours, h = 1.96 m, because MSL = PWD + 0.46 m.

It follows: Δh_{max} = 0.03 m, H_B = 2.3 m, U_{max} = 0,72 m/s.

2.2 Springtide

In this case Hz = 4.0 m. Given the sill at PWD – 1.5 m, the tidal range for the basin will be from PWD – 1.54 m to PWD + 2.46 m.

Take Hz = 4 m and T = 12.5 h; h = 1.96 m but calculation is only possible if h = 2.01 m (which value has been taken).

One finds: Δh_{max} = 0.10 m, H_B = 3.99 m, U_{max} = 1.28 m/s.

3.1 Flow gap varies in width (during closure operation); sill at PWD – 1.5 m, A = 4.8 km^2

Neap tide

The initial length B of the closure gap is 83 m at MSL; B decreases gradually during the closure operation.

Input for all situations: Hz = 2.3 m, h = 1.96 m, T = 12.5 hours.

The output for different values of B is given in Table 15.1:

Table 15.1 Value of output parameters for different values of length B of closure gap

B (m)	Δh_{max} (m)	H_B (m)	U_{max} (m/s)
83	0.72	1.79	3.39
50	0.97	1.24	3.92
10	1.14	0.27	4.26
5	1.15	0.14	4.27

Note: The area of the tidal basin during neap tide is possibly smaller than during springtide (4.8 km^2). In that case maximum current velocities will also be smaller. For instance if A = 3.5 km^2 and B = 83 m, the value in the Table of U_{max} changes into 2.85 m/s.

3.2 Springtide

The input values are as given in Sub-section 2.2 above except that now B = 83 m.

The output: Δh_{max} = 1.50 m, H_B = 2.64 m, U_{max} = 4.89 m/s.

4 Stability of geo bags in the current

Oberhagemann & Hossain (2010, 12)[7] have summarized the state of knowledge as at 2009 regarding the stability of geo bags in currents as follows:

- Geotextile bags of 126 kg filled weight are stable against near bed velocities of up to 2.9 and 2.6 m/s on slopes of 1V:2H and 1V:1.5H respectively. This corresponds to depth-averaged velocities of more than 3 m/s.
- Geotextile bags provide high stability against currents. The main reason is the substantial difference in shape: flexible geotextile bags lying flat on the bank show high resistance to flow forces. Also, bags are flexible and over time form a smooth layer, so that they do not project into the flow and generate local turbulence.
- So far there are no generally used formulae for calculating the resistance of geotextile bags to flow forces, as exist for riprap. Existing formulae require adaptation to reflect the specific characteristics of geotextile bags. Neill et al.[8] (2008), Stevens (2006), and Pilarczyk (2000)[9] make some suggestions. For present purposes, 126 kg geotextile bags are considered to be stable as revetments for a depth-averaged flow velocity of 3 m/s, based on studies and investigations mentioned earlier.

If the criterion of Isbash is applied on a 126 kg bag one finds a critical velocity of 2.7 m/s. Apparently, (see above) the acceptable depth averaged velocities in reality are some 15 % higher. From the above one can provisionally conclude that 250 kg geo bags, dumped to form a horizontal sill and having a critical velocity (Isbash) of 3.0 m/s will stay stable in currents of up to 3.5 m/s. During springtides a number of bags will be removed from the sill by the currents. No information has been traced about the (lack of?) friction between layers of geo bags.

[6] Calculations made by Van Duivendijk within the context of the dissertation; these calculations are not part of the draft Report.

[7] Oberhagemann, K., Hossain, M.M., Geotextile bag revetments for large rivers in Bangladesh, Geotextiles and Geomembranes (2010), doi:10.1016/j.geotexmem.2010.12.003

[8] Neill, Charles; Mannerström, Monica; Azad, Abu Kalam (2008); Model Tests on Geo bags for Erosion Protection. In: Sekiguchi, Hideo (Hg.): Proceedings 4th International Conference on Scour and Erosion (ICSE-4). November 5-7, 2008, Tokyo, Japan. Tokyo: The Japanese Geotechnical Society. S. 404-411.

[9] Pilarczyk, K.W. (2000); Geosynthetics and Geosystems in Hydraulic and Coastal Engineering; Rotterdam/Brookfield , A.A. Balkema.

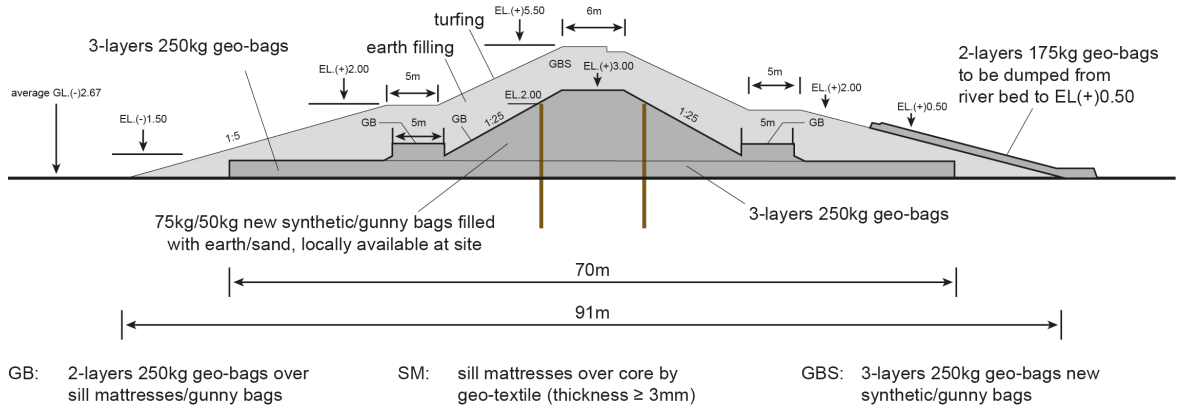


Fig. 15.5 Cross-section over sill, closure bund and final dam profile as initially designed; cross-section of closure bund was later changed; source: RHDHV & Partners (2021, 10/22)

15.3.2 Structural aspects of the sill

The dumping of geo bags would be done over a length of 340 m and a maximum width (i.e. perpendicular to the dam axis) of 70 m (Fig.15.5). The layer thickness of the geo bags would be 2 – 5 m. First, three layers of 250 kg geo bags were dumped over a width of 70 m. Subsequently, 50 kg synthetic bags would be dumped up to a level of PWD – 1.90 m and this had to be done carefully (especially at the location of the closure gap) in order to arrive at a more or less horizontal sill. The width of the crest of dumped 50 kg bags at that level would be approximately 36 m. Finally, a geotextile filter fabric would be placed over the full width of this crest. The geotextile filter fabric was subsequently to be ballasted by one layer of 250 kg geo bags in the centre of the sill and three layers at the edges. As measured in Fig. 15.5, this left a sill at PWD – 1.5 m having a width of some 28 m. Theoretically, this implies a volume of some 70 000 m^3 of geo and synthetic bags in the sill . The Contractor reported the following about the number of bags incorporated in the project:

- synthetic bags 75/50 kg 1 054 360 nos.
- geo-bags 175 kg 24 226 nos.
- geo-bags 250 kg 49 244 nos.

15.3.3 Structural aspects of the closure bund

The cross-section of the closure bund as well as the final dam profile are shown in Fig. 15.5. Apparently, this concerns an initial design which, later on, has been adapted.
The Bullah piles as shown in Fig. 15.5 seem to have the correct length of 11.7 m. According to the text of the draft Report the distance centre to centre of the piles is 1.0 m.
From what has been shown in the Figure one can conclude that the initial closure bund would have a width between the two rows of piles of 8 m and a height of 4.5 m (given a sill at PWD -1.5 m and a crest at PWD + 3.0 m). The 50 kg synthetic bags inside the two rows of piles would be dumped by manual labour while the bags outside the piles would be dumped from barges. Probably, it was the intention to carry out these operations simultaneously. Below, it will be seen that the actual closure works differed from what is shown in Fig. 15.5 and elaborated upon above.

15.4 Construction programme

In Fig. 15.6 the construction schedule, as submitted by the Contractor and copied in the draft Report, is shown.

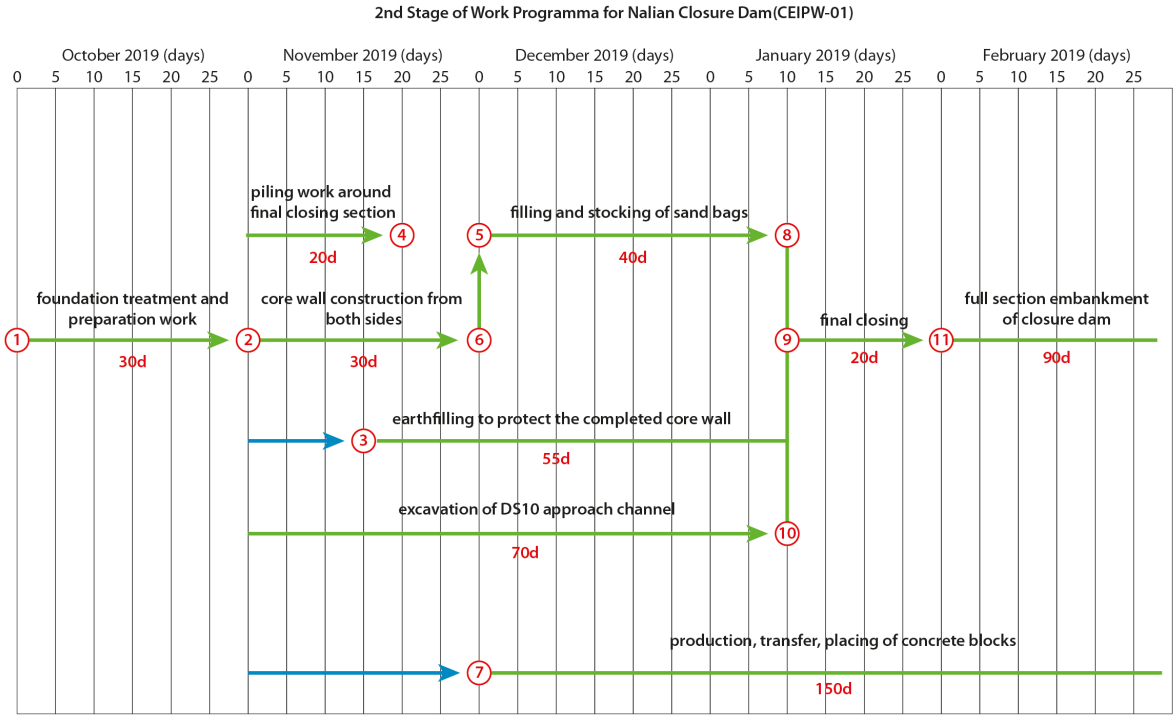


Fig. 15.6 Contractor's construction schedule; source: CHWE

This was the original plan prior to construction and there have been some deviations. The most realistic chain of events is described in the text here in this chapter.

According to Sub-section 4.2.2 of the draft Report the dumping of synthetic bags and geo bags started in October 2018 and continued (including intermediate additional dumping of bags as part of repair works) until the end of November 2019. This follows also from Table 4.1 in the draft Report where foundation works are scheduled for the period Sep. 2018 – Nov. 2019. This is understandable as consolidation of the soft layers, in so far as not 'washed-out', will take time and does not happen overnight.

The closure of the last gap (long 75 m) was, according to the schedule, planned for the last week of January 2020 but was shifted to 17 February 2020 as repairs of the sill as well as mobilization of the required manpower (800 labourers) took more time than expected.

Fig. 15.6 schedules the construction of the 'core walls'[10], placing 50 kg synthetic bags (event 2 – 6) and earthworks from both banks (event , 3 -)[11], for the period Nov. 2019 – early Jan. 2020. The excavation of the drainage channel to the newly constructed sluice (event -10) was planned for the same period.

Piling works for the final closure were planned for Nov. 2019 (event – 4). The filling and stockpiling of 90 000 nos. 50 kg synthetic bags to be used for the final closure would take place during 40 days in Dec. 2019 – early Jan. 2020.

[10] A cross-section of the 'core wall' is not shown in the draft Report and there is no further explanation about its shape and function in the text.

[11] I.e. the narrowing of the channel from both banks over 270 and 155 m, respectively, as noted in Sub-section 15.3.1.

15.5 Closure of the final gap

15.5.1 General

Below, the preparations for the final closure are described followed by information and remarks about this closure. Apart from the draft Report (Sub-sections 7.5.1 and 7.5.2), the information has been derived from the film made on the day of closure by means of a drone[12].

15.5.2 Preparations for final closure

According to Figures 15.7 and 15.8 the dike heads at both sides of the closure gap are constructed as vertical abutments: semi-circular double rows of Bullah piles transferred into a cage by horizontal bamboos; the space between the rows (2 m wide) is filled up with 50 kg synthetic bags.

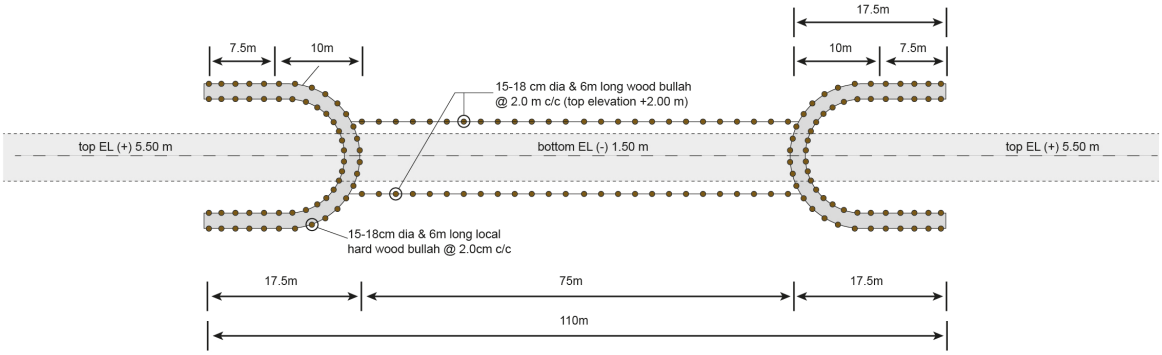


Fig. 15.7 Top view of abutments and sill with four rows of piles; source: RHDHV & Partners (2021, 16/22)

Fig. 15.8 Top view of abutment at edge of closure gap; photograph made by M. van Ledden

[12] The film has been made by the Contractor CHWE and has as title 'Closure of the Nalina Closure Dam'; its duration is 34 minutes.

The distance between inner and outer rows in the closure gap is 1 m. The piles in a row are driven at centre to centre distances of 1 m (both can be seen in the film). The distance between both inner rows is 6 m. Apparently, the piles in the inner rows are there to prevent the 250 kg geo bags from moving away and to stabilise the piles in the outer rows by means of steel wire connections. As the actual closure progresses, 4.5 m long bamboo grills are fixed horizontally against the outer row of piles at the river (= ebb) side to prevent the dumped 50 kg bags from washing-out from the sill (Fig. 15.10 and Fig. 15.11).

15.5.3 The actual closure operation

In this case a horizontal closure is carried out by carrying 50 kg synthetic bags filled with sand from the stockpiles on both abutments to the place of dumping. The operation started at 9.35 a.m. on 17 Febr. 2020. At that time the water level at the basin side was PWD + 0.50 m and PWD + 0.16 m at the river side[13].

Fig. 15.9 Actual closure in progress in the morning of 17 February 2020; source: RHDHV & Partners, (2021, 18/22)

The 75 m long gap was successfully closed, by stopping the flow at 11.37 a.m., i.e. 2 hours after start of the operation. At that time the water level at the river (ebb) side was stated to be PWD – 0.29 m. The dumping was said to be on the principle of non-wading, i.e. the local crest height of the dumping depends on the water level. During the period of two hours the water level at the upstream side (basin) went down from PWD + 0.50 m at both ends of the sill to (approximately) PWD + 0.20 when the dumping reached the centre of the sill. This means that during two hours:
2 x 40 (length of sill) x 6 (width) x (2.0 + 1.7)/2 (average height) x 35 (number of bags in 1 m^3) = 31 080 bags are dumped. This comes to 19 bags/per manhour which is possible because nobody is yet tired and walking distances are shorter than later on when the bags in the nearest parts of the stockpiles have already been taken.

[13] Also called upstream side and downstream side, respectively.

In the afternoon at 4.27 p.m. the closure bund had reached a crest level of PWD + 2.20 m over it full length. The width of the closure bund of dumped bags was 6 m and its sides were practically vertical. Its average length being 84 m (i.e. 75 m at sill level and 93.5 m at crest level) and its height 3.7 m (sill at PWD -1.5 m and crest of bund at PWD + 2.20 m), the volume of the closure bund must have been 1 865 m^3. Assuming that 1 m^3 contains 35 bags the number of bags dumped in the closure gap is 65 275. According to the draft Report 800 labourers were involved. The overall closure period was 7 hours. If a rest period is assumed of ½ hour[14], the dumping activities took place during 6½ hours which implies an hourly dumping production of 10 042 bags or 12.55 bags per manhour, which means that a labourer dumps a bag every 4.8 minutes. The average distance between stockpiles and place of dumping (incl. walking back) is 125 m. If a labourer walks at a speed of 1 m/s, then it takes 125 seconds to walk, which leaves 2.7 minutes for loading a bag.

Fig. 15.10 Closure works in progress seen from the ebb (river) side; source: RHDHV & Partners, (2021, cover)

Fig. 15.11 Dumping of bags after the flow has been stopped; source: RHDHV & Partners, (1921, 18/22)

[14] In the film it can be seen that during a certain period the dumping from one side has been halted.

15.6 Summary and Conclusions

1 In this Chapter a recent tidal closure has been described and analysed in which manual labour played a major role.

2 Although the sources (film and draft Report) are not always fully consistent, these have been useful and insightful to study the Nalian Closure in detail.

3 Prior to the actual final (horizontal) closure, a horizontal sill has been constructed (by vertical and horizontal method), subsequently followed by narrowing of the 500 m wide river channel to a width of 75 m at sill level.

4 The sill has been built up by dumping 250 kg geo bags and smaller (50 kg) synthetic bags filled with sand; the narrowing of the tidal river was by way of placing synthetic bags and earthworks; for the final closure 65 000 synthetic bags were carried and dumped during a period of 6.5 hours by 800 labourers.

5 Contrary to closures described in Chapters 7 to 11 (incl.) no classic bed protection has been applied to prevent scour of the easily erodible subsoil. Instead, geo bags and synthetic bags were placed to (a) prevent scour of the subsoil, (b) to consolidate the deposits of very soft clayey silts under the base of the future dam and (c) to form a horizontal sill for the final closure.

6 As such, the closure method using geo bags requires less input of skilled labour than, for instance, required to construct, transport and immerse floating fascine mattresses and/or to construct a jetty across the closure gap as described in Chapter 11.

7 A disadvantage of this method is the time it takes to construct the closure. The Nalian closure from start of geo bag dumping until closure took 17 months. The closures of Chakamaya and Amtali, described in Chapter 11, took each less than 4.5 months[15].

8 During the closure of the final gap in the Nalian cross dam a production of 12.6 and 19 bags per manhour was achieved in loading (on the head), transporting and dumping of the bags. Comparable figures have been found for the three closure projects in Bangladesh as described in Chapter 11:

- Chakamaya 12 bags/manhour
- Amtali 13 bags/manhour
- Feni 9 bags/manhour

Differences in production figures are mainly due to differences in walking distances between the projects and whether bags are dumped (Nalian, Chakamaya and Amtali) or neatly positioned as directed by supervisors (Feni).

During the closure of the Kedzi Canal in the Keta lagoon in Ghana production figures seem to be much lower (6 bags/manhour). But these bags were twice as heavy and had to be hand carried either by two adults or four young students.

9 This closure project has been tendered as a lump sum contract without an item-wise build-up of quantities and unit rates. A detailed analysis by Van Duivendijk of the construction method and the financial features of the various items involved in the construction of the closure is therefore not possible without additional information (like a detailed works diary).

[15] In all three cases the periods mentioned exclude the construction of the final dam profile.

CHAPTER 16 The closure of the Dagenham Breach in 1720[1] (project no. 72)

16.1 Introduction

On 29 October 1707 a small sluice[2], which was draining the marshes behind the northern flood embankment along the river Thames at Dagenham, 'blew up' and the Dagenham and Havering Levels[3] were flooded. The breach in the embankment caused by the defective sluice was initially only about 4.5 m long and could easily have been repaired but no action was taken and the tidal movement resulted in a flow gap which, ultimately, had a length of 120 m. Several attempts have been made to close the breach but to no avail. As such, closure of the breach was difficult because of high tidal ranges at that point in the Thames River (6 to 7 m during springtides).
Finally, in 1716, John Perry was assigned to close the breach and he succeeded, after many setbacks, to have the marshes drained in June 1720.
Below, first of all the site conditions are described. Then, briefly, the failed attempts to close the breach. Finally the work carried out by Perry will be discussed as well as the mishaps encountered by him.

16.2 Site conditions

16.2.1 Hydraulic and topographical conditions

Perry mentions 22 ft. (6.7 m) as the tidal range during springtide. Downing (1864, 59) states that the meadows are more or less at the level of high water neap (HWN) and that there is 4 ft. (1.2 m) of water on the meadows during spring tides. Tidal ranges have been found on the internet for the river Thames at Cross Ness which is near Dagenham (Table 16.1)

Table 16.1 Some water levels forecasted for Cross Ness in the year 2021

type of tide	date	high (m)	low (m)	tidal range (m)
spring	18 March	6.9	0.7	6.2
neap	23 March	5.3	2	3.3
spring	2 April	7.3	0.5	6.8
neap	6 April	5.6	1.9	3.7
neap	20 April	5.6	1.6	4.0

From the figures in this Table one can gather that in 2021 tidal ranges during springtides are in the order of 6 to 7 m and during neap tides 3 to 4 m. Whether this also applies to other months of that year and, more in particular, to the first half of the 18th century is not at all sure[4] but that is less relevant for the current discussion. Some hydraulic calculations, using the figures in the Table, confirmed the statement of Downing about the water level on the meadows during a springtide. Following the extent of the flooding and tidal channels, as shown on the map in Fig. 16.1, one can calculate the tidal volumes to be in the order of 2 million m^3 during neap tides and 15 million m^3 during springtides. The area of the tidal basin is approximately 10.6 km^2 and that of the tidal channels 1.1 km^2.

[1] Most of the information in this Chapter has been derived from the book by John Perry (1721) 'An account of the stopping of the Daggenham Breach' and the article by Samuel Downing 'Dagenham Breach' which appeared under the heading 'Hydraulic Earthworks' in the May and June 1864 issues of The Practical Mechanic's Journal.
[2] According to Downing (p. 31), this sluice, a hollow tree trunk, was originally constructed by Cornelis Vermuyden in about 1620.
[3] Level, In this case, means flat area, in the book and article mentioned in footnote 1 the area is also denoted as 'marshes' and as 'meadows'.
[4] For instance the stage hydrograph of upstream river flow may result in a higher or lower MSL in the course of a year.

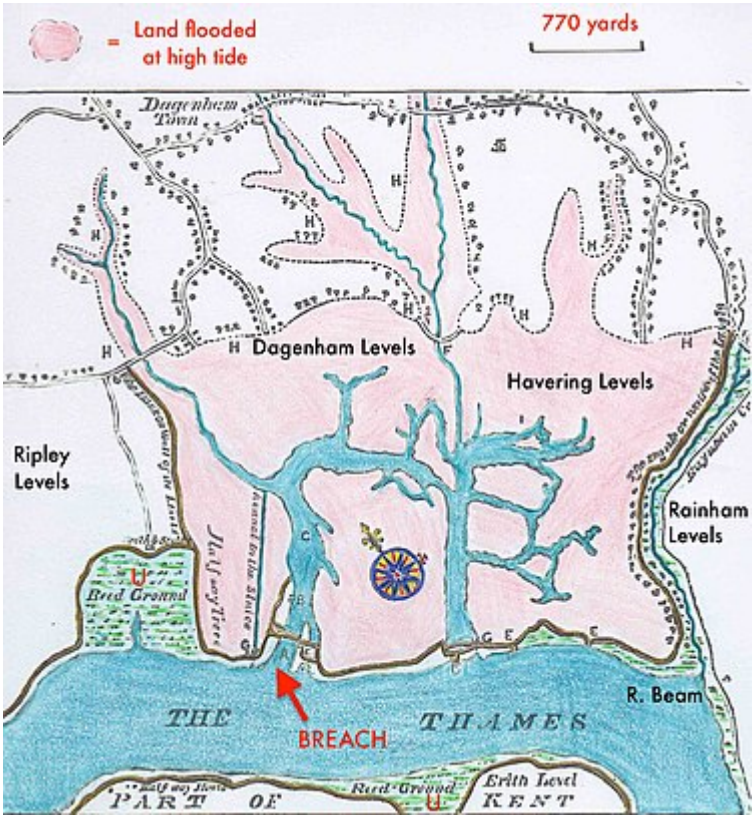


Fig. 16.1
A map of Dagenham Breach; Tidal waters scoured out a permanent lake. The pink area was flooded during spring tides. The channel marked G-F was due to an old breach, fixed by Vermuyden in 1621*;
source of map: Perry (1721)

*for the record: it is doubted by Harris (1953, 37) whether Vermuyden was successful in closing that breach.

16.2.2 Geotechnical conditions

By the time that the tides had widened and deepened the breach, the clay had scoured away, rapidly followed by the layer of peat. This enabled the tidal currents to scour deeply the underlaying yielding strata of sand and gravel. Perry (p. 50) states that when he started the closure works the breach had scoured to a depth of 7 m below LWS.
A similar development took place in the tidal channels which developed in the meadows. It was computed (Downing , p. 31) that 120 acres (50 ha) of land had disappeared from the meadows into the Thames, causing “a shoal halfway across the stream, extending nearly a mile, in both the higher and lower reaches of the river” having an injurious effect on navigation.

16.3 Failed attempts to close the breach prior to the year 1716

16.3.1 Attempts by the landowners

The landowners started their work by narrowing the breach from both sides by:

> “running forward a jetty of timber pile work” (Downing, p. 31) [5].

The remaining channel then was rapidly closed:

> “by sinking the hulls of old vessels and large rectangular chests, laden with stones, on each side of which were thrown in baskets filled with chalk, and bags filled with earth and gravel”.

Such operations were carried out during neap tides. But it turned out that:

> “the tide always succeeded in boring through below the obstructions which had been placed in its way”.

This exercise was repeated a number of times without a positive result. Then all efforts

> “on the part of the landowners ceased as the value of the property injured would not have warranted further expenditure”.

[5] Probably, a cofferdam is meant.

16.3.2 Attempt by Mr. Boswell

The destructive effects of the shoaling:

> "in the river caused the House of Commons to pass a bill[6] for effectually stopping the breach, removing the shoal deposited in the river, and making good the adjoining banks, at the public expense".

Mr. Boswell offered to execute the works for £ 16 500 while Captain Perry's quotation was £ 24 000. The lowest offer was accepted by the Trustees and Perry (p. 30) notes in this respect:

> "without taking upon them to examine whether his (i.e. Mr. Boswell's) skill was equal to his proposal."

Boswell started his Contract in a manner similar to that of the landowners but with some improvement, i.e. by narrowing the breach "by running out timber piers[7], from each side, as far as feasible".

In this flow gap:

> "specially formed flat-bottomed chests, or as they were named at that time, 'ponts', 18 m long, 9 m wide and 6 m high, with pointed ends like the cutwaters of the piers of a bridge, and for the same purpose. These chests, six in number, were intended to have been placed transversely to the line of the river bank, or fore and aft in the run of the tide through the breach, and at a clear distance of 12 ft. (4 m) intervals, each chest being provided with sluices or draw-doors[8], fore and aft, so that when sunk in place they yet permitted the tide to ebb and flow at will, and which, on being shut down[9], were wholly and at once to exclude the water. The intervening 12 ft. spaces were to have been made up close with piles and other timber work previously, and then the sluices being shut at low water, it was hoped that the work of excluding the tide would have been completed…
>
> But no sooner had the timber jetties contracted the gap, than the increased velocity of the current scooping out the soft bottom of the channel created such a depth of water as to render the plan impracticable." [10].

Boswell then contracted the gap even further and tried to place one enormous caisson but this only increased the depth and the bottom got quite irregular to which the flat bottomed chests could not conform. Perry (p. 31 – 38) describes Boswell's attempt in some detail, including other mistakes made by the latter and notes as a result of Boswell's work:

> "to render the Breach much more chargeable and impracticable to be stop'd than when he began it."

16.4 The plan for the closure as prepared by Captain John Perry

After Mr. Boswell's failure the trustees asked again proposals for stopping the Breach. Among those proposing Security, was Perry, offering to undertake the work for 27 000 pounds. But the trustees, after having studied all proposals, formulated more stringent conditions and asked for new proposals. Perry then made a new offer for 30 000 pounds and submitted his method statement on 6 December 1715 (Perry, p. 49 – 59). His method can be summarized as follows:

- Construction of two sluices, in undisturbed ground, each having a sill 12 m long, at an invert level at 30 cm above LWS[11];

---

[6] In April 1714.

[7] It is not completely clear what kind of construction is meant.

[8] *Hefschuiven*.

[9] The modern version of such 'ponts' is the sluice caisson (*doorlaatcaisson*).

[10] Downing (p. 31).

[11] Originally, Perry wanted to build one sluice, having a deeper invert level than finally constructed but because of bad ground conditions at lower levels, this idea was abandoned and he built two sluices instead.

- a cut-off wall using dovetailed[12] sheet piles (Fig. 16.2), in the centre of the future flood embankment, having its top at level at LWS + 0.45 or 0.60 m (Fig. 16.3 at B) to render the foundation absolutely water tight; thickness of sheet piling 17.5 cm near the dike heads and 20 cm in the centre of the breach; the sluices would be provided with draw-doors, which could be raised and lowered at will by rack and pinion[13];
- apart from the sheet piling no timber must be incorporated in the embankment, also no ships or chests because these constructions will always leave room for the water to flow through (i.e. piping);
- construct at both sides of the central row of sheet piles at 12 m distance a kind of low cofferdam 6 m wide, formed of vertical piles and horizontal boarding and filled with chalk (= soft limestone), shown in Fig. 16.3 at A and A; Perry calls these cofferdams foot wharfs. On their outside a wall of chalk rubble is to be made as further security;
- the dam itself must be composed entirely of clayey earth to be placed in 90 cm thick layers and forming steps C and C, 2 m wide;
- because of expected settlement the crest of the dam to be 45 cm higher than the adjacent embankments;
- after completion of the works the sluices must be removed because in the course of time they will decay and cause breaches to develop;
- in the course of its construction the sluices must be operated such that the water level on the meadows during the ebb period never comes below the crest of the dam in progress.

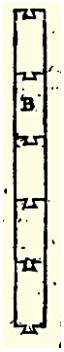


Fig. 16.2 Perry's dovetailed piles; source: Perry (1721)

Comment: This last condition is in fact the key to success as it prevents a large head in the nappe of the flow over the dam being constructed. Obviously, the gates of the sluices are fully opened for inflow and outflow of the tides at levels above the crest of the dam under construction.
Negotiations with Perry started on 12 December 1715 but, because of petitions from other tenderers, presented to the House of Commons, decisions were postponed. It was only seven months later that the contract was signed (8 June 1716). The Contract Sum was £25 000.
Meanwhile the price of timber and iron work had gone up by 20 % and spring as well as part of the summer of 1716 were lost.

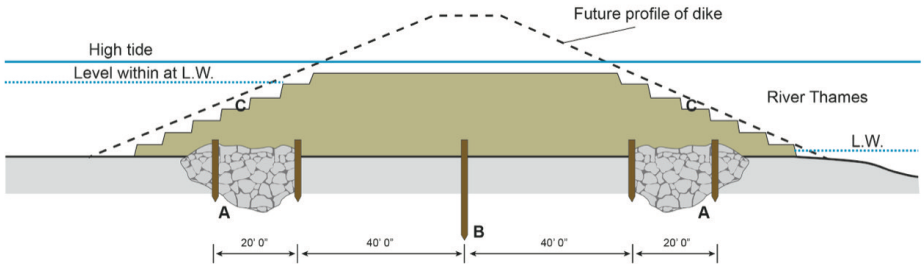


Fig. 16.3 Cross-section over closure works and final dam profile as proposed by Perry (1721)

16.5 Construction of the closure works

After Perry had constructed one sluice and the wet area of the breach had been decreased considerably by the dovetailed sheet piles, he noted that the force of the flow in the remaining passage

'began to operate with great violence'.

[12] Dovetailed: tenon shaped fitting in corresponding mortise & forming joint (*messing en groef*).
[13] Rack and pinion: *tandheugel.*

Accordingly, he decided to build a second sluice but , because of frost and drift of ice during winter, he was only able to finish that second sluice by spring 1717 and close the row of sheet piles in the beginning of June 1717.

> "And then the Time was near approaching, according to the Letter of my Contract, for turning the Tides out of the Levels" (Perry, p. 75).

The earthworks had to go on by day and night in order to take account of the low tides. As it was difficult to check the output of labour during the night it was decided to pay labourers:

> "by the tun (it being observable that they worked two or three times as much by Task-work, as by the day or by the tide)."

But the labourers:

> "wilfully brought soft and bad earth to the Works, especially in the night, mixed more or less with that which was good".

The situation got worse and worse, also because time was running out and Perry's assistants did not comply with his orders. But when Perry rejected barges filled with bad earth the labourers went on strike or riots broke out, etc.

The dam had hitherto from the foundation been carried up by set-offs on each side of about two m in width made by a drift of small piles, or boards or planks at the height of every 90 cm (Fig. 16.3). The fill brought in was every tide:

> "covered over at the top by reeds and hurdles to prevent the same from being gullied….But upon the turning of the tides out of the Levels (5 Sept. 1717), at the time of the neap, a naked wall of earth, without any piles driven, boards on the side, or covering on the top was proposed… to be carried up, by the great number of hands there was for it, faster than the spring could be apprehended to rise upon us; and, in accordance to the Expedition so much press'd to me, I agreed to this method , in the hope of success."

This relatively high (1.2 m) narrow bund settled, and then, on 10 Sept.:

> "an extra ordinary high tide, (one day after springtide) occasioned by a storm at N.W. overtopped the unprotected bund by 15 cm and within two hours it tore down the body of the dam, until stopped by the heads of the dovetail piles…" (Perry, p. 81-82).

During the winter months repairs were carried out to the sluices and to the foot-wharfs in the breach. Early in the spring of the year 1718 the earthworks were resumed in the same manner as before but with better earth. In the second half of June:

> "the tide was a second time turned out of the Levels in the time of the neap as before."

Inspection by a committee, appointed by the trustees, was on 20 July 1718.

After the inspection Perry started:

> "to dam up the two canals at the insides of the sluices and add more earth to the dam in the breach, as it was found to settle and as a new body of earth was natural to be expected." (Perry, p. 90 – 92).

On 22 August, during neap tides, Perry departed for Dover to advise about problems in the port. Afterwards, he fell ill and he visited the dam again on 25 September. On 30 September, during the night:

> "a high tide, which flowed 15 to 18 cm above the lowest part of the great dam in the breach cut down and destroyed the same….".

Perry then tried to limit the damage by removing the dams in the canal and by opening the sluices. During the sixth tide after the breaching the row of dovetailed piles were torn up, as well as the foot-wharfs at both sides destroyed, over a length of 30 m. In fact the scoured depth was 6 m more than when the works had started.

Perry did not understand what had happened:

> "Whether this misfortune took its rise from any leakage, or whether the hurdles were made loose, or how the water came to gull the earth , which they covered from underneath them, is not known; it is however evident , that there was a neglect of duty on this occasion: either the watchman must have been awake upon the works, and did see the tide rise higher than his orders was to have done, before he called or gave notice thereof, to my assistants; or was asleep, or absent at an ale-house, that was near…"(Perry, p. 99)[14].

Perry, in order to finance the repair of the breach, asked the trustees to pay him the remaining £ 4 000 of his Contract, but the trustees refused.

> "And upon the estimate which I afterwards produced to my Securities, that it would require between 6 and 7 000 pounds to make good the damage sustain'd , I found great difficulty to unite them to raise a Sum amongst themselves for it; and had not my good friends, the Russia Merchants, who knew me in the Czar's service, stood firmly by me, it is certain I should never have been able to have gone through with my undertaking.
> It became now a question by those who declined to advance money, whether the method I proceeded in was proper? And there were many proposers and new schemes now set up for repairing the works in a different manner from what I had laid down in my estimate."
> (Perry, p. 103)

The disputes about the method to repair the breach were time consuming and also the funds were coming available only slowly. As a result the repairs only started in February 1719 and advanced slowly because of the great depth to be filled and lack of money. In September of the same year the sluices were repaired, the foot-wharves completed and the missing sheet piling installed. Early in spring 1720 Perry started to place the fill above the level of LWS and once again had to experience damage to the earthworks by tidal flow going over the non-completed dam but on 18 June 1720 the tide was for the third time turned out of the Levels. Perry then doubled the watch during all springtides and also ordered his assistants to take their turns.

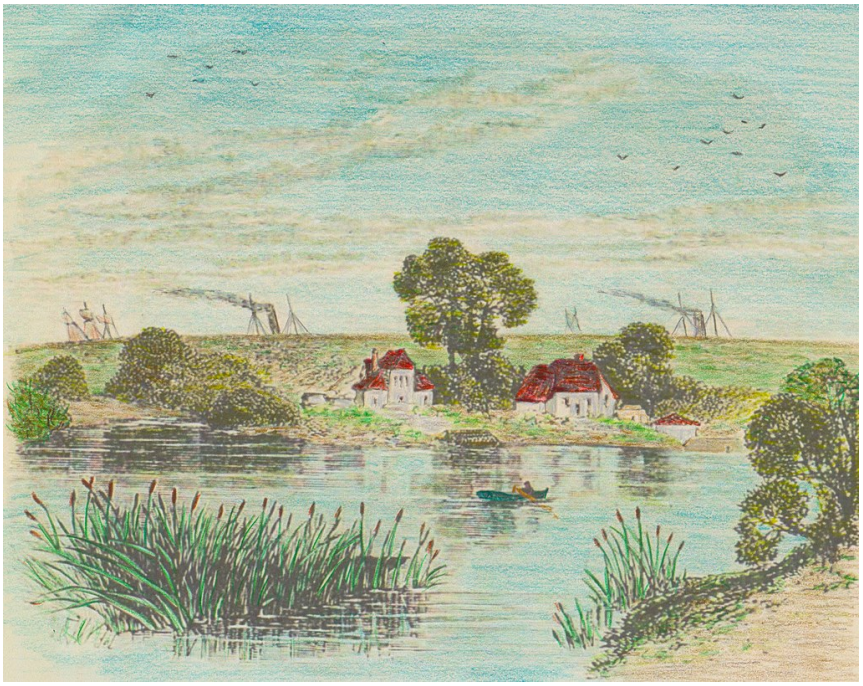


Fig. 16.4
Dagenham lake about 1860; Thames shipping passes on the other side of the repaired flood embankment;
source: hand-coloured version of public domain engraving (from Smiles, Samuel, "Lives of the Engineers", London 1861)

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[14] See also Section 16.6 for a possible explanation of what caused the breach in the dam.

In November 1720 a very high tide occurred but his dam still had a freeboard of more than 30 cm while, according to shipbuilders along the river, it was the highest tide on record in the Thames. Still Perry decided to heighten the crest of the dam by another 20 cm, having taken already account of the settlement of the dam body.

Finally, Perry hopes he will receive a reasonable sum of money in addition to the £ 25 000 laid down in his Contract. He states in this respect:

> "….have moreover been obliged to enter into very considerable debts and engagements, to enable me to complete my undertaking, very much beyond the Sum I contracted for with the Trustees; which, without I am relieved in, I must be liable to continual actions and imprisonment, and debarred from any further industry the future part of my life: therefore I hope …the Trustees in their justice to me , will now please to remember their promise, or agreement with me, upon my entering into Contract with them for twenty-five thousand pounds, that in case I did effect the work, and that the said sum should by means of unforeseen accidents, prove deficient , they would recommend me to the …House of Commons, to be reimbursed such money as my Contract should fall short of, and for such a reward to myself as the House should please to order."

It could not be ascertained whether his request has been successful.

16.6 Some remarks about the works carried out by Perry

Perry made his plans at a time when the knowledge about soil mechanics and fluid mechanics was still very limited. He also could not make reference to similar projects elsewhere. But he must have had the intuition to do the right things. Still, one would like to have an answer to such questions as:

- why were the sills in the sluices each 12 m long;
- how could the dam crest overtop three times, in each case due to a high springtide and…. after the tide had been turned out of the Levels; one would have expected more care after the first overtopping; also a final freeboard of 50 cm is not very much;
- was any compaction done of the 90 cm high layers of clayey earth.

After the third overtopping Perry was at a loss as regard the cause of the breaching. But a thin horizontal layer of reed on top of the 5 m wide crest does not necessarily prevent scour of the earth layer beneath. The current velocity over this clear weir is already more than 1 m/s. Moreover, as the water in the Levels had been removed, the water cascaded down over the steps on the land side over a height of some 5 m and will have undercut the heel of the dam.

When looking at this closure, carried out in an environment of springtides ranging on average up to 6.7 m, one can only admire Perry for his bold undertaking. But, admittedly, two factors helped to make the closure successful:

- the meadows were at a rather high level: according to Downing during springtides there was only 1.2 m of water on the meadows and nil during neap tides;
- the sluices restricted the flow over the embankment under construction and operated such that the water level on the meadows during the ebb period never came below the crest of the dam in progress.

The last point, however, may also has been a disadvantage: when compaction is lacking, the whole earth body will have got saturated during the continuous high water level inside the flooded area. Subsequently, when the dam was (nearly) at its final crest level, the tides were 'turned out of the Levels' and during a high tide the dam was overtopped. Only by 15 cm! But in view of the said saturation it is understandable that "within two hours it tore down the body of the dam, until stopped by the heads of the dovetail piles".

Another weak point must have been the supervision. Perry was not always at site and his assistants were supposed to prevent the labourers from bringing: “soft and bad earth to the Works, especially in the night, mixed more or less with that which was good”. But the assistants did not comply with Perry’s orders!

16.7 Cost of construction

Though no final figure about the cost of construction has been given it can be assumed that the overall construction cost of the project was in the order of £ 40 000. Such figure in the year 1720 has a purchasing power[15] in the year 2017 of approximately £ 4 644 300, which in that year is equal to € 5 301 004 or € 5 603 905 in the year 2020.

[15] https://www.nationalarchives.gov.uk/currency-converter/#

CHAPTER 17 A cofferdam in 1862 in the main outfall drain of the Middle Level (England) (project no. 73)

17.1 Introduction

The case history presented in this Chapter is based on a paper[1] presented by John Hawkshaw on 21 April 1863 at the Institution of Civil Engineers, London.

On 4 May 1862, the St. Germains'sluice at the confluence of the main outfall drain of the Middle Level and the tidal River Ouse, gave way. Consequently, the tide was then able to flow in and out of the Drain over a distance of more than 30 km. Several breaches developed in the banks[2] of the Drain. These were successfully stopped except one which caused flooding of 24 km^2 of land (Fig. 17.1). Apart from this flooding, tidal currents were scouring the bed of the Drain, in one place 4.5 m of scour was measured over a distance of 500 m.

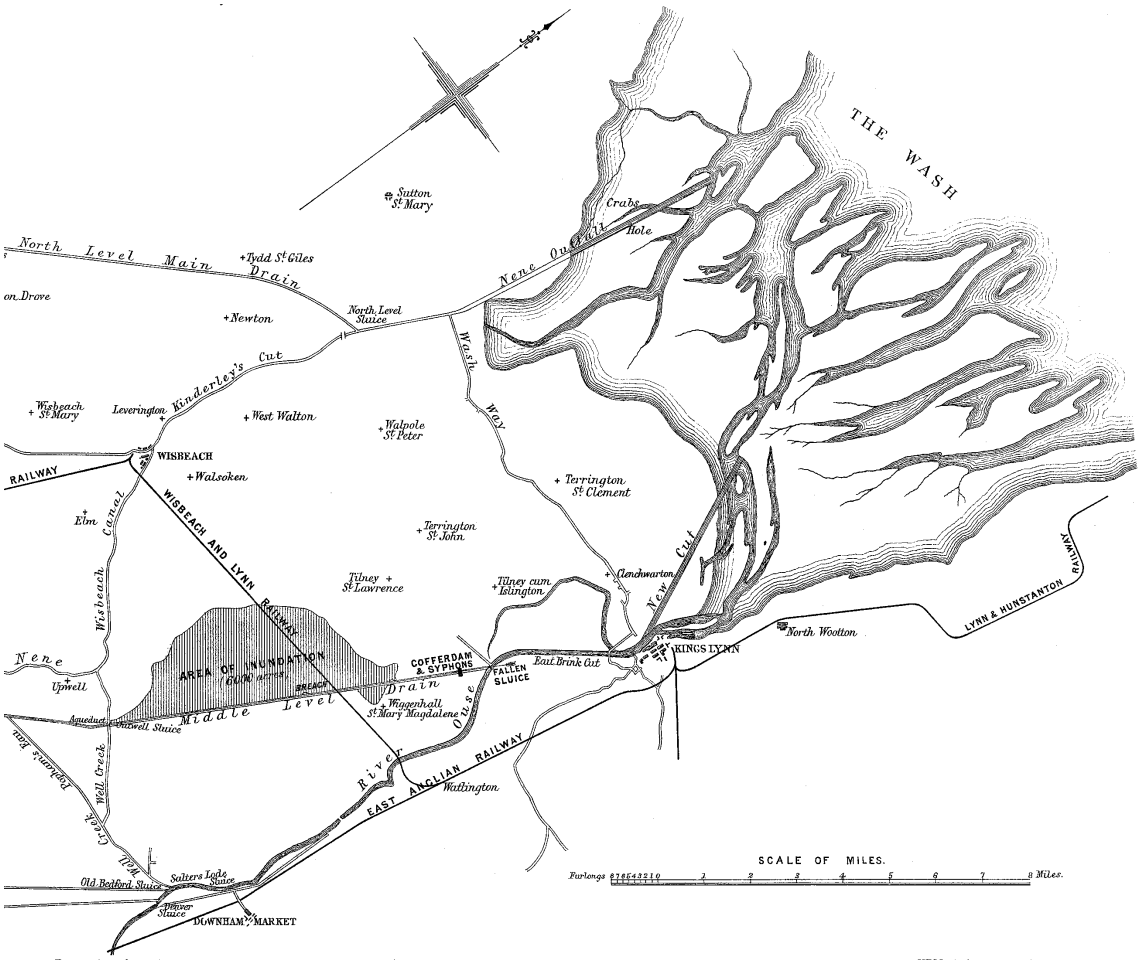


Fig. 17.1 Location cofferdam and syphon in the Middle Level Drain and area flooded; source: Hawkshaw (1863)

It was decided to close first of all the Drain, subsequently followed by repair of the breach in the bank through which the area of 24 km^2 was flooded. The area flooded operated as a safety valve as it

[1] Hawkshaw, John (1863); 'Account of the Cofferdam, the Syphons, and other works, constructed in consequence of the failure of the St. Germains' Sluice of the Middle Level Drainage'; Minutes of the Proceedings of the Institution of Civil Engineers, 1862 – 1863, pages 497 – 508.

[2] I.e. the flood embankments at both sides of the Drain.

dampened (too) high water levels in the relatively narrow Drain. If the breach had been repaired first, there was a possibility that the banks would have breached elsewhere. Ultimately, the breach was repaired which required 20 000 m^3 of stones, clay and clunch (pieces of hard limestone).
Below, first of all the site conditions are described, then construction of the cofferdam will be described as well as the difficulties encountered. Finally, some remarks will be made regarding the method used (or not used) to close the Drain.
Design and construction of the syphons (*hevelinstallatie*) replacing the sluice, as described in the Paper, will not be addressed.

17.2 Site conditions

17.2.1 Hydraulic and topographical conditions

Only the tidal range in the Ouse at the failed sluice during spring tide is given: 5.8 m. The sill of this sluice was at LWS – 1.8 m while the bed of the Drain (Fig. 17.2) is at LWS – 2.10 m.

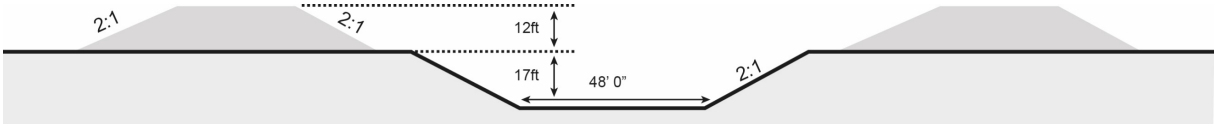


Fig. 17.2 Cross-section of the Middle Level Drain near its lower end; source: Hawkshaw (1863)

In the flooded area, the water level reached during spring tide is not more than LWS + 2.7 m while the variation in water level in the flooded area due to this tide is only 15 cm. This implies a tidal volume of 3.6 million m^3 for the flooded area only.
The interval of repose (*kentering*) in the Drain is of very short duration: only a few minutes. Current velocities at the site of the (future) cofferdam were about 2.7 m/s during springtides and 1.8 m/s during neap tides.

17.2.2 Geotechnical conditions

The bed of the Drain at the site of the (future) cofferdam consists of

> "soft blue clay while the slopes from bed level up to ground level consist of variable thicknesses of soft blue clay, peat, yellow clay and surface soil. Of these materials the side banks above the level of the ground are composed."

17.3 Fascine dam

Immediately after the inundation and prior to Hawkshaw assuming the direction of the works, the construction of a fascine dam across the Drain had started at 450 m from the failed sluice. The fascine mattresses were ballasted with clunch and bags filled with clay.
Hawkshaw writes the following about this dam:

> "the Author not wishing to abandon an experiment, which he did not originate, without having fairly tried it. It is clear, moreover, that if the dam had be well carried up even to a moderate height, which was the utmost the Author ever expected, or desired to accomplish, it would have had the effect of shutting out a certain quantity of tidal water, which would to that extent have relieved the banks of the drain, and have assisted the construction of the permanent cofferdam. On the other hand, by continuing this dam, the introduction of successive layers would pen up an increased quantity of water in the drain; and the more it was raised, the greater would be the scour from the water flowing over it at ebb tide, which scour was considerable when the dam was only raised a few feet. Numerous barges also were required in its construction, any one of which would, if it had broken loose when the tide was running up the Drain, have ruined the permanent cofferdam. Great caution was therefore necessary in pursuing its construction ; and not long after the permanent cofferdam was

commenced, the Author determined to relinquish altogether proceeding with the earthwork and cradle dam[3], tidal observations having shown that it was unnecessary, when the construction of the permanent cofferdam had advanced to a certain stage. There is no doubt, that the system of raising such a dam by cradles of fascines might be introduced with advantage, in many situations, and under certain circumstances. The Author intrusted the superintendence of sinking the cradles to Mr. Müller, who had considerable experience in raising banks in this manner[4], …"

It does become clear from the text that Hawkshaw did not see this fascine dam as a permanent solution or even a realistic means of stopping the tide[5]. He therefore speaks of the 'permanent cofferdam' that was to bring about closure.

17.4 Construction of the permanent cofferdam

Initially, a wooden road bridge 900 m away from the failed sluice was to be used as a staging for pile driving but that bridge was destroyed by a dislodged barge. It was then decided to locate the permanent cofferdam 780 m away from the former sluice.

Construction of the cofferdam began on 16 May 1862. First an auxiliary scaffold was built using screw piles. Then two rows of sheet piles were driven (7.5 m apart) on either side of the Drain from the crests of the dike down to 6 m from the bottom. The width of the closure gap was thus 2 x 6 + 14.7 = 26.7 m. Then vertically rectangular beams (Hawkshaw calls these twin piles) were driven 2.3 m centre-to-centre such that openings were created with rebates into which wooden panels (sliding gates) could be lowered, 18 cm thick, having a width of 2.21 m (Fig. 17.3).

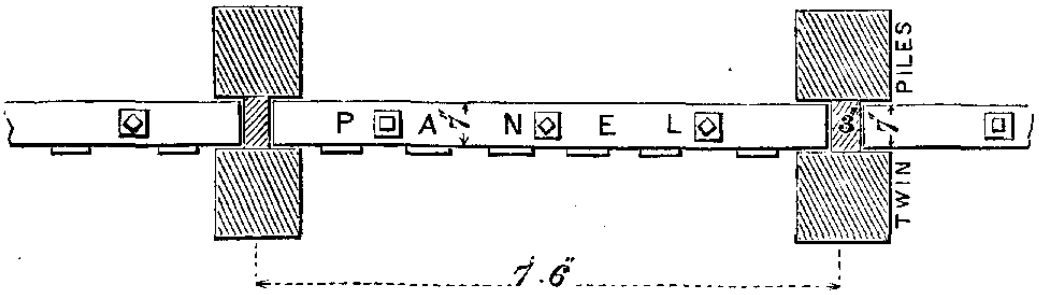


Fig. 17.3 Plan showing a sliding panel between two twin piles; source: Hawkshaw (1863)

"As the piles were driven, proper struts and ties were inserted to stiffen and strengthen the dam, and strongly framed raking struts were fixed, abutting against the dam and against abutments, composed of piles backed with concrete, on the land."

To hang the panels, a frame was erected with pulleys to lower or lift the panels (Fig 17.4. and Fig. 17.5). When this work was more or less complete, the difference in water level between the two sides of the cofferdam in progress was sometimes 1.2 to 1.5 m.

To minimise the scour at the cofferdam, aprons (*stortebedden*) were formed by depositing bags of stone, gravel and clay on both sides of the cofferdam within poured dams of clunch at the edges of the aprons. The problem was the space inside the cofferdam where, to avoid piping in the future, one did not want to deposit stone or gravel and where, consequently, scour kept occurring. The clay deposited there before, during and after the short slack water soon washed away again: current speeds reached 3.5 to 4 m/s. With great difficulty, the level inside the cofferdam could be maintained at the level of the channel bottom, and in the aprons at 30 cm above that level.

[3] 'For cradle dam read fascine dam.

[4] Muller was employed by the Netherlands Land Enclosure Company for the reclamation of the 'English' polders (Ch. 9, Section 9.8); see also his paper (Muller, 1862).

[5] See also the comments made in Sub-section 17.5.1.

The first panels had a height of 2.10 m. When those were in place, a second set was lowered with variable heights to create a horizontal overflow threshold (Fig. 17.5).

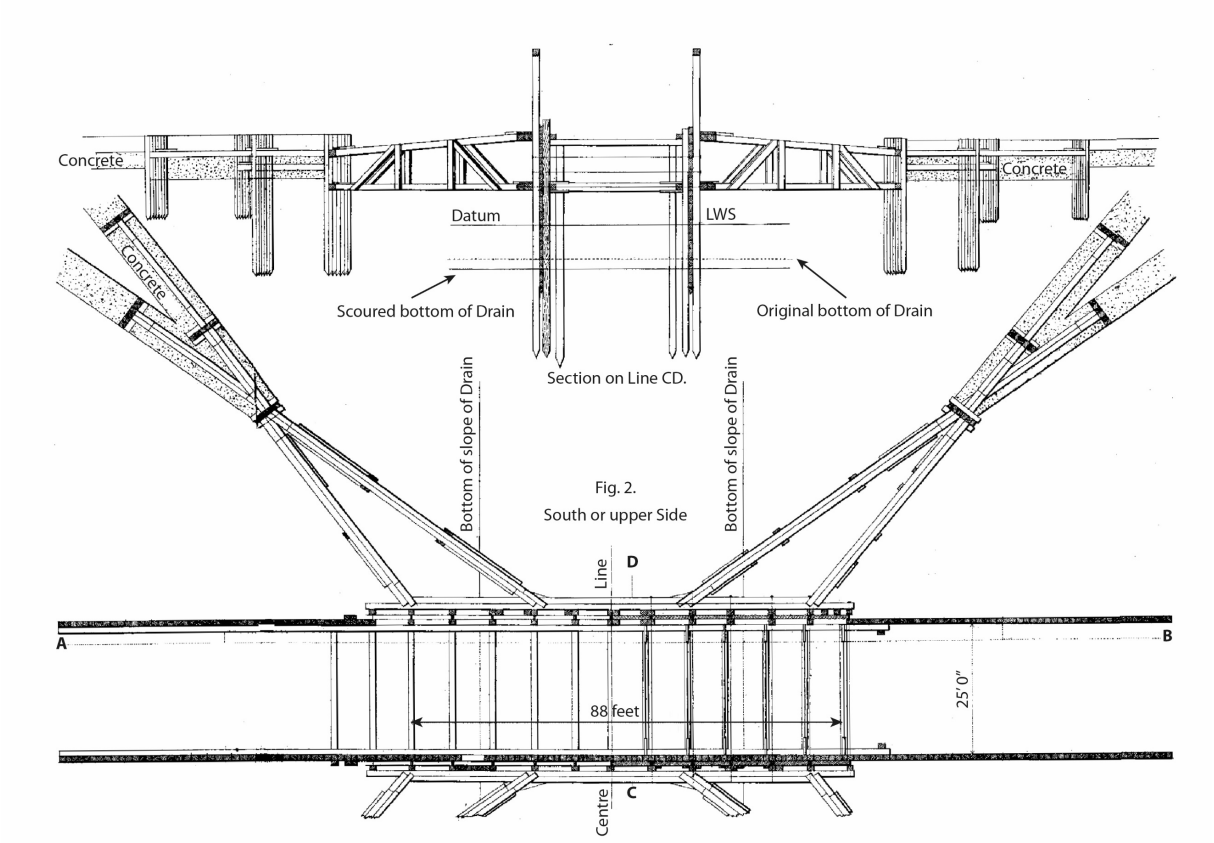


Fig. 17.4 Plan and section on line C-D of stiffening and strengthening measures taken for the cofferdam; source: Hawkshaw (1863)

It was understood that one had to act quickly now to avoid a total failure of the closure. Therefore, in one tide, one would have to lower panels around and near the high water mark over the entire width of the closure gap on both sides of the cofferdam over a height of 3.60 m. The first attempt to do this was on 10 June. That attempt failed because three of the composite piles (marked X on Fig. 17.6) with rebates broke (presumably already damaged during the pile driving), dislodging some of the already placed lowest panels at the bottom.

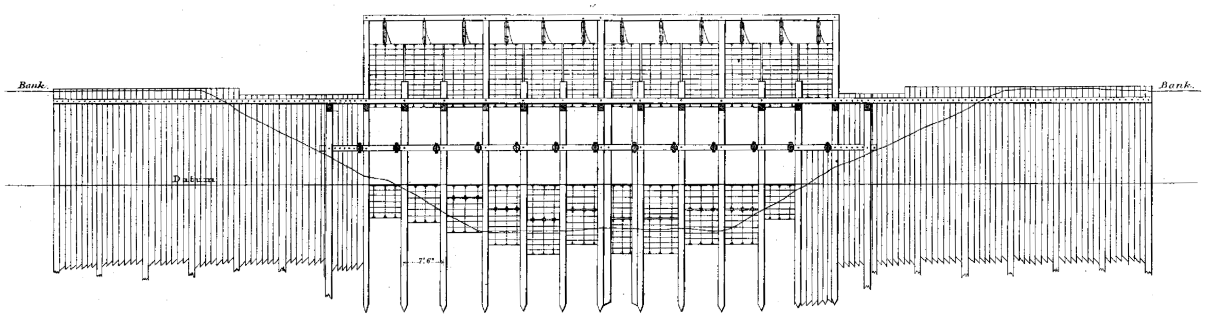


Fig. 17.5 Section on line A-B, showing panels suspended, ready to be dropped in place; source: Hawkshaw (1863)

The water had then subsided 8 cm but the head across the cofferdam was 2.4.m. Panels already placed were then lifted and removed[6]. In front of the broken piles, wooden sheet piling (marked Z on Fig.17.6) was then driven across a width of 1+ 3 openings to a height of the lowest waling pieces. This meant a further narrowing of the closure gap but a diver had found that the apron was quite damaged by the panels being torn out and measures had to be taken.
The next attempt was on 19 June during a neap tide and took place at low water slack. This was a complete success. Lowering the panels took 20 minutes. Clay, bags of gravel and clunch were then dumped in the cofferdam and it was possible to stay ahead of the rising tide. Both aprons were also steadily raised and extended. A total of 2,300 m^3 of clay was available and also several hundred bags of gravel and clunch. These materials were transported by using barrows and carts and then dumped. 800 men were available for this purpose.
Finally, a continuous row of wooden sheet piling was driven on the North side of the cofferdam and the space between this row of sheet piling and the cofferdam was filled.

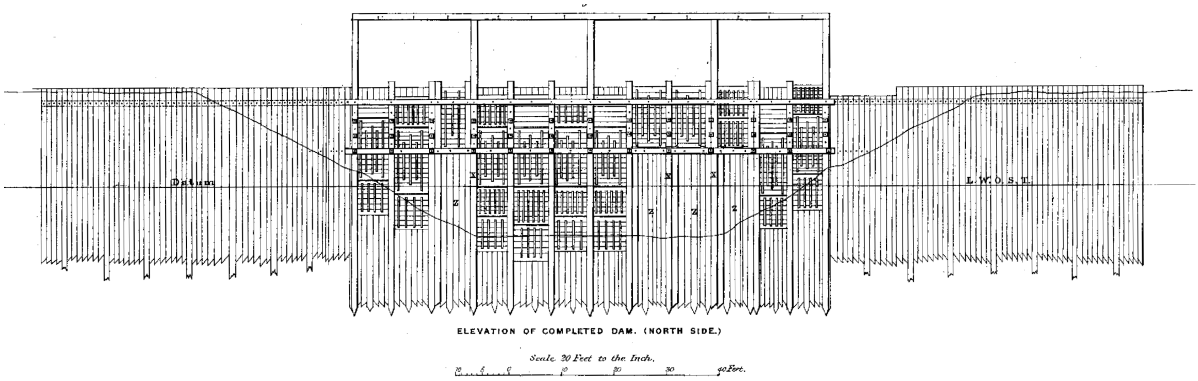


Fig. 17.6 Elevation of completed cofferdam (north side); source: Hawkshaw (1863)

17.5 Some remarks about the works described in Sections 17.3 and 17.4

17.5.1 The fascine dam

From the text quoted in Section 17.3 it is clear that Hawkshaw never intended to consider the fascine dam, which was under construction when he arrived, as a permanent structure. But the only negative point he mentions is the scour. This scour easily could have been prevented by a longer bed protection of fascine mattresses (as shown in Fig. 17.7). In any case, given the high tidal differences, two fascine dams would have been required as shown in Fig. 17.7. Caland made this design for a tidal difference of 4 m (also shown).

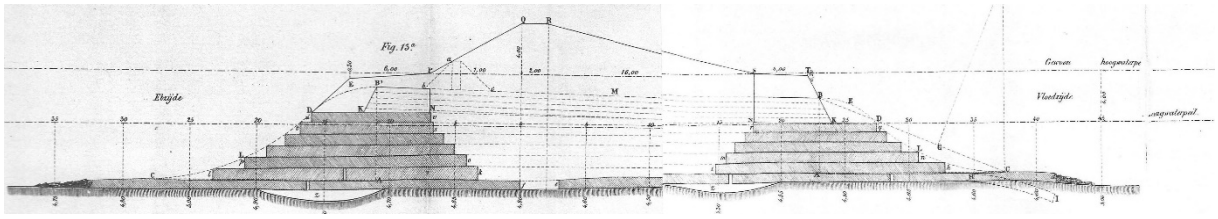


Fig. 17.7 Cross-section of a closure dam incorporating two fascine dams; source: Caland (1833, PL.II)

[6] From the Paper it did not become clear whether all placed panels were removed or only those which had been placed that very day, probably the latter.

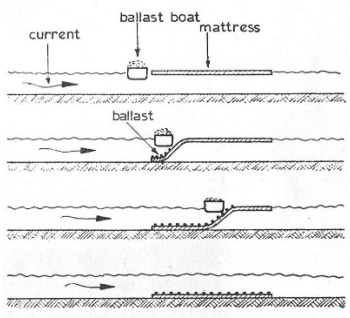


Fig. 17.8 Fascine mattress being ballasted and submerged in the current

As the water depth is shallow (only 2.10 m at LWS), you can gradually ballast and submerge the fascine mattresses during the ebb or flood (Fig. 17.8) and there is no need to submerge the whole ballasted mattress instantaneously during the turn of the tide, which was, according to Hawkshaw, very short. Still, a good reason to reject the idea of (a) fascine dam(s) could be the absence in England at that time of contractors experienced in this type of work.

17.5.2 The cofferdam

The closure of the drain by means of a double wall of sliding panels lowered into the grooves of twin piles (i.e. a speedy closure) is inventive. But one cannot consider it as hydraulic engineering, it looks more like the work of a furniture maker. In this respect it is odd that the notion 'tolerance' is neither mentioned in the text nor shown on the drawings: the sliding panels fit exactly in the space available between the grooves. It is not surprizing that four out of 2 x 12 bottom panels were torn out of the grooves of three twin piles "owing probably to their having been injured in driving". It is in fact rather amazing that apparently all the other panels could be lowered without problems!
It is not understood why Hawkshaw is so anxious about the bottom of the inside of the cofferdam: to avoid piping in the future, one did not want to deposit stone or gravel and where, consequently, scour kept occurring. But:

> "after these panels were placed sheet-piling was driven behind them on the lower side of the dam, thereby filling in all the spaces between the twin piles, and converting the lower side into one continuous row of sheet-piling…".

17.5.3 Quantities and production

It is assumed that, on 19 June, the whole cofferdam had to be filled from scoured bottom (at LWS - 3 m) and slopes of drain up to the top of the placed panels (LWS + 3.7 m). This implies a height of 6.7 m. The quantity of fill to be placed during six hours inside the cofferdam would then be 1 436 m^3. A global calculation of the fill to be placed in the aprons comes to 834 m^3. Probably, both figures are too high because Hawkshaw writes:

> ".. so perfect were the arrangements, that the raising of the dam inside was kept in advance of the rise of the tide".

As the low water level during a neap tide must have been about LWS + 0.5 m (i.e. 3.5 m above the bottom of the scoured drain), the cofferdam, probably, had already partly been filled on the day before.
Anyhow, Against the maximum quantity of fill to be placed: 1 436 + 834 = 2 270 m^3 stands Hawshaw's statement:

> "About 3 000 yards of clay with several hundred sacks of gravel and clunch were prepared for the operation".

"About 3 000 (cubic) yards" comes already to 2 200 m^3 and "several hundred sacks of gravel and clunch" to 50 – 100 m^3.
As 800 men were available for the operation, the maximum output per manhour could have been around 0.5 m^3. But probably the overall quantity to be placed was less than 2 270 m^3 and, accordingly, the output per manhour was less.

CHAPTER 18 The repair of breached dikes in Japan[7]

18.1 Introduction

Also in Japan dikes breached in the past and, subsequently, were repaired. Little information could be traced about such repairs. But, as, once again, the method(s) utilized differ from those applied elsewhere, it is worthwhile to have these methods, though rather sketchy, described.

18.2 Materials and constructions used in closures

18.2.1 Materials as described by De Rijke

De Rijke (1900a) published an article in the weekly De Ingenieur entitled '*Banjirs en vloeden in Japan'*. He states that during floods huge masses of stones, gravel, sand and silt are transported which results in rising beds of rivers followed by overtopping and breaching of flood embankments. Various constructions are used for bank protection and repair of breached dikes.
Here, only the constructions used for repair of dike breaches are shortly described.

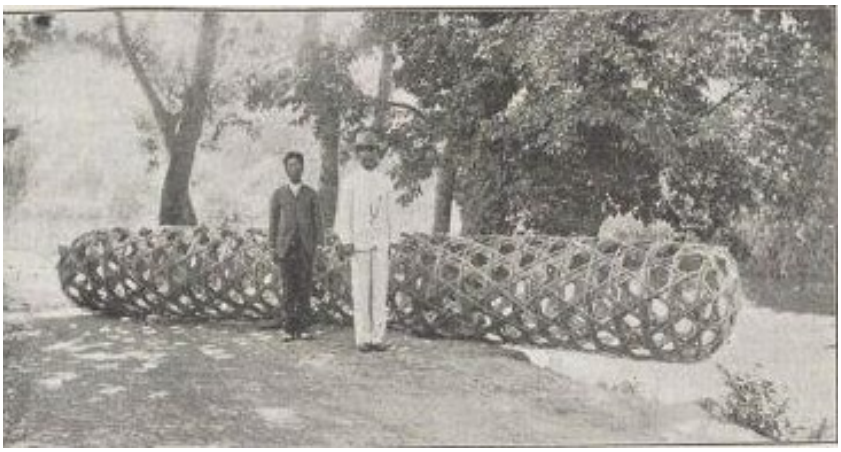


Fig. 18.1
Yakago, bamboo basket to be filled with boulders; source: De Rijke (1900, 8)

Yakago[8] (stone baskets)
These are elongated cylindrical baskets made of bamboo (Fig. 18.1) which are filled with boulders. The length is approximately 10 m and the diameter 0.70 to 1.20 m. The yakago shown in Fig 18.1 has a mass of 15 000 kg. Therefore, a yakago, when placed horizontally at the desired location, is still empty and subsequently filled through the meshes between the bamboo strips.
When it is placed during a flood along an eroding embankment the position is vertical or nearly vertical and only the lower end is in first instance filled with stones to keep the yakago at the desired location. Subsequently, when positioned at the right angle against the bank, the yakago is filled from the top. De Rijke notes that two or three rows of these yakago placed close together, provide a very good bank protection but as the bamboo will deteriorate in time the protection is of a temporary nature. For that reason yakago, like the one in Fig. 18.1, is sometimes reinforced by a net of galvanized wire.
Ushi (cows)
Ushi are sloping frameworks of tree trunks and thick bamboos (Fig. 18.2). A number of yakago are laid on the long tree trunks as shown in Fig 18.3 and filled with boulders. Shorter yakago are tied to the end of the ushi. The flexibility of the yakago is clearly demonstrated (right side of Fig. 18.3).

[7] The author is greatly indebted to Satoshi Nakazawa (Associate Professor, Graduate School of Humanities and Social Sciences, Hiroshima University, Japan) who provided illustrations and quotations from old Japanese books concerning the subject of this chapter.
[8] De Rijke (1900).

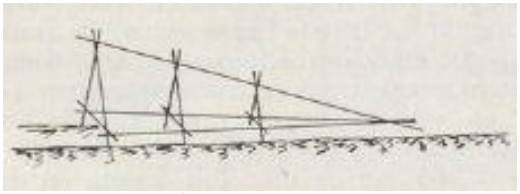


Fig. 18.2
Ushi (cow), a frame work of tree trunks and thick bamboos; source: De Rijke (1900a, 546)

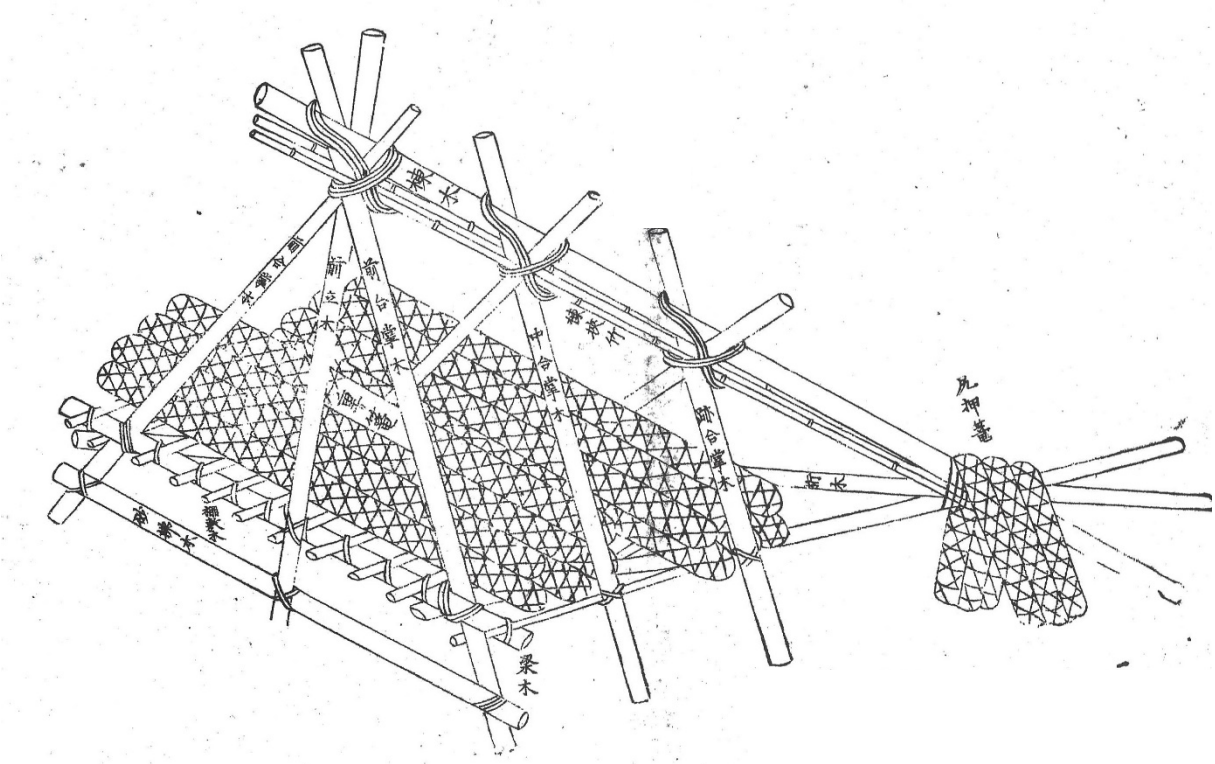


Fig. 18.3 Ushi, filled with yakago; source: Takatsu Giichi (1881, Vol. 3, fig. 1)

Fig. 18.4
Yakago applied in a Japanese groyne; Source: National Archives, The Hague; Access code 2.21.371, Inventory of the archives of the Family Escher, 563-b, photographs Grandfather Escher, Japan

18.2.2 Ishidawara

An ishidawara is a bale filled with stones. Ishi means stone while a tawara is a kind of large bale made of rice-straw, strenghened by rope (Fig. 18.5),normally used to pack rice or cabbages. When no stones are available the bale is filled with earth, sand or even grass. The tawara's , filled with stone, together with the branches of a felled tree (to which they are attached) function as a kind of bed protection (Fig. 18.6).

Fig. 18.5
Tawara, bale made of rice-straw and rope; accessed on 1 Nov. 2023 at: http://komedawara.net/product/米俵/

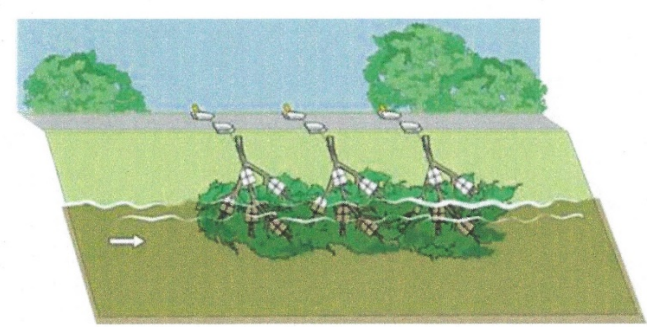


Fig. 18.6
Kinagashi-work, comprising felled trees and tawara's attached to the branches to keep the branches firmly on the river bed, thus preventing scour;
accessed on 1 Nov. 2023 at: https://www.ric.or.jp/bousai/suiboutxt/03-1kinagashi.html

18.2.3 Scaffold-type structure made up of piles, walings and struts

The function of the scaffold – type structure in Fig. 18.7 is to prevent the tawara's and other closure materials, like brushwood and bamboos, from being removed by the current.

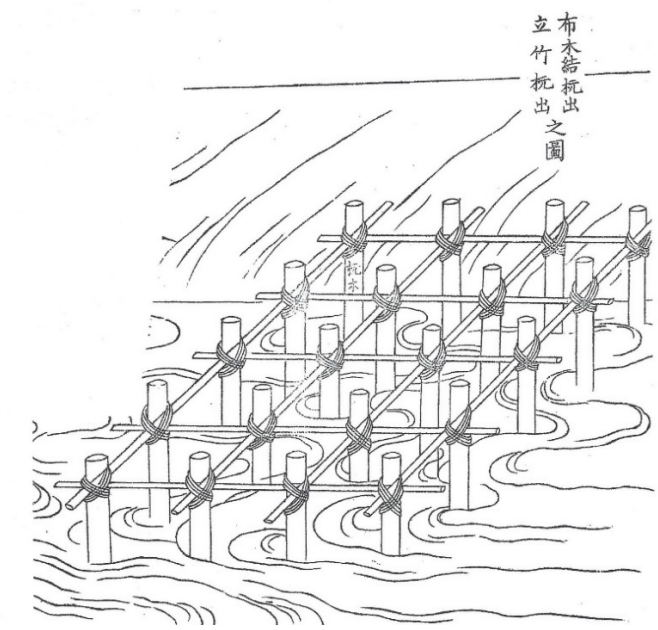


Fig. 18.7
Scaffold-type structure constructed in the current through the breach in a dike;
source: Takatsu Giichi (1881, Vol. 3, fig. 75)

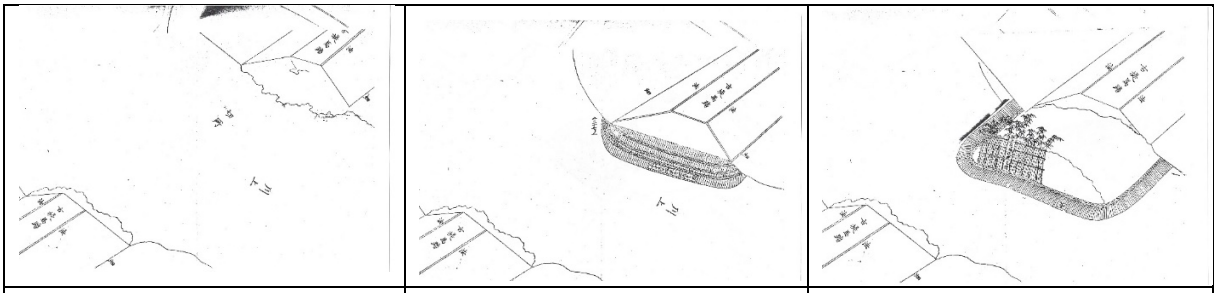
18.3 Closure methods

18.3.1 Perimeter bund

If placed in a row, close together, the ushi form the front of a perimeter bund around the scour hole of a breach. It is assumed that this perimeter bund would be constructed after the flood has passed and the base for the bund is more or less dry[9].

18.3.2 Closure of a flow gap (project no. 74)

Regrettably, De Rijke does not mention how one used to close a flow gap in Japan. However, some illustrations were found in a book[10] which' title 'Doboku Ko Yoroku' can be translated as 'Review of civil engineering works'. The various stages of the closure of a flow gap in a breached flood embankment are clearly shown (Fig. 18.8). It could not be established, by what materials or constructions thereof, the narrowing of the gap was achieved; but for the final closure 36 nos. yakago have been used[11].This would (for instance) enable closure of a gap, 3 m deep, length at river bed 6 m. slopes 1;1, length at crest level 12 m.



breach	start of closure	closure works well advanced
only a narrow gap left	closure and earth works complete, slope protection in progress	

Fig. 18.8
Five stages of the closure of a flow gap in Japan;
source: Takatsu Giichi (1881, Vol. 3, figures 66 - 70)

18.3.3 Closure of a flow gap (project no. 75)

The afore mentioned book also shows various stages of a closure comprising the use of tawara's and scaffold type structures.
In Fig. 18.9 the flow channel outside the actual breach has been protected against scour by dumping the kind of kinagashi as shown in Fig. 18.6.
In Fig. 18.10 the impact of the current on the main scaffold has been reduced by filling up a scaffold placed upstream with bundles of bamboos.

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[9] This assumption does not necessarily match the statement made by De Rijke (1900a, 546) which reads: "I have seen that such a dam, having a length of 200 m, was made in three days, while the current velocities in the river were still quite high." But is also possible that, initially, only the ushi were placed and the yakago inserted later.
[10]Takatsu Giichi {Naimusho Dobokukyoku (Public Works Department, Ministry of Internal Affairs)} (1881); Doboku Ko Yoroku; Tokyo, Ministry of Internal Affairs.
[11] Takatsu Giichi (1881, Vol. 1, sheet 139, right)

Finally, in Fig. 18.11 the breach is closed by dumping tawara's attached to branches of brushwood. Apparently, the brushwood prevents the tawara's from being transported away by the current.
In Appendix M some quotations translated from texts in various old Japanese books have been copied.

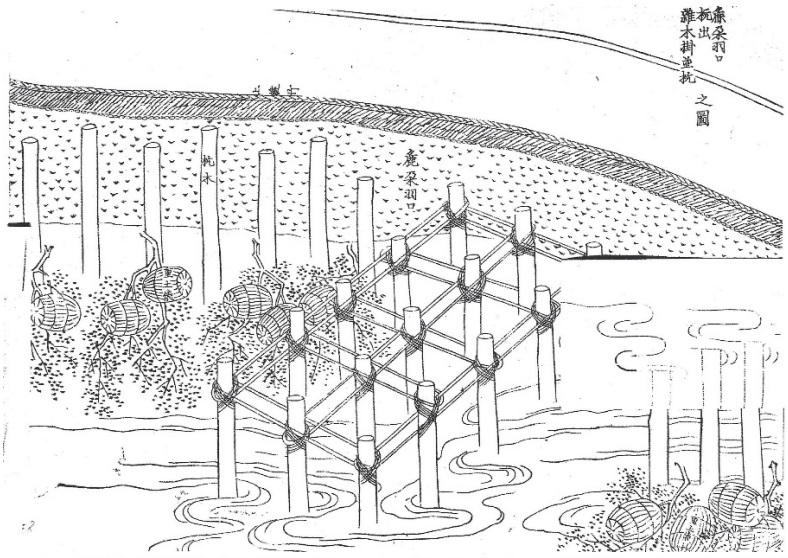

Fig 18.9
Kinagashi dumped outside actual breach to reduce or prevent scour

Fig 18.10
Scaffold filled with bundles of bamboo upstream of scaffold in breach

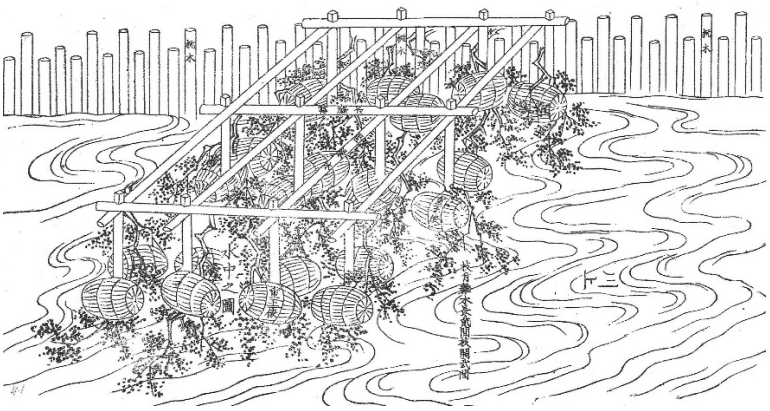

Fig. 18.11
Flow gap closed by placing tawara's and brushwood inside scaffold; note the row of piles along the dike head preventing widening of the breach by scour

Source of Figures:
Takatsu Giichi (1881, Vol. 3, figures 72, 75 and 76)

18.4 Repair works of the flood embankment of the river Yodo at Hirakata

In a letter dated 11 April 1886 from J. de Rijke to G.A. Escher, the former writes about the breaching of the flood embankment of the river Yodo at Hirakata. Van Gasteren et al (2000, 333) mentions this letter and shows a drawing of the repair works being carried out. The drawing (Fig. 18.12) is not very informative but in any case it is clear that labourers are carrying tawara's to the closure site and that tawara's have already been incorporated in the slope protection of the repaired section of the embankment.

Fig. 18.12 Repair works at the flood embankment of the river Yodo at Hirakata; source: Osaka – Nakanoshima Prefectural Library, Osaka, Japan

PART IV Synthesis of the case histories discussed in Parts II and III

Table IV-1 Overview of the case histories discussed in Parts II and III

project no.	country	project	year of completion	purpose of project
1	Nether-lands	Oost- & West- Hinkele polders, island Zuid-Beveland		reclamation for agriculture
1a		Dam 2	1263	
1b		Dam 4	1269	
2		Groote Waard, new dike Broek-Zevenbergen	1410 – 1412	flood control, peat digging
3		Groot Zuid-Beijerland		reclamation for agriculture
3a		Borrekeen, Dam A	1631	
3b		Grote gat, Dam B	1631	
4		Jonkvrouw Anna polder, island Noord-Beveland	1727	
5		Anna-Frisopolder, island Noord-Beveland	1747	
6		Reigersbergsche polder, island Zuid-Beveland	1773	
7		Henriëttepolder, island St. Philipsland	1776	
8		Eendragtpolder, Oost- Zeeuws-Vlaanderen	1777	
9		Wilhelminapolder, island Zuid-Beveland	1809	
10		Catharinapolder, Oost Zeeuws-Vlaanderen	1846	
11		Mortierepolder, island Walcheren	1846	
12		Perponcherpolder, island Zuid-Beveland	1847	
13		Anna-Jacoba + Kramers polder, island St. Philipsland	1847	
14		Willem III polder, Oost Zeeuws-Vlaanderen	1861	
15		Zwinpolder, West Zeeuws-Vlaanderen	1864	
16		Schenge polder, island Zuid-Beveland	1874	
17		Van Lyndenpolder, Oost Zeeuws-Vlaanderen	1876	
18		Sluispolder, island of Tholen	1877	
19		Dames polder, island Zuid-Beveland	1884	
20		Hertogin Hedwige polder. Oost Zeeuws-Vlaanderen	1907	
21		Hoogerwaardpolder, (mainly) island Zuid-Beveland	1912	
22		Dijckmeester polder, Oost Zeeuws-Vlaanderen	1918	
23		Kreekrakpolder, island Zuid-Beveland	1923	
24		Hellegatpolder, Oost Zeeuws-Vlaanderen	1925	
25		Biesboschpolder, Zuid Holland	1926	
26		Abraham Wissepolder, island St. Philipsland	1935	
27		closure tidal inlet Dijkwater	1954	flood control
28		Statendam, between islands Goeree & Overflakkee	1751 – 1765	transport. navigation, accretion
29		Kreekrakdam, between isl. Zuid-Beveland & N. Brabant	1867	railway and road link
30		Sloedam, between islands Walcheren & Zuid-Beveland	1871	railway and road link
31		IJdam, enclosing the tidal inlet IJ at Schellingwoude	1872	navigation and road link
32		Dam from Frisian coast to the island Ameland	1878 & 1882	to further accretion (failed 1882)
33		Damming the Slaak between the island St-Philipsland &		
33a		Noord-Brabant Slikdam overflow dam	1858	to further accretion (failed 1863)
33b		Slaakdam non-overflow dam	1884	road link & to further accretion
34	Germany, Schleswig -Holstein	Nordstrander-damm overflow dam	1907	to further accretion (failed repeatedly)
		non-overflow dam	1935	road link & accretion
35		Hindenburgdamm	1925	railway link
36	Nether-lands	repair of breach in Spaarndammer dike, Zuid-Holland	1510	flood control
37		flow gap at Leverhuyzen near Zierikzee	1578	flooding at time of war
38		repair of breach sea dike near Scharwoude, N-Holland	1676	flood control
39		repair of breach in Lekdijk, bovendams, Wyck, Utrecht	1747	
40		repair of breach in dike Nieuwerkerkepolder, Walcheren	1807	
41		repair of breach in sea dike Kruininger polder, Z- Beveland	1808	
42		repair of breaches in dikes 'English polders', Zuid-Beveland	1856 - 1864	reclamation for agriculture
43		repair of breach in sea dikes south of Edam, Noord Holland	1916	flood control
44		repair of breach, Noorddijk, Papendrecht	5-2-1953	
45		repair of breach, Nieuwe Stadspolder, island of Dordrecht	25-2-1953	
46		repair of breach at Jillesweg, Polder Oudeland, Goeree	27-2-1953	
47		repair of two breaches Kleine Zuiderpolder, Goedereede	21-4-1953	
48		repair of eastern breach Everingerpolder, Zuid-Beveland	8-3-1953	
49		repair of breach Zuidvlietpolder, Zuid-Beveland	7-2-1953	
50		repair of western breach Reigersbergse polder, Z-Beveland	19-2-1953	
51		compartmentalization, 4de weg inside Reigersbergse polder	11-3-1953	
52		repair of breaches in Kadijk, between Kruiningen & Waarde	23-4-1953	
53		repair of breach no. 13, Lavendeldijk, sec. dike, Z-Beveland	12-3-1953	
54		repair of breaches Kl. Van Steelandpolder, island of Tholen	5&7- 2-1953	
55		repair of breach Anna Vosdijkpolder, island of Tholen	20-2-1953	
56		repair of two breaches Willempolder, St. Philipsland	7-3-1953	
57		repair of breach Suzannapolder, island of Walcheren	7-1953	
58	Ghana	closure of flow gap at Keta lagoon	1969	
59	Bangla-desh	closure of Chakamaya creek	1979	reclamation for agriculture
60		closure of Madargong creek	1980 (failed)	
61		closure of Amtali creek	1982	
62		Feni River closure dam	1985	reservoir for irrigation
63	China	closure of breach at Lai-t'ung-chai	1889	flood control in the Yellow river
64		closure of breach at Liu-wang-chuang	1903	
65		closure of breach in Eastern Shantung	1923	
66		closure of breach in Western Shantung	1925	
67		closure of breach east of Litsin, Shantung	1928	
68		closure of breach at Tung Chuang	1936	
69		closure of breach near Kai Feng Fu	1947	
70	Bangla-desh	closure of Gangrail creek	1977 (failed)	reclamation for agriculture
71		closure of Nalian river	2020	
72	England	closure of the Dagenham breach near London	1720	flood control
73		cofferdam in the main outfall drain of the Middle Level	1862	
74	Japan	closure of flow gap nos. 1 in a breached river embankment	probably 19th century	
75		closure of flow gap nos. 2 in a breached river embankment		

Chapter 19 Review of the outcome of the case histories

19.1 Introduction

In Parts II and III of this Dissertation an overall total of about 75 case histories regarding closures has been presented (see Table IV.1 for an overview). The information, available to carry out an analysis on certain technical aspects of these closures, varied from 'very limited' to 'nearly complete'. Nevertheless, an effort will be made to compare individual and/or groups of closures with one another. Moreover, some general conclusions will be drawn regarding the possibility to apply simple means to closures and what conditions have to be fulfilled by a closure project to have it carried out by simple means.

19.2 The flow through the final closure gap

19.2.1 River closures

Out of 75 case histories only seven concern a river closure, i.e. a closure in which tidal flows do not play a role and only river discharge flows through the breach.
The information available about the closure of the Lekdijk bovendams (Ch. 9, Section 9.5) is not sufficient to make remarks about the discharge through the breach during closure. But from Woertman's notes[1], it would appear, that, at the time of closure, discharge must have been small or even nil.
Also the information about two river closures in Japan (Ch. 18) and three (out of four) in China (Ch. 13) is too limited to draw quantitative conclusions.
River closures are difficult because they have to be made in a continuous flow going in landward direction through the breach without any period of slack like a tidal flow. But the advantage is that, as the gap narrows, the flow through the breach increasingly diverts towards the river channel downstream. This is very well demonstrated in Ch. 13, Sub-section 13.6.3.2 (Fig. 13.16) and in Appendix K which present information about the closure of the 'crevasse' in the embankment of the Yellow River at Tung Chuang. The south flood embankment breached at six places during a river discharge of 14 000 m^3/s. After narrowing and closing most of the breaches a last one, long 834 m, was left. In order to bring the river back in its original bed six cut-off channels were dug and the aforementioned breach was narrowed to 40 m. Closure of this gap, using sink-fascines, took nearly six days. Appendix K shows water levels and discharges for the last three days of the closure. At the start of the measurements the discharge through the river, upstream of the gap, is 840 m^3/s of which 310 m^3/s flows through cut-off channels downstream and 530 m^3/s through the gap. On the day of final closure this discharge through the cut-off channels gradually increases from 600 to 890 m^3/s while the discharge through the gap in that same period (18 hours[2]) goes down from 150 to 20 m^3/s. Still, at the same time the head over the gap increases from 1.9 to 2.25 m.
An effort was made to calculate the current velocity through the gap at the start of the hydrometric measurements and this resulted in a current velocity of 5.3 m/s. As the initial horizontal closure during the last day will become a vertical closure, the current velocity will decrease as the height over the 'sill' will decrease[3].
It is interesting to note that a river closure using sink-fascines is possible, even when current velocities are in the order of 5 to 6 m/s, always provided that the sink-fascines are anchored at the upstream side.

[1] Woertman (1747, 65).
[2] In that period the upstream water level in the river goes up by six cm which explains the increase in overall discharge.
[3] For further details see Chapter 13, Sub-section 13.6.3.2.

19.2.2 Tidal closures

Table 19.1 Particulars of successful tidal closures selected from the case histories presented in Parts II and III

Project no.	country	project	purpose	area tidal basin (km^2) during neap tide	tidal volume in million m^3 through: breach or tidal channel neap tide	tidal volume in million m^3 through: breach or tidal channel spring tide	tidal volume in million m^3 through: final closure gap (neap)	length of final closure gap(s) (m)	max.-current velocity in final closure gap (m/s)	type of closure	chapter & section or Box
	1	2	3	4	5	6	7	8	9	10	11
1 1a 1b		Hinkele Dam 2 Dam 4	reclam-ation for agricul ture	 0.40 0.54 0.46 0.61			 0.8 1.1 0.9 1.2	 40 40 40 50	 1.1 1.4 1.2 1.3	probably: - vert. / hor. - gradual as well as speedy	7, Box 7.1
2		Broek-Zevenbergen	flood contr. peat digging	15			4.65	87.5	3.64	- horizontal - gradual and instantaneous	7 , Box 7.2
3 3a 3b		Groot Zuid-Beijerland Dam A Dam B		 not calc. 0.214			 not calculated 0.37	 25	 0.9	probably: - vert. / hor. - gradual as well as speedy	7 , Box 7.3
4		Jvr Anna polder		0,01			0.18			no information traced, proba-bly horizontal	7, Section 7.6
5	Nether-lands	Anna-Friso plr.		0.009			0.02	69			
8		Eendragt plr.		0.057			0.11				
13		Anna-Jacoba + Kramers plr.	reclama-tion for agricul-ture	1.2			3.1	300 & 600		-vert. / hor for south creek, - hor. north crk.	
16		Schenge plr.		0.53			1.7			- vert./hor.	
19		Dames plr.		0.21			0.28	25 & 350		no information traced, proba-bly horizontal	
20		Hedwige plr.		0.32			0.42				
21		Hoogerw. plr.		0.14			0.19				
22		Dijckmeester polder		0.5 (est.)0,5			2.0	65 TB & 78.5 TB	2 (in 65 m gap)	mainly horizon-tal; speedy	
24		Hellegat polder		not calc.			not calculated	35		mainly vertical, speedy	
25		Biesbosch plr.		not calc.			not calculated	55 TB & 77 TB		- hor. /vert. - speedy	
26		Abr. Wisse plr.					0.05				
27		closure tidal inlet Dijkwater	flood control				1.1	60 TB		- vertical - speedy	7, Sub-section 7.10.2
30		Sloedam	railway link	not applicabl e			not calculated	140	3 -4	- vert. /hor. - gradual	8, Section 8.4
36		Spaarn-dammer dike	repair of breaches	150	25.5	31.5	3.0	10	1.6 – 2	- horizontal - gradual & instantaneous	9, Section 9.2
38		sea dike near Scharwoude		100	20	26.7	23.6	23 + 54	1.5 – 1.6	- horizontal - gradual	9, Section 9.4
41		sea dike Kruininger plr.		13.2	9	33	9	1130	1.5 – 2	- vert./ hor. - gradual	9, Section 9.7
43		sea dikes in 1916		140			42	120 + 47	1.7 – 1.9	- vert./ hor. - gradual	9, Sub-sect-ion 9.9.4
44 to 57 (incl.)		closures in 1953		0.1 – 21.5 average 6 (28 breaches)		0.1 – 7 average 1.3 (25 breaches)	0.1 – 7.1 average 1.7 (11 closures)	16 – 260 average 60 (14 clos-ures)	0.2 – 4.95 average 21 (11 closures)	elaborated in the text	10, Sub-section 10.16.3, Tables 10.1 & 10.3
58	Ghana	flow gap at Keta lagoon	repair of cut, sand ridge	< 300			4 to 5	45	2.5	- horizontal - gradual & speedy	11, Section 11.2
59		Chakamaya	reclamati-on for a-griculture	3	4	10	4	213 TB	2 – 3	- vertical - hor. or v/h - gradual	11, Section 11.3 Table 11.4,
60		Madargong		2	3.3	7	3.3	160 TB	1.6 – 3.5		
61		Amtali		2.3	2.5	6	2.5	150 TB	2 – 3.7		
62	Ban-gla-desh	Feni River closure	reservoir for irriga-tion	12	3.9	25	nearly nil (as most of ebb flow diverted through sluice)	775 (21 open-ings, long 25 to 45 m)	3	- vertical - speedy	11, Sub-section 11.4.2
70		Gangrail	reclamati-on for agri-culture	1.15	2.4	3.5	1.7	15	3.45	hor./vert. - gradual & speedy	14, Section 14.3
71		Nalian		4.8	12	19	9.1	83	4.4	vert./hor. - gradual and speedy	15, Box 15.1
72	Eng-land	Dagenham near London	repair of breach	1.1	4	16.4	12,9 (springtide)	122	5,35	vert./hor. gradual	16, Sections 16.4 and 16.5
73		Middle Level Drain, Lincolnshire		35	18.3	20.5	7.3	15	5.6	hor. / vert. instantaneous	17, Section 17.4

19.2.2.1 Tidal volumes through breaches, tidal channels and final closure gaps

In Table 19.2 the figures of Table 19.1, column 7 and some additional ones derived from Ch. 10, Table 10.3 have been summarized.

Table 19.2 Tidal volumes through final closure gaps from Tables 19.1 and 10.3

volume in million m^3	from Table 19.1	from Table 10.3	totals	as percentage of overall total
≤ 2	17 nos.	8 nos.	25 nos	61 %
2.1 – 5	7 ,,	2 ,,	9 ,,	22 %
6 – 42	6 ,,	1 ,,	7 ,,	17 %

The 'large' tidal volumes (say ≥ 2 x 10^6 m^3) are either due to a low ground level in in the area (flooded or to be reclaimed) or to a water body (creek, lake) present behind the closure. Nevertheless one may say in general that closures can be constructed by using simple means when tidal volumes are lower than 5 million m^3. As follows from the case histories presented in Table 19.1, larger tidal volumes can also be accommodated when maximum current velocities are lower than 3 m/s in the closure gap.

19.2.2.2 Selection of closure projects from the case histories

From the 75 case histories mentioned earlier, 68 concern tidal closures. In Table 19.1 a selection[4] is presented of 31 closure projects[5] from the case histories given in Chapters 7 – 11 and 14 – 17. In the columns 4 to 10 of this Table the most important technical particulars of tidal closures are shown. Below these particulars are discussed in more detail.

19.2.2.3 The area of the tidal basin during a neap tide

In most cases the area of the tidal basin (column 4 in Table 19.1) during a neap tide is smaller than 2 km^2. If the area is large (for instance the lakes behind the breach in the Spaarndammer dike or the polders behind the breached sea dike at Scharwoude), it does not mean that the tidal volume (columns 5 to 7) will increase in proportion to that of the size of the area. In fact the tidal volume through the final closure gap (column 7) is much more determined by the ground level of the area in its relation to the tidal range. The ground level in the reclamation schemes of former centuries and in the polders behind the dikes breached during the storm surge flood of 1953 was relatively high[6] and this explains for the greater part the possibility to construct closures by applying simple means.

19.2.2.4 Length of final closure gaps

As can be seen in Table 19.3, the length of the final closure gap (column 8 in Table 19.1) is in almost half of the cases quite modest (less than 50 m). But when a jetty (temporary bridge, marked as TB in column 8 of Table 19.1) is used in order to transport closure materials for dumping in the closure gap, it usually will be longer than 50 m.

Table 19.3 Lengths of final closure gaps

length in m	from Table 19.1	from Table 10.3	from Table 10.1	totals	as percentage of overall total
0 – 50	13 nos.	10 nos.	1 nos.	24 nos.	48 %
51 – 100	9 ,,	3 ,,		12 ,,	24 %
101 - 1130	11 ,,	2 ,,	1 ,,	14 ,,	28 %

Perimeter bunds as part of breach repairs (like the one at the Kruininger polder: 1130 m) are usually quite long. But in case of a horizontal closure and a modest tidal range (as at Spaarndammer dike and the breach at Scharwoude) the final closure gap can still be limited in length.

[4] A selection based on the availability of relevant information.

[5] In which the 14 closures of breaches caused by the storm surge flood of 1953 are considered in the Table as one project, so actually 44 closure projects are included.

[6] This statement is only valid for the case histories about breaches and closures discussed in this dissertation. It does, for instance, not apply to the polder Schouwen.

19.2.2.5 Maximum current velocities in final closure gap

The maximum current velocities (column 9 in Table 19.1) in the final closure gaps for 50 % of the cases, are smaller than 2 m/s and for 38 % between 2 and 4 m/s. Only in four closures higher current velocities (4.4 to 5.6 m/s) were observed (or calculated).

Table 19.4 Maximum current velocities in 32 final closure gaps

max. current velocity in m/s	Table 19.1	Table 10.3	totals	as percentage of overall total
0 – 2	10 nos.	6 nos.	16 nos.	50 %
2.1 – 4	8 ,,	4 ,,	12 ,,	38 %
4.1 – 5	3 ,,	1 ,,	4 ,,	12 %

The current velocities in the closure gaps exceeding 2 m/s could be accommodated by taking special measures like screening of the area of dumping or other actions as described in the various case histories.

19.2.2.6 Types of closure

Types of closure were discussed in Ch. 5, Section 5.6. A distinction was made between vertical and horizontal closures and combinations thereof. A distinction was also made in terms of time: one can speak about gradual, speedy and instantaneous closures. But, as shown in Table 19.1 (column 10), a closure starts sometimes as a gradual closure and ends up as a speedy closure. In the case histories, it is demonstrated that what is done in practice depends on the particular circumstances. To mention a few of the latter: depth and length of the breach or tidal channel to be closed; tidal range; geotechnical characteristics of the subsoil; available closure materials and equipment, skills and available manpower. A few general rules, regarding the type of closure to be applied, are the following:

- When the breach or tidal channel runs (nearly) dry during Low tide, a horizontal closure can be applied. In case the bed of the tidal gully is easily erodible a bed protection must be made prior to the start of narrowing. The narrowing of the closure gap will induce an increase in the current velocities. When these current velocities start to damage the bed protection and/or prevent further narrowing (closure materials being washed-out) the gradual closure must change into a speedy closure.
- When the breach or tidal channel has a depth below Low tide, a horizontal sill should be constructed, having its crest more or less at the level of MLT. This is in fact a vertical closure carried out by means of waterborne transport. Also here, a bed protection preventing scour and, in addition, stabilising the foundation, might be required. The closure works can now be continued by, either a vertical closure by dumping closure materials from a jetty across the final closure gap, or by a horizontal closure from both ends of the closure gap. In the former case the closure is gradual, in the latter speedy. A combination, i.e. horizontal/vertical closure on this sill is also possible.

19.2.2.7 When is a closure by simple means possible?

From the above it can be concluded in general that tidal closures can be constructed by simple means, provided:

- tidal volumes are ≤ 5 million m^3, 34 nos.
- length of final closure gap is ≤ 100 m, 36 nos.
- maximum current velocities in closure gap are ≤ 4 m/s, 28 nos.

But there are always exceptions to the rule! And measures are then taken to mitigate the disturbing factor. For instance, in closures at Feni (Ch. 11, Section 11.4) and Dagenham (Ch. 16) the tidal flows were diverted through sluices. And the long closure gaps at the Kadijk breaches (Ch. 10, Section 10.10) and Feni (Ch. 11, Section 11.4) were sub-divided into smaller closure gaps.

Also, the long closure gap created by the perimeter closure bund at Kruiningerpolder was no problem because of the high ground level.

19.3 Closure materials

19.3.1 General

In the case histories a great number of different closure materials are mentioned. The closure materials discussed are mainly natural materials, i.e. either inorganic materials, like stone (sand, gravel), clay, bricks, or materials of a vegetal nature like wood, reed and plant fibres (sisal, hemp, and jute). These materials have in common that they have to be available at or near the particular closure site. This condition is imposed by the need to limit as much as possible the cost of a closure. But it also implies that the type of closure material available is a function of the particulars of the geographical area and its climate. In Bangladesh, for instance, no brushwood, for incorporation in 'fascine' mattresses, could be found which would float, and palm leaves and reed were used instead. In China millet stalks had the role entrusted in Europe to brushwood (*rijshout*).
More recently (1985 and 2020) also synthetic fibres have been used like polypropylene (as geotextile membrane in bed protection mattresses), polyester (as geotextile, woven or nonwoven, in geobags, synthetic bags) and nylon (ropes). In this section an effort is made to group all closure materials, and the combinations thereof, on the basis of their function. The following functions can be distinguished:

- prevention of scour;
- 'fill' in the closure bund;
- 'impermeable' retaining wall as achieved by sheet piling, sliding panels or sunken ships;
- ballast on or inside light constructions like fascine mattresses and sink-fascines;
- screening of the dumping area;
- structural components in an abutment, scaffold, jetty, cofferdam, etc;
- keeping materials together: bags, tawara's (Ch. 18, Fig 18.5), ropes;
- 'anchoring' of closure constructions to a fixed point: hawsers, ropes.

Below, these functions will be discussed in more detail and the relevant materials, as well as the combinations thereof, mentioned. Also the future applicability of these materials and their combinations will be looked at in the light of local availability, possible alternatives, local skills, cheap labour and cost.

19.3.2 Prevention of scour[7]

The various measures to prevent scour are summarized in Table 19.5.

Table 19.5 Measures to prevent scour

prevention of scour is achieved by:	proj. no.	project	year	reference
immersion of suspended mattresses	-	Huisduinen, coastal protection, NL	1664	Ch. 4, Section 4.3
immersion floating 'fascine'[8] mattresses	30	Sloedam, NL	1871	Ch.8, Section 8.4
ballasted straw or reed mats	58	Keta lagoon, Ghana	1969	Ch. 11, Sub-section 11.2.3
placing sink-fascines or mata's	2	new dike Broek-Zevenbergen, NL	1410- 1412	Ch. 7, Sub-section 7.4.5
dumping of a layer of clay clods	3b	Groot-Zuid Beijerland, Dam B, NL	1631	Ch. 7, Sub-section 7.5.3
a row of sheetpiles up to MLT	72	Dagenham breach, England	1720	Ch. 16, Section 16.4
dumping a layer geo- and synth. bags	71	Nalian closure, Bangladesh	2020	Ch. 15, Sub-section 15.3.2
kinagashi work	75	closure of a breach, Japan	19th cent.	Ch. 18, Fig. 18.6

19.3.3 Fill in the closure bund

The various possibilities to fill the closure bund are summarized in Table 19.6.

[7] The emphasis in this and following sub-sections is on items actually used within the context of simple means as presented in the case histories. So the dumping of stone as a protection layer or the immersion of polypropylene bed protection mattresses (like done for the closure of the Feni River) will be ignored.
For each item one or more references are given but this list of references is not necessarily complete.
[8] The 'fascines' in the mattress may consist of brushwood (*rijshout*), palm leaves or reed.

Table 19.6 Fill in the closure bund

fill in the closure bund may comprise:	project no.	project	year	reference
stacked immersed floating fascine mattresses	30	Sloedam NL	1871	Ch. 8, Section 8.4, Fig. 8.10
	41	dike Kruininger polder NL	1808	Ch. 9, Section 9.7, Fig. 9.25
imm. suspended fascine mattress(es)	-	(no example) NL	prior to ± 1700	Ch. 4, Section 4.3
dumped clay clods	22	Dijckmeesterpolder NL	1918	Ch. 7, Fig. 7.21
straw packs	48	Everingepolder NL	1953	Ch. 10, Section 10.6
pakwerks (fascines and earth)	30	Sloedam NL	1871	Ch. 8, Section 8.4, Fig. 8.15
pakwerks (millet stalks and earth)	64	Liu-wang-chuang, China	1903	Ch. 13, Section 13.6.1
dumped/placed sandbags or clay bags	58	Keta Lagoon, Ghana	1969	Ch. 11, Section 11.2.5
	59, 60, 61	Chakamaya, Madargong, Amtali, Bangladesh	1979, 1980 1982	Ch. 11, Sub-section 11.3.5.2
	62	Feni, Bangladesh	1985	Ch. 11, Sub-section 11.4.4
	71	Nalian closure, Bangladesh	2020	Ch. 15, Sub-section 15.5.3
miscell. materials inside a cofferdam	38	sea dike Scharwoude NL	1676	Ch. 9, Section 9.4
sink-fascines (= mata, rolled kids, 'willow-stone sausages') [9]	2	new dike Broek-Z' bergen	1410-1412	Ch. 7, Sub-section 7.4.5
	70	Gangrail, Bangladesh	1977	Ch. 14, Sub-section 14.2.3
	68	Tung Chuang, China	1936	Ch. 13, Sub-section 13.6.3

19.3.4 'Impermeable' retaining wall

In a few cases a flow gap is closed by driving sheet piling or a row of piles, closely placed to each other. Such a 'impermeable' wall can also be made by lowering sliding panels or by sinking ships. In these last cases it concerns an instantaneous closure.
For relevant references see Table 19.7.

Table 9.7 'Impermeable' retaining wall

'impermeable' wall:	project no.	project	year	reference
sheet piling, row of piles	35	Hindenburgdamm, Germany	1927	Ch.8, Section 8.9
	36	breach Spaarndamse dijk NL	1510	Ch. 9, Section 9.2
	38	sea dijk Scharwoude NL	1676	Ch. 9, Section 9.4
sliding panels	35	Hindenburgdamm, Germany	1927	Ch.8, Section 8.9
	73	Middle Level cofferdam England	1862	Ch. 17, Section 17.4
Sunken ships	2	new dike Broek-Zevenbergen	1410-1412	Ch.7, Sub-section 7.4.6
	36	breach Spaarndamse dijk NL	1510	Ch. 9, Section 9.2

19.3.5 Ballasting on or inside light constructions like fascine mattresses and in sink-fascines

Ballast materials for light constructions can be earth (in mata's), clay clods, sand- or clay bags, boulders or imported stone. The latter is expensive and prior to the 20th century it has been only been used in a few closures (for instance at the Sloedam). The relevant references are presented in Table 19.8.

Table 19.8 Ballast materials for light constructions

ballast material	project no.	project	year	reference
earth (in mata's)	70	Gangrail, Bangladesh	1977	Ch. 14, Sub-section 14.2.3
clay clods	-	(no specific project)	19th century	Ch.4, Sections 4.2, 4.3 & 4.4
sandbags or clay bags	58	Keta lagoon, Ghana	1969	Ch. 11, Section 11.2
	59-61	various projects in Bangladesh	1979-1982	Ch. 11, Section 11.3
boulders	62	Feni, Bangladesh	1985	Ch. 11, Section 11.4
stones (rock)	30	Sloedam, the Netherlands	1871	Ch. 8, Section 8.4

19.3.6 Screening of the dumping area

Screening of the dumping area inside the profile of a closure bund will be required to prevent dumped materials from being washed-out by the current. In this case references will solely refer to photographs as they illustrate the screening concerned very well (Table 19.9).

[9] In fact all four are sink-fascines, see Ch. 4, Table 4.1.

Table 19.9 Screening of the dumping area

material for screening	project no.	project	year	reference	
				Chapter	Figure
vertical bamboo stakes placed in advance	61	Amtali, Bangladesh	1982	11	11.27
fixing wire mesh to bamboo grating of mattress	62	Feni, Bangladesh	1985	11	11.42
bamboo grills fixed horizontally against row of piles	71	Nalian, Bangladesh	2020	15	15.11
hurdles placed against inside pile rows of cofferdam	-	(no specific project)	prior to 1900	3	3.3

19.3.7 Structural components in an abutment, jetty, cofferdam, etc.

This topic has only been added for the sake of completeness. What kind of structural components are required can easily be derived from various drawings and photographs. (Table 19.10).

Table 19.10 Structural components

structure being part of:	project no.	project	year	reference	
				Chapter	Figure
abutment	71	Nalian, Bangladesh	2020	15	15.8
scaffold	75	closure of a breach, Japan	19th century	18	18.7
compartments for mata placing	70	Gangrail, Bangladesh	1977	14	14.6
Jetty (temporary bridge)	22	Dijckmeesterpolder, Netherlands	1918	7	7.20
	59	Chakamaya, Bangladesh	1979	11	11.21, 11.22
cofferdam	-	breach at the Spees, Netherlands	1855	9	9.15
	73	Middle Level, England	1862	17	17.4

19.3.8. 'Keeping together' and 'anchoring' of fill materials

The important role in closures of natural or synthetic fibres shaped as bags, ropes or hawsers cannot be emphasized enough. Three examples are given here of 'keeping together':

- The floating fascine mattress comprises a filling of (mostly) three layers of brushwood (*rijshout*) compressed between two gratings of fascine strings; the compression is reached by numerous ropes connecting both gratings. The fascine strings in the gratings are in fact also fixed to each other by means of the same ropes. It is shown in Appendix D, Fig. D.5.
- The so-called wing kids, rolled kids and 'plugs' in Chinese closure works can only be constructed by using thick ropes and hawsers as described and shown in Ch. 13, Sub-sections 13.6.1. and 13.6.3 (Figs. 13.8. and 13.12) and in Appendix J.
- The sink-fascine, called mata, can only be made by wrapping the cylinder of earth and palm leaves in numerous ropes (Ch. 14, Fig. 14.6).

'Anchoring' is demonstrated in Ch. 13, Figs. 13.13. and 13.14 and in Appendix D, Figs. D.8 and D.9.

19.4 The material output in tidal closures

19.4.1 General

In Table 19.11 the production in terms of dumped or placed closure materials to fill the flow gap to be closed is presented for a number of selected projects. Based on this Table the characteristics and variables of closures will be discussed.

19.4.2 Length(s) of final closure gap(s)

Lengths of final closure gaps (column 4) were already discussed in Sub-section 19.2.2.4 of this chapter. But here the topic is presented again as it has a direct relation to the volume of fill to be placed/dumped during closure. Apparently, the longest gaps to be closed were those in Bangladesh in the period 1978 – 1982: 105 to 200 m. This was possible because (a) use was made of a jetty and (b) the jetty was transferred into a cofferdam. Moreover, all three were gradual closures.
The seven flow gaps in the Netherlands closed by making use of a temporary bridge also had lengths of up to 70 – 80 metres. But to carry out these closures within one tide (i.e. as a speedy closure) was

only possible because of the utilisation of trains of side tip wagons, crossing the tidal channel on a narrow gauge railway track mounted on the temporary bridge.

19.4.3 Area of cross-section to be filled

The areas of the cross-sections (column 5) to be filled have been derived from the text in various reports, mentioned within the context of case histories or by measurement from figures in these reports. This did not always result immediately in a satisfactory outcome as information in most cases was incomplete. When looking at the figures in column 5, one should also realize that closure materials, like clay clods, dumped from a temporary bridge need a larger (trapezoid) cross-section than closure materials dumped into a cofferdam having vertical sides. The ratio between the two could be as high as two. Compare for instance the Figures 7.27 and 7.32 in Section 7.10.2.

Table 19.11 Production during a number of tidal closures selected from Parts II and III

Project no.	country	project	purpose	length of final closure gap(s) (m)	area of cross-section (m^2)	volume to be filled in final closure gap (m^3)	type of fill	nos. of bags	time required for final closure (h)	nos. of labourers involved	volume (m^3) & bags both placed per manhour	chapter & section or Box
	1	2	3	4	5	6 = [4] x [5]	7	8	9	10	11	12
22	Netherlands	Dijckmeester-polder (1918)	reclamation for agriculture	65 78.5	18.7 13.8	1 218 1 080	clay clods	n.a.	12 11	600	0.33	7, Section 7.10, Table 7.16
23		Kreekrakpolder (1923)		65	25.3	1 650			no information found			
24		Hellegatpolder (1925)		35	24	840			< 12	80 or more	<0.88	
25		Biesboschpolder (1926)		55 77	38.2 24.9	2 100 1 915		- 7 000	11.5 11.5	144 158	0.64	
27		Dijkwater (1954)	flood contr.	70	14.3	1 000 [10]		n.a.	5	60	3.33	
36		Spaarndammer dike (1510)		125 to 235	560 (incl. losses	70 000		n.a.	10 467 labour days		0.53	9, Section 9.2
40		Nieuwerkerker - polder (1807)		19	16.8	320		n.a.	one tide	?	?	9, Section 9.6
52		Kadijk (1953)		13x20 = 260	7	1 820	sand- or clay-bags	91 000	5	1 000	0.36 & 18	10, Sections 10.10, 10.12, & 10.14
54		Kl.aas van Steelandpolder. (1953)		64 +5 16	22.3 57.8	1 440 +100 924		110 000[11] 130 000[12]	10 20	600[13] 600	0.26 & 18 0.07 & 5	
56		Willempolder (1953)		46.5	12.2	570		40 000	6	?	?	
58	Ghana	Keta lagoon (1969)		45	8.2	371		6 000	4	250	0.37 & 6 [14]	11, Sub-sections 11.2.7, 11.3..6, 11.4.5
59	Bangladesh	Chakamaya (1979)	reclamation for agriculture	200	24.8	4 968	clay bags	327 500	appr. 5 x 5 = 25 h	5 620 labour days	0.18 & 12	
60		Madargong (1980)		105	26	2 730		312 000	no detailed information available			
61		Amtali (1982)		120	42.8	5 143		190 000	5 + 4 = 9	1 600	0.36 & 13	
62		Feni (1985)		775	19.4	15 000		675 000	7.5	10 000	0.20 & 9	
71		Nalian (2020)		84	22.2	1 865		65 275	6.5	800	0.36 & 13	15, Sub- sect. 15.5.3
73	England	Middle Level Drain (1862)	flood contr	26.7		≤ 2 270	clay gravel/ clunch	- 200-300	6	800	≤ 0.50	17, Sub-section 17.5.3

19.4.4 Volumes to be filled in final closure gaps and number of bags (Table 19.11)

There is a direct relationship between the volume of fill (column 6) required and the number of bags (column 8) dumped or placed to acquire this volume. The difficulty is the unknown number of bags which goes into one m^3. This problem has been discussed earlier (Ch. 10, Sub-section 10.16.4, Table 10.5) and it

[10] Volume in cofferdam.
[11] This figure is excluding losses during closure works because these were considered negligible.
[12] A loss factor of 2 is assumed.
[13] From the text (RWS, 1961, 618) it cannot be established whether the 700 labourers were only involved in carrying and dumping clay bags or also in the filling of these bags and miscellaneous works (like dumping rock or driving vertical beams using hand-pile-drivers).
[14] These bags were much larger and heavier than the bags applied in the other projects listed in this Table. The number of bags in one m^3 is here 16. Accordingly, a crew of seven persons (four young students, three adults) placed 42 bags in one hour. This represents a volume of 2.6 m^3. When there are 50 to 70 bags in one m^3 this would imply a production of 19 to 26 bags per manhour.

was decided to adopt a figure of 70 bags in one m^3 in case the number of bags actually placed/dumped was not known. This decision is, however, only applicable to the case histories of Chapter 10[15]. In other case histories (Ghana, Bangladesh) the figure of 70 bags in one m^3 does not apply. In this respect reference is made to closure projects nos. 58 to 62 (incl.)
Another uncertainty is the loss of bags during closure. In this respect assumptions have been made explained in the relevant footnotes.
In order to bypass the uncertainty about the number of bags in 1 m^3, the volume of material placed per manhour (i.e. m^3/manhour) has also been inserted in column 11 (figures in red)[16].

19.4.5 Time required for final closure, number of labourers involved

The time required for final closure (column 9) is in most cases (for instance, Ghana, Amtali, Feni) well known and in others (for instance Klaas Van Steelandpolder) rather vaguely described. If it is stated that the closure took two days or two shifts, it has been assumed that 10 hours are worked on the closure during a shift or day. But in Bangladesh the number of hours (column 10) worked is less because the labourers have to walk to the site from their home and back. Moreover, the energy spent during the closure operation is higher than in Europe in 1953 because the bags can be twice as heavy. The number of labourers involved in the closure works is known in most cases but it is not always clear how many labourers are actually carrying bags. In those cases assumptions have been made.

19.4.6 Number of bags carried per manhour

The time it takes to close a final gap by utilizing manual labour as much as possible is very dependent on the number of bags carried per manhour (column 11). Obviously, the distance between stockpile of bags and place of dumping determines this output rate to a large extent. Given the various assumptions made and the differences in the said distance for the various projects, it is surprizing to find rather small differences between the manual output rates. The, apparently low, output rate for Keta is due to the volume and weight of the bags[17], while the low output rate for Feni is due to the fact that bags were placed in the bund profile rather than dumped (see Ch. 11, Fig. 11.54). Four output rates are between 11 and 13 bags/manhour. The rather high (18 bags/manhour) output rates for Kadijk and Klaas van Steelandpolder must have been due to the small transport distances.

19.5 Unit rates and cost of construction

19.5.1 Unit rates

19.5.1.1 General

Unit rates for closure materials have been presented in various chapters[18]. The unit rates concern very different projects, constructed under very different circumstances, in different locations and over a period of 205 years (1764 – 1969). It is therefore difficult to draw firm conclusions even after transferring rates in guilders (or other currencies) of the past into euros of the year 2020. Nevertheless a few general comments can be made.

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[15] In Chapter 10, closures in 1953 in the Netherlands are discussed. The bags used in those closures in most cases were smaller than bags used in Ghana and Bangladesh. They were also not completely filled because the average Dutch labourer does not carry a bag having a weight (= mass) of 35 kg or more.
[16] The volume in column 11 for project no. 27 is so high because the side tip wagons were filled by draglines. Draglines were not operated in the closures effected prior to the Second World War.
[17] It has been explained in Sub-section 19.4.4, Table 19.11, footnote 14.
[18] Please refer to Ch. 7, Section 7.9, Tables 7.11 to 7.14 (incl.); Ch. 8, Section 8.4, Table 8.4; Section 8.7, Table 8.6; Section 8.10, Table 8.8;Ch. 9, Section 9.6, Table 9.5; Ch. 11, Section 11.2, Table 11.3.

The unit rates in Table 11.3 (Ghana) will not be discussed as these were valid in circumstances (economic, climatic, geographical) completely different from those valid for the Netherlands on which the other Tables have been based. The said general comments are given in the following sub-sections.

19.5.1.2 Digging, transporting and placing of 1 m^3 of earth fill

If wheelbarrows are used (Tables 7.11, 8.4 and 8.6) the unit rate for the first 40 m varies between € 1.69/m^3 and € 2.29/m^3. But at the Sloedam this rate is € 4.36/m^3 (no distance specified). One sees immediately the advantages of other transport means (hand carts and horse-drawn carts, side tip wagons on narrow gauge rail track): for distances from 300 m up to 1 500 m the rates vary between € 4.55/m^3 and € 10.28/m^3 (Tables 7.12 and 8.4). In this respect, the introduction of bucket dredgers (Sloedam, Table 8.4) is even more convincing: € 3.69/m^3 (i.e. compared to the aforementioned rate of € 4.36/m^3).
Digging, transporting and placing/dumping of 1 m^3 of clay is more expensive as, usually, it requires transport by ship from a nearby salt marsh to the dike alignment: € 6.22/m^3 to € 15.03/m^3 (Tables 7.13, 8.4, 8.6).

19.5.1.3 Turfing, cramp matting, fascines and stone pitching on the seaward slope of a dike

Turfing of 1 m^2 varies between € 0.37 and € 2.00 but the large difference can be due to the variation in thickness of the sods as specified (Tables 7.14, 8.4 and 8.6).
The cost of 1 m^2 of a summer cramp mat varies between € 0.78 and € 1.22 while a winter cramp mat comes to: € 1.09 to € 1.53. (Tables 7.14 and 8.6).
It stands to reason that the application of 1 m^2 of the heavier (and more durable!) fascine slope protection is much more expensive: € 7.44 to € 11.14 (Table 7.15).
Even more expensive is 1 m^2 of stone pitching incorporating cramp matting, rubble sub-layer and a 25 cm thick layer of pitched stone (basalt or lime stone from Lesse or Tournai): € 40.72 (Table 8.6).

19.5.1.4 fascine constructions

Last but not least a few remarks about fascine constructions. Caland (1833, 206 et seq.) calculated the cost of 1 m^2 of a floating fascine mattress, thick 50 cm or 40 cm, at € 15.38 and 13.83 (i.e. as the figures would be in euros for the year 2020). These rates are including fabrication, transport and labour for ballasting but excluding the cost of purchase and transport of clay clods or stone required as ballast. These rates compare quite well to the rates given for the closures required for reclamation of the Schengepolder (Table 7.15): € 18.89/m^2 for a 45 cm thick mattress and € 12.78/m^2 for a 30 cm thick mattress. In Table 8.6 a 30 cm thick *kraagstuk* (fascine mattress for protecting the toe of the dike) is priced at € 16.54/m^2 and this is probably including ballast. Finally, Table 8.4 gives a rate of € 53.64 for 1 m^3 of fascine dam which implies € 37.55 for 1 m^2 of fascine mattress having a thickness of 70 cm (incl. thickness of ballast layer). In Table 8.4 also the cost of a pakwerk in mentioned: € 31.85/m^3.

19.5.2 Cost of construction

Analyses about cost of construction of reclamation and dam projects were made in Ch. 7, Section 7.7 (Table 7.9) and Ch. 8, Section 8.10 (Table 8.7) respectively. It could be concluded that in most cases no parameters (which these projects haven in common) could be defined, determining the cost of construction. The same type of analysis was carried out for the case histories discussed in Chapters 9 to 16 (incl.) and the result was even worse. Economic circumstances, availability and access to closure materials, magnitude of the breach to be repaired/tidal channel to be closed, differ much from country to country while also the year of achievement plays a major role.
This statement is illustrated in Table 19.12 presenting a selection of projects about which construction costs and the value of other parameters are (more or less) known.
The purpose of the project concerned is given in column 2. For each purpose the tabled construction costs per linear metre of the projects can be compared.

This results in the following:

- Reclamation is the purpose of eleven closure projects. The cost of construction per linear metre of dike varies between 79 and 8 571 if expressed in euros of the year 2020. The three projects in Bangladesh in the period 1979 – 1982 are expensive if compared with the other eight: € 5 200 – 8 571 against € 79 – 1 067. But the projects in Bangladesh are tidal closures over their full lengths while the cost/m[1] of the other reclamation projects also include the much cheaper construction costs of dikes on the high salt marshes (*schorren*).
- Flood control is indicated as overall purpose for the projects concerning the repair of breached dikes (4 nos.) and of repair of the cut in the sand ridge of Keta lagoon. If the cost of closing the Dagenham breach (€ 46 700/m[1]) is excluded, the repair works vary from € 1 765 to 4 719 per linear metre. The cost of repair of the Dagenham breach is so high because of the height of the dike, the high tidal range at springtide (6 – 7 m) and the repeated failures. Moreover, the method of transfer from GBP of the year 1720 into euros of the year 2020 may have contributed to the high cost in euros of 2020.
- Finally, six projects have as purpose either railway/road link, navigation or irrigation. For reasons explained in footnote 21, the cost of the IJdam per linear metre is extremely high. For the other five projects the construction per linear metre varies between € 816 and € 18 531. If the Slaakdam is excluded (which' construction in terms of tidal closure was an easy one) the range in construction cost per linear metre is € 4 457 to € 18 531.

Table 19.12 Cost of construction for a number of tidal closures[19] selected from the case histories presented in Parts II and III

Project no.	country	name of project (R = reclamation, F = flood control, L = railway/road link, N = navigation, I = irrigation)		year of completion	length (m) of dike or dam	cost of construction		cost per m[1]	height (m) from OGL to crest dike/dam	Chapter and Section or Table
						in currency of period	in € of 2020	€ of 2020		
	1	2	3	4	5	6	7	8	9	10
2	Netherlands	R	Groote Waard, new dike Broek-Zevenbergen	1410-1412	7 330 or 10 100	£ 8 474 – 5 – 2	6 191 000 [20]	845 or 613	not known	7, Sub-section 7.4.8
3		R	Groot Zuid-Beijerland	1631	3 250	f 223 291	2 330 000	717	not known	7, Sub-section 7.5.6
72	England	F	Thames River, Dagenham	1720	120	GBP 40 000 (est.)	5 604 000	46 700	aver. 11	16, Section 16.7
6	Netherlands	R	Reigersbergsche polder	1773	14 082	f 349 115	3 343 000	237	10 (est.)	Sub-section 7.8.4
40		F	Nieuwerkerkepolder	1807	28	f 13 000	105 000	3740	max. 8.5	9, Section 9.6 Section 9.7
41		F	Kruininger polder	1808	1 130	f 506 000	3 937 000	3 485	aver. 10.2	
9		R	Wilhelminapolder	1809	7 190	f 420 465	3 301 000	459	>7	7, Sections 7. 6, 7.7 & 7.8
11		R	Mortierepolder	1846	350	f 2 775	28 000	79	6.5 (est.)	
15		R	Zwinpolder	1864	700	f 39 000	452 000	646	9 (est.)	
29		L	Kreekrakdam	1867	3 640	f 1 500 000	16 224 000	4 457	max. 10	8, Section 8.3 Section 8.4 Section 8.5
30		L	Sloedam	1871	965	f 1 100 000	12 292 000	12 737	max. 17	
31		N	IJdam	1874	1 360	f 4 500 000	49 999 000[21]	36 764	≥ 10	
16		R	Schengepolder	1874	2 500	f 240 000	2 667 000	1 067	max. 3.5	7, Sections 7.6 – 7.8
33b		L	Slaakdam	1884	2 770	f 334 700	2 259 000	816	aver. 6	8, Section 8.7 Section 8.9
35	Germany	L	Hindenburgdamm	1925	11 000	f 11 100 000	87 807 000	7 982	aver. 7.4	
25	Netherlands	R	Biesboschpolder	1926	8 260	f 864 500	7 136 000	864	max. 7.8	7, Sub-section 7.10.2
68	China	F	Yellow river, Tung Chuang	1936	2 745	HK $ 2 638 000	12 953 000	4 719	max. 20	13, Sub-section 13.6.3
58	Ghana	F	Keta Lagoon	1969	170	40 250 cedis	300 000	1 765	max. 6.5	11, Section 11.2
59	Bangladesh	R	Chakamaya creek	1979	213.5	f 1 000 000	1 110 000	5 200	max. 11.7	11, Section 11.3
60		R	Madargong creek	1980	152.5	f 940 000	980 000	6 426	max. 11	
61		R	Amtali creek	1982	140	f 1 300 000	1 200 000	8 571	max. 10	
62		I	Feni River closure dam	1985	3 007[22]	f 65 830 000	55 722 000	18 531	aver. 7.3	11, Section 11.4

[19] Except project no. 68 which is a river closure.

[20] Dike was 95 % complete; roads and ditches etc were never made.

[21] Incl. cost of three navigation locks and sluice.

[22] Length is including the so-called flank embankments. Length of dam through the river is 1330 m.

19.6 The future of construction by simple means

The case histories presented in Parts II and III of this dissertation reflect the art of river and tidal closures by simple means over a period of nearly 760 years (1263 – 2020) in six countries, i.e. in alphabetical order: China, East-Bengal (now Bangladesh), England, Germany, Japan and the Netherlands. But most of the simple means described can be considered as history. Already in the 20th century in many countries tidal closures, if required, were constructed by utilizing large water-borne equipment and large constructions like (sluice-) caissons. There are, however, countries where manual labour is abundantly available and tidal closures could still be (and have been in fact) constructed by simple means. Such closures by simple means have been described for the years 1969 (Ghana) and 1979 – 1985 (Bangladesh). But one should realize that after 1985 not less than 35 years passed before another successful tidal closure by simple means (2020, Nalian closure) was made.
A typical aspect in the past was the abundant use of vegetal materials which required much periodic maintenance after the actual construction of the closure works. This applied in particular to the slope protection which comprised materials which had to be renewed regularly : straw in crampmatting, fascines in the tidal zone (Netherlands), millet stalks protecting the flood embankment against scour (China).

In BWDB (1982a) a 'Review of the Programme on the Closure of tidal Channels in Bangladesh' was published. In this Review recommendations were formulated about many aspects related to the design and construction of tidal closures to be carried out in future in accordance with the 'Dutch' method. But no follow-up materialized. The introduction of, for instance, the bed protection against scour using a specially developed floating mattress comprising locally available materials as well as the gradual closure applying a jetty transferred into a cofferdam, has not sustained.
If one compares the closures of 1979 – 1985 with the Nalian closure of 2020 it is noted that three aspects have stayed: (a) the construction of a horizontal sill prior to any narrowing of the tidal river/creek, (b) the screening of the dumping area, and (c) carrying and dumping of fill material contained in bags.
At Nalian river the sill was constructed by placing/dumping geo-bags and 50kg synthetic bags. The sill functioned as bed protection and consolidated also the weak deposits in the river, comprising very soft clayey silts. The use of geo-bags and synthetic bags in hydraulic engineering has become quite common in Bangladesh since the nineties of the 20th century. These items replace sandbags and clay bags as well as heavier items like crates filled by boulders or concrete blocks. As such, there is nothing against the newly introduced closure materials but one would like to know more about the possible lack of friction between units during closure operations.
For bed protection at the Feni River closure (1985) floating mattresses, comprising polypropylene fabric having a bamboo grating, were applied. The reason behind this decision was the impossibility to acquire the large quantities of materials, like reed and palm leaves, which would be required if bed protection mattresses would be constructed as applied in earlier closures (Chakamaya, Madargong, Amtali). Obviously, polypropylene mattresses could also be applied in future tidal closures. For ecological reasons the polypropylene fabric could then be replaced by a fabric of natural fibres.

Finally, a few recommendations:

- -- try to obtain and use site investigations data to identify, for instance, potential erodibility or compressibility of the natural bed;
- keep current velocities during closure operations as low as possible;
- take measures to prevent or minimize scour prior to and during closure operations;
- have sufficient and good quality closure materials in stock near the closure site;
- keep the labour force happy in order to prevent riots and strikes;

- always try to avoid the closure of a deep gap; this is preferably done by constructing a horizontal sill having its level around Low tide, over the full length of the closure gap prior to final closure;
- enlarge the mass of a closure item by making use of geo-bags, jute bags or beet nets;
- prevent the deposited or dumped closure materials from being washed away by the current either by screening the dumping area or by tying (anchoring) the materials to a stable point.

CHAPTER 20 Conclusions

20.1 Introduction

In Ch. 1, Section 1.3, research questions have been formulated. During the studies these questions have all been answered but the answers are scattered through the text of this dissertation and sometimes an answer has been presented in parts, given one by one in different places. Therefore, a comprehensive list of answers is presented with reference to the relevant parts in the dissertation for further details. This will be done in the next section.
For easy reference the research questions will be repeated, using the same numbering as before.

20.2 Questions and Answers

Q.1 Why did dikes breach so frequently during past centuries?

A.1 In most cases dikes breached because they were too low, or the slope protection against wave attack was inadequate or the geotechnical condition of the dike's cross-section led to piping, seepage and instabilities during storm surges. The questions remain why the dikes were too low and why the slope protection and geotechnical condition was not up to standard. In Ch. 2, Sub-section 2.7.2 six basic causes for dike breaching are given:

- the initial product (i.e. the newly completed dike section) is not to specification;
- the unusual external loading of the dike is higher than the design resistance;
- there is no periodic or regular maintenance; no timely repair of damages;
- lack of regular inspection and survey of the dike which may lead to a long-term deterioration and to hidden faults;
- a change in the original natural conditions;
- intentional breaching in time of war.

For a more detailed discussion about these causes reference is made to Ch.2, sub-sections 2.7.3 – 2.7.8 (incl.).

Q.2 What kind of closure methods were used and what are the characteristics and differences between the various methods?

A.2 Depending on the length and depth of a closure gap, the tidal range, the materials to be used and the measures taken in advance, a closure can be gradual, speedy or instantaneous.
One speaks about a gradual closure when it takes more than one day to close the last gap. Speedy means that the closure is carried out within 4 to 8 hours, but always within one tidal cycle. An instantaneous closure will not take more than one hour.
It also has to be decided whether the closure will be a horizontal or a vertical closure, or a combination of both. Differences between these types of closures are spelled out in relevant literature and will not be repeated here.
Until the second half of the 19th century, closures were in most cases gradual and a combination of a vertical and a horizontal closure. Usually, one started to prepare a horizontal sill over the full width of the closure gap by dumping clay clods from small ships or by placing sink-fascines in a vertical position. From the end of the 17th century the sill could also be formed by stacking fascine mattresses on top of each other. When the sill had reached the level of Low tide the works continued as a horizontal closure, by narrowing the gap by dumping clay clods from both ends. When currents in the final narrow closure gap were high an instantaneous closure could be required by lowering a fascine construction called 'sow'.
When tidal ranges were small, a breach in a dike or a tidal channel could be narrowed by driving a row of (sheet) piles followed by the sinking of ships in the final gap.
A closure could also be made by constructing a cofferdam across the gap; the cofferdam was subsequently filled by dumping layers of clay clods (vertical closure).

By the end of the 19th century the speedy vertical closure was developed. A jetty was constructed across the closure gap and the dumping of clay clods from side tip wagons on a narrow gauge rail track enabled the closure to be achieved within one tide.
Details are discussed in Chapter 5. More in particular reference is made to the Sections 5.5 (Type of closure) and 5.6 (Tidal closure concepts). Also, the Summary in Section 5.7 provides a good overview.

Q.3 What materials, tools, equipment and temporary works were applied for these closures and what was the role of labourers?

A.3 In order to close the flow gap in a breached dike or to dam the last gap in a tidal channel, one will need, apart from much manual labour:

- construction materials,
- tools and equipment,
- temporary works[1].

These three topics are discussed below bearing in mind the focus in this dissertation on closures carried out in whole or in part by using simple means.
One could in fact read simple means as 'to be carried out at low cost' because closure operations in former centuries always had to be carried out requiring a low investment[2].
Low cost implies in fact:

- cheap labour,
- construction materials to come preferably from the neighbourhood of the closure site;
- tools and equipment to be cheap and simple;
- temporary works to comprise cheap materials, easy to construct and easy to remove after closure.

For construction materials one must make a distinction between materials available in nature and processed/manufactured materials.
Materials, suitable for closure works and available in nature, are granular/ stony materials like clay, sand, boulders and rocks and (parts of) plants like brushwood, wooden sticks, poles and stakes, reed, straw, bamboo, palm leaves, millet stalks.
The said (parts of) plants are at the base of, what one can call, 'composite' fascine constructions:

- rubble or clay-filled fascine 'sausages' (*doodemans,* translated as sink-fascines*)*
- suspended fascine mattresses ('*zeugen'*, translated as 'sows')
- floating fascine mattresses *(zinkstukken)*
- *pakwerken* (no proper English translation found)
- *baardwerken* (,, ,, ,, ,, ,,)

All these fascine constructions have played in the past a major role in closures in the Netherlands and nearby parts of Belgium. Thanks to the research needed for this dissertation the composition and application of the suspended fascine mattress in tidal closures was discovered and described.
Some materials are obtained by relatively simple processing or manufacturing like jute bags (to be filled with sand or clay), rope and hawsers (needed to make the aforesaid fascine constructions).

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[1] Temporary works are all the works which are temporarily required to construct the permanent works, like tool sheds, canteens, a jetty across the closure gap.

[2] Dike repairs are inherently unplanned. Breaches occurred nearly always in a particular polder district and the responsibility for funding breach repairs was with the landowners and farmers. Such funding could be very difficult to raise at short notice because the landowners in most cases were not living in the polder and the farmers were poor. The central government in principle did not finance such repairs. but could give dispensation on taxes for a certain period. But if financing would be arranged it would cost time and therefore repairs could not start.

The tools used in the closures by simple means are shovels, forks, spades, sacks, baskets, barrows, aprons, head pans. The equipment can be bicycles, wheelbarrows, shovel shaped sleds (*molborden*), carts, side tip wagons, boats, manually operated piledriving installations.
Because of the importance of this topic Chapters 3 and 4, a total of 29 pages, as well as some Appendices, have been fully dedicated to the aforementioned items. Moreover, in Ch. 19, Section 19.3, the various functions of the closure materials have been linked to individual case histories presented in Parts II and III of the dissertation.

Q.4 Is there a certain relationship between the closure method(s) in use and the natural resources/social conditions in that particular region or country?

A.4 Thanks to the study of closures constructed in other countries, this question could be answered. The selected closure methods are highly dependent on local resources and social conditions, and no uniform relationships could be derived. However, a number of similarities have been found and a few illustrative examples will be given.

- The use of the sink-fascine has been identified in five countries (Bengal, China, Germany, Japan and the Netherlands). Its size turned out to be different in each country. This also applied to its composition: in each case the 'fill' as well as the 'skin' differed. The fill could be earth, straw, clay clods, gravel or stones. The skin comprised fascines, fascine strings, palm leaves, millet stalks or bamboo netting. More information has been presented in Ch. 4, Table 4.1.

- In the Netherlands ballast for floating fascine mattresses for many centuries have been clay clods but in Bangladesh clay clods could not be found and bags filled with clay were used instead, or boulders (Feni closure).
- Social conditions have had, through the centuries, a great impact on closure methods used. The change in these conditions is the basic reason that 'simple means' in closure methods in most countries is a thing of the past: the wages of labourers have become too high to apply 'simple means' involving abundant manual labour.

Reference is made to Ch. 19, Section 19.3 for additional examples in Tables 19.5 to 19.10 (incl.).

Q.5 How successful were the closure methods used in the Netherlands and in other countries during past centuries? And if they failed, what was (were) the reason(s) for failure?

A.5 In the past various closure methods were applied in the Netherlands for closing tidal creeks in reclamation schemes, for the closure of flow gaps in dike breaches and for damming tidal channels when dams had to be built to connect islands with each other or to the mainland.
The closure by simple means of tidal creeks for reclamation turned out to be relatively easy. The same was more or less true for damming tidal channels as these dams were built on or near the tidal divide. But closure of dike breaches could turn out to be problematic when tidal prisms of flooded areas were large and related current velocities in the closure gaps high. During past centuries in many cases simple means were not adequate to close such breaches and, consequently, the closure methods failed and flooded polders could not be reclaimed and had to be given back to the sea. Typical examples are the land losses which occurred at the south coast of the island Schouwen-Duiveland and those which led to the *Verdronken Land van Saeftinghe* in Oost- Zeeuws-Vlaanderen.
Special mention must be made of the low overflow dams, built to further accretion at both sides of those dams for future reclamation. In all four cases recorded these dams failed repeatedly and, finally, were abandoned or a dam having a non-overflow crest, was constructed instead. The reason for these failures was first of all lack of knowledge about the impact of storm surges on tidal flow and, secondly, a reluctance to publish about such failures. This in turn resulted in a repetition of the event.

Q.6 Has there been any technical development in the closure methods in the course of time?

A.6 In the answer to Research Question no. 2 an overview has been presented of the closure methods used in the Netherlands in the course of time. One can easily deduce the development of these methods from the overview. But for easy reference some aspects of this development are highlighted here for physical parts of the closure works.

First of all, protection was required of the easily erodible subsoil of the breach or tidal channel. During the period 1200 – 1700 one must have dumped a layer of clay clods as bed protection. But in heavy currents this was not enough and one had to roll sink-fascines (fascines imprisoning a gravel core or clay clods) into the gap. This was a complicated operation as, prior to the placing of sink-fascines a kind of dike head had to be constructed by means of a cofferdam. After the introduction of the floating fascine mattress (by the end of the 17th century) bed protection was not anymore a problem. These mattresses were ballasted by dumping clay clods and from the middle of the 19th century by dumping stones.

The actual closure used during the period 1200 – 1700, nearly always started by dumping a horizontal sill of clay clods up to the level of Low tide. Subsequently, the closure, after narrowing the gap by dumping clay clods, was effected by lowering constructions (called 'sows') made up of sink-fascines and clay clods. Also, such sink-fascines could be rolled into the gap by manual labour or placed in a vertical position when the gap was too deep and current velocities too high to construct a sill. Alternatively, a cofferdam could be constructed across the flow gap, the cofferdam was then filled by manual labour dumping clay clods or stones inside the cofferdam. In case of low tidal ranges, a row of sheet piles or piles, closely placed to each other, was driven across the gap. Sometimes, ships were ballasted and sunk on top of the sill.

By the middle of the 18th century the sill was built up by stacking fascine mattresses on top of each other, again up to the level of Low tide. Subsequently, the closure was performed gradually in a horizontal direction by constructing a bund in sections, comprising fascines, clay clods, stones and wattle work (such a bund section was called a pakwerk).

From the end of the 19th century up to the middle of the 20th century jetties were constructed across the gap to be closed. A narrow gauge railway track on the jetty enabled the dumping of clay clods or stone in the gap from side tip wagons. This was a vertical closure, not necessarily with a sill.

Q.7 Is it possible to compare physical output by labourers during closures in different centuries and different countries?

A.7 In Chapter 19, Section 19.4, the findings presented in various case histories regarding the physical output of labourers have been summarized and compared. Though there are many unknowns and, repeatedly, estimates had to be made regarding losses of closure materials during closures, there is a remarkable agreement in physical output by labourers. In Table 19.11, column 11, the figures in red give the output expressed in m^3/manhour for 13 case histories. If the lowest (0.07, 0.18 and 0.20 m^3/manhour for projects nos. 54, 59 and 62, respectively) and highest (3.33 m^3/manhour for project no. 27) are ignored[3], the output rates vary between 0.26 and 0.64 m^3/manhour. There is no significant difference in output during different centuries or between countries.

[3] The reasons for two low output rates have been explained in Chapter 19 (in foot note 14 and Sub-section 19.4.6 respectively). The high output rate is explained in foot note 16 of the same chapter.

Q.8 Is it possible to define, on the basis of closures studied, certain factors stimulating the success of manual closures, also at present?

A.8 At the end of Ch. 9, Section 19.6, recommendations have been listed which stimulate the success of manual closures. These are repeated here for easy reference:

- try to obtain and use site investigations data to identify, for instance, potential erodibility or compressibility of the natural bed;
- keep current velocities during closure operations as low as possible;
- take measures to prevent or minimize scour prior to and during closure operations;
- have sufficient and good quality closure materials in stock near the closure site;
- keep the labour force happy in order to prevent riots and strikes;
- always try to avoid the closure of a deep gap; this is preferably done by constructing a horizontal sill having its level around Low tide, over the full length of the closure gap prior to final closure;
- enlarge the mass of a closure item by making use of geo-bags, jute bags or beet nets;
- prevent the deposited or dumped closure materials from being washed away by the current either by screening the dumping area or by tying (anchoring) the materials to a stable point.

20.3 Dissemination of technical know-how during past centuries and the lack of such exchange if failures are concerned

One of the aspects in this dissertation, which has only been mentioned in passing, is the exchange and distribution of technical know-how about hydraulic engineering during the past centuries. One must think in this connection first of all about the exchange inside countries but also between countries. An example of the (lack of) such exchange is for instance the repetition of failures when building a non-overflow dam.

In the past exchange of technical know-how was in the first place verbally and secondly by correspondence. Already in the 12th century Dutch engineers travelled to Schleswig-Holstein to carry out drainage works. In the 17th century Dutch engineers travelled to Schleswig Holstein (Leeghwater), England (Vermuyden) and Italy (Cornelis Meijer) to advise about the design and construction of hydraulic engineering works. The involvement had sometimes been inspired by Dutch capitalist entrepreneurs who intended to finance a reclamation scheme and needed Dutch expertise to carry it out. But it could also happen that the local ruler asked specifically for technical assistance from a Dutch expert. Vermuyden was asked by King Charles I to make Hatfield Chase suitable for agricultural use. Rolwagen became dike reeve general in the dutchy of Holstein and was general advisor of the duke of Holstein.

In the 18th century, engineers started to communicate internationally by exchanging and discussing notes, reports and books. For instance the German engineer Woltmann received comments from the Dutch engineer Brünings on his 'Theory on construction of dikes' (± 1890) and Brünings in turn got 70 pages of comments on his submission for a contest about the current velocity of flowing water. The German engineer Von Wiebeking, after a breach in a dike along the river Lek in the Netherlands (1795) refers to a report made by the Dutch engineers Goudriaan and Conrad.

Both German engineers mentioned have written very comprehensive books about hydraulic engineering as practised in their own and neighbouring countries[4]. Apparently, they travelled to these

[4] Reinhard Woltmann: '*Beyträge zur Hydraulischen Architectur'*; four volumes, published in the years 1791 – 1799, together some 1 400 pages.
Carl Friedrich von Wiebeking: '*Theoretisch-practische Wasserbaukunst'*; four volumes, published in the years 1811 – 1817, together some 2 400 pages, also two atlases with maps and figures.

countries, collected technical information as laid down in books, notes and drawings and then started to write about it. In the atlases being part of the books one find plans of major and minor harbours in England, France, the Netherlands and Belgium, drawings of bridges in England and France, drawings of canals and navigation locks in France, sea defences in the Netherlands, etc., etc.
Also books published in other countries are extensively discussed. Woltmann (1792, Vol. 2) dedicates 21 pages to the French book of Bossu & Viallet (1764) and 11 pages to the Dutch book of Ypey (1777). All these books helped enormously in the dissemination of knowledge about hydraulic engineering.
In the 19th century institutions have inter alia, been founded in England[5] and in the Netherlands[6], which gave engineers the opportunity to present their project to colleagues and to have discussions about problems and mishaps which had been encountered during construction. In the 19th century engineers also started to work more and more overseas in their particular field of expertise.
Mr. C.B. Vignoles (1793 – 1875) is a typical example; He worked as a railway engineer in more than six countries[7].
Still, a culture in which engineers are willing to admit their failures and publish about their mistakes is seldom found. In this dissertation various projects are mentioned in which mistakes led to failures or mishaps and publication thereof did not occur or was ignored.
These failures and mistakes are summarized in the following paragraphs.

- The low overflow dams[8] built in the period 1757 – 1921 at four different locations all failed repeatedly. At the start (1757 – 1765) this was due to lack of knowledge of fluid mechanics during storm surges, but in 1883 the cause of failure was found in the Netherlands as follows from remedial measures proposed. Apparently, this was not communicated internationally and in the period 1906 - 1921 the mistakes were repeated in Schleswig-Holstein.
- In 1871 during construction of the Sloedam a 151 m long floating fascine mattress having a limited width (20 m) turned upside down in the strong tidal current during ballasting. The engineer supervising the construction of the dam reported about it and recommended measures to avoid this in future[9]. Some 80 years later the same happened nearly at the same spot (i.e. reclamation of the Zuid-Sloepolder). The engineer in charge of TBD[10] (who must have read about the mistake made in 1871) wrote an article about the closure in 1950 but did not mention the mistake now made a second time!
- In 1954, it was the intention to close the tidal inlet Dijkwater by filling a cofferdam with clay, dumped from a temporary bridge across the closure gap. When the closure was nearly complete one of the dike heads collapsed[11]. This had nearly happened in a similar situation in 1926[12] during closure of the Noorderdiep for reclamation of the Biesboschpolder and measures could have been taken to prevent this mishap at the Dijkwater closure. In the article written about this failed closure a completely different cause is suggested which led to the failure. The supervisory staff at the closure site was apparently not willing to publish the real cause of failure[13].

[5] The Institution of Civil Engineers (ICE) was founded in 1818.
[6] The Institution of Engineers (now KIVI) was founded in 1847.
[7] Reference is made to Ch. 4, Sub-section 4.2.1, footnote 29, for more details.
[8] Reference is made to Ch. 8, Sections 8.1, 8.2, 8.6 to 8.8 incl. and 8.11.
[9] Reference is made to Ch. 8, Section 8.4 and more specifically to footnote 77.
[10] TBD = Technisch Bureau der Domeinen.
[11] As described in Ch. 7, Sub-section 7.10.2 under the heading 'Damming tidal inlet Dijkwater…..'.
[12] See Ch.7, Sub-section 7.10.2 under the heading Biesboschpolder and more particularly the caption of Fig. 7.26.
[13] Van Duivendijk attended the closure as a 19 year old student of Delft University and saw it all happen. The suggestion in the article about the cause that led to the failure is definitely not correct.

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[4] For Volume I, please refer to De Bruin & Wilderom (1961)

GLOSSARY (English – Dutch)

English	Elucidation /Toelichting	Dutch
apron	for instance a fascine mattress, bag work or rip-rap	stortebed, maar ook berm onderaan een dijk (oud-Nederlands)
artisan skilled in the art of making fascine constructions		rijswerker
bank	bank line at mean low water	oever (–lijn)
bank full stage	river channel is completely full up to crest of summer dike	zomerbed gevuld tot de kruin van de zomerkade
bank over spill		water flowing over the flood plain
barrow	voor grondtransport	draagberrie
basket	oud-Nederlands: hot, bot	draagkorf
basket (large)	misschien vorm als een wasmand	grote pakmand (Vierlingh, 1920, 330)
brushwood	as material	rijshout
boulder	oud-Nederlands: cattebol	kei, rolsteen
boulder-clay		keileem
brick		baksteen
bund		kade, lage dijk (± 1-2 m hoog)
bund on part of crest of dike	oud-Nederlands: wolster	tuimelkade (Danner et al, 2009, 141)
camber		tonrondte
caterer (on a building site)	oud- Nederlands:	zoetelaarster
clay core	kleikern	kleikist
(clay) clod (fat clay clod)	lump of earth	(schor-) kloet, (kloeteling)
clear weir, perfect weir, clear overfall, free flow weir		volkomen overlaat
closure bund		kade door sluitgat (bij dijkdichting/geulsluiting)
closure gap		sluitgat
cofferdam	In original sense	kistdam
cofferdam type spur		kistpaalhoofd
compartmentalization dike	oud-Nederlands: vercortinghe	compartimenteringsdijk, delingsdijk
cramp matting		krammat
day works (construction in ..)		uitvoering in regie
dike fill		dijkspecie
dike head		dijkskop
dike reeve		dijkgraaf
estuary dam		afsluitdam in een estuarium
faggot	fascine, bundle of twigs bound together	bos rijshout
fascine	faggot, bundle brushwood	bos rijshout
fascine dam		rijzen dam
fascine mattress for protecting the toe of the dike		kraagstuk

English	Elucidation /Toelichting	Dutch
fascine pole, sink-fascine	fascine imprisoning a gravel core	zinkfascine, doodeman
fascine spur		rijshoofd
fascine string	fascine 'rope' (long and thin)	wiep
fascine construction		rijshoutconstructie
flat-boat	with flat bottom for transport in shallow water	platbodem
flat deck barge		zolderbak
floating fascine mattress		zinkstuk
flooding		overstroming
flood embankment		rivierdijk
flood plain		uiterwaard; winterbed
flow gap	also called: tidal gully (oud-Nederlands: doorgaand gat, spoelgat, speelgat)	stroomgat
flow slide	see also slip	zettingsvloeiing, oeverval
foreshore		vooroever; onderzeese oever
grating (under-, upper-)	also: grillage	onder- en bovenroosterwerk bij een zinkstuk
ground level		maaiveld
groyne	in river, as part of river normalization scheme	krib
hard point	miniature spur	bol, triangle
hawser	dik touw of kabel	tros
head pan	round container, like a bowl, used in construction work	geen Nederlands equivalent
hurdle		dijkhorde
hydraulic jump		watersprong
interval of repose	turn of the tide	kentering
inundation		inundatie
liquefaction		zettingsvloeiing
marsh grass	(see also rice grass)	schorgras
mean high tide (MHT)		gemiddeld hoog water (GHW), volzee
mean low tide (MLT)		gemiddeld laag water (GLW)
mooring post	bollard	prop (op een zinkstuk)
mud flat	see shore	
narrow gauge railway		smalspoorbaan
naval shipworm	see also shipworm	paalworm
neap tide		doodtij
necessitous	needy, nooddruftig	calamiteus
(osier) withe, binder	thin flexible willow twig, sprig	kruisband, knijpband, wiepband
non-overflow dam		hoogwatervrije dam
osiers	twigs, sprigs	rijshout
overflow dam		overloopdam
peat		derrie, turf
perimeter closure bund	oud-Nederlands: vingerling	ringkade
railway track		spoorbaan

English	Elucidation /Toelichting	Dutch
range pole, ranging rod		jalon
reclamation scheme		inpoldering
revetment	covering to a soil or rock surface to protect it from scour (Scott, 1991)	bekleding van een grondvlak ter voorkoming van uitschuring
rice grass		spartina townsendii
rip-rap	wide-graded quarry rock	los op een talud gestorte laag rotsblokken met brede gradatie
salt marsh	buitendijks begroeid gebied	schor, kwelder
scour hole	as a consequence of a dike breach; oud-Ned. : waal	wiel, kolkgat
sea defence		zeedijk, zeewering
sea-weed		wier
secondary sea defence		inlaagdijk
shipworm	also: marine borer, pile-worm	paalworm
shore, mud flat	area along the coast between low and high tide	slik, (deel van het) voorland
shovel		schop
shovel-shaped sled	drawn by horses	molbord
side tip wagon	loopt op smalspoor	kipkar
sill (in tidal closures)	also: threshold; oud-Nederl.: berm	drempel
sink-fascine, fascine pole	fascine imprisoning a gravel core	zinkfascine, doodeman
sink line		zinklijn
sink strap		zinkstrop
slack water at High and Low tide	also found: interval of repose	kentering bij hoog- en bij laagwater
slack water on the flood and on the ebb	turn of the tide	kentering van de vloed- en ebstroom
slightly silty clay	heeft blauw-grijze kleur	spier
slip	sliding earth mass	grondafschuiving
slope in dike body	oud-Nederlands: dosering	helling, talud
soggy		drassig
springtide		springtij
spur, wingdam (USA)	in rivers and tidal waters	hoofd
stake	has a number of different lengths and diameter's, all having different names: paaltjes, tuinlatten , bleeslatten, gaarden, slieten	staak
stone from quarry	rock	natuursteen uit steengroeve
stone pitching		gezette steenglooiing
storm surge[1]	a rise in water level caused by storm winds (Scott, 1991);	stormvloed

[1] In the Netherlands a storm surge is defined as the storm which rise' is exceeded once every two years (*grenspeil*)

English	Elucidation /Toelichting	Dutch
summer dike		zomerkade
surge		stormopzet
tender documents		bestek en voorwaarden
tidal difference	tidal range	tijverschil
tidal lands, salt marshes	areas along the coast above mean high tide (MHT)	schor, kwelder(deel van het voorland)
tidal divide, tidal meeting point		wantij
tidal gully closure, tidal closure		stroomgatsluiting
tidal prism	storage of tidal flow in basin	komberging
under-seepage		onderloopsheid
wattle work		tuin, betuining
wave deflector wall	wave return wall	muraltmuur
weir (clear or submerged)		overlaat (volkomen of onvolkomen)
wharf		zate
wheelbarrow	in oud-Nederlands ook genoemd: schuierwagen, cor-dewagen, kordewagen, piepe-gaal	kruiwagen
wind set-up		windopzet (opwaaiing)

APPENDIX A[1] The protective 'skin' of the dike body

A.1 Introduction

Apart from the dike fill, as described in Chapter 2, Section 2.5, other materials have been used in the past (i.e. prior to the 20th century) as part of the dike structure. The function of these materials, in most cases, is to protect the dike body against eroding forces: rain, wind, currents caused by tidal or river flow, ice floes, wave attack, carts, horses, pedestrians, grazing cows or goats.

These other materials are: fat clay clods (*kloetelingen*), fat clay sods, straw, reed, fascines, sea-weed, piles, timber, bricks, boulders, stones.

In most cases the materials are applied in combination with each other and this leads to certain constructions in which each material has its own specific function.

Below a distinction is made between:

- constructions which prevent erosion of the dike body against (mainly) wave attack; these constructions have in common that they are providing a 'skin' protecting the dike body (Sections A.2 to A.7 incl.) or a 'wall', having the same purpose (Sections A.8 to A.11 incl.);
- various constructions carried out in the past on the (fore-)shore; these works are constructed in order to protect the dike body either against wave attack or against scour by the current and ice floes, or both (Sections A.12 to A.17 inclusive).

Each of these materials / constructions will now shortly be discussed.

A.2 Fat clay clods

Probably the first protected dikes are so-called '*slikkerdijken'* (Thierry, 1930, A.49-50). A *slikkerdijk* (or *slijckerdijck)* is a protection at the seaward side of a dike body which has been built up using any fill material locally available. The *slikkerdijk* consists of a pile of regularly stacked clay clods leaning against the aforementioned dike body. See also Vierlingh (1920, 301).

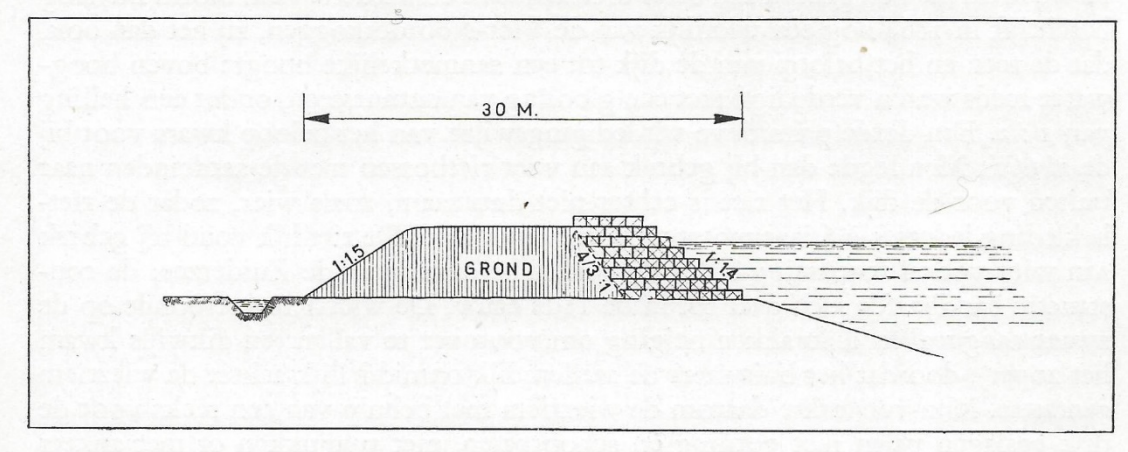


Fig. A.1 Cross-section of a *slikkerdijk;* the blocks at the right side are clay clods, protecting the dike body against erosion by currents and wave action; source: Dibbits (1950, 93)

A.3 Fat clay sods

Provided, our ancestors had the right tools to cut the sods (20 or 30 cm sides, thick 4-5 cm, Huitema, 1947, 102), from the ripe mud-flats; the placing of these sods at the dike slopes must have been the first effort to protect the dike against erosion. These sods are first of all meant to get a good grass cover on the dike slopes and, secondly, to avoid erosion by rain. When, in autumn, the grass of the

[1] Appendix A is referred to in Chapter 2.

sods has not yet rooted sufficiently in the clay underneath, the sods are cramped by means of straw (Fig. A.2, left). One should not mistake the clay sods for the clay clods (*schorkloeten*, *kloetelingen*) which are cube-shaped (15-20 cm sides) and are used to protect dike bodies (*slikkerdijken,* see above) as well as to ballast a fascine mattress (Huitema, 1947, 3).

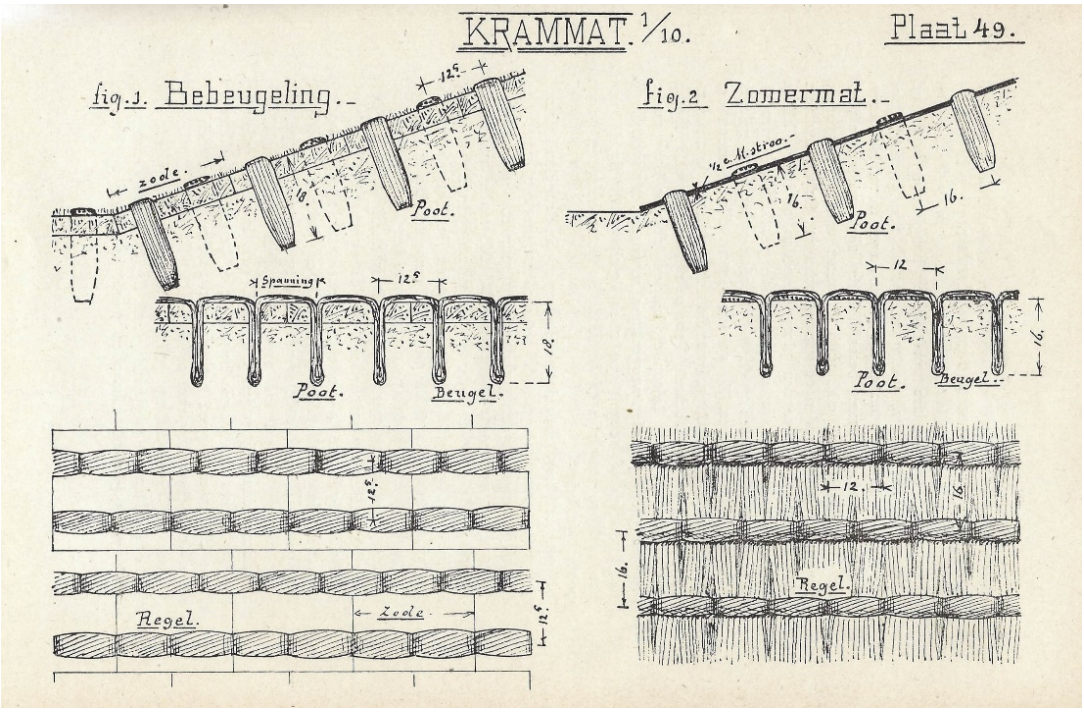


Fig. A.2 Left: cramped sods; right: cramped straw mat: source: Van Breen (1920, 205)

A.4 Straw

Straw is already being used in the fourteenth century. Mention of the use of straw on dikes is, inter alia, made in Hamaker (1880, vol. II, 360) for the year 1344 (straw for a dike near Brouwershaven) and by Gottschalk (1983, vol. I, 184) for the end of the 14th century when she mentions the use of large quantities of straw for cramp matting *(krammatten*).

Cramp matting is applied when a good grass cover is still missing in the zone above average high tide or when the placed clay sods have not yet rooted into the under laying clay. Preferably, straw from rye is used but failing its availability, also straw from wheat and reed may do. A thin layer of straw or reed is laid on the slope. It is fixed to the clay surface by placing parallel rows of straw whisks (Fig. A.2, right) using a special tool (as shown in Chapter 3, Fig. 3.5 right).

Schraver (1807, 90-91) states that a well- designed and constructed cramp matting will be able to prevent damage to the slope by wave run-up during heavy wave attack, provided the slope is very gentle. In this connection he mentions the slopes of the sea defence at Westkapelle (Ch. 2, Fig. 2.2) and refers to Van Bleiswyk (1745) and Velsen (1768).

This praise of cramp matting might be correct but a cramp matting is not very sustainable: it has to be replaced twice a year by a new one to be effective.

Further details can be found in Schraver (1807) and Van Breen (1920).

A.5 Fascine slope protection (Fig. A.3)

In the tidal zone, between low and high tide, a heavier protection is required. In the south-western delta one used to have the following construction: a thin cramp matting, on which a 5 cm thick layer of reed is placed in transverse direction (i.e. parallel to the dike crest) on top of which a layer of fascines is placed (thick 12 -16 cm) in the direction of the slope. Finally, the layers are fixed to the subsoil by rows of wattle work (Fig. A.4).

This type of protection in any case is already in use in 1510[2] in the Vierambachten in the later Zeeuws-Vlaanderen (De Bruin, 1955, 6). The cramp matting and reed prevent the washing out of soil as well as the loosening of the soil by crabs, while the fascines offer a flexible protection against wave attack.
But also this type of protection has its disadvantages: the cramp matting and reed are far from sustainable and have to be repaired/replaced regularly. The fascines, placed in the tidal zone are also not durable and, moreover, from ± 1730 onwards, are destroyed by the naval shipworm (De Bruin, 1955, 6).

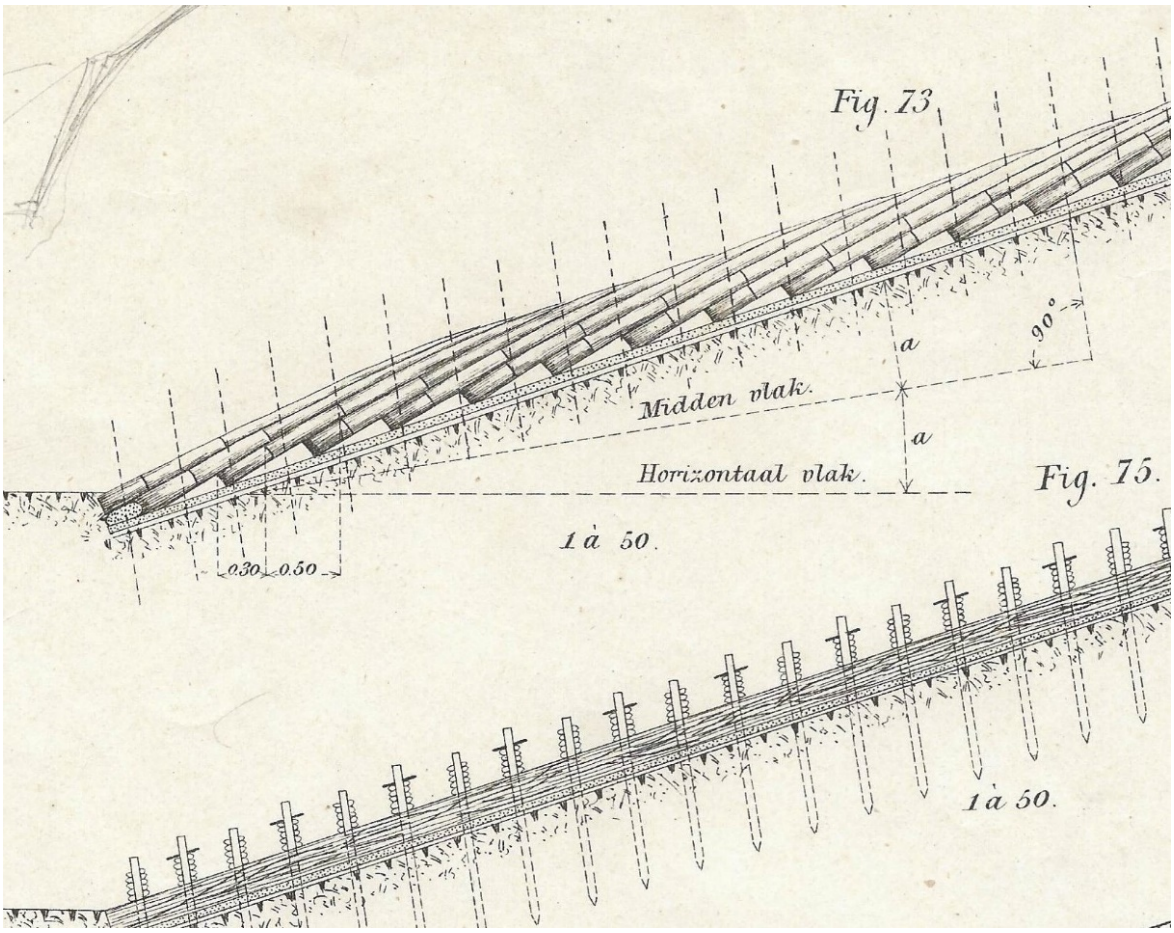


Fig. A.3 Fascine slope protection; details on the way of placing the fascines are also shown; source: L.R. Plasschaert, Atlas (1902a, Plate VII)

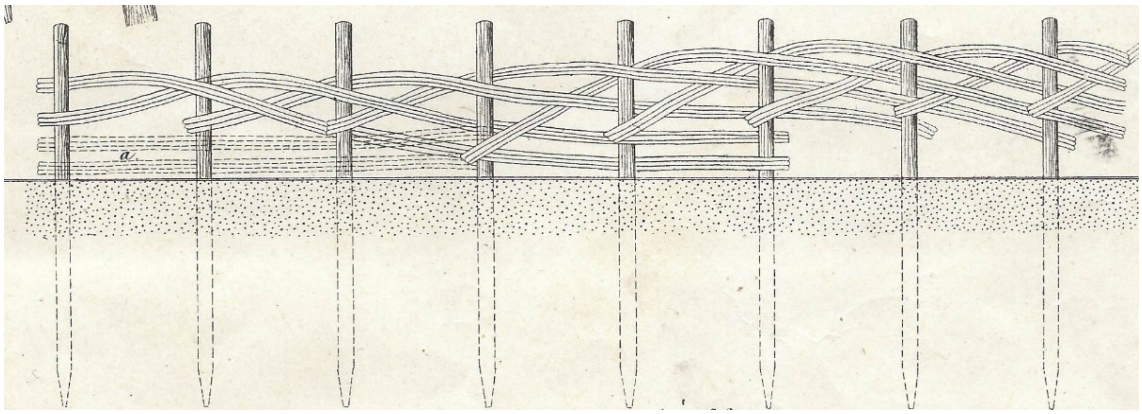


Fig. A.4 Wattle work; source: L.R. Plasschaert, Atlas (1902a, Plate VII)

[2] Van de Ven (ed.) (2004, 133) mentions 1450 as the start of using fascines but does not give a source.

Still, the fascine slope protection is in use for a long time: the Waterboard St. Maartensdijk only replaces the fascine slope protection by a stone pitching in 1876 (Hollestelle, 1920, 226). On the island of Walcheren a fascine slope protection is replaced by a pitching of stone from Vilvoorde in 1860 (De Waard, 1914, 427, Inv. 2411).

A.6 Fascine slope protection with stones on top

Already in the 17th century (Kool-Blokland, 2003, 79), and most probably earlier (Vierlingh, 1920, 269-270), in the province of Zeeland, stone is being placed on top of the fascine slope protection to cope with heavy wave attack. But this is not the rule because stone is costly. It is essential to fix the stone to the fascine layer and this is done by clamping one or more stones between rows of wattle work or stakes (Fig. A.5).

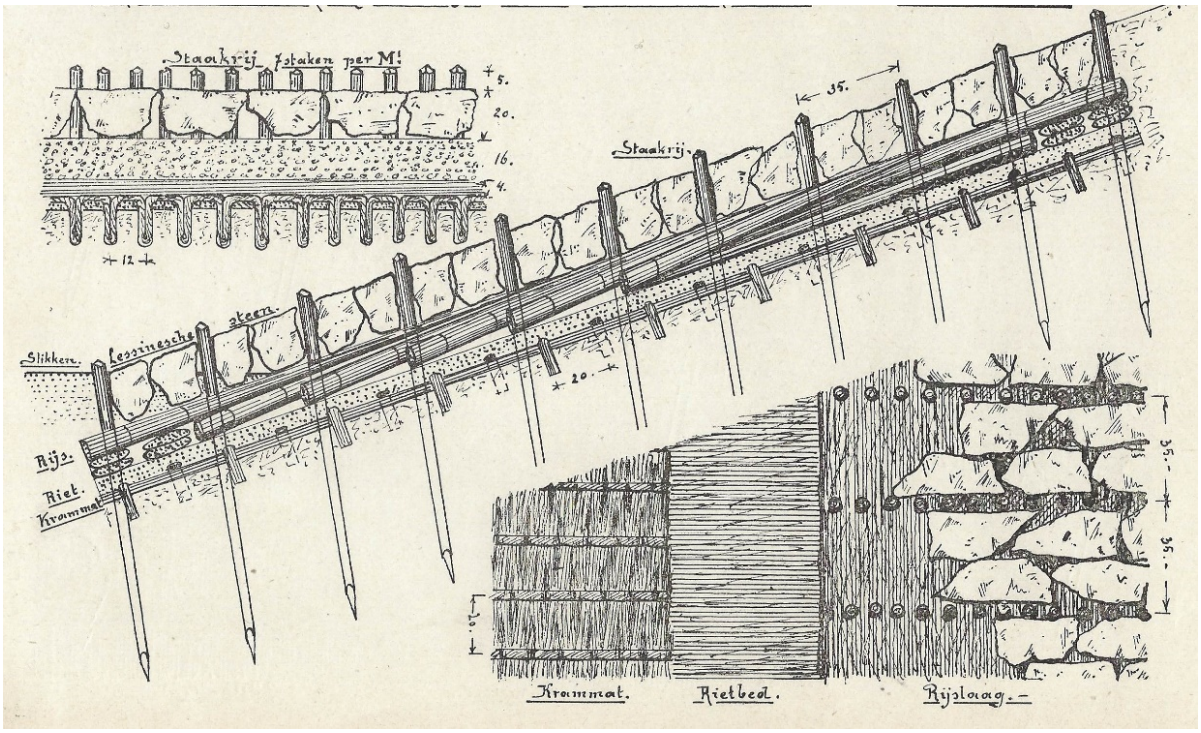


Fig. A.5 Stones on top of a fascine slope protection, the stones are clamped between stakes; source: Van Breen (1920, 212)

The stones come from nearby Belgium, mostly from Vilvoorde and, to a lesser extent, from Lessen, Doornik and the area of the river Meuse. In 1803-1804 such a protection is still being introduced on the island of Tholen at Moggershil (Hollestelle, 1920, 331).
Caland (1862, 115) states that In the year 1851 not less than 57 750 m^2 of 'old dilapidated fascine slope protection' in the province of Zeeland is being replaced by stone pitching.

A.7 Stone pitching

A stone pitching (*gezette steenglooiing*) comprises a number of layers: first a cramp matting is applied, then two layers of old bricks (laid flat) and on top broken bricks (*puin*) to a total thickness of 20 cm; finally the stones are pitched. The problem is that an individual loose stone will move easily during wave attack and though the stones have a more or less regular 'block' shape, it is still difficult to place them such that they are acting together as one solid slab of stone. One of the methods used is the placing of parallel rows of stakes to clamp the stones to the subsoil (Fig.A.6).
By the middle of the 19th century basalt becomes available and the regularity of these stones is much better than that of the type of stone mentioned above (Fig. A.7).

Prior to the 20th century fascines used for slope protection and stone pitching are typical for sea defences in the provinces of Zeeland and Zuid-Holland. In West-Friesland sea-weed is important for slope protection (Section A.8). In the province of Friesland wooden pile structures are in use (Section A.11).

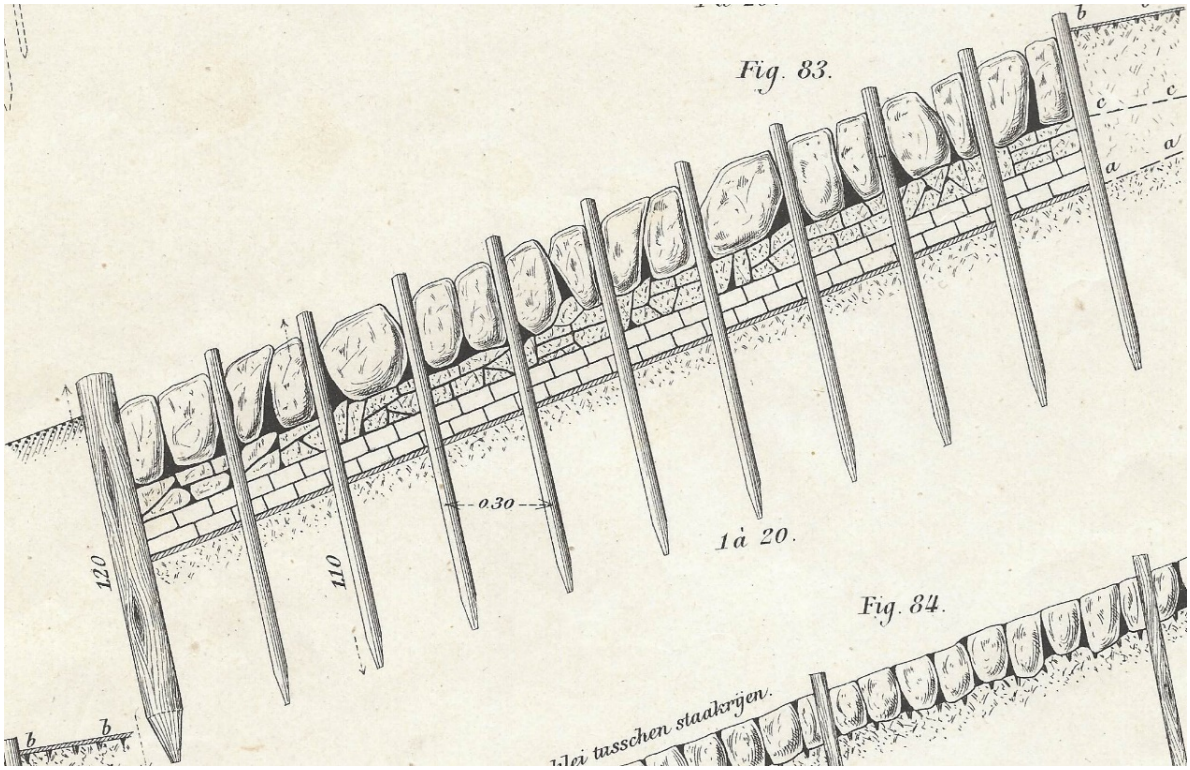


Fig. A.6 Stone pitching on cramp matting, two layers of bricks and broken bricks; the stones are clamped between stakes; below: rejected stones are placed directly on the clay sub soil; source: L.R. Plasschaert, Atlas (1902a, Plate VII)

A.8 Sea-weed dike (*wierdijk*)

In the past in West-Friesland the seaward slopes of dikes were protected by a solid 'wall' of sea-weed (Van Bleiswyk, 1745, 27-29). According to Dibbits (1950, 94-95) this type of protection is mentioned for the first time in a decree of 1319.

In a Charter of Philips de Goede of 5 July 1466 it is recalled that the protection of the West- Frisian dike by sea-weed has come in jeopardy due to scarcity of this material. Therefore it had been decided to combine 50 % sea-weed with 50 % reed, but it is stated in the Charter that this is now also not any more possible; see Gottschalk (1975, Vol. II, 212-213).

In 1467 a new type of protection is introduced: a frame work of oak beams and ironware (p. 214). From the text one would get the impression that the new type of protection replaces the wall of sea-weed but this is not the case. The frame work is meant to prevent the mass of sea-weed to topple in the scour hole formed at the toe by wave action. In Fig. A.8 the construction in the background marked 5, 6, 7 comprises iron rods with hooks to keep the sea-weed in place and 8, 9 is a timber construction (vertical pile and strut) to prevent the said toppling of the sea-weed mass. A 'box' of boulders (*krebbeling*), marked 10, prevents scour at the toe of the sea-weed dam.

The sea-weed dike has many disadvantages but for a long time, apparently, this is not a reason for change(Thierry, 1930, A 50-51).

After the 'immigration' of the naval shipworm (± 1730) it is proposed to place vertical piles behind the boulders and behind the sea-weed mass and connect these by means of the horizontal beams marked 1, 2, 3, 4. But this is an intermediate measure. Ultimately, it is proposed to adapt the cross-section as shown in in Fig. A.9.

Schilstra (1987 ,84) points out that the original *krebbeling* contains boulders transported from Greenland and Scandinavia as ballast in ships. But after 1730 much more boulders are required. For a period of more than 100 years these boulders are collected in the province of Drenthe and transported by various means (Fig. A.10) to West-Friesland.

Apparently, the sea-weed dike still exists in the middle of the 19th century (Fig. A.11).
Also in the province of Friesland dikes of sea-weed and reed are mentioned (Van Buijtenen & Obreen, 1956, 38). But it is also stated that these are probably earth dikes having a cramp matting or toe protection of sea-weed and reed (p. 39).

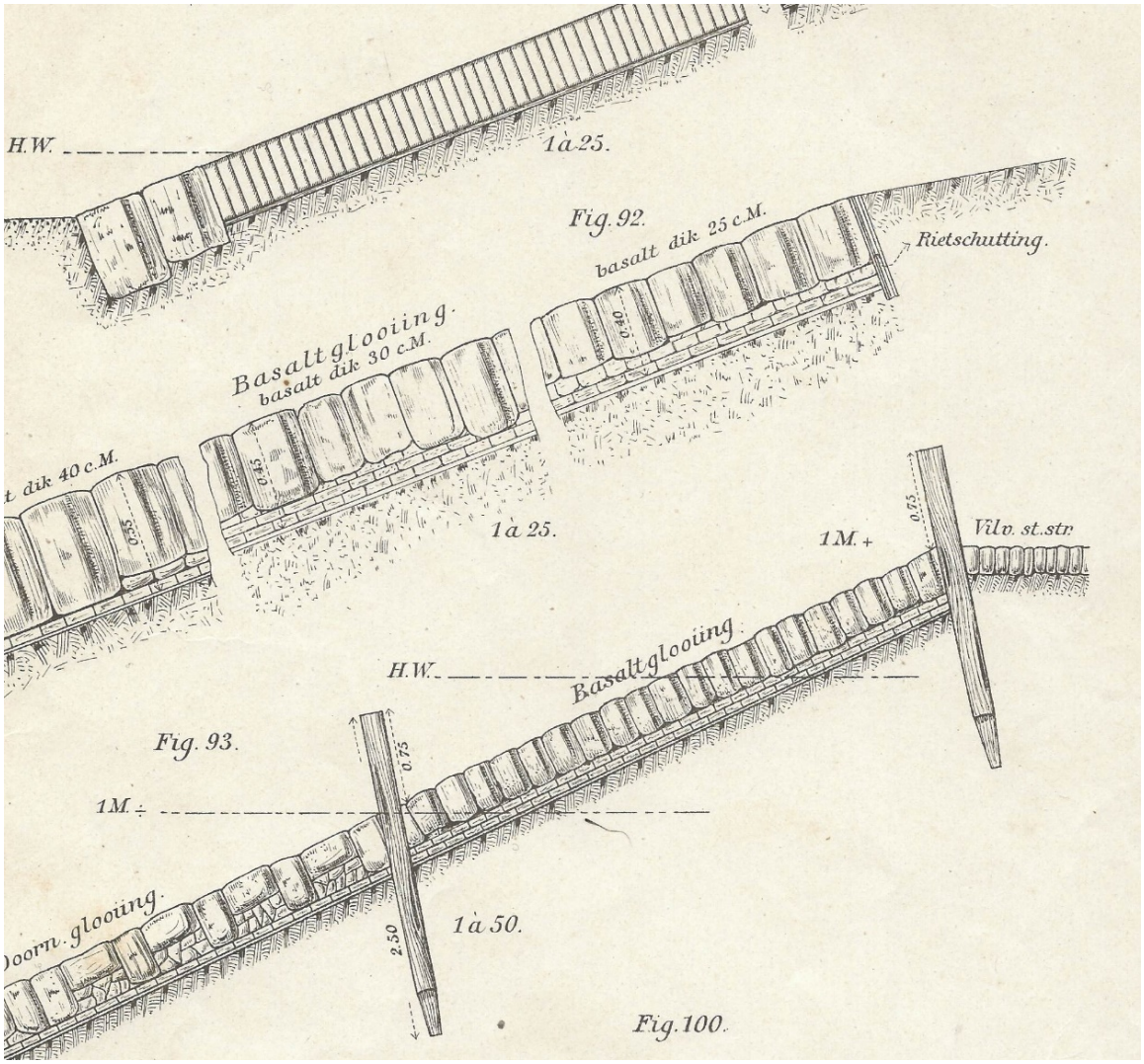


Fig. A.7 Below left: pitching of stone from Doornik, Belgium;
below right and in the centre: basalt stone pitching;
top: brick slope protection;
note the adaptation of the thickness of the sub-layers to the varying height of the stones;
source: L.R. Plasschaert, Atlas (1902a, Plate VIII)

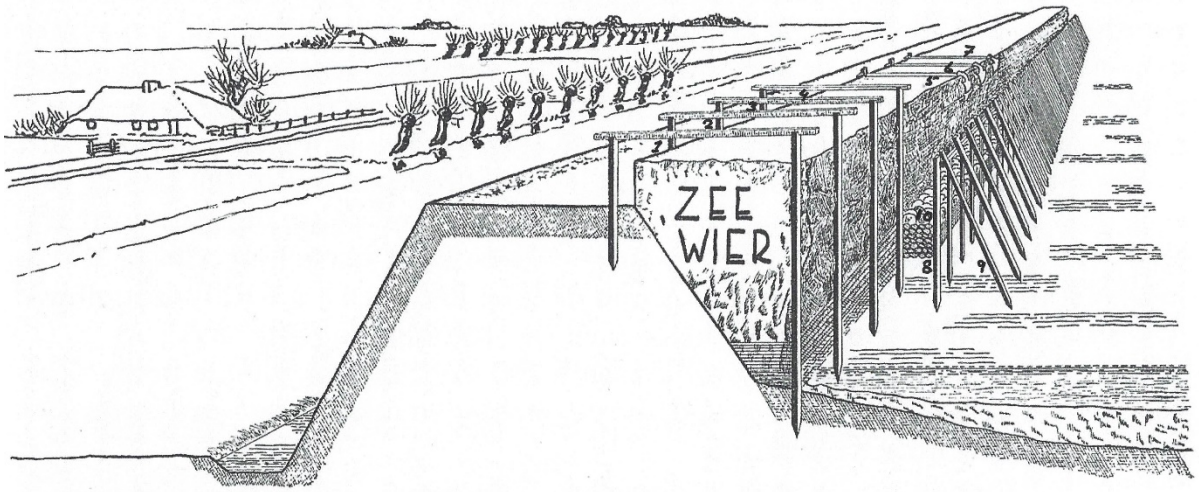


Fig. A.8 Sea-weed dike for the Noorder Koggen; because of the invasion by the ship- worm possible modifications to the support structure are indicated;
source: Dibbits (1950, 95) with adaptations

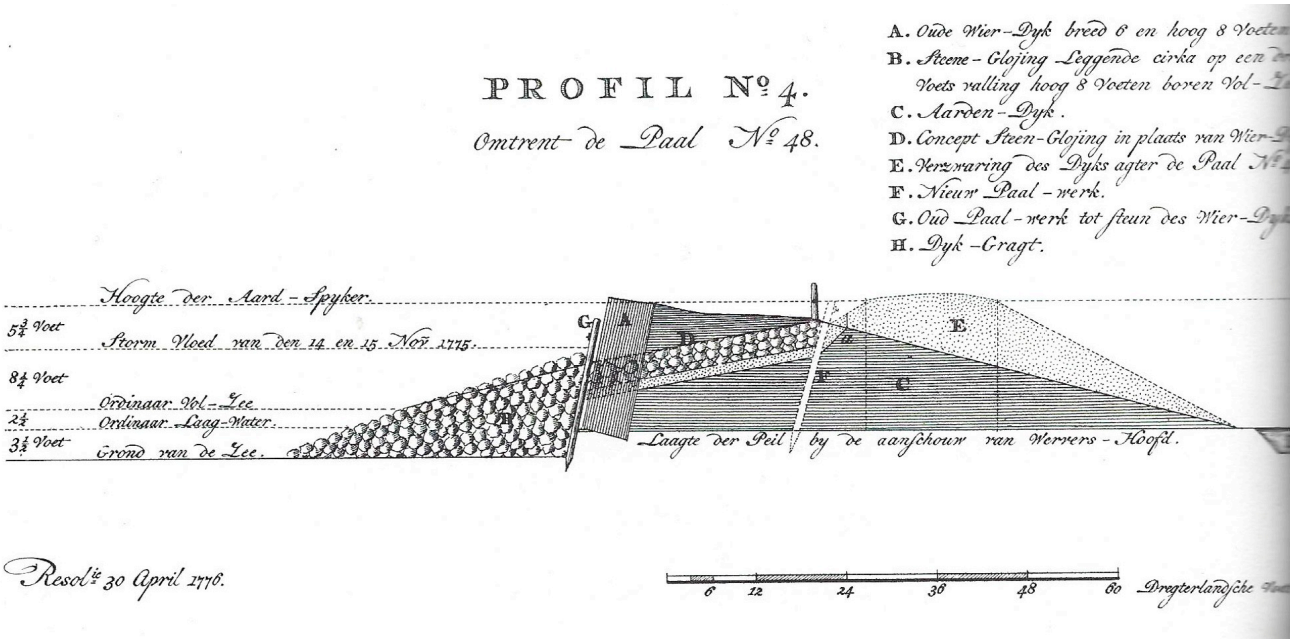


Fig. A.9 Adaptation of a sea-weed dike as proposed in 1778; source: Schilstra (1987, 82)

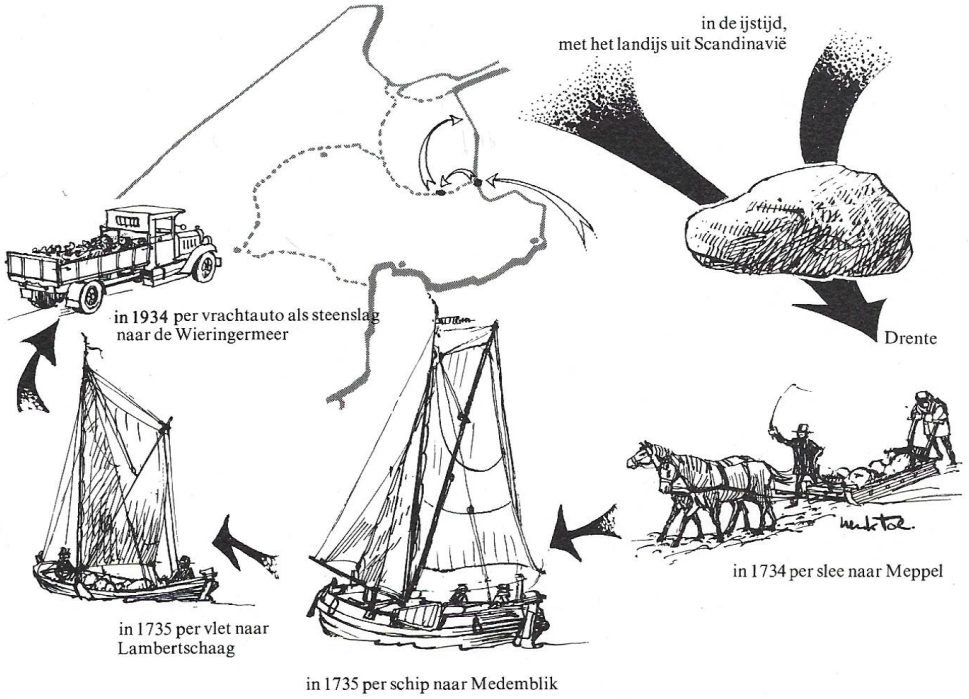


Fig. A.10 The transport of boulders from the province of Drenthe to West-Friesland in the 18th century and in the 20th century; source: Schilstra (1987, 73)

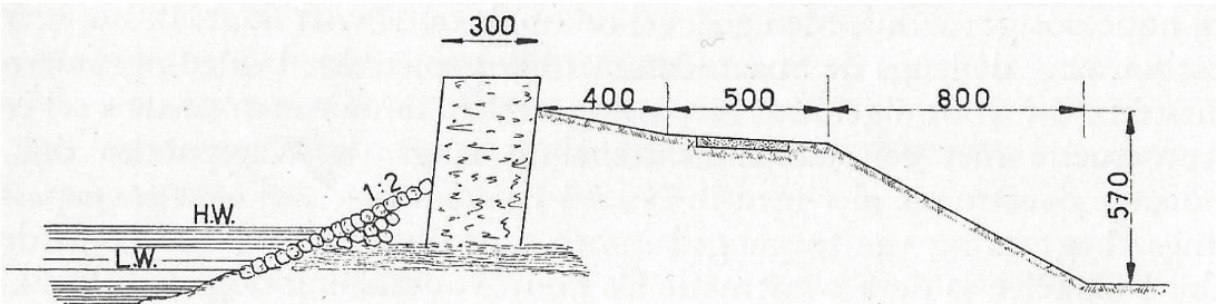


Fig. A.11 A sea-weed dike still existing in the middle of the 19th century; source: Storm Buysing, Atlas (1864, Plate XLVI, Fig. 15)

A.9 Reed dike

Failing the availability of sea-weed, bundles of reed are stacked with the roots at the seaward side. But also this type of slope protection is not sustainable and has to be replaced every 5 to 6 years. The reed dikes were predominantly built along the southern coast of the Zuiderzee.

A.10 Pile screen dike

Another method of slope protection is the construction of a nearly vertical screen of wooden piles. According to a decree of Philip of Burgundy in 1440 the Diemer dike between Amsterdam and Muiden had been provided with such a screen (Fig. A.12). The same authority , in 1466, orders the construction of 4.5 km of pile screen for the dike near Medemblik (Thierry, 1930, A51). The piles are 7 to 10 m long, 30 by 30 cm in cross section (Fig. A.13).

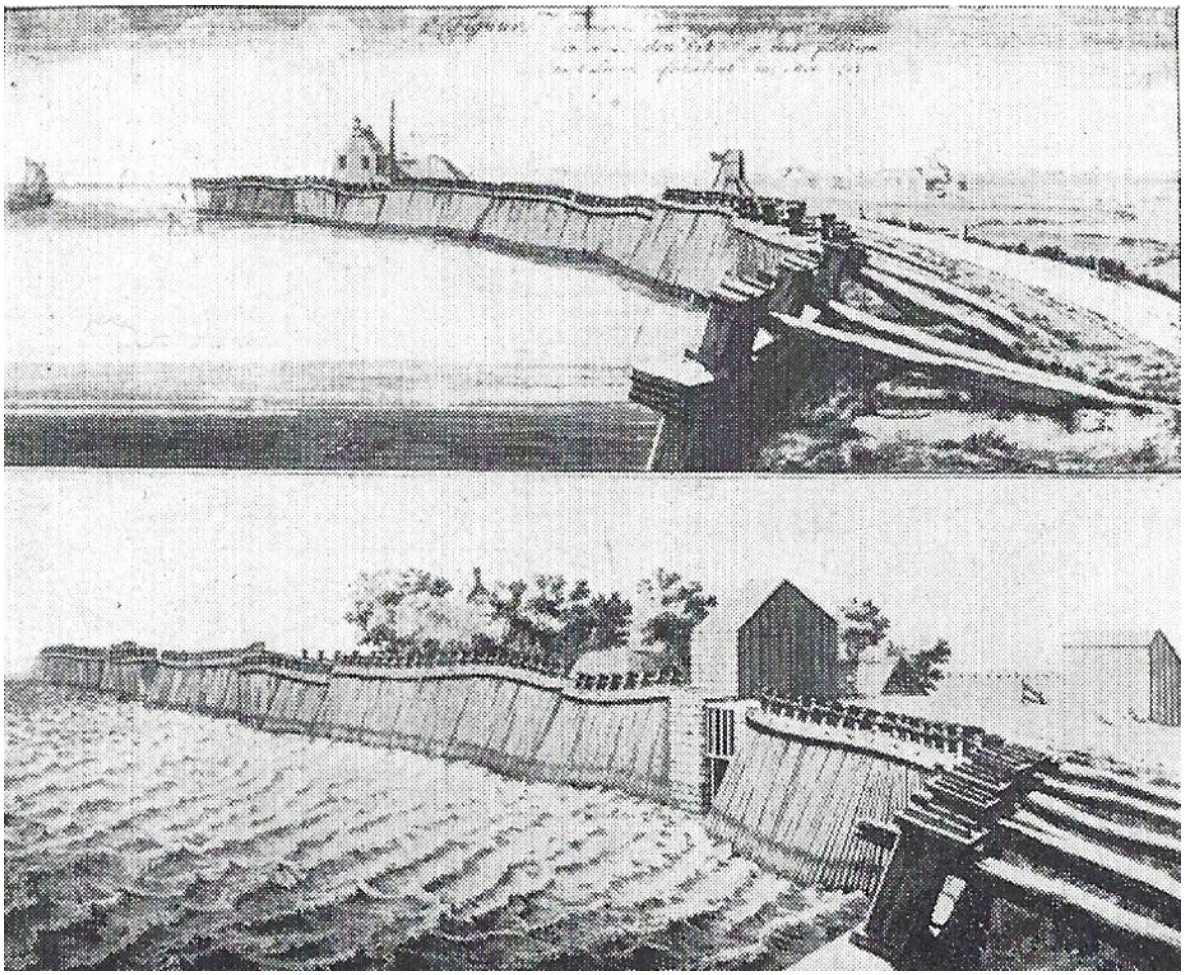


Fig. A.12 Pile screen dike between Amsterdam and Muiden; source: Dibbits (1950, Fig. 85)

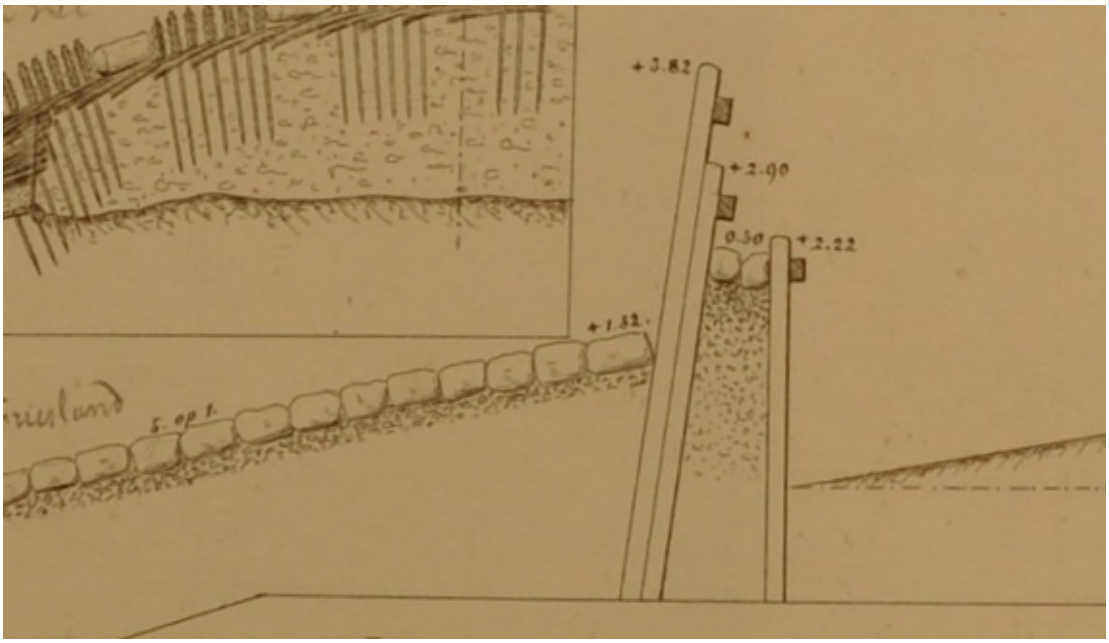
Fig. A.13 The former screen dike near Medemblik exposed; source: Arbeiderspers NV

A.11 Brick work or pile structure

In the province of Friesland, apparently, the wave attack is not as heavy as elsewhere because for a long time slope protection works consist either of a brick revetment (Fig. A.7, top), or a wooden pile structure (Fig. A.14 and A.15), or just a grassed slope. Pile structures are in use in that region since 1574 (Priester, 1915, 25). A detailed description is given in Huitema (1949, 192 – 195)

Around 1810, the Inspector- General of Rijkswaterstaat, A.F. Goudriaan, tries to persuade the waterboard of '*De dijk der vijf deelen*' to introduce a stone pitching because timber, at that time, is very expensive. The documents in the *'Verzameling van stukken betrekkelijk tot het aanleggen van steenglooyingen'* (1815) show that the said waterboard is very reluctant, if not completely unwilling, to accept such a proposal. From Van Buijtenen & Obreen (1956, 387) it follows that as recent as 1925 pile structures are still being renovated in Wymbritseraaddeels.

In 1827, Van Peyma (1827, 45) tries to prove that a grassed gentle slope without further protection is much cheaper than a brick revetment or a pile structure. The pile structure, in his opinion, has only to be replaced once every 40 years. But this might sound rather optimistic to the reader!
The grassed gentle slope of the earth dike, according to Van Peyma (p. 101), can be 1: 12 at places with heavy wave attack, 1: 10 for average wave attack and 1: 8 for moderate wave attack. Such earth dikes should have a height of ± 5 metres above ground level.



detail of the pile structure shown in Fig. A.14 below

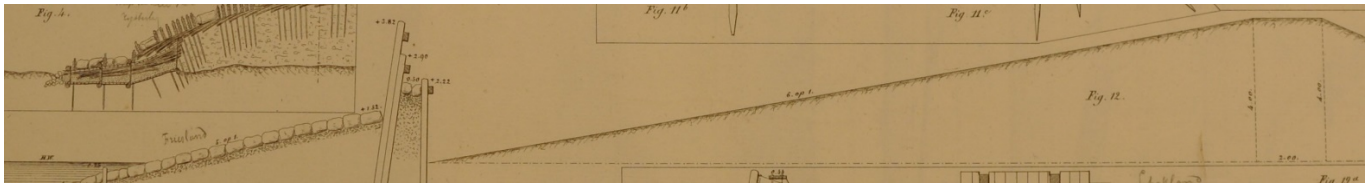


Fig. A.14 Pile structure dike along the west coast of Friesland;
source: Storm Buysing, Atlas (1864, Plate XLVI, Fig. 12)

Fig. A.15 The pile structure near Oude Wirdum (Province of Friesland) with 'krebbinge' at the toe (© Carel Blazer/MAI)

A.12 Spurs shaped and constructed like cofferdams[3] (*kistpaalhoofden*)

These spurs are applied in front of dunes and sea dikes. If placed in front of dunes they are supposed to prevent erosion of the beach proper, which is the sandy area laying above the low water level.
If placed in front of dikes, i.e. on the shore, their function is to prevent erosion of the shore (*voorland*) by shifting tidal channels. If a shifting tidal channel comes near the sea dike it may result in a flow slide (*oeverval*) followed by failure of the dike body because of a slip (*dijkval*).
Moreover, the spurs are supposed to decrease the impact of waves on the dike body and prevent movement of ice floes along the vulnerable slope protection.
In 1533 spurs are already in use along the northern coast of the province of Friesland (Van Buijtenen & Obreen, 1956, 101).

[3] Here, the word cofferdam is used in its original sense: *kistdam* but the name in Dutch of such a spur is *kistpaalhoofd*.

In 1546 an inspection of the coast of the island of Walcheren is carried out by the dike reeve Arent Jansz Boone and his assistants (De Waard, 1912). The purpose of the inspection is related to the possibility of a future military attack on the island. On the basis of this inspection a map can be drawn up (meanwhile lost, but redrawn in 1912!) showing , inter alia, the spurs existing at various points around the island. On the map no less than 12 spurs are shown at Westkapelle, 32 on the southern and south-eastern coast and eight near Veere. From the report made by Boone it would appear that these spurs solely consist of two rows of piles (he mentions in this respect *'ijspaele'*) with stones and fascines in between (p. 20) and that many spurs have to be repaired.

This type of spur (Fig. A.16), apparently applied up to the end of the 18th century, nearly always has the shape of a long cofferdam. It is built up of two or three rows[4] of long driven wooden piles and/or wooden sheet piles. The two rows are firmly connected to each other by a system of walings, struts and braces, bolted together. The rows are filled In between by horizontally placed layers of fascines[5] ballasted by rock. Sometimes the fill only comprises rock (Van Bleiswyk, 1745, 30, 33 et seq., Van Breen, 1920, 202).
These spurs are very costly, as well in initial construction as in maintenance. Moreover, their effectiveness turns out to be questionable. The (nearly) rectangular cross section as well as the vertical ending (head) cause strong eddies to develop resulting in scour around the head and considerable embayment threatening the 'stem' of the spur as well as the toe of the dike. Also the wave attack on the vertical sides causes scour at the toe of the spur. Finally, the wood work lower down is destroyed by the naval shipworm.
By the end of the 18th century it is realized that this type of spur seldom is a solution for the problem at hand. Caland (1833, 277 et seq), states that he, on purpose, will refrain from a discussion about this type of structure, which, in his opinion, is inappropriate. In this respect he also refers to Goudriaan (1798) and to Woltmann (1799, vol.4, 22 et seq.) for a more elaborate presentation about this topic.

A.13 Fascine spurs (Fig. A.17) (*rijshoofden)*

A less elaborate type of spur is also referred to in the literature. In the publications of De Kraker (1997) and De Vleesschauwer (2013) the fascine spurs (*rijshoofden*) are regularly mentioned. They are applied along the coast of Zeeuws-Vlaanderen already at the beginning of the 16th century. Apparently, their structure is not as solid and heavy as that of the cofferdam type of spur. They consist of stacks of fascines kept together by piles driven into the foreshore. Stone can be used for ballasting but that is not always done (De Vleesschauwer, 2013, 205).

De Kraker (1997) gives various details about their application (and failure): In 1545 fascine spurs are covered by clay sods (p. 81). In the same year a new fascine spur (long 54 m) is constructed at another location along this coast but this one is ballasted with stone (p. 84). De Kraker: "After having tried unsuccessfully during seven years to fight coastal erosion near Terneuzen by constructing secondary defences, it was decided in 1559 to construct large fascine spurs instead." (p. 92) Other, already existing spurs get a 'stiffening' *(verstijfuynghe*) of stone (p. 93).

[4] But also some times only one row of piles is constructed.
[5] Or reed, Van Buijtenen & Obreen (1956, 102).

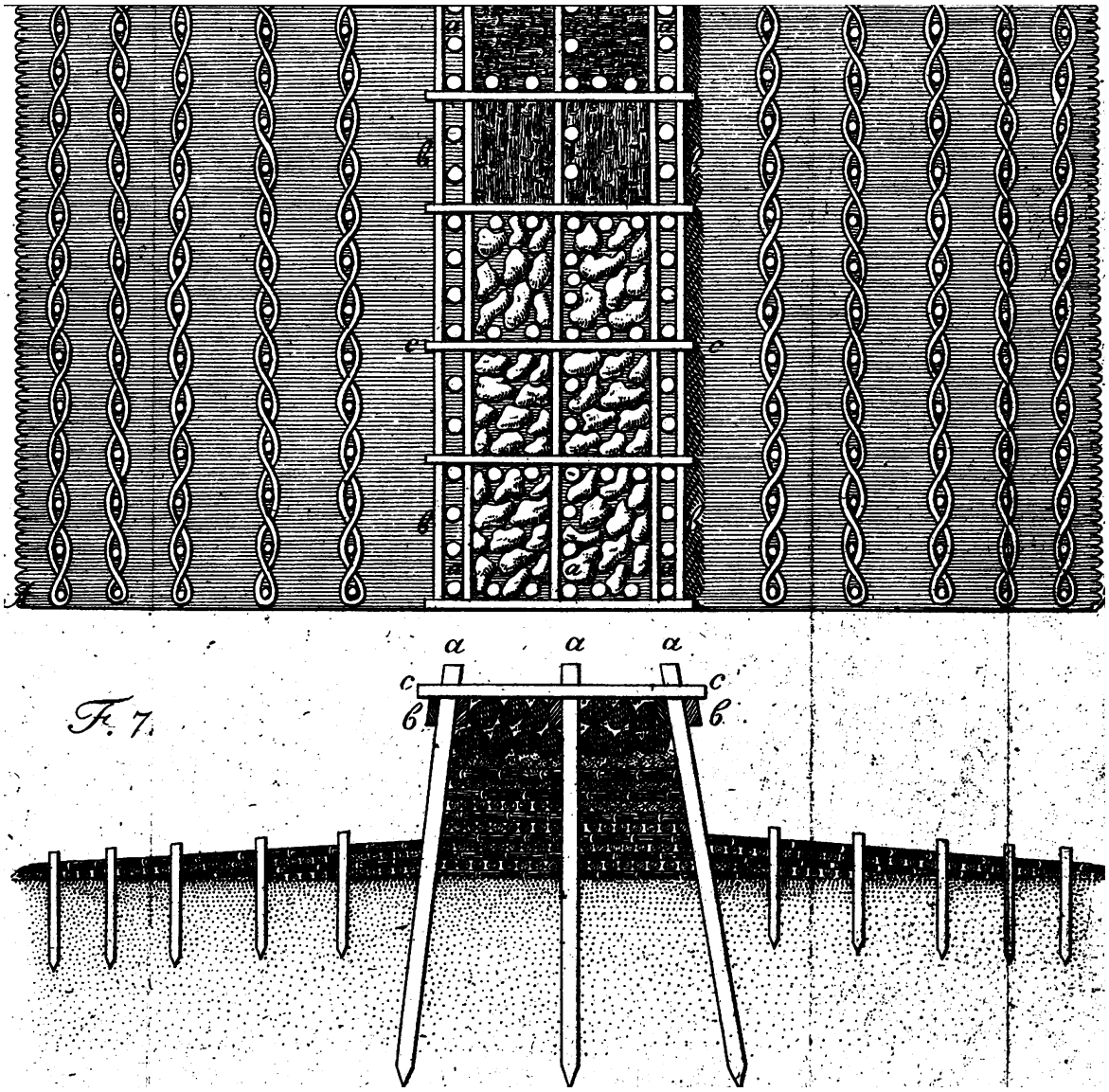


Fig. A.16 A cofferdam type of spur on the island of Walcheren at the end of the 18th century; the base is a fascine mattress provided with wattle work; width on top of cofferdam is 3 to 4 metres; source: Woltmann (1799, Vol. 4, Plate I)

In 1570 some spurs are damaged by ice floes but, in general, the spurs have stopped coastal erosion. For the time being, that is! Because De Kraker (p. 161) concludes that, during the period 1583-1600 the coastal defence in this region has failed completely: all fascine works (including spurs, berms and slope protection) have been seriously neglected.

Though fascine spurs are cheaper than cofferdam type spurs they are more vulnerable for moving ice floes and require more maintenance.

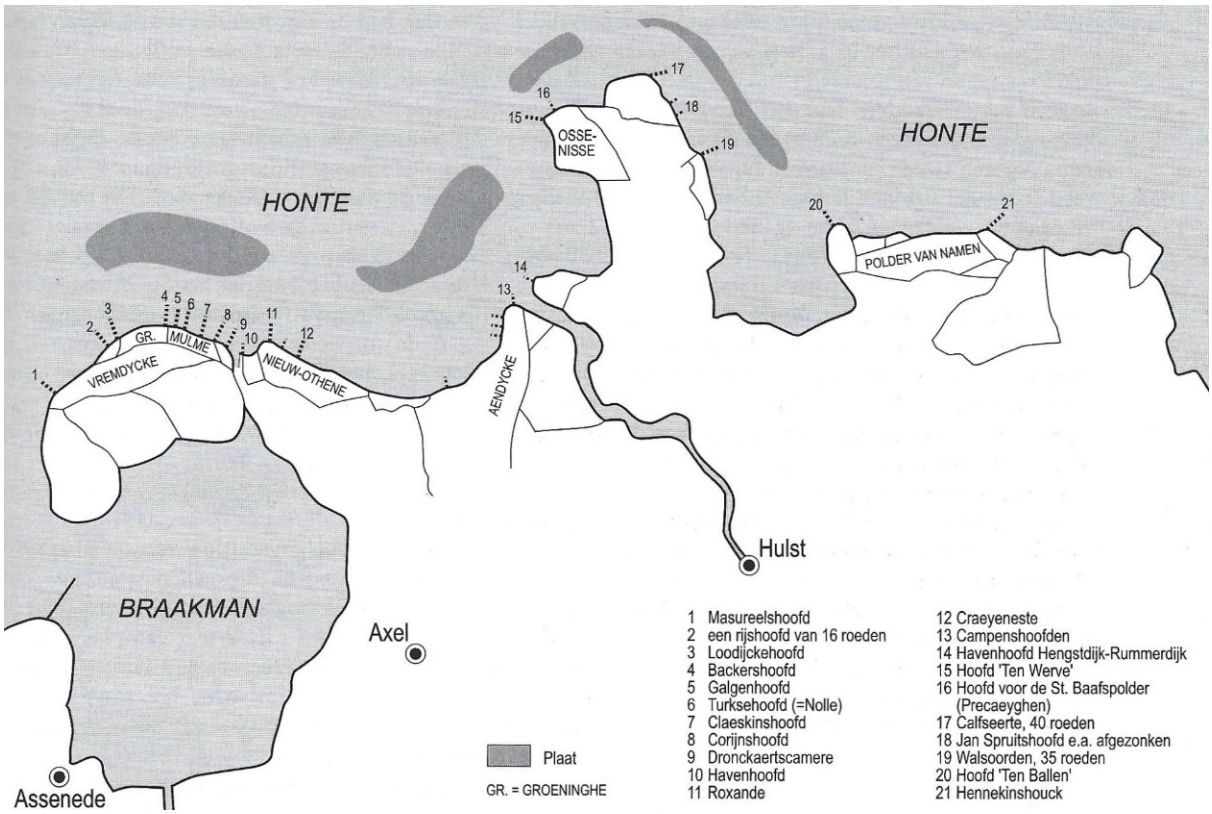


Fig. A.17 Reconstruction of fascine spurs along the Westerschelde (Honte) in 1565; source: De Kraker (1997, 95)

A.14 Spurs along river dikes (Fig. A.18)

During past centuries spurs (*hoofden*) are common along river dikes. They can have a horizontal crest (nearly as high as the river dike proper) or are sloping down from the dike crest to the level of the flood plain (*uiterwaard*). In the first case they are only armoured at the head. The body of the spur consists of earth and the 'stem' of the spur is usually protected by fascines; see for instance Van Vliet (1961, 39). She mentions for the *Lekdijk Bovendams* spurs having lengths between 75 and 100 m.

The function of these spurs is to keep the current and moving ice floes away from the (unprotected) slope of the flood embankment.

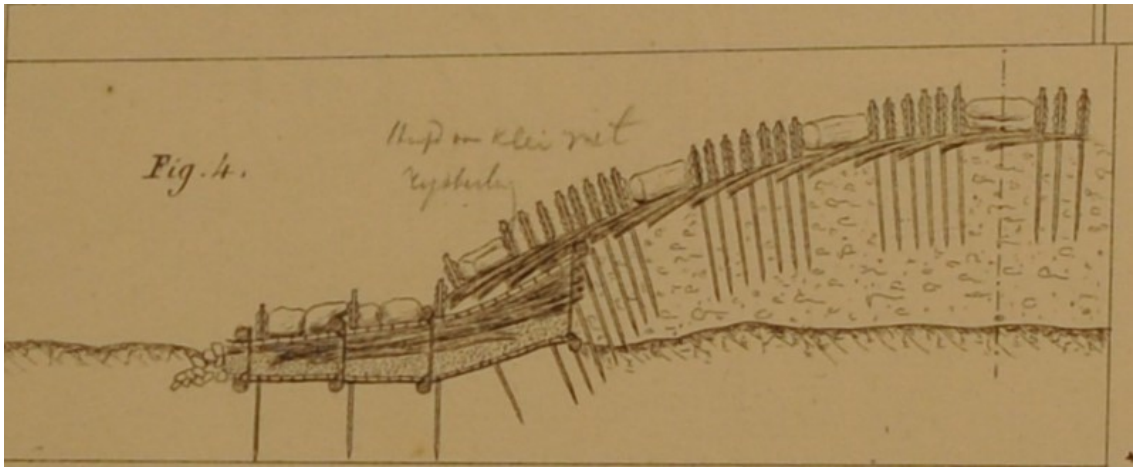


Fig. A.18 Longitudinal section of head of spur made of clay protected by fascines; source: Storm Buysing, Atlas (1864, Plate XLVI)

A.15 Hard points (*bollen*)

This type of spur is mentioned by Van Bleiswyk (1745, 191 et seq.). He states the following:

> "*Bollen* or *hoofden* are not more than a kind of small groynes and are constructed in the same manner but differ from these only in size: as well in mass of the body as in length; they are built from the bank into the river at such places where it is 'not as much the current but the scour which erodes the bank' (sic); this erosion is the more harmful and has to be prevented if there is not much shore between the bank and the toe of the dike; such prevention is achieved by constructing *bollen*, as said, from the banks into the river like groynes, at such distances and direction in relation to each other that the current does not have the opportunity to come near to the bank."

Also Van Breen (1920, 112) defines *bollen*:

> "Old groynes may consist of so-called *bollen, bolletjes* or triangles, which might be considered more as a local bank protection than as part of river normalization works. The horizontal projection of these *bolletjes* is in most cases an isosceles triangle having its obtuse top angle directed towards the river and its base placed against the bank. They are positioned much nearer to each other than normal groynes, this in order to reduce the depth of the embayment as much as possible. The shape of these *bollen* had been chosen well; the water along the upstream side was directed towards the middle of the river and also the overtopping water was diverted from the downstream crest towards the river."

Finally, Kool-Blokland (2003, 76) writes:

> "In the 16th and 17th century many *bolle stukken zinkwerk* were made, called *bollen* or triangles. But they were not always successful, they slid down, broke and floated away in the current. A big disadvantage of their round (*bolle*) shape was that they caused a strange flow pattern."

This is all not very clear and unambiguous and also no drawing of these *bollen* has been found. The reason for the discrepancy between Van Breen and Kool-Blokland might be that Van Breen refers to the use of *bollen* in rivers while Kool-Blokland relates experience in tidal waters with waves (around the island of Schouwen, now part of Schouwen-Duiveland).

A.16 Protection of the foreshore (*vooroever*)

The name of Caland has been mentioned already in Section A.12. As he does not agree with the pre-19th century ideas about, what he calls 'beach and bank defence', he dedicates 140 pages to this subject in his book, which' title can be translated as 'Manual about dike building and the art of sea defence' (Caland, 1833).

Now Caland (p. 146, foot note) considers the bank (*oever*) to be the area in front of a dune, which he calls beach (*strand*), as well as the area in front of a dike (*voorland*), which, depending on its elevation, is called either mud flat (*slik*) or salt marsh (*schor, kwelder*)[6].

Caland makes a distinction between:

- defence against waves, in so far as they tend to erode the 'dry' beach,
- defence against eroding currents,
- a combination of both.

[6]See in this respect also Ch. 2, Section 2.2 and note that there the definition of bank is different.

This is not the place to present a complete summary of all assumptions, considerations, arguments and proposals put forward by Caland. One can however conclude that most of his text about defence against waves concerns beach defence and this topic is not relevant to this dissertation.
Consequently, this type of defence and also a defence against the combination of waves and currents will not be discussed here.
Caland states also (p. 235) that one does not completely understand the effect of waves on beaches and banks and that defence of the banks against erosion by the tidal currents is more appropriate. Eroding currents are due to shifting tidal channels and Caland admits that the shifting as such cannot be stopped (p. 175). He explains the disadvantages of spurs (cost, bad hydraulic shape) as described in Section A.12 above and proposes (p. 177 et seq.) to defend especially that part of the (fore-)shore which is permanently under water[7], i.e. below the low water (or bank-) line. In this area the currents are high and tend to erode the underwater slope of the foreshore. This underwater slope would be protected by a strip of fascine mattresses parallel to the bank up to a level a little bit above the mean low water level (Fig. A.19). At regular distances this strip is connected to the toe of the dike by means of low dams (Fig. A.20) (*slikdammen*).
To save on cost, if required, Caland, subsequently, suggests to leave unprotected areas in the strip (i.e between the individual fascine mattresses). Alternatively, fascine mattresses can be alternated by areas of dumped rubble or broken bricks.

In 1862 the *Raad van de Waterstaat* publishes its report about the protection of the foreshore in the province of Zeeland (*Verslag Oeververdediging in Zeeland*). The authors recall (p. 12 et seq.) various protection projects as have been initiated and constructed from the first half of the 18th century until the middle of the 19th century, including those proposed by Caland. They have meanwhile observed (p. 80 et seq.) that a non-continuous strip of fascine mattresses is not a good solution. Also the durability of already placed fascine mattresses is doubtful because of insufficient ballasting with stone. The reasons in this case are (a) lack of funds and/or (b) stone could at the time of construction not be imported from Belgium (after 1830). They propose a continuous strip comprising a layer of stone , 60 cm thick on top of fascine mattresses (overall thickness of fascine layers 40 to 50 cm). They are not in favour of major spurs (p. 84) but, never the less, indicate how these spurs should be built up, if they are locally needed.
The developments in the 20th century concerning the protection of the foreshore are summarized in Ch. 2, Sub-section 2.7.7, par. 1 of this dissertation.

A.17 Toe protection (Fig. A.21)

Dikes constructed as part of reclamation schemes are normally located on salt marshes only flooded during very high tides. This implies that the seaward slope of the dike in most cases ends on the dry shore. Because of wave attack during storm surges the transition from (protected) dike slope to unprotected shore is a vulnerable spot which erodes by the scouring effect of the waves. For that reason and also as a support for the toe of the dike and its slope protection a fascine mattress, made in situ, is normally placed in front of the toe. This mattress has, depending on the wave attack expected, a width of 5 to 10 metres and contains only two layers of fascines. It is called *kraagstuk* .
Obviously, it is also possible that the shore is at a much lower elevation and that shore and lower part of seaward slope of the dike are permanently under water. In that case the '*kraagstuk* ' will be constructed on a wharf and transported to the dike in floating position[8]. It will then be positioned with its long side parallel to the dike, ballasted and immersed .

[7]Called in Dutch: *vooroever* or *onderzeese oever.*
[8] i.e a floating fascine mattress (*zinkstuk*).

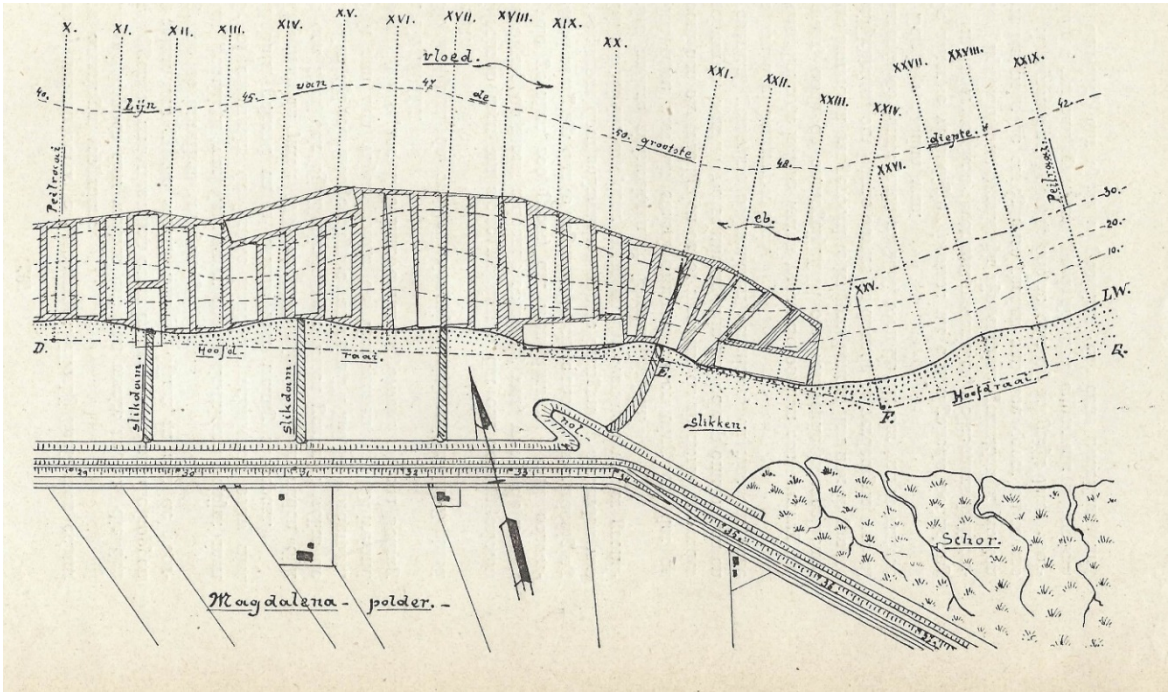


Fig. A.19 Continuous protection of the foreshore by means of fascine mattresses; note the low spurs (*slikdammen),* (see also Fig. A.20 below) connecting the sea dike and the protected zone; source: Van Breen (1920, 168, Plate 38)

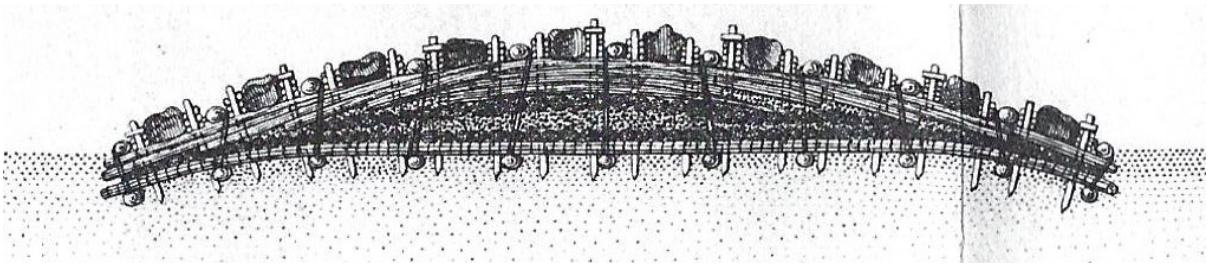


Fig. A.20 Cross section of low spur (*slikdam*); source: Caland (1833, Plate III, Fig. 20)

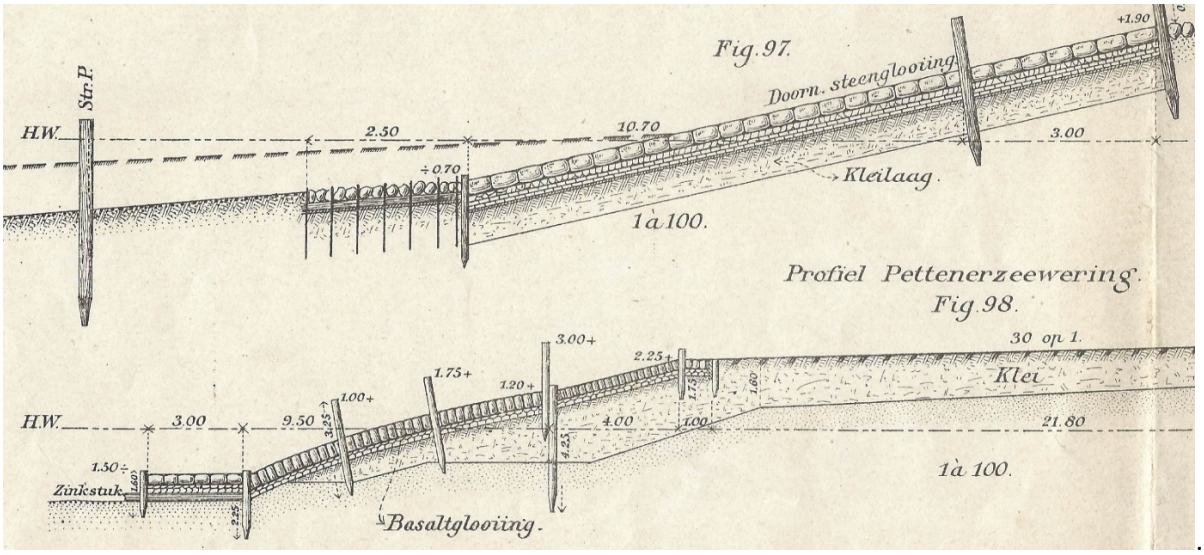


Fig. A.21 Two examples of a toe protection (*kraagstuk,* in the figure called *zinkstuk)*); in this case the widths are only 2,50 and 3 metres; source: Plasschaert, Atlas (1902a, Plate VIII)

APPENDIX B[9] The suspended fascine mattress as described by Smallegange (1696, 197)

‘De Zee- werken worden meest gemackt van rijs, op verscheide lange welien gelegt, en dan met goede, taeye , en stijve sooden gevult ; en daer over wederom rijs gelegt; dan eindelijk de welien by een gebragt , en wel gesloten zijnde , geheel het werk gesonkken; sulks dat de meeste konst bestaet , in dat het gewicht met goede over- een-koming tegens worde gelijk gemackt. De steen in dese sijnk-werken is beter als sooden , maer valt vele polders te kostelijk.’

[9] Appendix B is referred to in Chapter 4.

APPENDIX C[1] Extract from the Report by the Gecommitteerde Raden van West-Friesland en het Noorderkwartier te Hoorn aan de Staten van Holland, March 1664[2]

Van de zeugen

De seugen ofte sinckwerken om de schadelijcke cours van de stroom te stuijten worden in Zeeland gemaakt op deze wijze

Twee scheepen een met rijs en 't ander met steenen gelaaden worden door uijtsteeckende houten aan malkanderen vastgemaakt soo nogtans, dat tusschen beijde een wijde openinge blijft van 15 a 16 voeten, tusschen 't welke op eguale distantie 6 touwen van 6 a 7 duijm dik in de scheepen overs handts wel vastbeleijd, met een bogt van seven (à) agt voeten diep ende op gelijcke wijse nog 24 a 25 willigen van bitterwillig gedrait[3] gehangen worden, waarop dan met magt van volk de reijsbosschen eerst langs op de voorn. touwen ende willigen tot op de hoogte van omtrent drie voeten, ende dan wederom dwars daarover twee a drie bossen dik gelijd worden het welke dan door kneppels, lang omtrent vier en een halve voet, in vier rijen aan een geslagen, ende met knijpwerk aan malkanderen gebreijd word; daarop wordt dan steen geworpen tot sinckens toe, ende voorts wederom met rijs langs en dwars over heen gewerkt met kneppels en tuijnwerk, wel vastgeslaagen en doorvlogten, tot dat de seuge 7 a 8 voet hoog ende 12 voeten breed word, op de lengte van 60 a 70 voeten, ofte soo lang als de scheepen sijn, als wanneer deselve dan met de voorschreve willigen omgord, met kracht toegedrait en wel vastgemaakt zijnde, de voorn. scheepen met hare anckers in zee, en trossen aan land, eeven op die plaatse daar men het stuk sincken wil, vast zijnde, op een geroep vant woord overal de boven genoemde ses touwen 't effens losgelaten ende alsoo gelijckelijk neder gesonken word; welke gelijt soude worden volgens nevensgaande afteeckeninge int Diep, op distantie van 10 a 12 roeden, uijtbrengen, de 181 stuks, en na de markt vant rijswerk, stene en arbeijtsloonen in Zeeland geraamt, te zullen bedraagen yder stuck omtrent 300 guldens, monterende op de voorn. 181 zeugen , een somme van vier en vijftich duijsent drie honderd guldens, segge 54300 gls

[1]Appendix C is referred to in Chapter 4.

[2] Noord – Hollands Archief, Haarlem, Access code 3: Gecommitteerde Raden van West-Friesland en het Noorderkwartier te Hoorn , inventarissen 338-823: Ingekomen stukken, plakkaten e.d van de Staten-Generaal en de Staten van Holland. The manuscript can be found in Inventory 583 (= jaar 1664), sheets 36 to 42 (incl).

[3] De schrijver zal bedoelen betrekkelijk dunne strengen samengesteld uit in elkaar gevlochten tuinlatten, De diameter van zo'n streng zal niet meer zijn dan 5 à 6 cm. Het is dus geen wiep (diam.12 à 13 cm). Zie voor de definitie van tuinlatten Van Breen (1920, 43).

APPENDIX D[1] Floating fascine mattresses[2]

D.1 Background

Of old fascine[3] mattresses composed of brushwood are being used in The Netherlands in the struggle against the sea. The mattresses are towed over water to their location and immersed by ballasting them with either boulders, sand bags, clay clods or combinations of same.
The fascine mattresses have a wide range of applications, such as:

- protection of embankments against scour and wave attack,
- foundation of dikes and sluice-caissons,
- fascine dams, by sinking in succession one mattress onto the other until a dam has been formed,
- protection of the bottom of tidal creeks against scour during enclosure operations, when high current velocities occur,

Through the ages, up to the mid-sixties of the 20th century the mattresses are used in their oldest form.
The construction is of such a simplicity and efficiency that up to the sixties of the 20th century no suitable substitute could be found, which could compete with the performance of the mattresses.
The construction of the mattresses is very labour-intensive and thus offers an opportunity for unskilled unemployed labour to earn an income.

D.2 Construction

Prior to giving a description of the construction method of a 'standard' mattress, the various materials etc. being part of it will be described.

D.2.1 Brushwood materials (Fig D.1)

Brushwood, in a bundle, is called a fascine, it comprises tough branches of willow, osier, oak or hazel, bound by two withes.
Fascines come in different sizes (length and circumference) depending on the application envisaged but also on the kind of wood and the region where the wood grows. The longest fascine measures 3.75 m and has a circumference of 70 cm.
There are many more parts of the brushwood which have a function in fascine works. Caland (1833, 388), and also Watson (1875, 158 et seq.) mention, inter alia, '*staken, tuinlatten en gaarden'* (stakes having lengths of 1.20, 2.20 and 3.00 m respectively). Also are mentioned '*wiepband, knijpband en kruisband'* which Watson translates 'as osiers to be used as withes and binders'.

What is written above is only meant to be indicative. The dimensions quoted are those of the 19th century and they may have changed in time. There are also accepted tolerances in these dimensions and, moreover, the dimensions differ for each kind of brushwood (*rijshout*).

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[1] Appendix D is referred to in Chapter 4.
[2] This document has been based on the Appendix written by Van Duivendijk as part of his First Interim Report on the 'Study of methods used for closure of tidal channels' in Bangladesh (1976). The original Appendix has been adapted and corrected in a number of places. It has been added to Chapter 4 of the dissertation to provide outsiders, in the matter of fascine application in closure works during past centuries, some background in this specific field. As, originally, the Appendix was drafted bearing in mind Bangladesh' circumstances, certain statements may sound odd to Dutch readers. For instance in the Netherlands a labour-intensive construction is not considered an advantage but in Bangladesh it is. Also the use of sandbags as ballast on fascine mattresses has been mentioned because in Bangladesh rock and boulders are either not available or expensive.
[3] Throughout the text the word fascine is used but in the Figures the word fascine as well as the word faggot is found which, actually, means the same. In the text the word grating is used and grillage in the Figures. Finally, in the text the words binder and withe appear while in the Figures one uses the word sling.

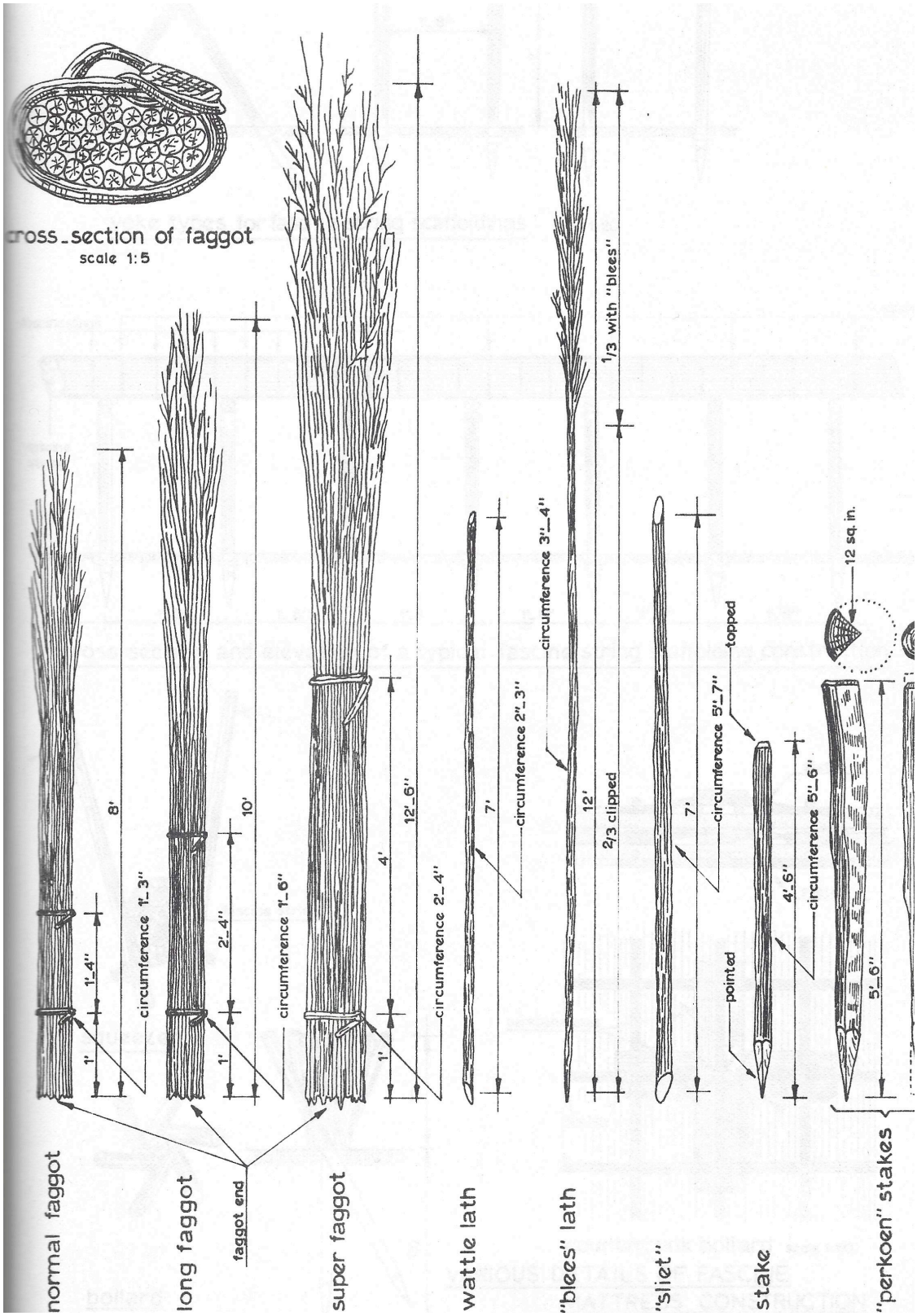


Fig. D.1 Brushwood materials; source: BWDB (1976)

D.2.2 Construction parts of fascine mattresses (Fig. D.2 and Photo 18)

D.2.2.1 Fascine_strings

Fascine strings are long flexible strings (*wiepen*) made out of fine tough branches. The normal circumference is 40 cm.

The fascine strings are made on a scaffolding, which consists of yokes placed at intermediate distances of 50 cm. The yokes are constructed using stakes.

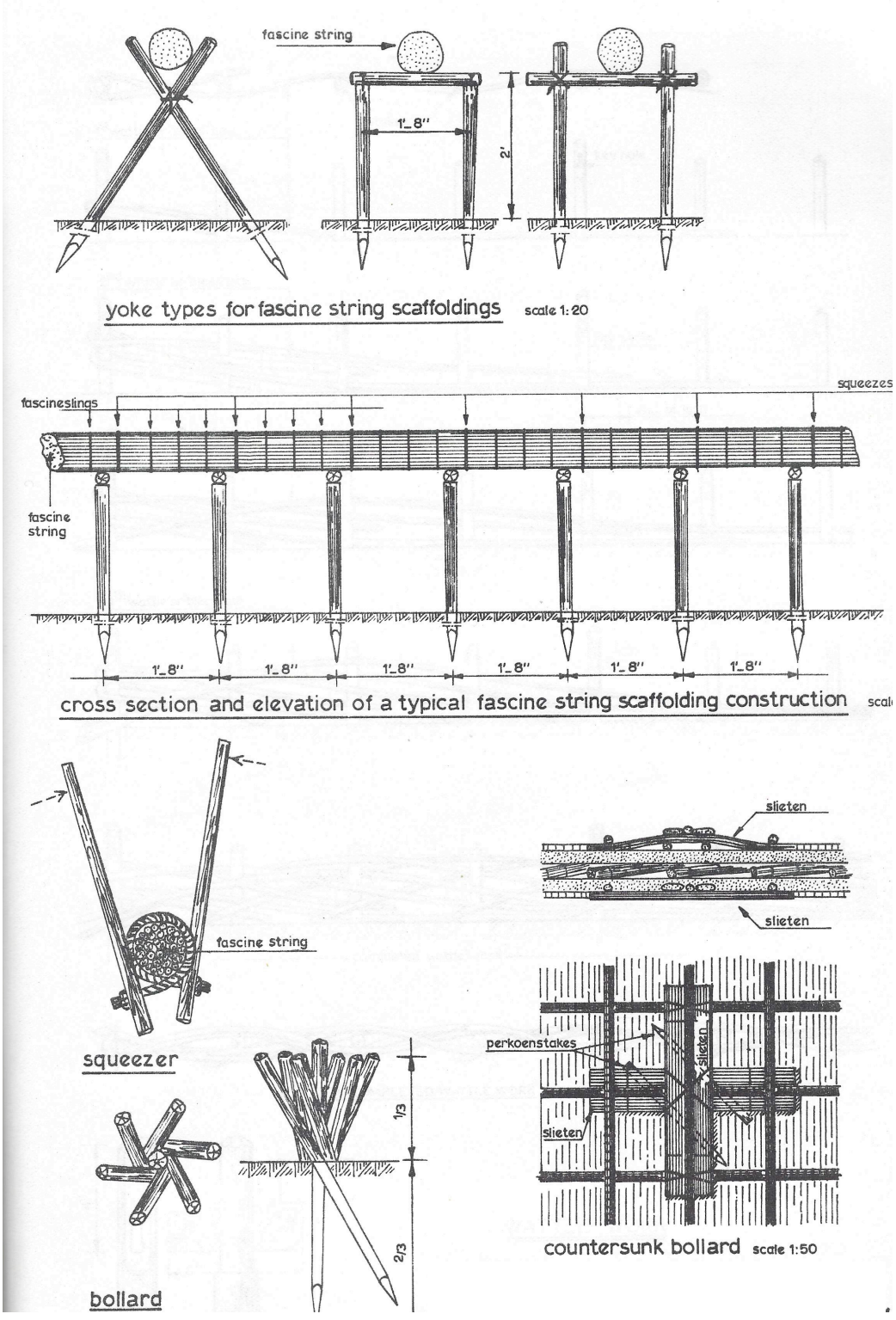


Fig. D.2 Various details of fascine mattress construction; source: BWDB (1976)

When starting the 'spinning' of a fascine string (*wiep*) a normal fascine forms the initial stage of the string. This fascine is extended into a fascine string, with branches from fascines of which the withes have been cut.

The branches are arranged in such manner, that the finer twigs (*blees*) come at the perimeter of the fascine string (*wiep*), which results in a relative smooth surface.
First labourer in the production line of a fascine string is the so-called "*spinner*" who arranges the twigs in the shape of a fascine string, second comes a 'worker' who bundles the string with a squeeze binder at the location of each yoke.
In between the squeeze binders, which are applied every 50 cm, three fascine withes are placed by other labourers, Accordingly, the fascine string (*wiep*) when completed is bundled every 12.5 cm.
The fascine strings can be reinforced by incorporating a 3 mm diam. wire in the centre of the string. This reinforcement might be needed when excessive tensile forces are to be expected, for instance due to high current velocities during transport and sinking of the fascine mattress, or due to a heavy swell.
When constructing fascine strings having a circumference exceeding 30 cm a "squeezer" (see Fig. D.2) is used to squeeze the string sufficiently when fixing the withes.
Fascine strings are made in lengths of max. 5 m. If the length is longer transport of the strings becomes too difficult and buckling might occur.
The fascine strings are to be considered as the cables or framework of the mattress. For the construction of a fascine string five labourers are needed, one for the spinning and four for bundling the string.
An experienced crew of 5 labourers is able to produce 450 m of fascine strings during a day.

D.2.2.2 Bollards (Photo 21)

Bollards consist of 7 short poles arranged as shown in Fig. D.2. The poles are driven into the mattress in order to have a fixing point for anchor cables and hawsers on the mattress.
When mattresses are sunk one onto the other (sandwich wise) the bollard arrangement with poles protruding out of the mattress cannot be used and a "countersunk bollard" is applied (see Fig. D.2).
A protruding bollard, as mentioned above, would result in some room between the various mattresses which would result in piping or even worse: mattresses could be washed away as the currents would get a grip on the mattress.

D.2.2.3 Wattle work (Fig.D.3)

Wattle work is made out of stakes and wattle laths or *blees*- laths. The stakes are driven into the mattress up to certain height and between the stakes wattle- or *blees*-laths are plaited. After the wattle laths have been fixed, the stakes are driven deeper (when the mattress is afloat) until their height above the mattress is 12 to 15 cm. Every fifth stake is a key stake having a protruding key or wedge at the top. When driving these stakes down the wedge compresses the wattle work into a compact wicker work.

The reason that the stakes are driven down only after the plaiting is complete is to ensure ample working space during the plaiting works and to avoid that the mattress is pinned down to the wharf on which it is made.
The wattle work fixed to the mattress gives an extra reinforcement to the construction and keeps the ballast from rolling-off the mattress. -
When large forces are anticipated, heavy wattle work is made by plaiting bundles of two or three laths at a time.

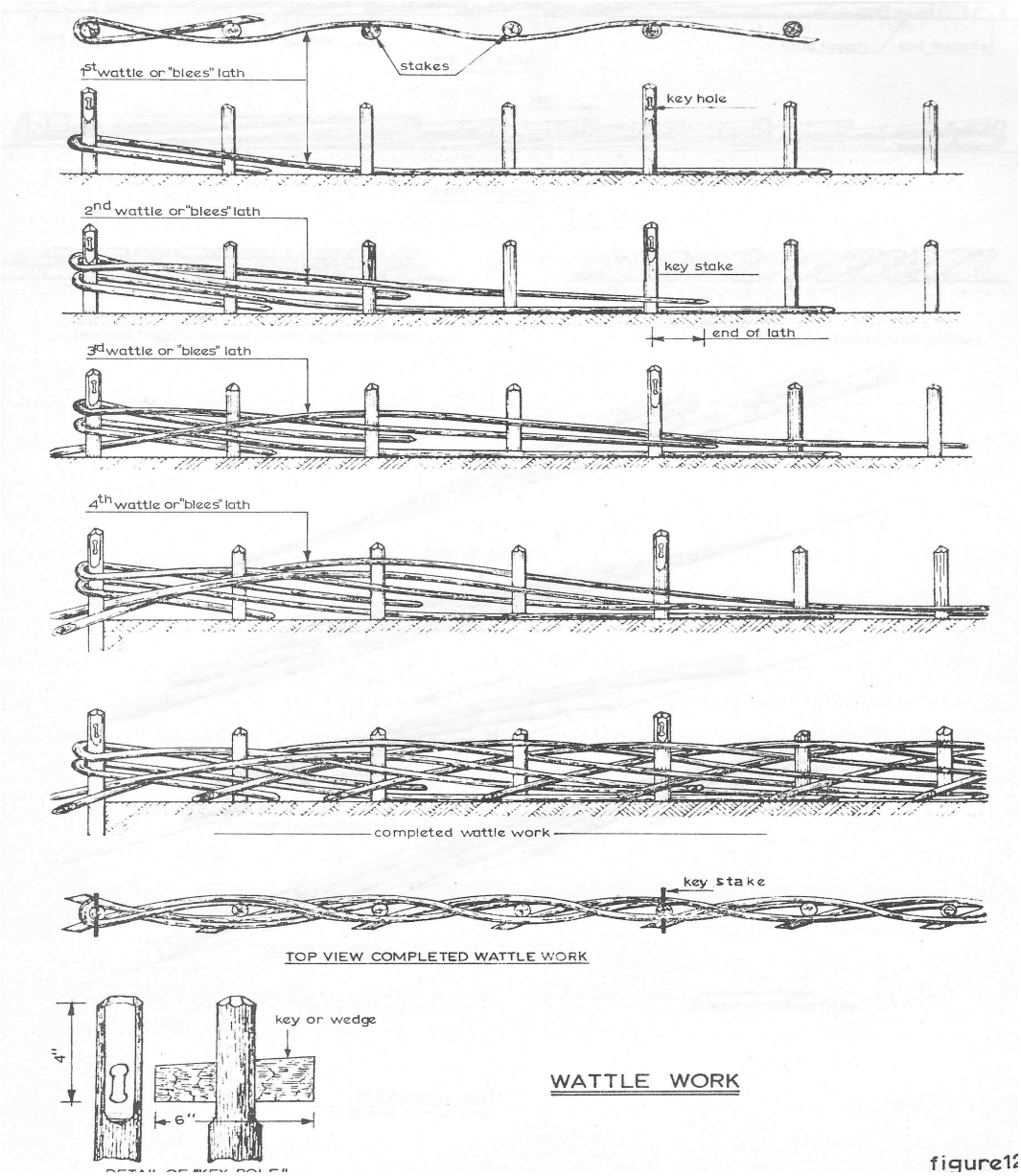


Fig. D.3 Wattle work; source: BWDB (1976)

Apart from the stakes the wattle work is fixed to the mattress with cross-binders.
During the sinking operations or because of wave action the mattress will be subject to bending. Consequently the wattle work will be alternately compressed or stretched. In order to follow these movements, fixing of the wattle work to the mattress has to be thorough (to avoid buckling) and the ends of the wattle laths have to be placed sufficiently beyond the poles to avoid that the wattle work springs open when stretched.

D.2.2.4 Fill of the mattress by placing layers of fascines (Fig. D.4, Photo 20)

Inside the mattress, fascines can be placed in different bonds, each with its own typical characteristics:

'top up bond' the tops of the fascines are placed on the end of the underlaying fascines with the tops pointed to the crest of the embankment[4]. By placing the fascines in a top-up bond a smooth surface of fine twigs is created;

'top down bond' the tops of the fascines are placed on the end of the underlaying fascines with the tops pointed to the toe of the embankment. By placing the fascines in a top down bond a smooth surface of fine twigs is created;

Note: the top of the fascines is to be placed in the direction where the greatest force (current, wave action) is to be expected to avoid damage. When placing the above mentioned bonds horizontal the bond is called 'top up bond'.

'end up bond' the ends of the fascines are placed on the tops of the underlaying fascines. By placing the fascines this way a smooth dense base layer is created. A good connection with the subsoil is achieved, which reduces washing away of fine particles in case of under- currents.

D.2.3 Composition of mattress

D.2.3.1 Site selection

Mattresses are constructed on a construction site ('*zate*') which will be above water level during low tide, and just one metre below the water level during high tide.

During high tide the mattress will become afloat automatically and thus can be towed by one or more tugboats or by other means to the area where it will be immersed.

In view of the rough underside of the mattress high friction and thus high towing forces will have to be anticipated. In order to decrease the towing power needed, the mattresses, if possible, should be transported in a downstream direction.

For instance, when sinking a mattress during slack water at high tide, the mattress will preferably be transported during high tide in the direction of the flood current and, consequently, the zate should be selected upstream of the , immersion' site.

Because of the limited time available between tides, the greatest possible number of men has to be set to work to construct the mattress.

D.2.3.2 Assembling the mattress (Figures D.5, -6, -7, Photos 19 and 20)

A 'standard' mattress consists of three layers of fascines between an under- and top-grating of fascine strings (Fig. D.5).

On part of the top grating, wattle work is placed to reinforce the mattress and to fix the ballast materials.

The mattress is provided with bollards and sinking straps (Figure D.5).

In the old days, to avoid problems when transporting and sinking the mattresses, the maximum mattress dimensions were approx. 150 by 15 m. By using modern equipment larger mattress dimensions can be realized.

As soon as the dimensions of the mattress are known, the outside dimensions are pegged out on the *zate*. Along the perimeter, poles are placed at a distance of 90 cm centre to centre to indicate the position of the fascine strings, which have been prefabricated the previous day. The bottom fascine strings are placed on the zate in parallel lines and on top the subsequent layer is placed at right angles (ref. Fig. D.5).

This grating of fascine strings is called the under-grating.

[4] Obviously this remark only applies to fascines in a slope protection.

The bottom fascine strings are placed such, that after sinking the mattress, the fascine strings are at right angles to the under current, this to avoid a water current between the strings and to avoid subsequent scour.
If in one direction high tensile forces are expected, the mattress can be reinforced by reducing the distance between the strings in that direction.
The crossings of the fascine strings are fixed with lash- ropes and cross withes (Figures D.5 and -7). Lash ropes are used at the crossings of the two outer fascine strings and moreover, at every other crossing. All other connections are made with two cross withes.

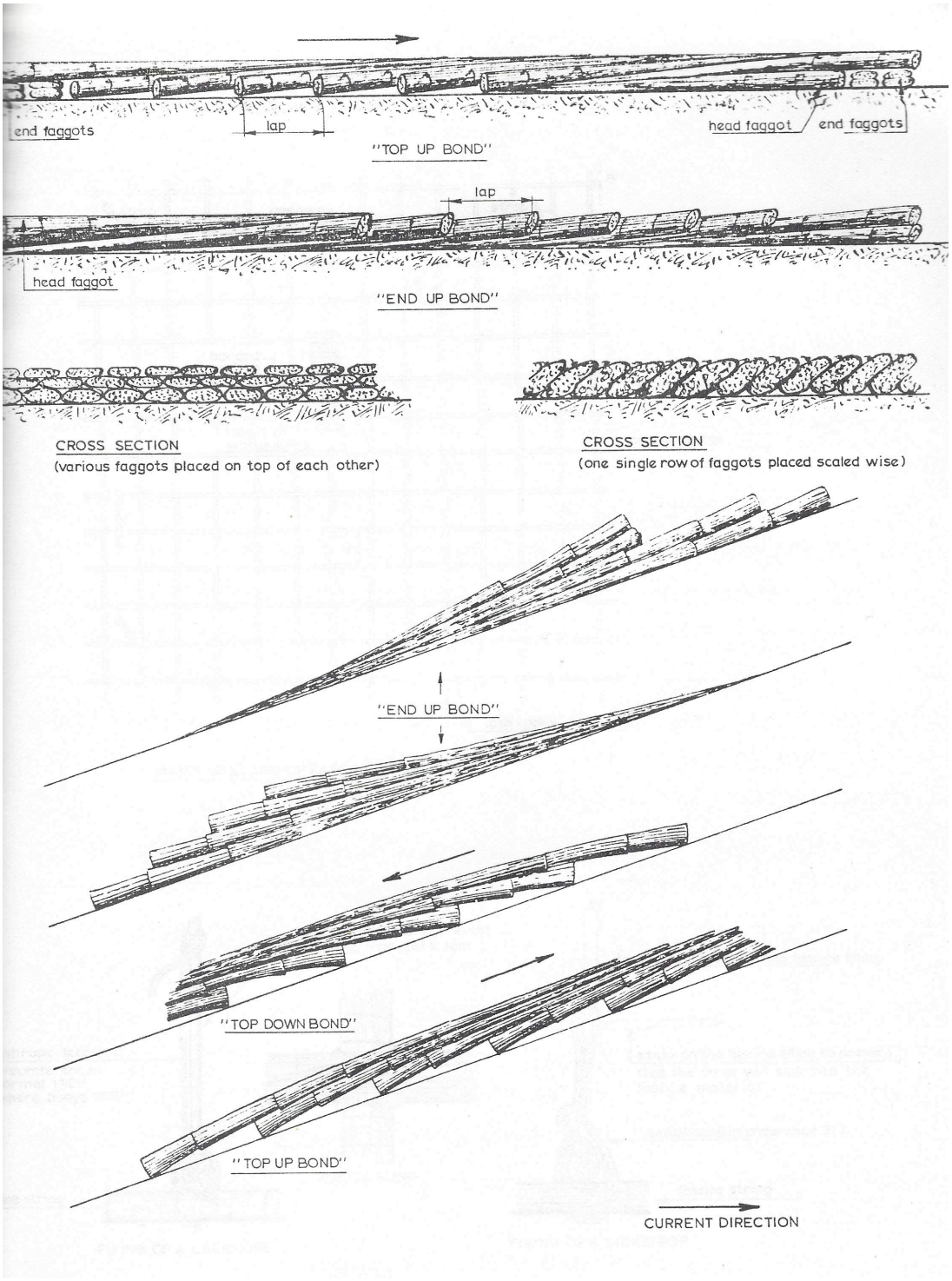


Fig. D.4 Various methods of layering the fascines; source: BWDB (1976)

Apart from the above mentioned lash ropes, sink straps (to be connected to sink lines) are taken up in the mattress construction. The sink straps are fixed to the under- grating at the corners and in between at a distance of approximately 9 m centre to centre.
During the construction of the under-grating the location of the bollards has to be decided upon, as at the location of the bollards, bundles of wattle laths are fixed to the under-grating, to reinforce the mattress and to fix the pole points of the bollard. (Figures D.2 and D.5).

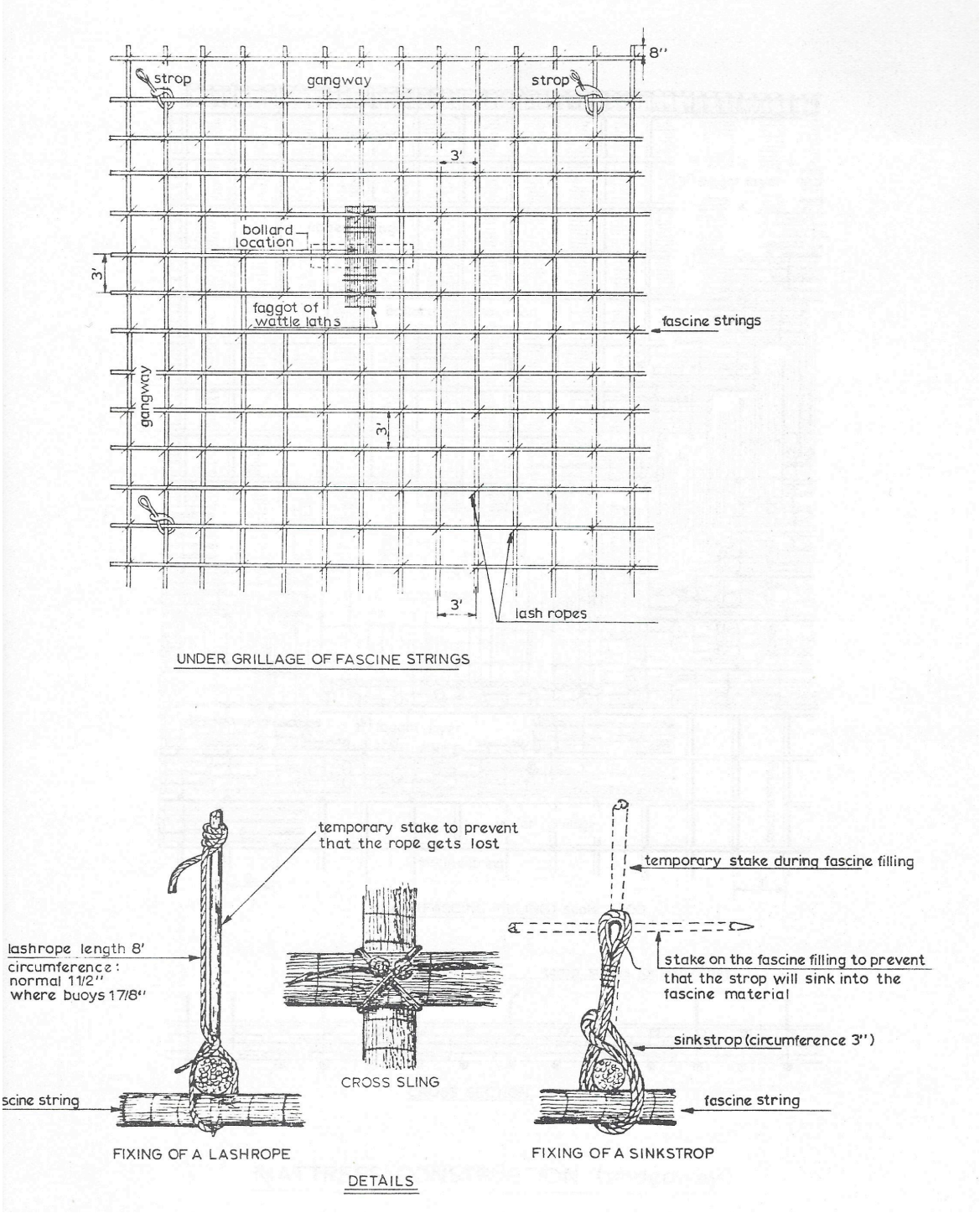


Fig. D.5 Mattress construction (start); source: BWDB (1976)

The bollards are placed at a minimum distance of 4.5 m from the perimeter of the mattress.
As soon as the under-grating is completed three layers of fascines are placed, with a total thickness of 40 to 45 cm (ref. Fig.D.6, Photo 20). The fascines of the bottom layer are placed between and

parallel to the fascine strings of the grating; the layer is placed in an "end up bond" and forms a direct cover of the (sea) bed.
To ensure sufficient support the fascines are placed with the end binders just above a fascine string, in this way the ends are protruding sufficiently beyond the strings and losing the fascines during transport or sinking operations is out of the question.

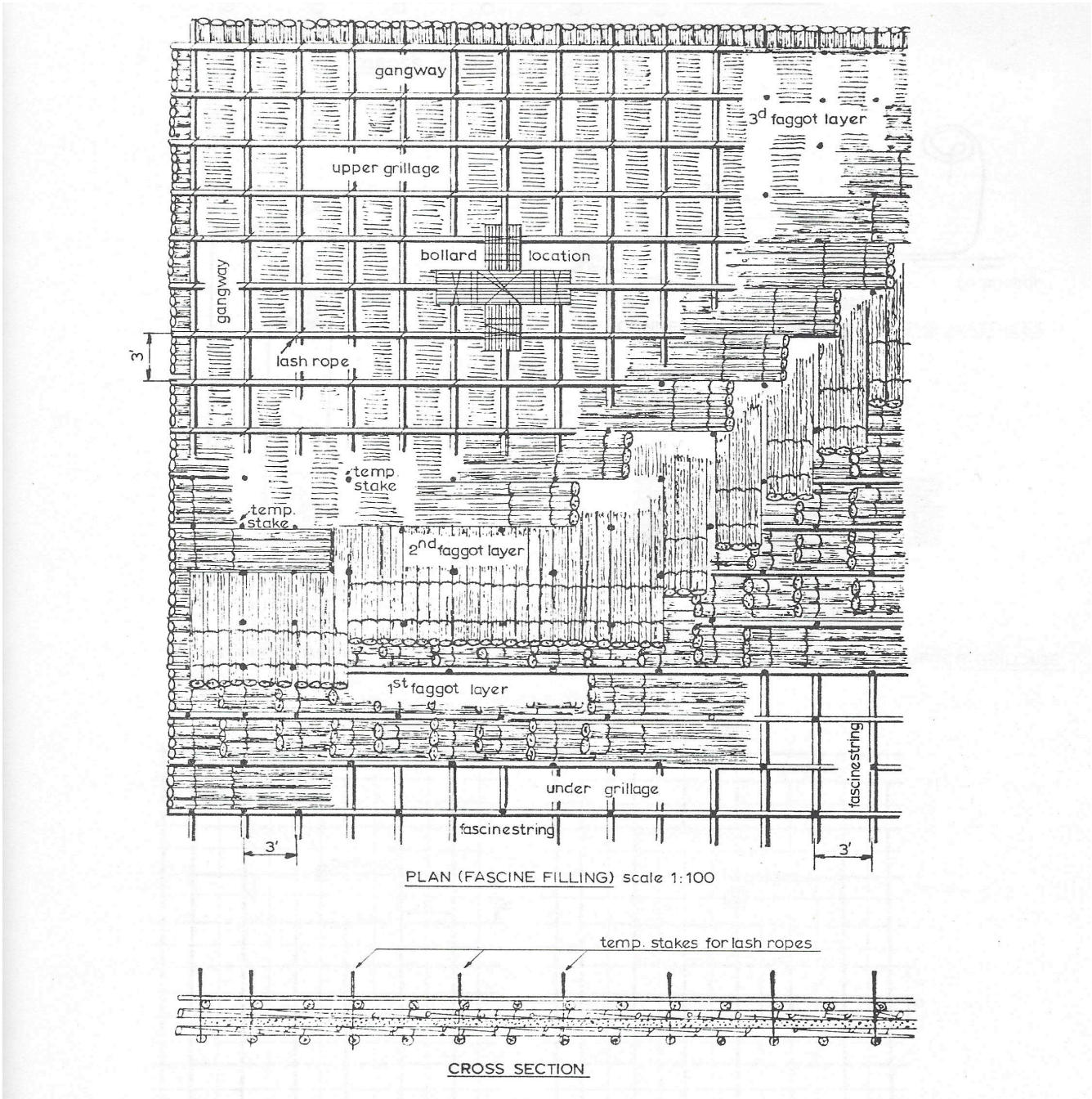


Fig. D.6 Mattress construction (underway); source: BWDB (1976)

At the perimeter of the mattress the fascines are placed more compact (scale-wise, see Fig. D.4), to reinforce the perimeter and to build-in a resistance against collisions. Besides, the buoyancy of the perimeter will be increased, which facilitates the sinking operations. Moreover, the perimeter will not dip below the water too soon, when several workers are walking on the gangway during manoeuvrings.

The second and third fascine layers are placed in a "top up bond" and placed each time at right angles to the underlaying course, with the end binders of the fascines just above the fascine strings. The location of the fascine strings (crossings) of the under-grating is indicated by the temporary stakes of the lash ropes, which protrude above the fascine filling.

As was the case for the first layer the fascines of the second and third layer are placed dense (scale - wise) at the location of the gangway in order to reinforce the outline and to increase the buoyancy of the mattress.

Although in general mattresses are made of three layers of fascines more layers can be applied if desirable. After completion of the fascine layers, an upper grating of fascine strings is placed. Please note that the bottom of this grating must be placed at right angles to the fascines of the underlaying fascines.

The fascine strings are placed just above those of the under grating. In order to render this possible the temporary stakes of the lash ropes are used as a reference.

After the fascine strings of the upper grating are lined up the lash ropes are disconnected from the temporary stakes and fixed to the crossings of the upper-grating (Fig. D.7**).** Under-and upper-grating are connected thoroughly and the fascines compressed in between.

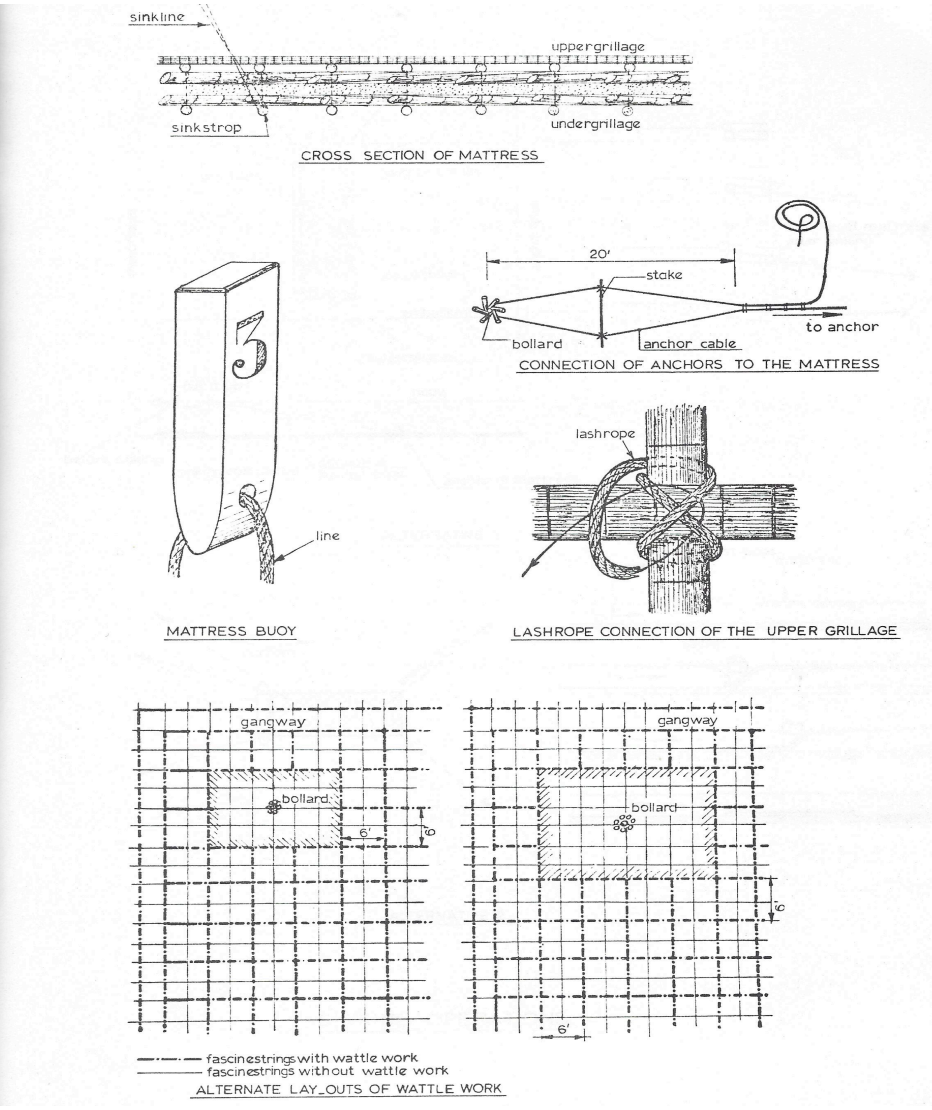


Fig. D.7 Plan of wattle work and bollard, various details; source: BWDB (1976)

At crossings where no lash ropes were placed, the fascine strings are connected with cross binders.
At the location of the bollards, bundles of wattle laths are fixed to the grating, to support the bollards, to reinforce the mattress and to spread the anchor forces.
On some fascine strings of the upper grating wattle work is fixed. The wattle work is placed on the two outer fascine strings along the perimeter of the mattress, forming the 'gangway' of the mattress. Between the gangways, wattle work is fixed at every other fascine string, forming squares of 1.80 m by 1.80 m in which ballast is thrown during the sinking operations {ref. Fig. D.7).
To locate the mattress during and after sinking, buoys are fixed to the mattress at the corners and in between at a distance of 30 m centre to centre. The buoys are fixed to the upper-grating with lines, which are exceeding the sinking depth by 2 m, in this way the position of the mattress during and after sinking can be checked under all circumstances.

NOTE
Instead of fascines reed can be used.
The top layer of the fascines, however, under all circumstances must be made of twigs as reed is damaged easily by walking men and ballast materials etc.
Reed has not been used frequently in sea works. Coverage of the sea bed is excellent and the material **is** not attacked by the ship-worms, but the material makes the mattress too light. The result is a greater uplift and the sinking of the mattress (after ballasting) becomes a too slow process.
Consequently, the possibility that the mattress is not reaching the intended location on the sea bed is increased and, moreover, toppling of the mattress might occur.
For temporary works, as second layer, materials such as straw can be used.
In order to reduce in the Netherlands the big labour force needed to fabricate a conventional 'standard' mattress, in recent years combinations of the conventional fascine mattress and synthetic fabrics are made. Spinning of fascine strings is modernized and the strings are now produced by a semi-mechanized procedure.
In Bangladesh, however, the laborious construction of the conventional mattress forms an asset, as many landless labourer's would find employment if mattresses are constructed in the conventional manner.

D.3. Immersion of a mattress

D.3.1 Transport

Transport of fascine mattresses takes place in the direction of the longest mattress side.
A barge, loaded with ballast materials is placed transverse wise in front of the mattress. At one side the bollards of the barge are connected with hawsers to the bollards of the tugboat, at the other side the bollards are connected to the bollards of the mattress (Photo 24).
The barge forms a stiff beam needed to transfer the hawser forces from the tugboat to the mattress.
As described previously (ref. site selection, Sub-section D.2.3.1), transport preferably takes place in the direction of the prevailing current.
When transporting large mattresses often a second tugboat is employed to push the mattress at the rear.

D.3.2 Positioning (Fig.D.8, Photo 21)

The ultimate location of the mattress is set out in advance and marked with beacons.
The mattress is floated to a spot upstream of its final location and provisionally anchored.
The place of the anchors is set out in advance and marked with beacons (ref. Fig. D.8, Anchor lanes).
As a next step the mattress is brought approximately in position by veering out the anchor cables and additional anchors are fixed in all other directions.
To facilitate the lifting after sinking operations the anchors are marked with small buoys.

Depending on the currents and the water depth, the anchor weight is 100 to 150 kg. and the cable length 100 to 300 m (min. 3 times the water depth).
On the mattress each anchor cable is connected to a bollard (ref. Fig.D.7 and -8) and the cables are pulled through manually until the mattress is just upstream of its final position.
During sinking, the mattress is displaced by the water current which will rectify the 'just u/s' position.

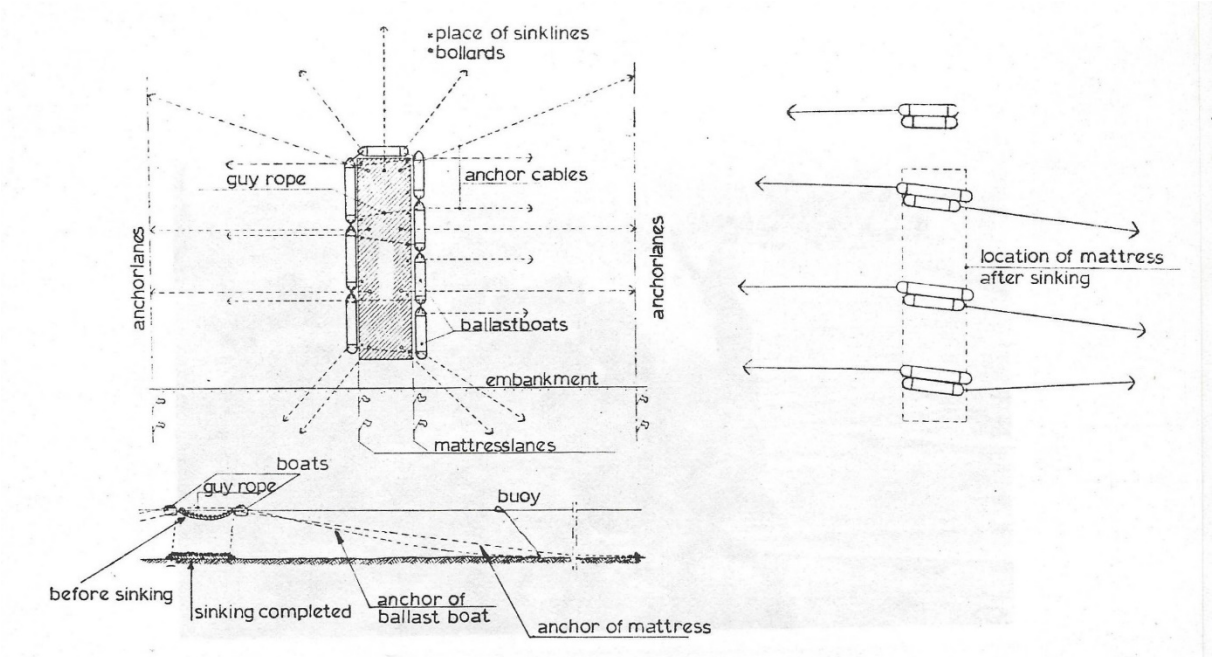


Fig. D.8 Overview of the positioning and anchoring of a floating fascine mattress in deep waters, followed by its fast immersion at the turn of the tide; source: BWDB (1976)

The off-set depends on: magnitude of the current, water depth, quantity of ballast, materials used, etc.
During the positioning of the mattress additional barges (or country boats) with ballast materials are moored to the mattress and put to anchor, the barges facing each other, are connected to each other by guy ropes.
Through the sock eyes of the sink straps of the mattress sink lines are pulled and fixed to the bollards of the 'ballast' barges.
The location of the sink straps on the mattress is related to the type (dimensions) of ballast barges used, therefore the type of barges has to be known before 'construction' of the mattress starts, in order to be able to fix the straps in the right place.
After the positioning of the mattress is completed, the ballasting operations can start.

D.3.3 Ballasting (Photos 22, 23 &25)

The mattress is ballasted to make it sink. The ballasting takes place from the middle of the mattress to the sides of the mattress, the workers form long lines of men and pass the materials from the barges to the dumping spot. The last worker in each line drops the ballast in the squares formed by the upper grating on the mattress and takes care that the material is spread evenly.
The mattress centre submerges gradually under the increasing weight and the mattress pulls with increasing forces on the anchor lines and bollards.
The ballasting is continued until the whole mattress is submerged but for the gangway (which has a greater uplift).
One may say that the whole mattress is hanging as a huge bag in the sink lines between the barges.

Then, awaiting the slack water at high tide or low tide, the ballasting is stopped temporarily. During this breathing space, ballast material is placed along the edge of the boats to facilitate and accelerate the final ballasting, when called for.
At slack tide, on the word of command, the gangways are ballasted in the shortest possible time and the whole mattress submerges, while in the meantime all the sink lines are disconnected. To avoid calamities due to a blocked line, each worker at the sink lines has a knife to cut the line in case of an emergency.
As soon as the mattress is sunk, the boats are shifted above the mattress and heavy rock is dumped. Because of the greater uplift of the gangway and the method of sinking, the centre of the mattress hits the sea bed first and the sides thereafter. This is the only way to ensure a good coverage of the sea bed.
If the sides would hit the bottom first followed by the centre part, the mattress would fold.
For the sinking operations of a normal mattress, four labourers per 90 m^2 of mattress are required.

Note on modified sinking operations
If a mattress is to be immersed in shallow water and the current is not too high, anchorage and sinking becomes a simple operation (ref. Figure D.9 and Photo 25).The sinking is then realized gradually during the last hour before slack tide.
Only one ballast barge, moored at the upstream side of the mattress, is used.
The strip adjacent to the boat is ballasted until it is submerged, the anchor lines are veered and the next strip is ballasted; this is repeated until the whole mattress is sunk.
The water current helps the sinking operation along, as the current is pushing the mattress downward during sinking.

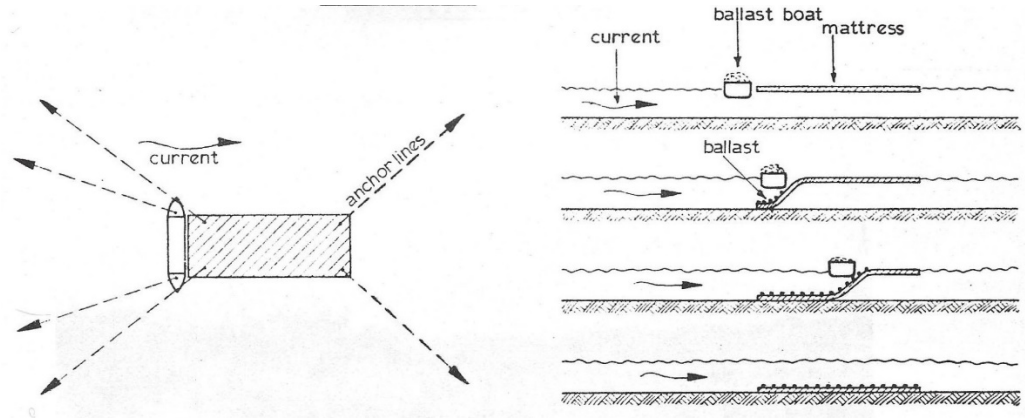


Fig. D.9 Gradual immersion of a floating fascine mattress 'in the current' (*op stroom*), in shallow waters; source: BWDB (1976)

D.4. Ballast materials

The individual weight of ballast materials is not to exceed 40 kg as the material is passed by hand to the mattress. The weight of the single ballast pieces is normally 5 to 40 kg or 10 to 60 kg.
For the rock dumped directly on the immersed mattress, after the actual sinking operation has taken place, heavier pieces are used, ranging between resp. 40 to 200 kg for light work and 60 to 300 kg for heavy work.

D.4.1 Rock

The ballast of a normal mattress (three fascine lavers) consists of 200 kg/m^2 of 'sink' stone and 800 kg/m^2 of 'ballast' rock.

For river works. 125 kg/m^2 and 300 kg/m^2 are used respectively. The exact amount depends on local circumstances and density of the ballast material.

D.4.2 Sandbags

Instead of stone, sand bags can be used. When using bags made of jute, it should be considered that this material is subject to decay. Ballasting with bags of jute, filled with sand or clay, is therefore only allowed for construction of closure bunds or for temporary slope protection of dikes.
In this respect the possibility of impregnating the jute fibers should be investigated[5].

D.4.3 Clay Clods

Clay clods are sods of cubic form with sides of 15-20 cm. The clods consist of tough clay having sufficient cohesiveness, the overgrowth (roots) is not removed as this will disturb the cohesiveness. Normally the clods are cut on the salt marshes. When ballasting is carried out using clay clods one layer of the sods is placed on the mattress in advance (on the '*zate*').
To prevent washing away of the relative light clods during sinking of the mattress, heavy material (sink stone, sand bags) is dumped on the gangways of the mattress. Ballast with clay clods can be used only in temporary works and in closure bunds as the material itself is subject to abrasion by scour.
The advantage of the use of clay clods is:

- the material is cheap
- the material is easy to handle
- cutting and transport is labour intensive
- the clay protects the mattress against shipworm attack

A disadvantage might be that the materials are pressed in the mattress and accordingly some settlement has to be expected.

D.4.4 Bricks

Instead of ballasting with expensive stones, bricks can be used instead. Individual bricks are too light, but by placing the bricks in "boxes" of wire mesh (gabions) units of almost any weight can be composed. Bricks are available in Bangladesh in abundance as they can be made on the site of the works in especially made field kilns. Moreover, the use of misshapen bricks (which are useless for building or road works) is no problem.

Photographs

Photo 17:
Zinkbaas Gerrit Visser in 1953; the famous sinking master who figures as Klaas Otterkop in 'Het verjaagde Water' * by A den Doolaard; in this respect reference is made to the ninth annotated edition (2001, 216 et seq.), as edited by K. d' Angremond and G.J. Schiereck.

* in translation: 'Roll back the Sea' (1948)

[5] This remark clearly was made when this Appendix was drafted in 1976. Nowadays for environmental reasons, such treatment would not be allowed.

Photo 18: Fascine string fabrication

Photo 19: Construction of under-grillage

Photo 20: Placing of fascine filling

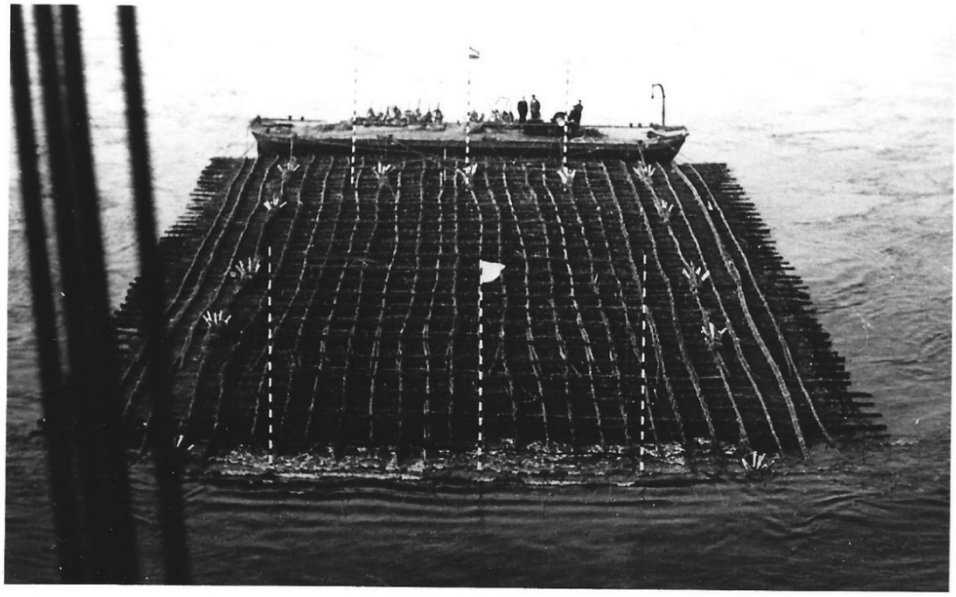


Photo 21: Floating mattress (note the bollards)

Photo 22: Ballasting of mattress

Photo 23: Ballasting of mattress

Photo 24: A fascine mattress ready for transport to final location (Schelphoek 1953)

Photo 25: A fascine mattress being gradually sunk in the alignment of the sill for the caisson closure; note wattle work on the fascine strings (Schelphoek 1953)

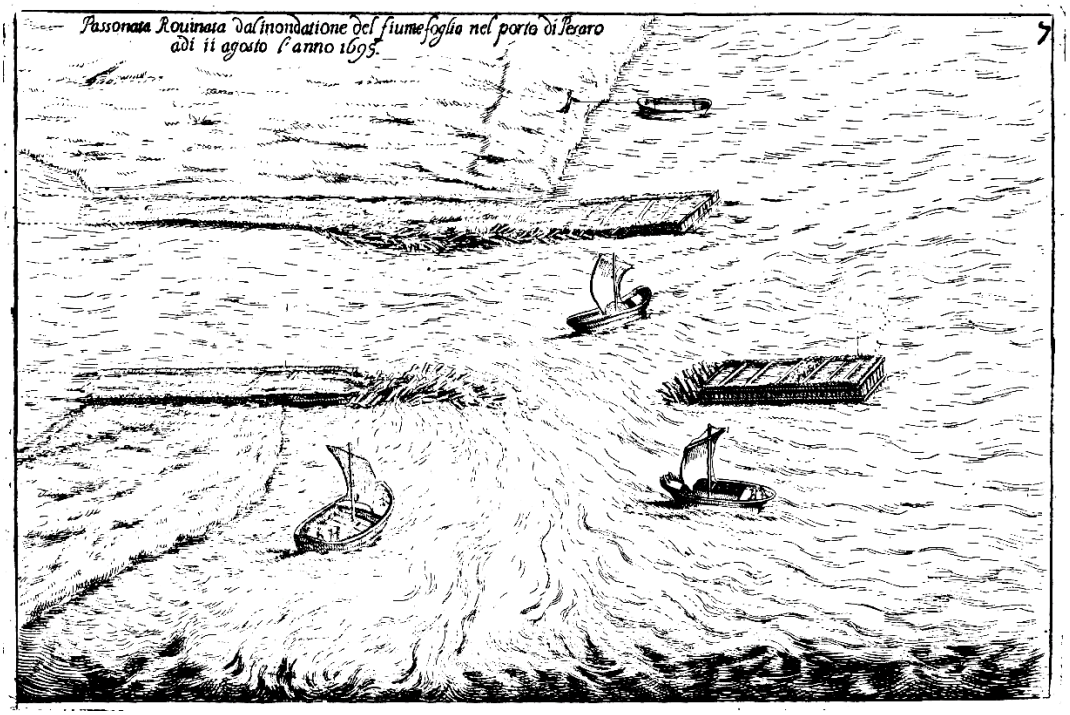
APPENDIX E[1] Cornelis Meijer about repair of harbour works at Pesaro, Italy in 1695-'96 [2]

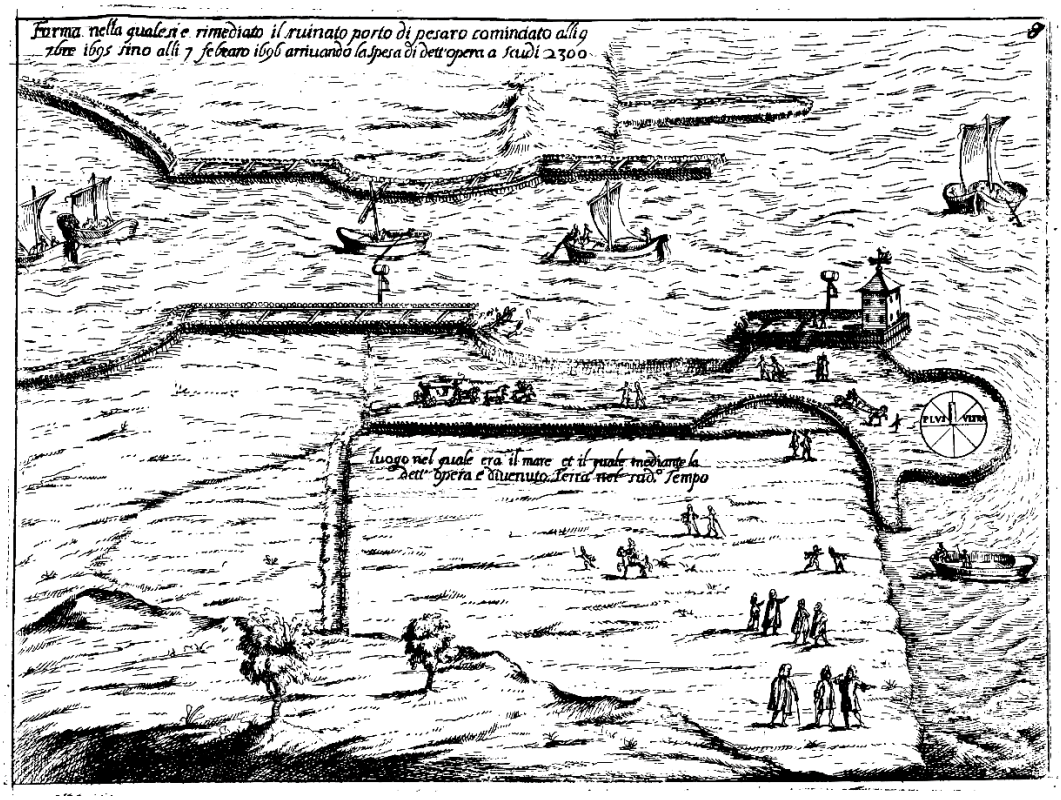
Figura nella quale si dimostra il modo come sia aggiustato il ruinato Porto di Pesaro Adi 9. Setembre 1695. comminciato	Afbeelding waarin de manier getoond wordt hoe de vernielde Haven van Pesaro hersteld is d.d. 9 september 1695. begonnen
Fu comminciato questo lavoro con admirazione d'ogn'uno, perche chi credeva essere il mio pensiero di farlo tutto di muraglie sodi e grandi, chi di palificate ben aggiustate, chi in un altro modo più fermo e stabile delli sudetti, fù però contro lor opinione principiato con sole fascine, Terra, e pochi sassi senza metterci passoni grandi, ni guide ne catene ò traverse, senza chiodi, si che credevasi che la primera fiumara overo borasca per certo haverebbe portato via tutto il lavoro, ma venendoli adosso l'escrescenze del fiume e le tempeste di Levante hanno fatto l'effetto tutto contrario a quello ch'eransi imaginati, anzi apportandoli utile non mediocre; perche essendo arrivato sin al fine della palificata rimasta ci venne tra la fascinatura tanta terra che si sono sparagnati molte fascine anzi havendo lasciato tra il mio lavoro una meza luna senza mettervi fascine d'alcuna sorte, si empì in pochi giorni di Rena dove per empirla non bastavono molte fascine hor essendo io arrivato sin al fine della palificata vecchia proposi di tirar avanti il lavoro nel mare sin alla longhezza di trecento piedi quando di nuovo fu soppreso di paura l'Illustrissimo Magistrato di Pesaro dicendo ch'il mare e le tempeste l'haverebbero daneggiate.	Dit werk werd begonnen met bewondering van iedereen, want [er was] wie dacht dat het mijn gedachte was om hem helemaal van zware en grote muren te maken, [er was wie dacht] van goed herstelde paalwerken, [er was wie dacht] op een andere steviger en stabieler manier dan de bovengenoemde [manieren]. Het werd echter tegen hun mening in begonnen met alleen bossen rijshout, aarde, en weinig stenen zonder grote matrassen van takken, noch geleiders, noch kettingen of dwarsbalken, zonder spijkers, tot het punt dat er werd gedacht dat het eerste hoogwater ofwel de eerste storm heel het werk zeker zou hebben weggevoerd. Echter, nadat de zwellingen van de rivier en de oosterstormen er tegenaan kwamen, hebben ze een effect bewerkstelligd dat helemaal tegengesteld is aan wat ze zich hadden voorgesteld, sterker nog, ze hebben baten gebracht die niet middelmatig waren. Omdat het werk tot aan het einde van het overgebleven paalwerk was gekomen, kwam er tussen de bossen rijshout zoveel aarde dat ze zich vele bossen rijshout bespaard hebben. Sterker nog, door tussen mijn werk een halve maan opengelaten te hebben zonder daar bossen rijshout van enige soort aan te brengen, vulde het zich in een paar dagen met zand waar vele bossen rijshout niet voldoende waren om haar te vullen. Nu ik aan het einde van het oude paalwerk was aangekomen, stelde ik voor het werk naar voren in zee door te trekken tot aan een lengte van 300 voet, toen de Hoogdoorluchtige Magistraat van Pesaro opnieuw door angst werd overweldigd dat de zee en de stormen het zouden hebben beschadigd.
Però in tre giorni ne tiravo avanti il lavoro sin'a sessanta piedi, donde l'Illustrissimo Magistrato credendo ch'il lavoro sarebbe stato troppo debole e leggiero ordinò per quella volta di non tirarlo più avanti ma fortificar ciò ch'era comminciato il quale secondo il desiderio di lui ho puntualmente esseguito.	Echter in drie dagen trok ik het werk tot aan 60 voet door naar voren, vanwaar de Hoogdoorluchtige Magistraat, gelovend dat het werk te zwak en te licht zou zijn geweest, beval voor die keer om het niet verder naar voren door te trekken maar om dat wat begonnen was te versterken, hetgeen ik volgens zijn wens punctueel heb uitgevoerd.
Nello stabilire dunque il sudetto lavoro ho messo al fine di esso una colonna con la soprascritta *Plus ultra*, per dimostrare che il lavoro debbasi tirar più avanti, & attorno la sudetta colonna ho fatto una bussola acciò che li marinari disponessero le lor bussole secondo la mia, ho de più sopra il residuo della palificata fabricato un casino di guardia e sopra questo hò messo una nova invenzione de penelli al quale d'ogni tempo si puo vedere che venti tirino e gran commodo con poche spese, e benche credevono, qualqueduno che la mia fascinatura in poco tempo sarebbbe infragidata non è però così perche le fascine poste sotto la superfitie dell'acqua rimangono sempre sane.	In het aldus realiseren van bovengenoemd werk heb ik op het einde daarvan een zuil geplaatst met het opschrift *Plus ultra*, om aan te tonen dat het werk verder naar voren doorgetrokken moest worden, en rondom bovengenoemde zuil heb ik een kompas gemaakt zodat de zeelieden hun kompassen afregelden volgens de mijne. Bovendien heb ik op de overblijfselen van het paalwerk een wachthuisje gebouwd en daarop heb ik een nieuwe uitvinding van panelen geplaatst waarop op elk tijdstip te zien is welke winden er waaien en groot gemak met weinig uitgaven. En hoewel enkelen geloofden dat mijn werk van bossen rijshout in korte tijd zou zijn verrot, is dat niet zo omdat de onder het wateroppervlak geplaatste bossen rijshout altijd gezond blijven.
La raggione poi perche questa maniera di lavorar nell'acque è migliore di quella delle palificate grandi è questa, ch'essendo le palificate large solamente vinti piedi & essendo riempita di fascine e sassi è facile al mare & alle borasche portarci per quella poca larghezza la rena nel porto, come chiaramente si vede all'altra banda del Porto (dove la palificata è rimasta sana) per la quale anch'hoggidì passa tanta rena ch'è capace d'empir tutto il porto.	De reden dan waarom deze manier van werken in het water beter is dan die van de grote paalwerken is deze, dat aangezien de paalwerken slechts 20 voet breed zijn & aangezien zij gevuld is met bossen rijshout en stenen, het gemakkelijk is voor de zee en voor stormen om er zand door die geringe breedte in de haven te brengen, zoals duidelijk te zien is aan de andere kant van de haven (waar het paalwerk gezond gebleven is) waardoorheen ook vandaag zoveel zand passeert dat het in staat is de hele haven op te vullen.
Si che dunque essendo la mia infascinatura larga più de cinquanta piedi non è cosi facile al mare farvi passare la rena perche resta subito morta fra tante fascine la qual medema rena asportata dal mare li serve per più sodezza, hor il più grande sparagnio	Omdat mijn werk van bossen rijshout dus inderdaad meer dan 50 voet breed is, is het niet zo gemakkelijk voor de zee om er zand doorheen te laten passeren omdat het meteen dood blijft tussen zoveel bossen rijshout, waarvoor datzelfde door de zee

[1] Appendix E is referred to in Chapter 4.

[2] Text and sketches derived from the book 'Nuovi Ritrovamenti' by Cornelis Meijer, published in 1696 in Rome; Dutch translation of this text by Alessandra Crosato and Erik Mosselman in 2021.

caggionato da questo lavoro è ch'il sudetto sia ridotto in longhezza di cinquecento vinti sei piedi in tempo di cinque mesi solamente dove e contrario faccendolo di passoni grandi e reducendolo in questa longhezza dovevasi spendere almeno un'anno e faccendolo di muraglie non bastavano ne meno due anni, de più ho conservato sempre la navigazione la quale nel farlo di palificate o muraglie se non sempre almeno per qualche tempo sarebbe stata interrotta.	aangevoerde zand dient voor meer stevigheid. Welnu de grootste besparing veroorzaakt door dit werk is dat het bovengenoemde in lengte 526 voet gereduceerd wordt in een tijd van slechts 5 maanden waar men in tegenstelling minstens een jaar moest besteden door het te maken van grote matrassen en door het in deze lengte te reduceren. En door het van muren te bouwen waren zelfs twee jaar niet genoeg. Bovendien heb ik steeds de scheepvaart in stand gehouden die door het te maken van paalwerken of muren onderbroken zou zijn, zo niet voor altijd dan toch minstens voor enige tijd.





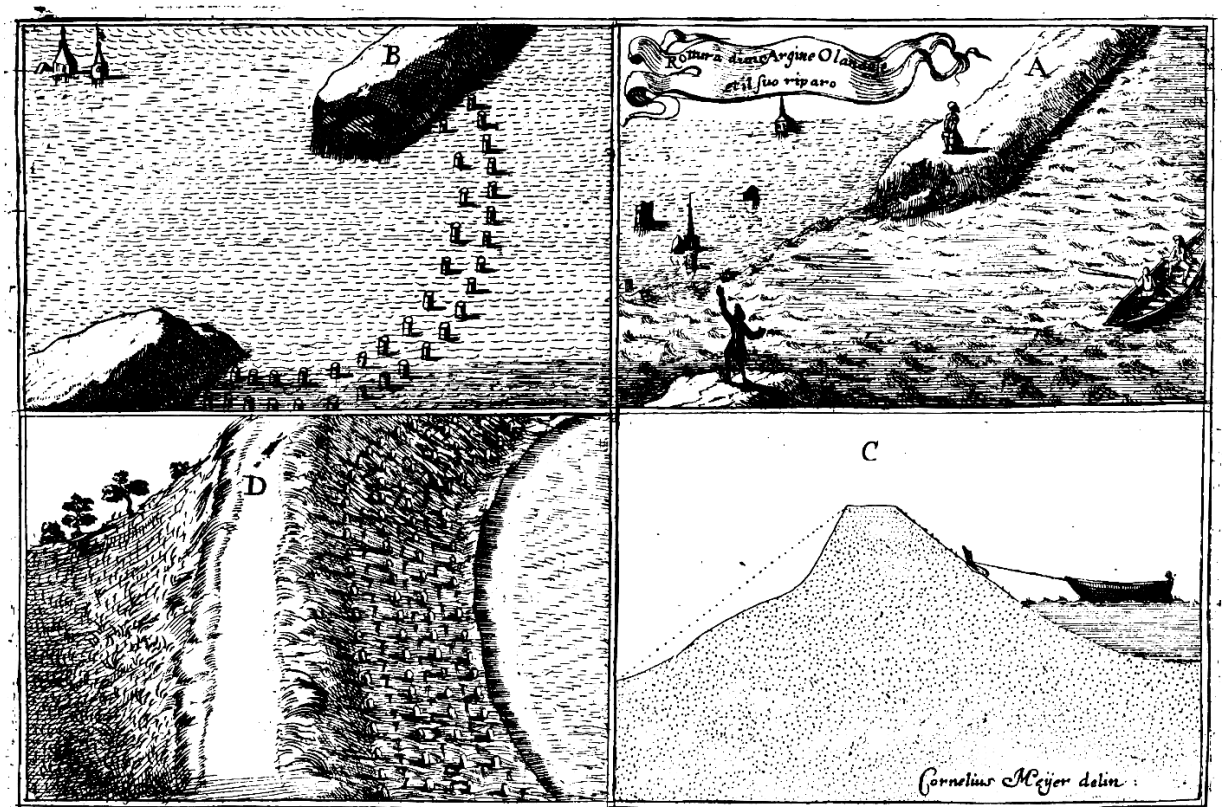
APPENDIX F[1] Cornelis Meijer about repair of breached dikes[2]

Degli argini	Over de dijken
L'industria humana attenta sempre à procurare, o ad cuitare il male, che soffre, ò d'essimersi da quello, che paventa, non ha fin'hora saputo trovare altro ripiego per liberarsi dall'inondationi, che di fabricare sopre le ripe de i fiumi, ò in vicinanza delle spiagge de i mari (dove li terreni à quelli confinanti sono mancanti della necessaria altezza) un aggiunta, overo alzamento di terra, che communemente argine si addimanda.	De menselijke nijverheid probeert altijd om voorzorgsmaatregelen te treffen, of om het kwaad te vermijden dat zij ondergaat, of om zich te onttrekken aan dat wat zij ducht. Tot nu toe heeft zij geen andere uitweg weten te vinden om zich te bevrijden van de overstromingen dan door een toevoeging te bouwen op de oevers van de rivieren, of in nabijheid van de stranden van de zeeën (waar de daaraan grenzende terreinen verstoken zijn van de benodigde hoogte), oftewel een verhoging van aarde die men gewoonlijk dijk noemt.
Mà per questo non resta che li argini medesimi non venghino tal hora superati dall'onde, ò squarciati dalla corrente.	Maar dat betekent niet dat diezelfde dijken niet soms overtroffen worden door de golven, of opengereten door de stroming.
Poiche l'elemento dell'acque tal volta ingrossandosi non riconosce ne confini, ne termini, che dall'huomo, ò dalla natura li sono stabiliti, mà superando ogni riparo, & abbattendo ogni ostacolo corre sfrenato per le Campagne contigue, con sommergere tal volta le Provincie intiere.	Aangezien het element van de wateren soms, zich opzettend, geen grenzen of eindpunten erkent die hem door de mens of door de natuur zijn vastgesteld, terwijl het daarentegen iedere beschutting overwint en ieder obstakel neerslaat, stroomt het ongeremd over de aanliggende landerijen, met soms onderdompelen van volledige provincies.
A simil male sono assai soggette le Provincie de i Paesi Bassi della Fiandra, e dell'Olanda, poiche il Mare ivi sovente s'inalza, in modo che ò sovravanza gli argini, ò infuriato con il corso dell'onde li sdrucisce, onde poi ne seguono danni cosi grandi, che per liberarsene, ò per cuitare, che non ne seguono de i maggiori, si tengono al'ordine in vicinanza degli argini quei materiali, & ordegni, che possono bisognare per un sollecito riparo.	Aan een vergelijkbaar kwaad zijn ten zeerste de provincies van de Lage Landen van Vlaanderen en van Holland onderworpen, aangezien de zee zich daar menigmaal verhoogt, op een manier die ofwel over de dijken heen gaat ofwel ze woedend met de loop van de golven openscheurt, waaruit vervolgens dermate grote schades volgen dat men, om zich ervan te bevrijden of om te vermijden dat er grotere uit volgen, in de nabijheid van de dijken die materialen en werktuigen gereed houdt die nodig kunnen zijn voor een prompte reparatie.
Per dar dunque qualche essempio delli ripari, che hò veduto pratticare in mia presenza in quelli Paesi suppongo nella sopraposta figura una rottura nell'argine A. quale si rimediò in breve tempo con una Palificata riempita di materiali, come al B. in forma di semicircolo, che s'intesta nelli lati dell'argine alquanto lontano dalla rottura, e ciò per doi capi:	Om dan een voorbeeld te geven van de reparaties die ik heb zien toepassen in mijn aanwezigheid in die landen neem ik in de bovenstaande figuur een breuk aan in de dijk A die men in korte tijd herstelde met een paalconstructie gevuld met materialen, zoals in B in de vorm van een halve cirkel, waarvan men de koppen aansluit in de zijkanten van de dijk behoorlijk ver van de breuk, en dat om twee hoofdpunten:
Uno, perche con un riparo nuovo non si puol cosi ben unire l'opera tirandola retta all'argine vecchio, che non passi l'acqua, oltre ancora che l'estremità della rottura dell'argine si trovano indebolite, & inzuppate dell'acque, e facile à cadere:	Eén, omdat men met een nieuwe reparatie het werk niet zo goed kan verenigen door hem recht te trekken naar de oude dijk, opdat het water er niet doorheen gaat, buiten nog dat de uiteinden van de breuk van de dijk zich verzwakt vinden en doorweekt door de wateren, en makkelijk om te vallen.
l'altro, perche poi si possa rifabricare à suo bell'agio la parte mancante dell'argine nella medesima linea di prima.	Het andere, opdat men daarna op zijn mooie gemak het ontbrekende deel kan bouwen van de dijk in dezelfde lijn als eerst.
Solevano ancora per il passato fabricare gli argini in quelle Provincie quasi senza scarpa nella parte, che guarda la Campagna, come si suppone mostrare con il profilo al C. onde sovente seguiva che quando il mare supera l'argine, la caduta dell'acque ruinava il piede dell'argine in modo che precipitava, quindi accortesi poi dell'errore,	Ze waren in het verleden nog gewoon om de dijken in die provincies vrijwel zonder glooiing te bouwen in het deel dat uitziet op het land, zoals verondersteld wordt te laten zien in het profiel bij C, waar vaak het gevolg was dat, wanneer de zee boven de dijk uitkomt, de val van de wateren de voet van de dijk zodanig vernielde dat deze

[1] Appendix F is referred to in Chapter 5.

[2] Text and sketches derived from the book 'Nuovi Ritrovamenti' by Cornelis Meijer, published in 1689 in Rome; Dutch translation of this text by Alessandra Crosato and Erik Mosselman in 2021.

costumano fabricare gli argini tanto verso la Campagna, quanto verso il Mare con scarpa proportionata alla di loro altezza, anzi la parte, che guarda verso il Mare, come si suppone mostrare al D. è munita di Passoncelli, attorno i quali sogliono industriosamente tessere la paglia, si perche l'onde del Mare dando in questi Passoncelli perdono di forza, come anco perche nel ritirarsi l'acque non tirano seco il terreno à piedi dell'argini.	instortte. Daarom, zich nadien bewust geworden van de fout, hebben zij het gebruik om de dijk net zo aan landzijde te bouwen als aan zeezijde met een glooiing in evenredigheid gebracht met die van hun hoogte. Sterker nog, het deel dat uitziet op de zee, zoals verondersteld wordt te laten zien bij D, wordt voorzien van matrassen van takken rondom welke ze gewoon zijn om nijver stro te weven, ofwel opdat de golven van de zee die in die matrassen van takken terechtkomen kracht verliezen, ofwel ook opdat ze het terrein aan de voeten van de dijken niet met zich mee trekken bij het zich terugtrekken van de wateren.



APPENDIX G[1] Proposed method of closure for the Amtali creek

1 The proposed method is similar to the method successfully applied for the Chakamaya closure.

2 The bed of the channel has to be protected by placing fourteen bed protection mattresses having a size of 54' x 150' up to 150' up and downstream of the centre line of the cross-dam (Fig 9).

3 The sill has to be protected by placing seven sill mattresses with a top level at PWD - 4'.

4 The mattresses are a sandwich construction composed of an under frame of full bamboo grating, spacing 2.5' x 3', one length-wise bed layer of reeds, one cross-wise middle layer of reeds, one length-wise top layer of reeds and a top frame of full bamboo grating, spacing 2.5' x 3' (Fig. 8). The middle layer will possibly be replaced by a goal-pata layer, provided it is available on 15th November 1981.

5 The connections at the crossings are to be made with jute binding rope. The rope used for the under frame must also be used for the top frame crossings. The under frame must be made on a platform 80' by 190' having a level of PWD + 7'. Sufficient hetal pegs will mark the spacing of the grating on the platform. The under frame is carried to the outside platform 80' by 100' which has a level of PWD + 2' at the river side and of PWD + 3' at the land side. The levels have been chosen such that sufficient time is available to make the mattress on dry land and to ensure it is in floating condition during High tide to enable transport of the mattress to the location where it will be immersed.

6 The mattresses are provided with anchor ropes of 1½' diam. fixed to crossings of the under frame. The mattress is fixed to two self-propelled barges as shown in Fig. 7 and transported to the sinking location by means of the tidal current or the engines of the barges. The upstream anchor ropes are fixed to the big anchor ropes of the concrete anchor blocks or the bullah anchor piles before the mattress is connected to the upstream barge. The mattress is kept in position by means of 7 anchor ropes as is shown in Fig. 7. The two barges are loaded with a total of 5 500 gunny bags filled with clay.

7 The immersion of the mattress is started by loading by headload the upstream half of the mattress with gunny bags. Walkways are necessary to enable the loading. The upstream end of the mattress is heavily loaded by dumping from the upstream barge. The sinking lines are then released and the upstream part of the mattress will sink to the bottom. The upstream barge is then pulled towards the downstream barge, which remains in position and the remaining bags are dumped continuously. The sinking lines connecting the mattress with the downstream barge are released after heavy loading of the downstream end of the mattress. The upstream barge is then pulled back to its start position. The downstream barge can then be pulled towards the anchored upstream barge, while the gunny bags are continuously dumped. The process is terminated by releasing all anchor ropes.

8 The underwater sill can be constructed by dumping the gunny bags from a barge. One barge can be loaded with 3 500 bags. The barges must properly be anchored to ensure the correct location of the dumped gunny bags. The sill must be constructed by dumping in layers having

[1] Appendix G is referred to in Chapter 11.

a maximum thickness of 2' while the layers must be dumped alternatingly with the barge parallel and transverse to the current. Small abutments can be made by earth fill as per direction of the Engineer-in-charge. The sill is covered by sill mattresses after reaching the final level. The final sill level is assumed to be PWD - 4', but could be changed in case it is found advisable by the Dutch experts.

9 The seven sill mattresses differ from the bed protection mattresses. The length-wise bamboo is half split instead of full. On top, additional half split bamboos will be laid in both directions between the normal spacings. The size of the mattress is 54' x 170'/150 (Fig. 10). The length is dependent on the bed level. A length of 170' has been assumed for the cost estimate.

10 The sill mattresses are further provided with cross-wise reed rolls, diam. 1¼', at centre to centre 5' distance, to prevent washing out of gunny bags due to the high current. The immersion of the mattress can only be done during High tide. The procedure is the same as described for the bed protection mattress. The cross-section of the sill is shown in Fig. 11.

11 The small abutments must be covered by gunny bags after the immersion of the adjoining sill mattress. The construction of the jetty on top of the sill and across the channel, as shown in Fig. 11, can start from one side after the immersion of sufficient sill mattresses. The ballast of the sill mattresses must be increased and maintained by dumping of additional clay- or brick-filled bags as per direction of the Engineer-in-charge.

12 The thread part of the bolts meant to join the horizontal girders and bracings with the bullah piles, must be long enough to fix the girders and bracings firmly against the piles. It is very important that the underside of the horizontal girders is above PWD + 8’ to prevent problems during the closing stage.

13 The first phase of the actual closure consists of dumping gunny bags from the jetty and from barges downstream of the jetty up to a level of PWD + 1' after bamboo piling has been placed[2] downstream of the jetty as shown in Fig. 11. The second phase of the actual closure (to stop the flow) consists of dumping of gunny bags up to a level of PWD + 8' between a third bamboo piling as shown in Fig. 11 and the upstream bamboo piling.

14 The final cross section of the cross dam must be obtained by dumping clay fill, carried in head pans, from the jetty.
The number of labourers required for filling gunny bags, making and immersing mattresses and dumping of the gunny bags for the sill is about 300.

15 The number of labourers required for the actual closure is 2 000. A labour count will be made to ensure that this number is available when starting the actual closure.

[2] The bamboo piling is required to form a cofferdam from which dumped bags cannot be removed by the current.

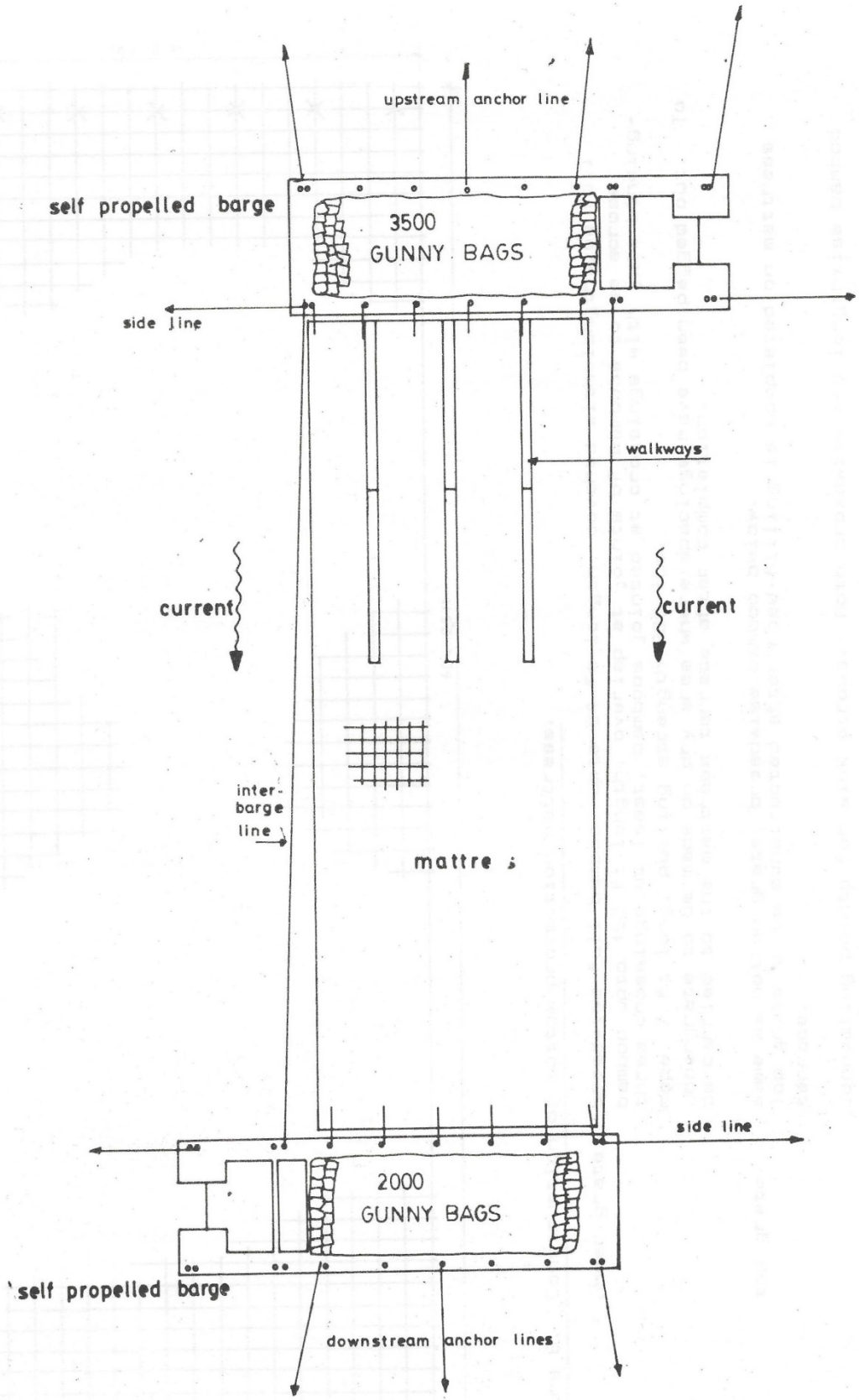


Figure 7. Lay-out of barges and mattress anchored in position prior to sinking operation.

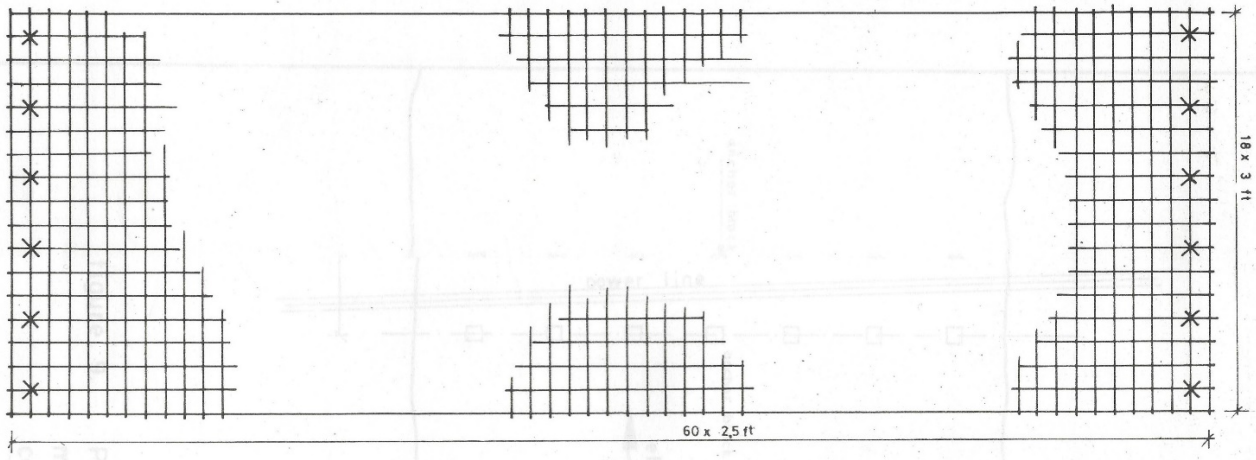


Figure 8. Construction of bottom protection mattress.

under grate: broadwise full bamboo, upto 54 ft length, covered with lengthwise full bamboo upto 150 ft length, overlap at joints of bamboos to be across three crossings at least, bamboos jointed at crossings with jute binding-rope, 7 ft long, pulling strength 300 lbs.
Undergrate to be made on dry area where spacings have been pegged out. To be carried to the mattress terrace after completion.

top grate : same as bottom grate; broadwise bamboo below.
Top grate to be constructed after reed-filling is completed on mattress terrace.

Connecting points for sink strops. Both broadwise and lengthwise bamboo should be made double upto 18' from crossings.

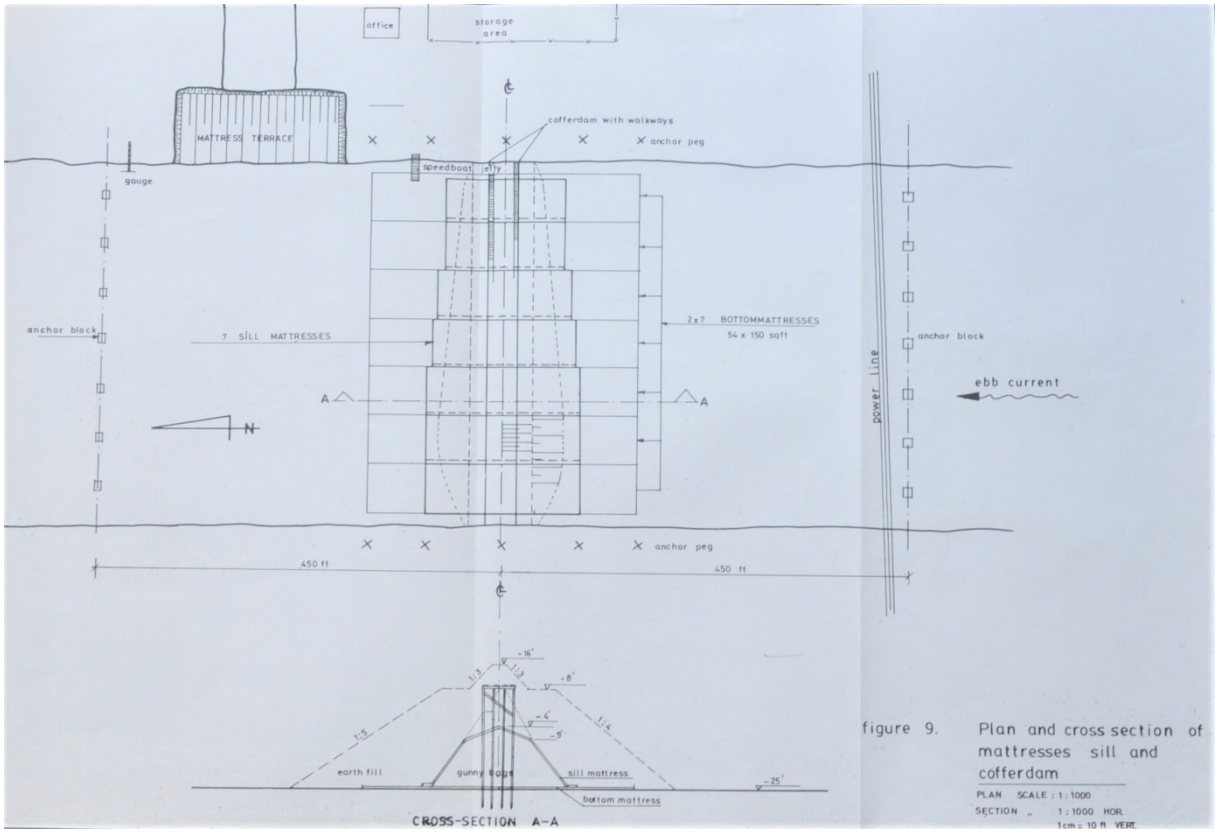


figure 9. Plan and cross section of mattresses sill and cofferdam

PLAN SCALE : 1:1000
SECTION „ 1:1000 HOR.
1cm = 10 ft VERT.

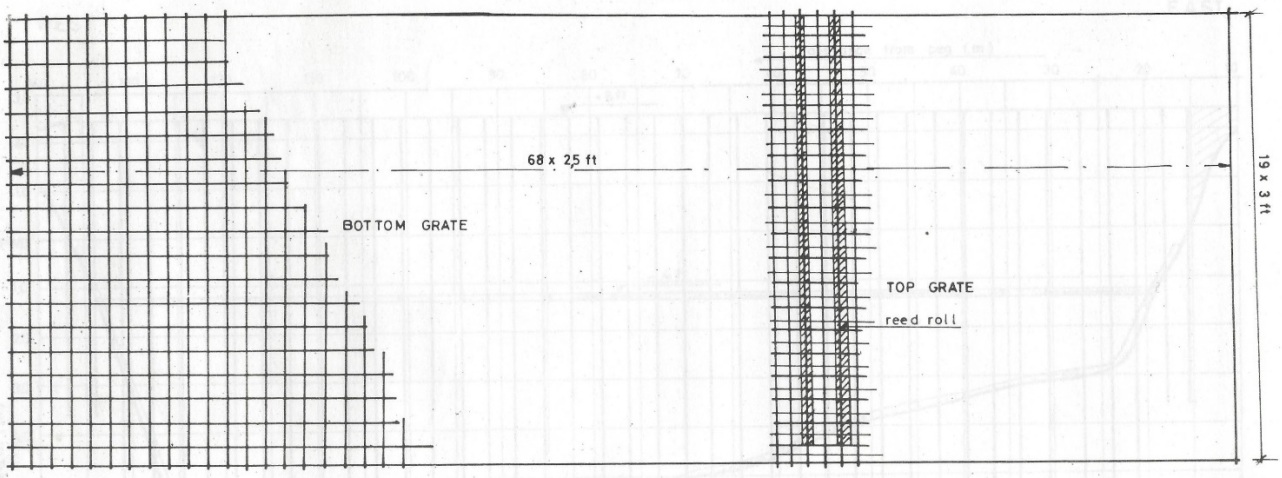


Figure 10. Construction of sillmattress.

undergrate : broadwise full bamboo below, on top lengthwise half split bamboo, overlapping joints across three crossings, bamboos to be jointed on crossings with jute binding rope 8 ft length, pulling strength 300 lbs. Under grate to be made on dry area where spacings have been pegged out. To be carried to the mattress terrace after completion.

top grate : broadwise half split bamboo c/c 2½ ft, at mid-grid; covered with lengthwise bamboo (half split), c/c 1½ ft, covered with broadwise bamboo, c/c 2½ ft. Every second full broadwise bamboo to be provided with reed roll dia 1½ ft, length 3 ft shorter than width of the mattress.

reed roll : dia 1½ ft Circular binding rope c/c 3 ft strength 150 lbs, tied to broadwise full bamboo with jute binding rope pull strength 300 lbs, c/c 3 ft.

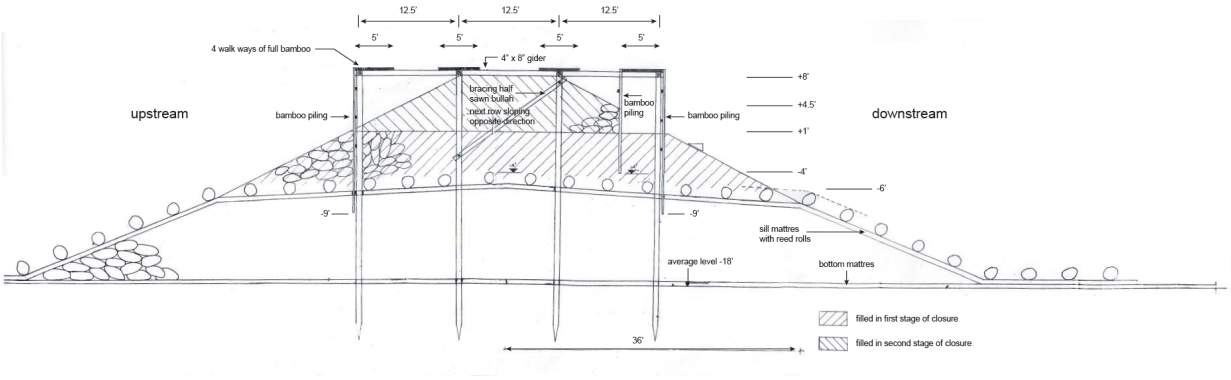
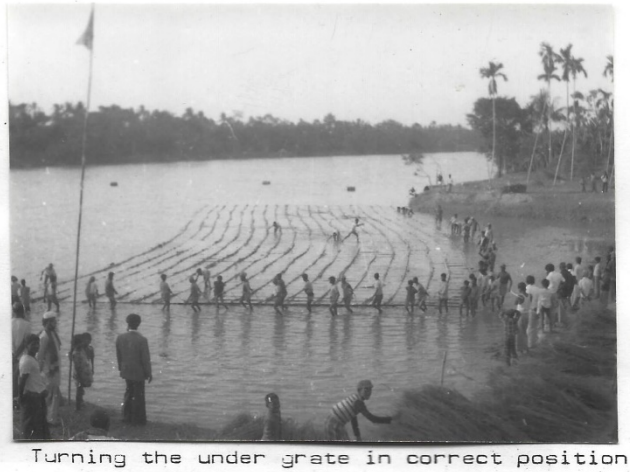


Fig. 11 Cross-section of sill and jetty/ cofferdam (with bamboo pilings and walkways)

PHOTOGRAPHS

Making under grating

Carrying under grating to submerged terrace



Turning the under grate in correct position

Placing hetal pegs and fixing binding rope.

Making middle layer of golpata leaves

Filling mattress with reed and golpata

Carrying reed roll to sill mattress

Sill mattress allmost ready

Positioning of mattress on sink location

Ballasting of mattress

Ballasting of mattress

Ballasting of sill mattress

Ballasting by head load allmost ready

Sinking and continued ballasting

Mattress just sunk

Pile driving for cofferdam-bridge.

Cofferdam-bridge near completion

In-coming tide

Finishing walkpaths and bamboo screen piling.

Start final closure operation, dumping gunny bags from walkpaths.

Dumping gunny bags over four walkpaths by 1600 labourers

Actual closure in progress

APPENDIX H[1] Quotations from: Delft Hydraulics Laboratory, Feasibility study Sandwhip Crossdam, Stability of clay-filled gunny bags

Report on investigations[2] R 1331-40 December 1985

Summary

The behaviour of the gunny bags under strong currents has been investigated in a flume on a scale of 1:1. In the first test series a dam was built up with gunny bags only. The bags in the dam are quite stable up till a velocity of ca 2.60 m/s. With larger velocities the current is removing the bags to such an extent, that the flow cross-section is enlarged, so the velocity decreases again to ca 2.60 m/s. During a second series of tests, the bags were dropped on top of a willow mattress at floor level of the flume.

In that case, velocities up till ca 3.00 m/s did no harm. However, at velocities of ca 3.80 m/s an avalanche effect was noticed and the flume was swept clean by the current removing all bags including the mattress.

1.1 Terms of reference

Building of dams in estuarine channels is one of the proposed activities in the framework of the cooperation between the governments of the Netherlands and Bangladesh.

Locally available material such as gunny bags, may be used for the closing of the channels. However, for an adequate use of the bags, the behaviour of the bags under strong currents, must be known. The bags are filled with a clay-silt mixture, which may influence the stability of the bags by a certain degree of consolidation.

Therefore tests were done on a scale of 1:1, in a flume where high currents can be created.

The flume, a disused fish ladder, forms part of the river regulation works in the river Maas, near the village of Lith in the Netherlands.

The Delft Hydraulics Laboratory was commissioned by Euroconsult to conduct these tests, by the letter, dated 26-08-1985 and with re. 222/4.68.057.

1.2 Scope of work

Two series of tests were held. In both tests gunny bags were used, filled with a material, that resembles as much as possible the characteristics of the locally available material in Bangladesh.

For the first test series, a dam was built inside the flume, consisting of gunny bags only.

This dam was to subjected to currents well over 3 m/s. For the second series of tests, a willow mattress was used as a foundation and support for a layer of bags. All alterations of the dam and the mattress, happening during the tests, were to be described, photographed and if possible to be checked by levelling.

2 Conclusions

The critical velocity for the gunny bags on the top of the dam in a situation similar as in the flume is ca. 2.60 m/s.

This velocity is only slightly higher than the calculated critical velocity determined for a stone with the same mass and volume as a gunny bag. The expected positive effect on the stability due to a certain degree of consolidation caused by the clay in the filling material of the bag is therefore small.

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[1] Appendix H is referred to in Chapter 11.

[2] The Report was written by R. Bruinsma of Delft Hydraulics Laboratory.

The supporting effect of the willow bundles of the mattress on the stability of the gunny bags could not be determined in the test.
Due to a loss of filling material of the bags and a too abrupt increase of the velocity in the last test, the entire structure became instable and was discharged from the flume after reaching a velocity of ca 3.80 m/s.

Under the current conditions during the test there is a considerable loss of filling material from the bags. Without supplementary measures (larger grain diameters of the filling material and/or finer meshes of the fabric) the bags cannot be used as ballast for a willow mattress at current velocities as large as adjusted in the flume.

3 Materials

3.1 Gunny bags

In … pictures of some gunny samples are shown. It was decided that sample A was the most suitable, because it was the heaviest material and had the thickest thread…….

3.2 Filling material

In the Netherlands the material that resembled the material from Bangladesh closest, was found in the southern part of the Netherlands. This material is locally known as "löss". The löss had a D_{50} of 0.02 mm. The D_{50} of material in Bangladesh is 0.01 mm.

4 Preparations for the test

4.1 Building of the dam

……..

After filling, the average weight of the bags was determined by weighing some sample bags.. The weight was between 42 and 50 kg. The upper part of the dam was built underwater by dropping the bags from a height of ca 2.00 m above the water level.

……..

The average level of the dam was calculated to be +2.74 m. Taking a level of +1.48 m as the reference, then the volume of the dam is ca 25 m^3, while ca 600 bags were used for the equivalent part of the dam. This leads to a volume of ca 42 dm^3 per bag, including empty spaces between the bags.

4.3 Willow mattress

The willow mattress was built up in a grid in such a way that pockets with an area of approx. 1 m^2 and a depth of approx. 0.35 m, were formed.
This mattress was laid down at floor level as shown in …. On this mattress, while submerged in the water, bags were dropped from a level of approx. 2 m above the water level, in the same way as the upper part of the dam was built. The mattress was completely covered by one layer of bags.

6 Results of the tests

6.1 Test of the dam structure

……

Test no. 4
The current velocity over the dam= 2.40 m/s.
After the test, it could be noticed that the upper layer of the dam, forming the crest, started to move with the current. But still the bags settle down on the downstream slope.
This is the velocity during which single bags cannot withstand the current on their own any longer.

Test no. 5
The current velocity over the dam reaches a maximum value of 2.80 m/s, but decreases after a short while to 2.60 m/s.
After the test it could be noticed that again more bags in the upper layer had moved, but they still settle on the downward slope, and so far no bags have been lost.
Now clouds of filling material can be seen, passing by in the water. Some bags showed loss of contents after the test which confirms the observation during the test.
It seems that the stability limit is reached now and that each individual bag cannot rely any longer on more support from its neighbours.

Test no. 6
Each time the current velocity was increased beyond 3 m/s, it settled down again after a short while to a velocity of 2.60 m/s.
After a maximum initial velocity of 3.40 m/s, and another failure to hold on the velocity for a longer period of time, above the critical velocity of 2.60 m/s, the test was terminated.
……..
Sometimes large clouds of material were visible in the water. It could be confirmed afterwards that this always coincided with movements of the bags. Some bags were found to be filled only for a quarter or less of their original size. Since not a single opened bag was found, it may safely be assumed, that the current was able to extract material from the bags through the gunny fabric.

6.2 Test on the willow mattress structure

……
Test no. 9
The gates were completely opened during this test.
The velocity above the mattress was measured at 0.37 d from the surface. The maximum velocity reached was 3.23 m/s.
After the measurement in section 3 and returning to the same position in section 7, the velocity had decreased to 2.98 m/s.
Although some bags had moved, including the control bag downstream, at the mattress position there was not much difference.
More downstream near the gate, a patch of 5 bags had disappeared.

Test no. 10
The stationary part of the downstream gate was lowered by dismantling.
The downstream layer of bags, starting from the mattress was secured by topping it with steel wire webbing.
After reaching a velocity of more than 3.80 m/s, everything started to move and it was not longer possible to maintain a stabilized velocity.
All material in the flume, including the complete willow mattress was discharged from it by the strong current.

7 Discussion on the results

The dam built from gunny bags appeared to be stable up till a velocity of ca 2.60 m/s.
At higher velocities the bags are removed until a flow cross-section is created, large enough to permit a decrease of the velocity to ca. 2.60 m/s again.
In the end, at high starting velocities, the dam was swept away completely.

Each test had a duration of approx. 30 minutes.
During that period, the bags started to lose their filling material. Longer periods were not tested, but it may be assumed that the loss of material will be a main factor to take into account.
The stability of the gunny bags may be approximated by regarding the bag as a stone with a mass of 45 kg and a volume determined from this mass and the volume density. The volume density is estimated to be 1600 kg/m^3.
The nominal diameter becomes:
$Dn = (volume)^{1/3} = (45/1600)^{1/3} = 0.308$ m
and with:
$\Delta = (\rho_{bag} - \rho_{water}) / \rho_{water} = 0.6$

The determining parameter ΔDn becomes:
$\Delta Dn = 0.185$ m
According to Izbash the critical velocity can be determined by:
$V_{cr} = 1.7 (\Delta Dn\, g)^{½} = 2.30$ m/s (broad crested dam)
The critical velocity found in the flume does not differ appreciatively from the calculated one and therefore the effect of consolidation due to the clay part of the filling material is only small and has no practical significance.
The critical velocity for the bags on the top of the willow mattress was found to be around 3.00 m/s for short periods. During longer periods, the loss of material from the bags will become eventually the deciding factor. Due to the loss of material the load on the mattress will also decrease.
At a certain moment parts of the mattress are becoming free from the underground and start vibrating, caused by the turbulence of the current. This increases the turbulence even more. Then after all the bags start moving, the mattress itself is taken along by the current. In fact all the bags, including the mattress as a whole were ejected from the flume.

The larger critical velocities found for the situation with the mattress may be an indication of some support by the willow bundles. However from the photographs it can be seen that on this stage of the test the upper layer of bags is still present and a maximum support by the bundles is not yet available.
The support from the bundles is however not the determining stability factor. More important for the stability is the loss of filling material from the bags under conditions with large current velocities. This loss causes an important restriction in the use of the bags in particular as ballast for a willow mattress. The loss on the spot can be even more due to a finer filling material for the bags in Bangladesh.

APPENDIX J[1] Constructions for the closure of breaches in dikes along the Yellow River in China[2]

J.1 General

The purpose of this Appendix is to explain to the reader the method of closing breaches in flood embankment along the rivers in China.

Various authors describe the closure of dike breaches in China. In CNCID (2005, 162) it is stated that the 'blocking technology' (i.e. the closure of a breach in a flood embankment along a river) became mature during the Yuan, Ming and Qing dynasties. These successive dynasties cover the period 1270 – 1912.

In the text of CNCID (2005) a distinction is made between three types of constructions: 'rolled kids'(p. 85), 'wing kids' (p. 85) and 'wing enclosure kids' (p. 164 et seq.) in use for the closure of dike breaches. But, apparently, the original Chinese text has been translated by a non-technical translator and this results in indistinctness and also in some obvious mistakes. In order not to confuse the reader too much, these have been rectified, in as far as possible. The translator uses the word kid probably in the sense of a 'bag' in which you can carry things (like in the word kit-bag)

Figures from CNCID have been added for easy reference.

J.2 Rolled kids

Rolled kids are in fact sink-fascines, having a diameter up to 1 – 2 m and a length of some 20 m, and they were already in use during the Song dynasty (960 – 1279). They are being used in the final stage of closure. One could describe it as a speedy closure method.

The procedure of making a rolled kid was described in *History of the Song Dynasty • Records about Rivers and Canals* as follows:

- Land selection: a wide and flat dike surface would be selected as the kidding site;
- lining: grass ropes would be laid down on the ground in an dense pattern with soft materials such as branches and reeds spread on the ropes, after which a layer of earth would be pressed onto the soft materials mixed with cobbles. Then a big bamboo rope would traverse the whole structure, i.e. the so-called core rope;
- rolling and binding: the layered kid materials would be rolled and bound while the two ends would be tied up with coarse ropes, thereby getting the kid bundle. Such a kid was fairly large in volume, usually several *zhang* in height with the length being twice the height[3]. It took several hundred or even several thousand people to move it;
- placement: the kid bundle would be moved to the weak point of the dike, pushed into the river and tied to the pillars of the dike bank with small bamboo chains; meanwhile, long wood piles would be hammered into the kid to make it penetrate into the ground and consolidate the kid. After the four steps, the kid bank was finalized.

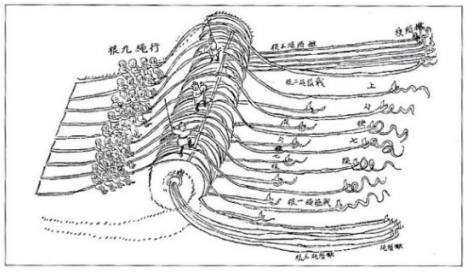

Fig. J.1

Sketch of a rolled kid as drawn during the Qing dynasty; the materials in this kid are millet stalks, earth, tree branches and ropes;

source: CNCID (2005, 84)

[1] Appendix J is referred to in Chapter 13.

[2] The text in this Appendix is an adapted version of relevant sections in CNCID (2005) 'History of Irrigation and Flood control in China'.

[3] This appears to be wrong. In view of the figures the dimensions mentioned in the first sentence of this section are adopted.

J.3 The making of wing kids in the Qing Dynasty

After Emperor Qianlong of the Qing Dynasty, the wing kids (also called soft wing) gradually replaced the traditional rolled kid. To produce a traditional rolled kid, a spacious construction platform was required. A 1 m diameter kid could be rolled tight and solid only by having available a 7 m wide kidding platform. Such platforms were time and labour consuming and therefore gradually the 'bundling boat' of the Qianlong Period was introduced with kids made between the dike head and the boat.[4]

Kaoliang stalks and hemp ropes were the main materials for constructing the main closure dyke except when the crevasse (i.e. the breach) was reduced to a very small width. Hemp ropes were first thrown into the river. The ends of the ropes on one side, however, were fastened to piles, horizontally arranged, driven into the newly constructed dyke behind; while the other ends were put on a boat which was well anchored in the crevasse in front of the dyke. On top of these ropes kaoliang stalks were laid which were later weighted down by many labourers and then a thick layer of clay was put over them.

Such a procedure was repeated alternately until the:·whole thing was sunk into the river and actually reached to the bottom of the river bed. Then, the ends of the ropes which were handled by men on the boat were thrown over this mass; and immediately tied to the original piles. A very thick layer of clay was finally laid on top of the dyke. Simultaneously, stone and willow mattresses (i.e. fascine mattresses) were used as revetment to withstand the strong current of the river outside. Clay was also necessary for "sealing'' the slope on the other side of the newly constructed section of the dyke.

In addition, as the stalks were fairly soft and could be obtained locally, large kid bodies could be made within a short time period. Therefore, the wing kid was more flexible and construction less labour intensive than the rolled kid.

J.4 The construction of the wing enclosure kid

When the blocking dam works on two banks went on to about 20 metres from each other, the blocking project entered its decisive stage. At this point the mouth was referred to as dragon mouth which was difficult to block.

In the Qing Dynasty, enclosure construction mainly took the two forms of rolled kids and wing kids. the former being major sunk fascine works passed down from previous times and the latter being invented during Emperor Qianlong's administration. However, the new invention did not fully replace the old kidding practices. Sometimes rolled kids were still applied at sites where the enclosure mouths were of torrential water regime and poor soil quality and where it was difficult to bundle wing kids. The enclosure kid was of heavy unit weight with the ratio of wicker to grass[5] being 7:3. As the positioning of sunken fascine works mainly depended on ropes rather than the piles that went through the kid body, the ropes must be "strong and numerous". After complete sinking of the kid pillow, the lower piles fixed the kid body to the river bottom while the ropes were tightly fixed to the wood piles at the two sides.

During the administration of Emperor Qianlong, stalks were used as major materials for sunken fascine works while kidding practice also changed from the previous rolled kids to bundled wings. Prior to enclosure, rope nets needed to be pulled at the two ends of the mouth, i.e. the so-called 'dragon cloth' (Fig. J.2). Tightly fixed to the enclosure cables by small ropes, the dragon cloth was spread

[4] The text which follows in CNCID is not very clear. It has therefore been replaced by the text (two paragraphs) found in Kung (1936, 14).

[5] Probably is meant: osiers to millet stalks.

with stalks and bags of earth (Fig. J.3). Subsequently, the construction labourers stepped on the layer of stalks and earth in order to compact it while the enclosure cables were eased off (Fig. J.4). Then the process was repeated, etc. until the enclosure cables reached the bottom of the closure gap and the gap was closed ……A common problem was tilting or twisting of the kid body because of non-synchronous slacking of the cables. Also, efforts must be made to ensure that the kid body is everywhere pressed to the bottom of the closure gap.

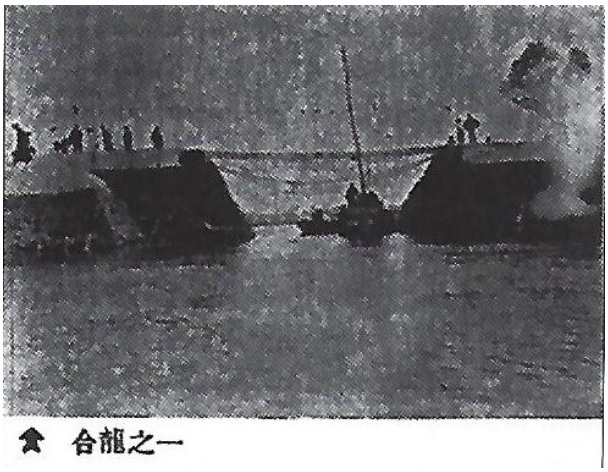

Fig. J.2
The 'dragon-cloth' has been positioned over the final opening to be closed

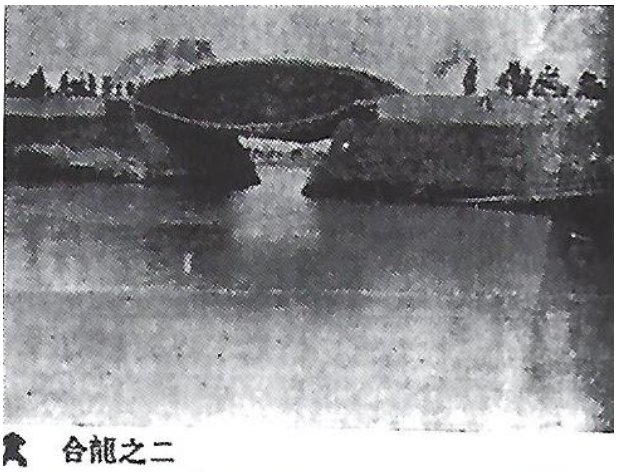

Fig. J.3
The first layers of kaoliang and (bags of) clay have been placed on the dragon-cloth and the 'plug' is ready to be lowered inch by inch by easing off the cables

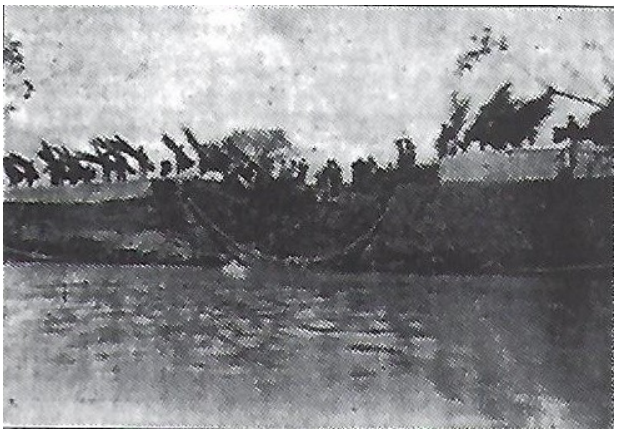

Fig. J.4
The 'plug' has now reached the water level; labourers are carrying bundles of kaoliang to the plug; labourers on the plug are jumping up and down to compact the kaoliang and the earth brought in;

source of all three photos:
CNCID (1936, 165)

APPENDIX K[1] Hydrometric measurements made on 25-27 March 1936 during final closure of the breach at Tung Chuang, China[2]

RECORD OF HYDROMETRIC MEASUREMENTS

(NEAR THE CREVASSE)

MARCH 25th

Time	Kiang Su Spur Dyke		No. 3 Cut-off		No. 4 Cut-off		Old River Course		Crevasse				Remark
									Water Level				
	W. L. meters.	Discharge m3/sec.	W. L. meters	Discharge m3/sec.	W. L. meters	Discharge m3/sec.	W. L. meters	Discharge m3/sec.	Up-stream	Down-stream	Diff. of W. L.	Discharge m3/sec.	
1	55.20								55.23				
2	55.21								55.22				
3	55.22								55.21				
4	55.22								55.19				
5	55.22								55.21				
6	55.23								55.21				
7	55.24	840			55.02	60	55.02	250	55.22	53.78	1.44	530	
8	55.25								55.23	53.75	1.48		
9	55.25	880							55.24	53.76	1.48		
10	55.26	880	55.24						55.24	53.75	1.49		
11	55.28	870	55.24	28					55.25	53.77	1.48		
12	55.31	860	55.24	28	55.08	65	55.08	280	55.27	53.75	1.52	515	
13	55.31	850	55.29	31	55.08	65	55.08	280	55.31	53.73	1.58	505	
14	55.34	810	55.31	32	55.12	70	55.12	290	55.31	53.72	1.59	480	
15	55.34	820	55.31	32	55.12	70	55.12	290	55.32	53.71	1.61	460	
16	55.33	800	55.33	33	55.12	70	55.12	290	55.33	53.67	1.66	440	
17	55.33	780	55.33	33	55.12	80	55.12	290	55.33	53.67	1.66	410	
18	55.33	780	55.34	33	55.12	80	55.12	290	55.35	53.65	1.70	410	
19	55.34	760	55.34	33	55.15	85	55.15	310	55.35	53.64	1.71	365	
20	55.36	760	55.34	34	55.15	90	55.15	310	55.36	53.64	1.72	360	
21	55.37	760	55.34	35	55.14	90	55.14	320	55.35	53.61	1.74	350	
22	55.37	760	55.31	36	55.13	100	55.15	330	55.36	53.59	1.77	330	
23	55.36	760	55.35	36	55.16	105	55.16	335	55.37	53.61	1.76	320	
24	55.36	760	55.35	37	55.17	110	55.17	335	55.37	53.62	1.75	315	

[1] Appendix K is referred to in Chapter 13.

[2] The tables have been copied from Kung (1936, 21-23)

RECORD OF HYDROMETRIC MEASUREMENTS

(NEAR THE CREVASSE)

MARCH 26th

Time	Kiang Su Spur Dyke		No. 3 Cut-off		No. 4 Cut-off		Old River course		Crevasse				Remark
									Water level		Diff. of W. L.		
	W. L. meters	Discharge m3/sec.	W. L. meters	Discharge m3/sec.	W. L. meters	Discharge m3/sec.	W. L. meters	Discharge m3/sec.	Up-stream	Down-stream		Discharge m3/sec.	
1	55.37	770	55.35	37	55.19	120	55.19	337	55.36	53.61	1.75	313	
2	55.37	770	55.35	37	55.19	120	55.19	337	55.37	53.58	1.79	313	
3	55.37	770	55.36	38	55.18	120	55.18	337	55.37	53.60	1.77	313	
4	55.36	760	55.35	37	55.17	120	55.17	340	55.35	53.55	1.80	300	
5	55.37	760	55.34	37	55.16	120	55.16	340	55.36	53.56	1.80	300	
6	55.38	750	55.34	37	55.16	120	55.16	340	55.37	53.58	1.80	290	
7	55.38	760	55.36	38	55.18	120	55.18	349	55.37	53.57	1.80	291	
8	55.37	750	55.36	38	55.19	120	55.19	350	55.35	53.55	1.80	280	
9	55.37	760	55.34	38	55.14	120	55.14	350	55.36	53.54	1.82	290	
10	55.37	750	55.34	38	55.14	120	55.14	360	55.35	53.53	1.82	270	
11	55.36	750	55.34	38	55.12	120	55.12	360	55.35	53.54	1.81	270	
12	55.33	750	55.34	38	55.10	120	55.10	360	55.33	53.54	1.79	270	
13	55.33	730	55.31	37	55.11	110	55.11	350	55.33	53.53	1.80	270	
14	55.33	720	55.30	36	55.10	100	55.10	350	55.32	53.52	1.80	270	
15	55.34	710	55.30	34	55.10	95	55.10	345	55.34	53.45	1.89	270	
16	55.34	650	55.32	34	55.09	80	55.09	320	55.34	53.44	1.90	250	
17	55.34	650	55.31	34	55.07	80	55.07	320	55.33	53.47	1.86	250	
18	55.31	680	55.29	33	55.07	80	55.07	360	55.34	53.49	1.85	240	
19	55.34	700	55.29	33	55.08	80	55.08	420	55.31	53.41	1.90	200	
20	55.33	750	55.29	33	55.10	80	55.10	500	55.34	53.43	1.91	170	
21	55.31	750	55.28	32	55.06	80	55.06	500	55.33	53.40	1.93	170	
22	55.32	770	55.28	33	55.05	80	55.05	520	55.33	53.40	1.93	170	
23	55.30	750	55.27	33	55.05	80	55.05	520	55.29	53.38	1.91	150	
24	55.29	750	55.25	32	55.05	80	55.05	520	55.27	53.39	1.88	150	

RECGRD OF HYDROMETRIC MEASUREMENTS

(NEAR THE CREVASSE)

MARCH 27th

Time	Kiang Su Spur Dyke		No. 3 Cut-off		No. 4 Cut-off		Old River Course		Crevasse				Remark
									Water level		Diff. of W. L.		
	W. L. meters	Discharge m3/sec.	W. L. meters	Discharge m3/sec.	W. L. meters	Discharge m3/sec.	W. L. meters	Discharge m3/sec.	Up-stream	down-stream		Discharge m3/sec.	
1	55.29	750	55.25	31	55.05	80	55.05	520	55.26	55.37	1.89	150	Crevasse closed at 5: 45 P.M.
2	55.27	730	55.24	31	55.01	78	55.01	510	55.24	55.37	1.87	142	
3	55.27	730	55.23	30	55.96	76	55.96	500	55.21	55.37	1.84	154	
4	55.22	710	55.21	30	55.95	76	55.95	500	55.19	55.37	1.82	134	
5	55.23	710	55.21	29	55.95	77	55.95	500	55.22	55.37	1.85	133	
6	55.23	710	55.21	29	55.94	75	55.94	500	55.21	53.35	1.86	135	
7	55.23	750	55.21	30	55.93	80	55.93	540	55.21	53.35	1.86	130	
8	55.21	760	55.20	30	55.92	80	55.92	550	55.21	53.35	1.86	130	
9	55.21	760	55.17	30	55.92	80	55.92	550	55.21	53.32	1.89	130	
10	55.24	840	55.20	30	55.94	100	55.94	640	55.24	53.31	1.93	100	
11	55.24	880	55.20	30	55.02	100	55.02	670	55.24	53.29	1.95	100	
12	55.25	880	55.22	30	55.02	115	55.02	685	55.26	53.29	1.97	80	
13	55.26	890	55.25	41	55.02	125	55.02	700	55.28	53.27	2.01	65	
14	55.27	900	55.26	44	55.01	130	55.01	720	55.28	53.25	2.03	50	
15	55.27	890	55.26	44	55.01	130	55.01	720	55.28	53.25	2.03	40	
16	55.30	900	55.28	46	55.00	130	55.00	740	55.32	53.17	2.15	30	
17	55.32	900	55.31	50	55.04	130	55.04	745	55.34	53.15	2.19	25	
18	55.35	910	55.34	53	55.07	132	55.07	760	55.36	53.11	2.25	18	
19	55.38	910	55.36	70	55.09		55.09		55.38	53.06	2.32		
20	55.42		55.37	80	55.10		55.10		55.39	53.01	2.38		
21	55.42		55.37	80	55.12		55.12		55.40	52.99	2.41		
22	55.42		55.37	90	55.12		55.12		55.41	52.97	2.44		
23	55.42		55.38	100	55.14		55.11		55.43	52.96	2.47		
24	55.42		55.40	120	55.15		55.15		55.43	52.92	2.51		

NB: please note a few typing errors in the water levels given in column 11, for instance for 55.37 m read 53.37 m.

APPENDIX L[1] Construction materials for closures using the mata method[2]

L.1 General

The materials used in general for the construction of mata closures are:

-	clay	-	golpata	-	sudari bullah
-	brushwood	-	jute	-	mata
-	straw	-	bamboo	-	gunny bags

L.2 Clay

The clay used is excavated from a borrow area on the river banks and preferably as close as possible to the dam site. The excavated material is transported to the dumping place manually. The system is such, that one man excavates and loads the clay into a basket, about 35 kilos at a time. The basked is then placed on top of the head of a labourer, who walks with it 30 feet to the next labourer and so on, forming a live chain to the dumping place. The output of one "excavator" is thus restricted by the speed of the carriers. Exact figures for the output are therefore hard to obtain.

L.3 Brushwood

The brushwood, used to fill up holes and to reinforce the face and the sides of the primary dam during construction, is cut from the shrubs growing on the riverbanks in the surroundings. At first near, but later, when construction is getting ahead and more and more brushwood is needed, the cutters have to go much further to obtain suitable material. The cutting, gathering and transporting to the site is done on a contract basis. Three to four men with a canoe can bring about 14 m^3 per day to the site. The brushwood is not selected on straightness and is also not bundled. This is not very handy during placing and the practice is, that one man carries one branch at a time when placing.

Among the different types of brushwood, that of the "Ora" tree is best fit for the job. It is similar to the wood of the willow- tree which is used in the Netherlands for this kind of work.

In order to obtain a better quality (mainly as far as straightness is concerned) it should be cultivated.

L.4 Straw

The straw used consists of the stems and sometimes of the whole plants, including roots, of the rice plants. It is bought on the local markets by agents of the Contractor and brought to site in bundles, having a length of approximately 1 m. From this straw also the ropes needed for the construction of mata's are spun at the site in the lengths required.

Straw is used in very large quantities, for making the mata's but also to fill up holes between the layers of clay to bring the dam to the required level.

L.5 Golpata

Golpata is a Bangladesh name for the leaf of the Nippa palm tree which grows in the swampy areas of the Sunderbans. In Bangladesh it is used in large quantities to make roofs on the houses in the villages. The leaf, which looks like the feather of a bird, is sliced into two over its stem, leaving a stem with fringes on one side only. The length used varies from 1.8 to 2.4 m. It is mainly cut in the Sunderbans area and brought to site in large country boats. The travelling time for these boats in

[1] Appendix L is referred to in Chapter 14.

[2] The contents of this Appendix have been copied (with small adaptations) from BWDB (1977), Appendix 3.

general is 3 to 5 days. Golpata is counted and marketed per "kahon'' which consists of 1280 (half) leaves. One kahon is sufficient to make the roof of a normal size Bangladesh house.

L.6 Jute

Raw jute (Bangladesh is a top producer in the world) is twisted to short ropes at the site and used to tie the bamboo framework together. Also the line to pull the block of the pile driving rig is made of jute.

L.7 Bamboo

The bamboo used mainly grows in the villages around the site, where it is bought by agents of the Contractor in the local markets. The bamboo is of a heavy type, with hardly any hollow space in the central part. Although the cut length is more, the working length is in general not more than 6-9 m. Bamboo is used for:

- Ganji piling,
- frame work to close compartments,
- anchoring and pinning down of matas.

L.8 Sundari bullah

These are trees growing abundantly in the Sunderbans area. The length, according to a forest engineer, ranges from 9 to 15 m.

Only exceptionally do they grow taller than 18 m and certainly this length is hard to obtain on the local markets. When the length is about 9 m, the diameter at the top is about 25 cm.

The wood of the sundari bullah is hard and tough and does not float in water. At Gangrail (as well as in 1978 at Madargong[3], difficulties were encountered in obtaining the long lengths required.

L.9 Mata

The composite construction material mata together with the clay forms the main body of the primary dam. Matas are made of straw rope, golpata, straw and clay. The length in which they are being used depends on the width of the channel to be closed, but in general lays between 30 and 45 m. The diameter is approximately 90 cm.

To construct a mata, first straw ropes having a length of 7.5 m are laid on a platform at distances of 25 cm apart and perpendicular to the long side of the mata.

With their length parallel to the long side of the mata, a layer of golpata is spread. The golpata leaves are laid at distances of 2.5 to 4 cm and cover a length of approximately 1.2 m, overlapping each other. On top of this layer comes a layer of straw, evenly distributed and loose, having a thickness of 5 to 8 cm.

This straw layer is then topped by a layer of clay, the lumps being cut to size. The thickness of this layer is hard to judge, but will be between 1 and 2 cm. This whole packet is then being rolled into a kind of sausage, whereby the ends of the ropes are tied to avoid the mass getting loose when rolling down the slope during the placing.

400 labourers can prepare 20 matas in a day and place them at the time of Low tide. The weight of a mata depends much upon the quantity of clay used, but on average varies between 6.6 and 7.3 kN/m^3. The diameter is between 75 and 90 cm.

L.10 Gunny bags

Gunny bags are made of jute. Mostly they are second hand, but still in good condition. Their capacity is 40 to 80 kilos (40 dm^3).

[3] BWDB (1978)

APPENDIX M[4] Texts in old Japanese books concerning the repair of breached dikes

M.1 From:

小林丹右衛門（安達満翻訳校注）『川除仕様帳』（『日本農書全集第65巻』農山漁村文化協会1997: 5-58.

Translation:

KOBAYASHI Tan'emon, *Kawayoke Shiyōchō* (Specification Book for River Protection, 1720) edited by ADACHI Mitsuru in *Nihon Nōsho Zenshū* 65 (Tokyo: Nōsangyoson Bunka Kyōkai, 1997), 5-58.

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出水のとき、切岸が掘り込まれたり、堤防が壊されたりした場合、すぐに蛇籠や尺木牛などで防ぐことはしない。水が深くては作業もできないし、かえって悪影響があるものである。そういうときは竹木を伐り、石がなければ俵に石砂や芝などを詰めてよくくくり、それを竹木の先にしっかりと結びつけ、竹木の先を川底に沈むようにいれ、川縁に雌鶏の羽のように並べて押しつけ、竹に結びつけておく。

Translation by Google

When the river bank is dug in or the embankment is broken, it is not immediately protected by gabions[5] or shakugi oxen[6]. If the water is deep, it is impossible to work, and it has adverse effects. In such a case, cut the bamboo, and if there are no stones, fill the bale with stone sand or grass and tie it well, tie it firmly to the tip of the bamboo, and place the tip of the bamboo so that it sinks into the riverbed. Arrange them like chicken wings and press them down and tie them to bamboo. By the way, it is good to tie 5 or 6 bamboos to the tail, or 10 bamboos in a bundle, and tie them with stones or bales.

Translation by DeepL

When the water rises, if a cut bank is dug in or a dike is breached, we do not immediately prevent it with a snake cage[1] or a shakugi cow. If the water is too deep, it is impossible to work, and this can have a detrimental effect. In such cases, cut down the bamboo trees, fill bales with stone sand or grass if there are no stones, tie them tightly to the ends of the bamboo trees, place the ends of the bamboo trees so that they sink to the river bottom, line them up like hen's feathers on the river edge, press them together, and tie them to the bamboo. By the way, it is good to tie 5 or 6 bamboos to the tail, or 10 bamboos in a bundle, and tie them with stones or bales.

[4] Appendix M is referred to in Chapter 18.

[5] Gabion = snake cage = yakago.

[6] Shakugi-ushi: a kind of *ushi* (Japanese groyne constructed with timbers) consisting of raftered *shakugi-gaki* (palings fixed on a jakago, Fig. 8).

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また、切れたところが川の本流になりそうな場合には、ほぼ水が引いたところで切れ口に笈牛[7]を入れるがよい。そのまま放置しておくとさらに瀬が掘れてあとで工事がしにくいものである。たいがい、出水のときは少しの被害にも大勢集まってあわてふためいて、手の届くところばかり大げさに補強し無駄なことをするものである。まして大きな被害のときはなおさらである。むしろそのようなときには、冷静に対策を考えて、ひと眠りするような気持でいるほうがよい。とはいえ、常にそうとばかりはいえず、場所やようすによることはいうまでもない。ところで尾につなぐ竹は五、六本か竹によっては一〇本を一束にして石や俵を結びつけるとよい。

Translation by Google

Also, if the cut is likely to become the main stream of the river, it is better to put the cow into the cut when the water has almost receded. If left as it is, the shallows will be dug further and it will be difficult to work later. Most of the time, when flood waters occur, even a small amount of damage causes a large number of people to gather and panic, exaggerating reinforcements only where they can reach them, and doing useless things. It is even more so when there is a great deal of damage. Rather, at such times, it is better to calmly think about countermeasures and take a nap. However, it cannot always be said that it is so, and it goes without saying that it depends on the place and the situation.

Translation by DeepL

If the cut is likely to become the main channel of the river, it is advisable to insert an oi cow[8] into the cut when the water has almost receded. If you leave it as it is, the rapids will be further dug out and it will be difficult to work on it later. When a flood occurs, even a small amount of damage is usually not enough to cause a panic, and people tend to exaggerate and wastefully reinforce only those areas that are within their reach. This is even more so in the case of major damage. It is better to calmly consider countermeasures and take a nap at such times. However, this is not always the case, and it is needless to say that it depends on the location and conditions.

[7] 牛枠（三本の木材で三又を組み、蛇籠で沈圧した水防施設）の前に前立ち木をつけたもの。谷川・小川に用いる。

[8] Oi-ushi: a kind of *ushi* with palings (Fig. 4).

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堤防が決壊したところを修復するにあたっては、まず川の流れの側から輪留[9]にする。ことに砂地の場所は切れ口がだいぶ掘り込まれるものであるから、すぐに修復しようとすれば人夫が大分必要となるものである。川岸から引き下がって築いてある土手ならば川の流れの側からでも土手裏からでも輪留にして修復する。切れ口が掘り込まれていないときはすぐに修復すること。

Translation by Google

When repairing the collapsed embankment, the first thing to do is to tether[10] it from the side of the river flow. Especially in sandy places, the cuts are dug in, so if you try to repair it immediately, you will need a lot of workers. If the embankment is pulled down from the river bank, it can be repaired from either the side of the river flow or the back of the embankment. If the cut has not been dug in, repair it immediately.

Translation by DeepL

When repairing a levee break, the first step is to start from the side where the river flows. In particular, since the cut was dug into the sandy soil, a lot of manpower would be required to repair it immediately. If the bank has been built down from the riverbank, it can be repaired from the side of the river or from the back of the bank. If the cut is not dug in, it should be repaired immediately.

M.2 From:

SATO Nobuhiro: '*Teibo Kokyoku Shi*' (History of Embankments and Water Courses, (1876); Tokyo

How to repair a dyke breach

In order to repair a broken part of a dyke, where water is deep, sink stone sacks as underwork, drive piles and make fences outside as well as inside the breach, then build the embankment. Sometimes twigs and fascine balls called "varia" are sunk under water with earth sacks where it is scoured very deep and the force of water is very strong. When the breach grows several hundred yards, however, the force of water inside and outside is too strong to be kept and whatever put in will be flushed away. In this case, fill the breach gradually from both sides, letting water flow through the opening in the middle and then close the middle finally.

In such places, only the true adept knows a better way than being broken several times. It is particularly difficult where fillers should be transported by ship. It should be well devised and the best workers should be employed. In order to estimate the underworks, the lengths, breaths and depths of the breach should be measured in several places according to the situation of breaking. For example, five ken[11] wide, one jo[12] deep and fifteen ken long at one place, four ken wide, one jo and two shaku deep and seven ken long at another place, four ken wide, seven shaku deep and eighteen ken long, and so on, and average them to estimate.

[9] 月の輪ともいう。堤防の切れ所を半月形に囲むように築いた堤防。

[10] Wadome: semi-circular dike constructed in an emergency to enclose the breach in the main embankment (fig. 7); comment Van Duivendijk: in fact a bund along the perimeter of the scour hole (old Dutch: *vingerling*).

[11] Old Japanese unit of length. 6 shaku make 1 *ken* (about 1.818 m); i.e. 1 shaku is about 30.3 cm.

[12] Old Japanese unit of length. 10 shaku make 1 *jo* (about 3 m).

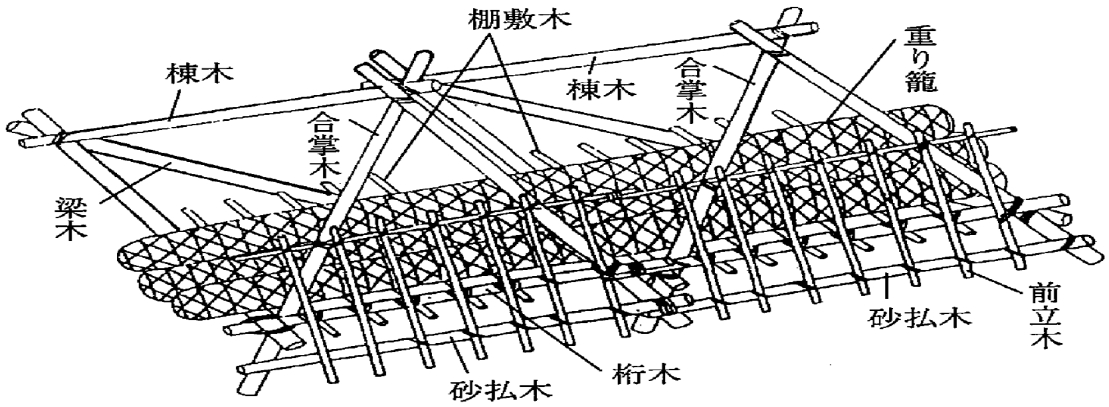

図 4　笈牛

Fig..4　A kind of ushi with yakago

図 7　輪留

Fig. 7　Wadome, i.e. a perimeter bund around a scour hole (*vingerling)*

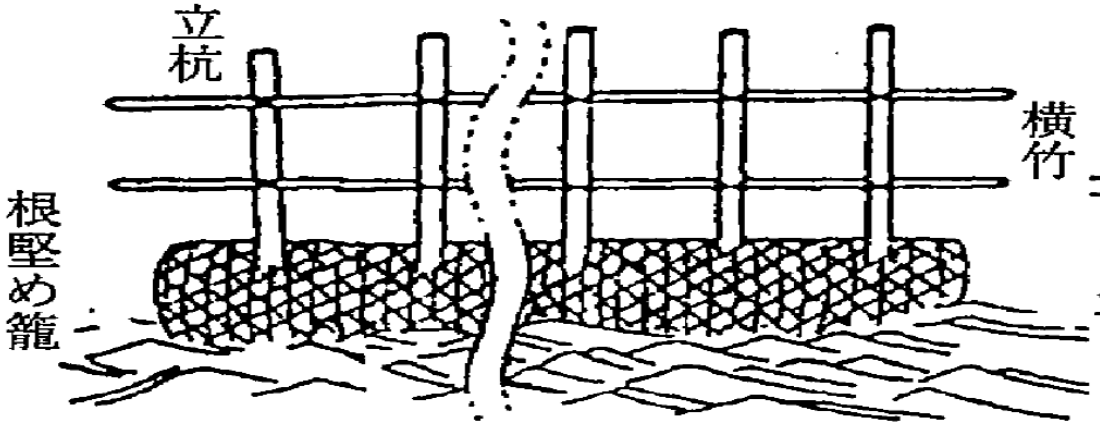

図 8　尺木垣

Fig. 8　Shakugi-ushi: a kind of *ushi* (Japanese groyne constructed with timbers) consisting of raftered *shakugi-gaki* (palings fixed on a yakago)

APPENDIX N Formulae as applied in CRESS: tidal flow through closure gap[1]

'Channel detention (storage area approach)
When assessing storage basins it is assumed that the surface of the water in the basin is flat and horizontal. The calculation rule determines the variations in water level in the basin that result from variations in sea level (for example tides). It is assumed that the variations in water level are sine shaped. The phase shift, the maximum fall over the flow opening and the maximum average velocity in the flow opening are also calculated.
Equations used:
The variations in water level of the sea in relation to the SWL are schematised as:

$$\eta_z = \tfrac{1}{2} \cdot H_z \cdot \sin\left(\frac{2\pi}{T} t\right)$$

The variations in water level of the basin in relation to SWL is schematised as:

$$\eta_B = \tfrac{1}{2} \cdot H_B \cdot \sin\left(\frac{2\pi}{T} t - \varphi\right)$$

with:

$$H_B = \sqrt{2} \cdot k \cdot \sqrt{\left(\sqrt{1 + \left(\frac{H_z}{k}\right)^2} - 1\right)}$$

For the phase shift:

$$\varphi = \tfrac{\pi}{2} - \arctan\left(\frac{2k}{H_B}\right)$$

and

$$k = \frac{6\ g}{32\ \pi} \cdot \left(\frac{c \cdot B \cdot h \cdot T}{A}\right)^2$$

The maximum hydraulic gradient/fall in level follows from:

$$\Delta h_{max} = \sqrt{\tfrac{1}{4} H_z^{\,2} + \tfrac{1}{4} H_B^{\,2} - \tfrac{1}{2} H_z \cdot H_B \cdot \cos(\varphi)}$$

and the maximum flow velocity follows from:

$$u_{max} = c \cdot \sqrt{2 \cdot g \cdot \Delta h_{max}}$$

in which:
A: storage area of the basin [m2]
B: width of the flow opening [m]
c: discharge coefficient for an short free-flow weir/spillway (ca. 0.9 - 1.3) [-]
H_z: tidal range at sea [m]
h: average water depth in the flow opening [m]
T: wave or tidal period [s]'

[1]Text and equations have been copied from http://www.cress.nl/About.aspx); CRESS Coastal and River Engineering Support System, A16.1 Flow through closure gap (2.0)/Help

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I have tried to find all copyright holders of the illustrations in this dissertation. In case I did not succeed, copyright holders are requested to get in touch.

Curriculum Vitae

Johannes (Hans) van Duivendijk was born in Zierikzee on 30 October 1934 where he attended the Zierikzees Lyceum. He got his diploma (HBS-B) in 1952 and started that same year his studies in Delft at the Technische Hogeschool, Afdeling der Weg- en Waterbouwkunde.

During his studies he worked in the summers as a trainee in the Netherlands (twice on tidal closures that were required after the 1953 storm surge flood: at Schelphoek in 1953 and at Dijkwater in 1954) and in Austria and Canada (hydropower construction sites). These traineeships are mentioned because they have had an important impact on his subsequent career and on the subject of this dissertation. He graduated as civil engineer in 1958 on a thesis that dealt with a design of the Grevelingendam at an alternative alignment. He completed his compulsory military service in 1960. Van Duivendijk was employed by the consulting engineering firm Haskoning (now Royal HaskoningDHV) from 1960 until his retirement in 1999. During this period he mainly worked on overseas projects in more than 20 countries. This embraced assignments of short duration (1-2 weeks) to much longer involvements (5 to 10 years). The assignments always concerned hydraulic engineering in all its variations: river basin development (large dams, flood control, river training), sea defences, ports and tidal closures. His duties could be of a technical nature (design and/or supervision of construction) but also in the managerial field. He was, for instance, leader of large multi-disciplinary study teams in Indonesia, Morocco and Bangladesh. His experience is reflected in the many articles and some books written by him. He has been a member of a number of international technical committees within the context of organisations like Nethcold, ICOLD and ICID[1].
In 1985 Van Duivendijk received a high award of the Association of Consulting Engineers of the Netherlands, the ONRI Riband, for his Feni River Closure concept. At that occasion he was also presented the statuette[2], shown on the back cover of this dissertation: a clay bag bearing Bangladeshi on the day of the Feni closure.
After his retirement he continued his part-time duties as associate professor, at Delft University, Faculty of Civil Engineering and Geosciences, in the dept. of Hydraulic Engineering. In this capacity he gave lectures in Water Power Engineering during a period of 23 years (1986 – 2009).
In 2010 he was appointed a Knight of the Order of Orange Nassau by Her Majesty, Queen Beatrix of the Netherlands.

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3. *"The Systematic Approach to Flooding Problems"*, in Irrigation and Drainage, Journal of ICID, Vol. 55, Supplement 1, July 2006 ,Wiley

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[1] Acronyms for: the Netherlands' Committee on Large Dams, International Commission on Large dams and International Commission on Irrigation and drainage, respectively.
[2] The bronze statuette was made by R.P. Pleysier, a Dutch sculptor.
[3] One publication (nos.16) is in the Italian language.

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Fighting
Against the Current

Restoring dike breaches and damming tidal channels by simple means – from past to present

Over the last centuries people have developed several ways to close dike breaches and to dam tidal channels. These closure methods were needed to repair damages after floods and to reclaim land and they were carried out by using simple means. By simple means one must think about the application of local materials and simple equipment by (mainly) manual labour.

This dissertation aims to provide a systematic overview and analysis of closure methods that have been developed in the course of time.
The research was in the first place aimed at Dutch closure methods as practised in the Netherlands, Germany (Schleswig Holstein) and Ghana/Bangladesh (20th century). But it was also considered useful to check the closure methods developed in other countries than the ones just mentioned. This resulted in the identification of closure methods practised in the past in Bengal, China, England and Japan.

J. van Duivendijk