CIE3330

Hydraulic Structures

Locks

version March 2011

W.F. Molenaar e.a.

Cover photo and artist impression: IJmuiden lock complex – the Netherlands Courtesy Rijkswaterstaat



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APPENDICES

A1 Inland waterway vessels, CEMT classes & transport capacity
A2 Water Management
A3 Ice control in locks
A4 Case study:Naviduct in Enkhuizen

PREFACE

Are these lecture notes finished for the next years? Probably not; big projects like the Panama Canal Lock project and the Seine-Nord project are knocking on the door and shall find an entry into the material written down here. But it is time to stick to the 80-20% efficiency rule.

And why did writing these lecture notes take so long if only an English translation of the already existing Dutch lecture notes had to be produced in 2004? The answer is that opportunity knocked on the door: there was a chance to include material from books published e.g. by Rijkswaterstaat, and the new material that would be gathered by the PIANC Workgroup for the revision of the navigation lock report; this update was recently published indeed, Of course, from the moment more considerable changes get into view, minor items like upgrading the illustrations is taken at hand as well. It all boiled down to a very substantial revision of the lecture notes on locks; it took the time that was needed, too long for the impatient mind, but it was done with pleasure.

These lecture notes were written upon the backs, shoulders and brains of many predecessors, if anyone, Kees Bezuyen has to be mentioned because he meant so much for many hydraulic engineering students, so long. Students, graduate students and young engineers also contributed and I hope that especially student readers will show the initiative to point the finger at mistakes, but what would be better, possible improvements in the future.

Delft, March 2011 Wilfred Molenaar

READER TO THESE LECTURE NOTES

These lecture notes on locks are part of the study material belonging to the course 'Hydraulic Structures 1' (code CT3330), part of the Bachelor of Science and the Master of Science, the Hydraulic Engineering track, for civil engineering students at Delft University of Technology. Many of the principles and engineering techniques treated in the BSc curriculum have to be applied when designing a lock. Everything has to be used, from construction mechanics to fluid and soil mechanics, and anything in between, or vice versa. A most important design aspect will be added: construction. Unfortunately, construction differs considerably from site to site, hence this important matter surely cannot be discussed in all its details and varieties, but an effort to show the most relevant items to be taken into account will be made. It should be emphasised that the designer has the opportunity, maybe the obligation, to combine all the imaginative powers, common sense and engineering judgement, to deal with the present day and future problems related to the project at hand.

In chapter 1 we will provide a general overview of the various sluice types that exist. Functions and their application in different types of sluices will be discussed. A (navigation) lock has to suffice to most, if not all the functions that will be reviewed. Therefore it is meaningful to produce a (preliminary) design of a navigation lock to find out into what trouble the designer gets combining all the functions. In order to be able to make a design the required theory will be presented in Chapters 2 through 4. Chapter 2 deals with general design matters and subjects that are especially important in the initiative and feasibility stages of a project. In following chapters the focus will be more and more on the structures of lock (complex) and the constituting elements. Chapter 3 the main items to determine the overall navigation lock layout are described, whilst Chapter 4 will treat the most important structural components (filling and emptying systems, lock chamber, lock gates, lock head, etc.), in more detail.

For Dutch BSc-students this is probably one of the first courses in the English language, so some very specific technical terms have been translated into Dutch *(indicated between brackets and in italics).*

1. Introduction to sluices, locks, especially navigation locks

According to Webster's Universal College guide sluices are defined as:

- 1. An artificial channel for conducting water often fitted with a gate at the upper end for regulating the flow.
- 2. The body of water held back or controlled by a sluice gate.
- 3. A channel carrying off surplus water.
- 4. An artificial stream or channel of water for moving solid matter

The definition of sluices used in hydraulic engineering is often the first. A lock is part of the sluice family, it consists of an enclosed chamber in a waterway (*vaarweg*) used to transport vessels from one to another water level.

The main functions of a sluice, as defined by the Dutch TAW (Technische Adviescommissie Waterkeringen; this committee was abolished in 2005) are:

keersluis

- Water retention (keren)
- Water locking (schutten)
- Water discharge (spuien)
- Shore connection (oeververbinding)

Before focusing on navigation locks some other sluices will be briefly described to illustrate the wide variety of structures used for, on one hand, water retention, on the other hand, passage of the same water and e.g. ships. The latter last but not least in many situations. Keep in mind that the word sluices is used to indicate that the lock belongs to a larger family of structures constructed for similar reasons. The following sluices will be dealt with in different sections:

- Dewatering gate uitwateringssluis
- Stop lock spuisluis
- Guard lock -
- Navigation lock schutsluis

1.1 Dewatering gate (uitwateringssluis)

Large parts of the Netherlands are situated below mean sea level. Therefore, the surplus of water in the polders (caused for instance by seepage or rainfall) has to be discharged into the channel surrounding the polder (*boezem*) or directly into sea by artificial means, e.g. using pump stations (*gemaal*). For these water management (*waterbeheer*) purposes, hydraulic structures such as dewatering gates (*uitwateringssluizen*) are built.

In polders the discharge of water (from low to a high) is separated in two steps. First, the water is pumped in a storage basin or reservoir, or discharge channel (*boezem*) located at a higher level than the polder. Next, as soon as the water level in the basin or channel exceeds the water level of the sea or the nearest river, the water is discharged through a dewatering gate. Typically such a dewatering gate is a duct (*duiker, (omloop)riool)*) with a valve that serves as (adjustable) closing element (*afsluitmiddel*). This implies that shipping through a dewatering gate is impossible.

Additionally, the dewatering gate may serve as fresh- salt water separator or retain water (for instance between salt sea water and fresh or potable water of a canal or polder system). In case high external water levels occur it acts as a barrier.

From the above the functions of a dewatering gate can be deduced, as follows:

- Discharge of water from the area taken into consideration (polder) when the basin level exceeds the target or allowable level
- Retain water at high external water levels
- Separation of salt water and fresh water

Single or multiple gates working by force of nature

Water is discharged through a duct in free flow or submerged condition. To prevent flow, for water retention, traditionally mitre gates are used. The principle is simple: difference in water level will keep the gate in a closed position at a higher exterior water level and open them automatically at high interior water levels. This principle is depicted In Figure 1-1. This combination of duct and mitre gates (duiker) is often used in areas with a low risk profile for flooding. In case of a higher risk profile other solutions (e.g. different type of gates) have to be applied.



Figure 1-1

Mitre gates (*puntdeuren*) that open or close depending on the water levels

Automatic dewatering gates

There is an increasing demand for a more accurate control of the storage basin levels. This requirement results in the application of gates that can be operated by will rather than as a result of coincidental differences in water level. Moreover, the gates must be able to withstand water level differences in both directions, generally lift gates (*hefdeur*) or rolling gates (*roldeur*) are more suitable than mitre gates. The whole is referred to as the automatic dewatering gate. An advantage of this solution is that it can be used for irrigation when the water levels are too low to meet for agricultural demands. Purpose built for that situation the dewatering is more accurately referred to as an inlet sluice (*inlaat sluis*). In arid countries these types of sluices are a common element of irrigation systems.



Obviously this doesn't apply to dewatering gates in coastal areas because of the salt content of the seawater (e.g. the *Haringvlietsluizen* where segment gates were used).

In the design of the Haringvlietdam the following main functions are combined:

- Protection, retain water at high external water levels (protection North Sea)
- Diversion of water, forcing more water into the Nieuwe Waterweg (Rotterdam) to prevent the salt water tongue to intrude too far landinward
- Discharge of water (river Rhine and Meuse) when the basin level exceeds the external water level

More info on the Haringvlietsluizen; see the website: http://www.haringvlietsluizen.nl (October 2005)

1.2 Stop lock *(spuisluis)*

s

=

The purpose of a dewatering gate is to control the water level and to refresh water in a closed water basin or channel system. Besides water level control the structure can be used to clear the basin or canals from sediments, debris and other pollutants while water is being discharged. In this case the sluice is referred to as stop lock. Although the stop lock resembles the dewatering gate, the main difference is based on the principle that a stop lock is opened when the reservoir level is (considerably) higher than the water level of surrounding water areas. The water level difference should result in sufficient water discharge to flush out the sediments, debris and pollutants.

The higher the water level difference, the higher the flush velocity u, which results in increased sediment transport s, which is illustrated by the following formula:

Where:

S	=	sediment transport
m	=	'garbage' coefficient or expression, not dimensionless, typically ≈ 0.01
u	=	velocity of water
n	=	constant, typically $3 \le n \le 5$

Since the gates have to be opened at a water level difference, mitre gates cannot be applied for a stop lock. An example of a stop lock in the Netherlands is illustrated in figure. In this particular case the stop lock is positioned adjacent to a navigation lock.



muⁿ



Figure 1-3 Stop lock, Zandkreekdam Katse Heule, the Netherlands

Stop lock, Afsluitdijk – Den Oever, the Netherlands

More detailed information on the design of the Zandkreekdam - Katse Heule stop lock, on the east side of the 'Veerse meer', can be found on the following website:

http://www.dubo-centrum.nl/projecten/project.php?recID=103 (Oktober 2005)

1.3 Guard Lock *(keersluis)*/ Storm Surge Barrier

Guard locks and Storm Surge Barriers *(keersluis, stormvloedkering)* generally have two conflicting primary functions:

- Passage of vessels
- Retention of water

The gate of the guard lock will be kept open as long as possible to allow ship passage, obviously, when opened the guard lock can not retain water. A water level being too high or too low, not necessarily extreme water levels, on either side of the lock may require closure of the gate, preventing further ship navigation. Conditions for closure may be either extreme low water levels due to draught or extreme high water levels due to a storm surge. Two types of guard locks exist: one-way retaining or both-ways retaining.



Figure 1-5 Guard lock Kromme Nol near Heuschden, the Netherlands

One-way retaining guard lock

In normal conditions, from water level point of view, the lock is open, only in certain extreme conditions (during a storm) it retains water in one direction. This type of guard lock is commonly found in areas with a small tidal range. The doors are closed when the outside water level reaches a certain predefined level. When closed, the doors are part of a flood defense system, which also includes adjacent dikes or other flood defense structures. The design height of the doors may be less than the design height of the surrounding dikes pending on berm design of the dike and the allowable amount of water overtopping the structure (*wateroverslag*).

Both ways retaining guard lock

In areas with large tidal ranges, as for instance along the English Channel, harbour entrances are usually equipped with guard locks retaining water both ways. For instance, at the harbour side of the guard lock, the water level is kept between certain boundaries, effectively a harbour basin is created. A minimum water level is defined to enable a vessel to manoeuvre safely without the fear of running aground. Obviously, also a maximum water level is defined to prevent flooding of e.g. the port facilities and the town.

The harbour basin level differs from the tidal range. During low tide the water must be kept within the harbour basin, and during high tide the water must be kept out. This implies that the guard locks are only opened during the period around the turn of the tide (*kentering*), between high tide and low tide. Ships can only pass during this period with, fortunately, limited flow velocities. There may be a more restrictive limit to the draught of vessels because the high water tidal wave, thus deeper water, cannot be used

As an alternative to double retaining mitre, rolling, or sliding gates, or a vertical lift gate can be used. These types are able to retain water in both directions. A disadvantage of the vertical lift gate to be taken into due consideration is the limitation to the air draught of ships.

Storm Surge Barriers - Extreme guard locks

Storm surge barriers are closed to provide protection against extreme water levels, e.g. in storm conditions in coastal areas. The main purpose of the structure is to protect the land from the sea in (extreme) storm conditions and to ensure the passage of vessels for the remaining time.

A well known example is the Maeslantkering, the storm surge barrier in the New Waterway the gateway to the port of Rotterdam.



Figure 1-6 Birds-eye view of a storm surge barrier; Maeslantkering, Rotterdam

For more information on the Maeslantkering, see:

http://www.keringhuis.nl/engels (January 2006)

1.4 Navigation Locks

A navigation lock, shortly referred to as lock, is the link between two sections of a waterway (river, sea or channel) with different water levels. The lock enables the transfer of the ship from one to the other section of the waterway. Obviously the water level difference between both sides of the lock, the lift, is one of the main parameters to be considered in the design of the lock.

Navigation locks and the required lift

Comparing rivers or waterways having a steep water profile with rivers or canals having a rather flat water profile results in the obvious conclusion that either a bigger number of locks, or a lock designed for a higher lift, will be required for the steep profile. The efficiency of navigation would benefit in general from locks being spaced as far apart as possible, which reduces shipping delays, but may result in high lifts that can not be accommodated in most of the (traditional) locks. Positioning a number, typically 4 or 5, locks behind each other, i.e. construction of a flight of locks or lock ladder, could solve this problem. However, in these cases the use of alternatives, e.g. a ship lift or pente d'eau, should be investigated. Typically the water level difference of the latter type of structures is > 25 meter.

Since the lock generally is part of the flood defense system along the river, the decision for the lift of the lock also has effects on the retaining height of those systems and vice versa. The costs of one single big lift lock and having to increase the top level of the (whole) flood defense system, to match the retaining heights, have to be weighed against the costs of a bigger number of locks and smaller or no changes to the flood defenses.

Locks for inland navigation and for coastal areas

Figure 1-7 shows a typical inland navigation lock in the Netherlands. Due to the Dutch topographical situation the required lift generally varies from say 3 to 5 meter. Following chapters will deal extensively with this type of lock.



Figure 1-7

Typical inland navigation lock



Figure 1-8 Princess of Scandinavia entering the Middensluis, IJmuiden, the Netherlands

Locks in tidal areas differ in many respects from those in inland waterways. The generally much larger dimensions, due to the larger size of maritime vessels, see Figure 1-8, is the first to be taken into account. Besides this they operate under water level differences in either direction; may have to be designed to reduce salt water intrusion and should be able to cope with more adverse weather circumstances, including wave attack.

The following navigation lock types, which are able to lift the ship over a bigger water level difference from traditional lock to e.g. the rotating wheel in Falkirk, will be described in the hereinafter:

- (Traditional) lock; for inland navigation and for coastal areas
- Lift lock
- Inclined plane
- Pente d'eau
- The rotating wheel at Falkirk Scotland

<u>Lift lock</u>

The principle of a lift lock is vertical transport of a vessel with the surrounding water in a movable closed lock chamber. Some times the lift lock has two movable chambers counterbalancing each others weight, see the description of the lift lock in Henrichenburg – Germany below, but this system can be considered as slightly outdated. More often the weight of the moveable lock chamber is balanced by a counterweight and the chambers are designed and constructed as independent systems. Provided the weights of the chamber and the counter weight are more or less in equilibrium, a relatively small 'force' is required to put the system into motion. For the latter mechanical devices, e.g. winch-cable or worm screw-engine systems, or electro-magnetic engines can be used. Note that the weight of the moveable chamber nearly always remains the same, since the total volume of water displaced equals the mass of the vessel entering the chamber.



Figure 1-9 Lift lock Strepy Thieu, Belgium

Old type lift lock – "floaters"

To reduce the total force in the chamber lifting system usually floaters are constructed under the chambers. These floaters move up and down in separate reservoirs by adjusting the water level. Imagine a piston moving in a cylinder. When floater and chamber 1 move down, water flows out of reservoir 1 into reservoir 2, where the increased water level lifts floater and chamber 2. Instead of under barometric pressure, the water can be kept under high pressure in the cylinder. By linking the cylinders through a pipe with a valve that can be opened and closed as required an alternative, very accurate motion system is created.

The chambers have to be closed at each end by a gate or bulkhead to prevent the loss of water during the lift, the vertical transport. Obviously the adjacent ends of the upper and lower canal reaches have to be equipped with gates as well to avoid draining the canal when the chambers are on the move. When the lower chamber is correctly aligned for opening the gates from the lower canal, the upper chamber has to be in the right position (level/height) for opening the gates from the upper canal as well. Before opening



the gates the bottom and walls of the chamber and canal have to be joined with a watertight connection and water level differences between the chamber and the canal reach have to be equalized as well.



Figure 1-11 Lift lock Luneburg – Germany. Areal view and cross sections

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Inclined plane

The principle of an inclined plane is the use of a natural slope in the terrain to overcome a height difference. A movable lock chamber is moved over a slope. The adjoining waterways must be sealed of with a water retention structure and gates (e.g. Ronquires 1968). Inclined planes can de divided in two types:

- Transport in longitudinal direction in the axis of the vessel (Ronquieres Belgium; not in these lecture notes
- Transport in transversal direction perpendicular to the axis of the vessel (Arzviller France, see below)

Depending on local circumstances a choice is made for the one or the other. Some advantages and disadvantages of these systems are listed below.

- Movement of water in the chamber during acceleration or slowing down, which is especially important for longitudinal systems
- Amount of required space
- Forces in the cable system
- Positioning in the landscape.



Figure 1-12 Inclined plane in sideways direction, Arzviller, France

<u>Pente d'eau</u>

The principle is the same as the inclined plane; the use of a natural slope in the terrain to overcome a height difference. In this case water is being moved using a more or less watertight door in a kind of canal pushing the water up or down a slope. The advantage would be a simpler construction than an inclined plane. The disadvantage is that more energy needed to transfer water plane and ship. Up to now only a few have been built (Montech 1974, Béziers 1983).



Figure 1-13 Pente déau, Béziers, France

On the following website a nice animation showing the working principle of the pente d'eau in Montech – France can be found:

http://www.canaux-historiques.com/d2m/ouvrage/montech/fiche_ouvrage/fo_00.html#

For improving the inland shipping connection between Paris and Belgium & the Netherlands, known as Seine-Nord, the feasibility of a large pente d'eau has been taken into consideration (2006).

Rotating wheel

The original concept of a wheel to act as a boat lift actually dates back to 19th century Europe; it was seriously considered by British Waterways as the solution for Falkirk in 1994.





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Artist impression
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Landrot: boatlift Schipper / Zeeman: shiplift

Figure 1-14 The wheel during construction, Falkirk, Scotland

The objective was to create a functional boat lift that could raise and lower boats swiftly, over a height of 24 m, with a structure worthy of a new millennium. The Falkirk Wheel lies at the end of a reinforced concrete aqueduct that connects, via the Roughcastle tunnel and a double staircase lock, to the Union Canal.

Ships entering the Wheel's upper gondola are lowered by a rotational movement, along with the water that they float in, to the level of the lower canal reach. At the same time, an equal weight is lifted up in the other gondola, which acts as counterweight. Archimedes Principle of displacement is applicable on the total weight of the gondolas. The mass of the boat sailing into the gondola displaces a volume of water having exactly the same mass. The total weight of the gondola remains the same.

The gondolas, or moveable lock chambers, have to rotate in the opposite direction of the wheel, otherwise the vessel and surrounding water would be thrown out like the water of a bucket being emptied. Therefore small wheels. running on a single curved rail fixed to the inner edge of the opening, have been assembled

to the gondola, at both ends. In theory, the downward weight of the gondola should be sufficient to always remain horizontal, but any friction or sudden movement could cause the gondola to get stuck or tilt. To ensure that this could never happen and that the gondola, the water and the boats, always remain perfectly level throughout the whole rotation cycle, an inventive series of linked cogwheels *(tandwielen)* is used as safety system.

Given the precise balancing of the gondolas and the relatively simple, nonetheless clever system of cogwheels, operating without too much friction, only a small amount of energy is required to turn the Wheel. In fact, it is a group of ten hydraulic motors located within the central spine that provide the small amount, just 1.5 kW, of electricity to turn it. The figure below shows their position.



Figure 1-15 Principle of the rotating wheel at Falkirk

More information on the Falkirk Wheel can be found e.g. on:

http://www.falkirk-wheel.com http://en.wikipedia.org/wiki/Falkirk_Wheel

2. Navigation lock design

2.1 Design process in general

Design starts with thorough analysis of the problem or project at hand. Is it really necessary to maintain a water level difference? Is the shipping intensity indeed high enough to warrant the investments in a navigation lock? Asking the right questions about the problem, the objectives, functions, operational aspects, constraints and assumptions should finally result in a good set of design criteria or in the Basis of Design (*Programma van Eisen*).

To produce alternatives, not only a systematic work method, but sufficient creativity, which may be best served by the opposite of systematics, is required as well. Politics comes into play during the selection of alternatives to be elaborated. Frequently multi-criteria analyses are used to weigh for instance safety, noise or panoramic pollution, think about the required towers for a lifting gate, against costs. Fortunately further



Figure 2-1

Common flowchart design process

elaboration of the selected alternative(s) is a more straightforward engineering matter.

During the whole design process the work proceeds from more general considerations and rule of thumb use into more detailed calculations. Results of more detailed calculations are feedback for the previous level. Use the feedback to decide whether or not design is still heading in the right direction.

At a certain moment the life stages of the project or structure have to be taken into account. It is easy to imagine that e.g. construction methods or maintenance requirements (operation phase) influence the design. The following phases should be distinguished in the life cycle of a navigation lock:

ldea Initiative	Feasibility	Design	Construction	Operation	Reuse Disposal
	F ierry				



Design plays its role from the idea/initiative into the reuse or disposal phase, but most dominant in the design phase itself. In this and the following chapter general and more detailed lock design issues will be dealt with.

2.1.1 Functions of a Navigation Lock

The functions of a navigation lock are analysed in the first place to determine the necessary lock components. The analysis also contributes to the preparation of the design criteria for the navigation lock or finding the (quantitative) requirements the lock should suffice to.

The three main functions of a navigation lock:

- 1. Water retention maintain a water level difference e.g. for water management or flood defense purposes
- 2. Ship passage the water level difference and the means to maintain the water level difference require solutions for vertical and horizontal transport of the vessels
- 3. Water quality management depending on the surroundings or environment of the lock care may have to be taken about e.g. sediment and debris discharge or separation of salt and fresh water

Note:

As mentioned in the Introduction the TAW defined 'Shore connection' as the 4th 'main' function of locks. This is a matter of definition indeed; what is a main function and what is a secondary function? Often it depends on a particular situation. One could argue that water quality management is not a main function at all. Another example would be to consider 'Provision of port facilities' as main function of a pleasure navigation lock, whereas this would not be applicable for commercial navigation.

It is easily observed that (structural) solutions for water retention may get in the way of ship passage and water management, and vice versa, which is illustrated by the (incomplete) table below.

Function				
Retain water	Transport vessels			
	Horizontal	Vertical		
Dike / levee	In water & using ship's engine	Adjusting water level		
Sheetpile wall	Sliding [*]	Lifting / lowering *		
Door / gate	Rolling *			
'Return pumping'				
* with or without the surrounding water or lock chamber				

Table 2-1 Functions and solutions

Depending on the point of view the lock will be considered as a flood defense structure or as transport infrastructure. In the following the emphasis will be on inland navigation, hence the transport function.

Ship passage function

Navigation locks play an important role in the inland waterway transport network for the transport of goods in Europe and in the Netherlands in particular. More than 35 000 kilometers of waterways connect hundreds of cities and industrial regions. In European inland waterway transport is a realistic alternative to road and rail transport. It is energy-efficient; the energy consumption per ton-kilometer is only 17% and 50% of the consumption of road and rail transport respectively. Noise and gaseous emissions are modest. Additionally, inland waterway transport ensures a high degree of safety, in particular when it comes to the transport of dangerous cargoes.

For the most common Dutch inland navigation lock the solution developed for retaining water and transport of the vessel is to use a lock chamber that can be closed by gates to maintain the water level difference. Within the lock chamber the water level can be adjusted to allow for the vertical transport of the ships, the horizontal transport is taken care of by the ship's own propulsion. Generally cut-off screens below and adjacent to the structure are used to prevent piping. The function analysis for the typical Dutch

lock would result in the following lock components being necessary: Gates and housing, lock chamber, a water leveling solution and cut-off screen.

It is left to the reader to do a more elaborate function analysis.

Note:

A clear cut between the analysis of functions and the analysis of operational aspects is not always possible, and again, depends on definition. In these lecture notes the lock chamber will be the result of the functional analysis, whereas the dimensions of the lock chamber will be the result of operational considerations, see section 2.1.2. The result, a comprehensive list or document of design criteria, matters much more than determining by which principal method or principal analysis it has been or should be obtained.

Water retaining function - maintaining a water level difference

A lock is situated in a waterway between two sections with a different water level. Combined with dikes and other structures it is part of a water defense system. The upper lock head (at the side of the highest water level) must be able to retain the highest water level. The lower head only retains the maximum chamber level, which is not necessarily the highest water level. In tidal areas lock heads are referred to as outer head (on the sea side), respectively inner head (on the land side).

As part of the water defense structure the navigation lock must be able to retain water under all circumstances. If for any reason (e.g. a ship runs into a gate) the lock is damaged a back-up system, a second set of gates or stop logs, must be available to take over the water retaining function.

Typically, water levels will be different at both sides of a lock. This will result in a groundwater flow under and around the sides of the lock structure. At the downstream side where the groundwater flow exits the soil, erosion will occur in case the flow velocity is high enough to induce soil movement. This is a self sustaining and enlarging process; since the first erosion will result in decreased flow resistance and subsequent increased groundwater flow, exit velocity and erosion. The phenomenon is known as seepage and /or piping, and may undermine the whole lock structure To avoid piping locks are provided with seepage or cut-off walls, often sheetpiles are used as cut-off screen, acting as an impermeable curtain perpendicular to the axis of the lock chamber. Alternatively filter layers and scour protection are used around the structure, provided the resulting seepage or piping remains within acceptable limits. These limits may be enforced from retention, water management or structural point of view.

Water management function

A lock can also be used for water management, for quantitative as well as qualitative aspects of a water management, or combinations. Qualitative aspects generally boil down to keeping water masses with different properties separated, for instance:

- Clean and polluted water.
- Salt and sweet or fresh water.

Quantitative aspects in the water management system are for instance:

- Discharge of a predefined amount of water within a certain period of time.
- Minimizing the fresh water loss from the upper canal reach.

Discharge of water through a navigation lock can be achieved in various ways, e.g. valves in the gates or a drain system. In tidal areas, the drainage system can be used to dispose salt water that entered the fresh water. At times of large river discharges (e.g. during a period of heavy rainfall) the full cross section of the lock may be needed for discharge. Flow velocities will be too high for safe navigation and locking is impossible in this situation anyway. The lock at Dintelsas, at the mouth of the Mark or Dintel to the Volkerak – the Netherlands, is an example of this type of lock.

In case of smaller discharges the lock's drainage system can be used to get rid of a surplus of water. However, sluicing and locking cannot take place simultaneously. Preferably the discharge of water is in "quiet" shipping hours, e.g. at night time or during the weekends. This to minimise hindrance for vessel traffic. In case of multiple lock chambers both functions are more easily combined. The lock near Wijk bij Duurstede – the Netherlands, is used to maintain the right waterlevel on the Amsterdam –Rijn canal. The whole lifting gate is used to manage the water supply; not just valves in the gate.

 $A \cdot z + W_{\mu\nu} - W_{down}$ The amount of water lost per leveling cycle is: = horizontal area lock chamber A = difference in water level, water head \overline{z} = amount of water which is displaced by ships going upstream W_{up} W_{down} = amount of water which is displaced by ships going downstream (4) 1 H=z W_{down} Wup 11/211= =///4/11= 2 5 Axz A×z ◀ =///411= =//ANE 3 6 W_{down} =/// =7/2\\

Figure 2-3 Locking cycle and water losses

Generally there is an imbalance in traffic, either a different number of ships sail up or down, using alternative routes to return to the point of origin or destination, or vessels are loaded in upstream direction whilst they sail in ballast in the downstream direction, or vice versa. To minimise the amount of water lost, the use of reservoirs or water saving basins could be considered, or much more rigorously, the amount of lost water could be pumped back to the upstream river reach.

2.1.2 Operational aspects

After the analysis of lock functions operational aspects are studied. Predominantly this will result in dimensions of the lock components (preparation of design criteria), however, it will extend the list of lock components as well. To mention only a few operational aspects:

- Navigation through the lock
- Operation of the discharge system
- Lock maintenance (regular and incidental)





Operation: Navigation through the lock

Analysing navigation through the lock results amongst other, in the following three design requirements:

- 1. Lock dimensions must be large enough to accommodate the governing *(maatgevend)* vessel for the considered waterway (design ship).
- 2. Lock capacity, defined as the number of ships passing the navigation lock within a certain time interval, must match present and/or future demand.
- 3. Even if the capacity is not fully utilised the time losses during passage should be kept at a minimum.

The intensity of a waterway is defined as the number of ships that pass a given cross section of the waterway per time interval. The capacity of a lock is the number of ships a lock is able to process. If the capacity is too small this will result in long waiting hours, if the capacity is too large this will result in an expensive navigation lock. For instance:

According to the client 12 800 vessels are expected to pass the waterway in one year. The navigation lock is available 7400 hrs per year. This results in an intensity of 1.7 vessel per hour. If the lock capacity is smaller than required the lock will not be able to handle the amount of vessels. To determine the lock capacity a quick-and-dirty method to estimate a figure would be to use a multiplication factor of 2 - 2.5. In this case this would result in a required capacity of 3 - 4 vessels per hour.

Capacity calculations and related subjects are treated more elaborately in the course on Ports and Waterways (CT 4330).

In the above the focus was on dimensions or the quantitative aspects relevant for lock design, now a brief illustration of more qualitative results of the operational analysis. In the following table the navigation process has been described, not even in the greatest possible detail, in the left column, whilst the right hand column shows required lock components or lock furniture:

Ор	perational activity	Required facilities
1.	Ship reports in	'VTS' room / Lock-Master room
2.	Ship decelerates; possibly extra propeller	
	action required for manoeuvring	Bottom and bank protection
3.	Ship waits / not	Waiting berths
4.	Ship sails into lock chamber	Guard / guide walls
5.	Ship berthing	Fendering
6.	Adjusting the water lock level	Bollards, mooring rings,
	 ship moored 'flexible' 	ladders, etc
7.	Deberthing	
8.	Sailing out & reporting out	Bottom and bank protection
9.	Acceleration 'at safe distance'	

Table 2-2	Operational activities and required facilities
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Operation: the discharge system

Every lock provides some discharge capacity for its surroundings. It depends on the specified function and the location of the lock whether or not this has a large impact on design of the lock; some examples:

- Lock complex IJmuiden: the Netherlands: the required discharge capacity is large since a number of polders surrounding the Northsea canal are dewatered by this system. The capacity requirement resulted in construction of a separate pumping station *(gemaal)* instead of combining the navigation and discharge function in one lock.
- Locks in the city of Amsterdam: the canal system in Amsterdam has to be flushed through at regular intervals (daily) to prevent al sorts of health risks in the city. A number of the stop locks are also available for navigation. Navigation is not possible when the gates are open for discharge, generally during night hours.
- The lock in Dintelsas: after large rainfalls the north western part of Brabant is dewatered through this lock. When the lock is used for dewatering, navigation is interrupted.
- The lock at Wijk bij Duurstede: to let water in into the Amsterdam-Rijn canal the lifting gates of the lock are used. The whole gate is lifted for 1 – 2 meters for this purpose, instead of using valves in the gates, or a culvert system around the lock heads.

See the (sub)section on 'Filling and emptying systems' in Chapter 4 to determine whether or not the functions navigation and discharge (in fact the water management function) can be combined in one lock from an operational point of view.

Operation: lock maintenance (regular and incidental)

To avoid too much interference of regular maintenance (annual) with navigation through the lock it may be decided to take some extra measures, e.g. providing extra concrete cover or an extra set of gates. Obviously the costs involved throughout the years would have to be carefully considered.

When irregular maintenance (groot onderhoud) (say once in a decade) is required generally navigation is influenced, occasionally ships may have to be rerouted when the lock is completely taken out of operation during the maintenance period. Suppose the irregular maintenance requires the lock chamber to be laid open (droog gezet), which would imply that a lock with a closed concrete section has to be checked for floating up. During design it could be decided to prepare the structure for this condition by adding, for instance, tension piles, extended bottom slabs, extra concrete weight or combinations. Often structural measures of this kind result in a substantial increase of the initial investment. Alternatively it could be decided to solve the possible floating up problem if/when it actually occurs in the future if it is ever (really) necessary to lay the lock open.

Nowadays it is more common practice 'not' to design and construct for being laid open. As a consequence maintenance to the lock chamber walls and floor has to be done underwater.

In the design stage a decision has to be made on the maintenance strategy taking into consideration the extra initial investments, the future costs and last but not least, the 'negative' revenues. Commercial navigation pays for the use of the lock. When the lock is not available due to maintenance it does not generate revenues and on top of this there are extra transport costs due to rerouting (extra costs for the society; not necessarily for private parties).

Suppose the evaluation method uses the Net Present Value (NPV) technique for costs, than the costs for regular, irregular or combinations of maintenance can be ranked as follows (indicative costs only):

Maintenance type / Maintenance strategy		Maintenance and incidental costs as percentage of initial investment in lock			NPV value	NPV value ***
Initia	al investment in lock: *	Annual	Incidental		Total initial investment	discount rate
€ 10	0 000 000,-		During constructio n	During** lifetime		8%
1:	Regular + No extra measures for prevention	1%	0	0	€ 100 000 000,-	€ 112 077 074,-
2:	Regular + Extra measures for prevention	0.5%	2.5%	0	€ 102 500 000,-	€ 108 538 537,-
3:	Incidental + Prepared structure	0	10%	5%	€ 110 000 000,-	€ 114 115 750,-
4:	Incidental + No extra measures during construction	0	0	15%	€ 100 000 000,-	€ 116 462 999,-
5:	2 + 3	0.5%	12.5%	5%	€ 112 500 000,-	€ 122 669 951,-
6:	1 + 4	1%	0	15%	€ 100 000 000,-	€ 128 571 401,-
*	 this is a rough guestimate for a Class V canal lock with 2 chambers, one for each direction of travel. It also includes landscaping of the surroundings and other related infrastructural works. 					

** assume a lifetime of 50 year; irregular maintenance in the 10th, 20th, 30th and 40th year

*** the selection of the discount rate is important; the higher the rate, the more attractive it is to reduce the initial investment as much as possible and move costs as much as possible into the future

Table 2-3 NPV's of maintenance strategies

For the above example cost figures have used partially from the writers own experience and partially to arrive at a plausible answer and conclusion. In any way it is left to the reader to dig up appropriate (cost) figures in real life and 'nowadays' design situations.

The above is a mere illustration of the fact that the operational aspect 'maintenance' will require considerable attention during design, most appropriate in the more detailed design stages. In the preliminary design stages the designer has to use engineering judgement.

2.1.3 Life cycle considerations regarding the Navigation Lock

Life Cycle Management (LCM) is a management approach to infrastructure construction to achieve optimum quality and minimum Whole Life Cost (WLC). Whole Life Cost relates not only to the direct cost of construction, maintenance, etc. of the structure itself but also to indirect costs and probable benefits related to its use and the environment in which it is located. It also examines appropriate life cycle stages such as design, construction, operation, maintenance (including inspection, evaluation and repair), re-use and /or disposal. In principle LCM is aimed at providing minimum Whole Life Cost. This topic is dealt with

in detail in the PIANC Report of MarCom WG42, Life Cycle Management of Port Structures: Recommended Practice for Implementation.

During the last 20 years Life Cycle Management has become of increasing importance. Increased costs of labour and mechanical parts lead waterway authorities to design more efficient locks (i.e. with a higher quality/cost ratio). The priorities of different owners of locks varies, with some requiring minimisation of maintenance effort and cost even if this leads to higher initial construction costs, while others require minimum construction cost even if this leads to higher maintenance costs. A third category aim to assess and minimise Whole Life Cost (including operational and maintenance costs) as their main criterion, and in the absence of other investment or maintenance constraints this should be considered to be the preferred approach.

For locks, a reduced WLC should imply optimum levels of reliability (which must be determined specifically for each project), and more efficient maintenance. This does not mean less inspection and survey or less maintenance. Locks normally have to be designed to allow efficient inspection and daily/weekly maintenance without stopping traffic. Other means can also be adopted to assist in achieving the objective of minimum WLC, including quantitative assessment of the costs of closures for inspection or maintenance, and use of standardization of components as a means of reducing construction and maintenance costs. Operating costs should also be considered as part of the optimisation process. These can be reduced by adopting automation or remote control. This subject has been covered by PIANC InCom-WG18 (1992) and PIANC WG96 (2008)

2.2 Navigation traffic and lock capacity

2.2.1 Duration of lock cycle and vessel passage

Taking the journey of a vessel as a starting point, which begins and ends with the vessel being loaded or unloaded somewhere, the lock could be considered as a part of the trip where delays are likely to occur. For the vessel's journey from port A to port B, the time needed to pass through a lock has a great importance (see Figure 2-4). For inland navigation some of the items mentioned in this figure are not applicable, as items 3, 4, 9 and 10.

This figure shows clearly that more time is required than just the time to navigate through the lock. What the figure does not consider is the waiting time induced by a queue. Especially when lock capacity is lower than the average traffic intensity, queuing time will be a substantial part of the delay faced by the vessel, which is generally unacceptable. Before getting into the laborious work of optimizing the lock cycle and reducing its duration, it is absolutely necessary to check, whether or not, the overall capacity of the lock is sufficient regarding the vessel traffic to be handled.

The duration of lock passage has to be considered in terms of the vessel's journey, which includes looking at the whole river or canal system, probably including more than one lock or even including series of locks.

Maritime navigation 1. Vessel reports in 7. Sailing out & reporting out 2. Vessel decelerates 8. Tuas loose 3. Pilot boarding Pilot disembarking 4. Tugs tying up Lock operation 9 10. Acceleration at 'safe' 5. Vessel standby /not distance 6. Entering lock chamber



The reduction of the lock cycle of few minutes does not really seem significant with regard to vessel journeys in the order of days. However, for container lines in ports behind a lock, fast locking and transit is required. These container lines consider a lock as a general risk regarding the timetable to be respected. The lock may even be considered as an unacceptable risk with regards of possible strikes or blockades, or the occurrence of operational breakdowns.

Table 2-4 gives some information on the time required for some typical lock operations. The activities required for locking have a duration in the order of ten's of minutes, except gate operation.

List of Events	Average time	% of the	Average time	% of the	Possibilities for
	Inland Navig.	total	Sea lock	total	optimisation to reduce
		time		time	the total time
TOTAL LOCKING	28 min	100%	45 min (*)	100%	
(1/2 cycle)	(20 – 40 min)		(40 – 90 min)		
Entrance / Exit	5 min	18%	15 min (*)	33%	Medium
	(3 to 10 min)		(10 to 20 min)		
Mooring	5 min	18%	7 min (*)	15.5%	High
	(3 – 10 min)		(3 – 10 min)		
Gate manoeuvring	3 min	11%	3 min (*)	7 %	Low
_	(2-4 min)		(2-5 min)		
Filling / Emptying	15 min	53%	20 min (*)	44.5%	High
	(8 – 20 min)		(10 – 25 min)		
(*) For Panama the figures are: 45-60 mins for the existing Panamax locks and 80 minutes for the third					
locks (in project)	-		-		

Entrance/exit: Existing locks 15 mins. Third locks 20 mins.
 Mooring: Existing locks 2 – 3 mins Third locks 5 mins.
 Gate maneuvering: Existing locks 2 mins. Third locks 5 mins.

Filling/emptying: Existing 8-13 mins. Third locks 10-17 mins.
Table 2-4 Duration of navigation through a lock (1/2 lock cycle)

Figure 2-5 shows the lock cycle, distinguishing vessel and lock operations, for single and dual direction use of an inland navigation lock. Looking for possibilities to optimize, read reduce the lock cycle in the following circumstances and possible (counter) measures will be discussed influencing the duration. In general, for single lock, dual direction use of a lock chamber is preferred because it maximizes the lock capacity (compared to a single direction use). For a fleet of locks, as in Panama, single direction is suitable.

In the absence of vessels in the lock chamber the emptying or filling operation can be executed much faster because limiting the water movement and resulting mooring forces in the lock chamber is not necessary.

Some ideas, which are not intended to limit, restrict or even prevent new innovative ideas, to reduce the duration of the navigation cycle are:

- Pre-positioning of ships in the lock chamber to avoid waiting time – use of radio/radar/VTS : Use of radio, radar or VTS to make sure the ship is being locked at the earliest possible lock cycle reduces the waiting time.

Either the vessel may have to increase its speed or the lock operation is delayed until the vessel arrives. The latter may be more feasible for a single direction lock chamber than for a dual direction lock chamber.

- Faster movement of gates: It is imaginable that selection of a more innovative type of gates, other than the traditional mitre, lift and rolling gate, reduces the time needed to open or close the gate.
- New concepts for the filling and emptying system to reduce the filling and emptying time: for instance a non linear opening of the valves.

Equipment: self releasing mooring hooks, magnetic or vacuum based mooring devices, may be used to reduce the time needed for mooring line handling and/or tending.



Figure 2-5 Lock cycle, indication of duration, distinguishing vessel and lock operations

In the above factors influencing the lock cycle and their effect on the lock capacity of an existing lock have been discussed qualitatively. Combination of the lock capacity and variation of the traffic will result in a certain performance level of the overall traffic handling system. To arrive at conclusions on the performance it may be necessary to use quantitative results of a traffic simulation model. In the Netherlands the Ministry of Transport and Water Management, Rijkswaterstaat, uses the SIVAK (2003) simulation model for this purpose.

2.2.2 Lock Capacity versus Traffic Intensity

Generally there is enough data available to express the traffic intensity, on a river or canal, in terms of number of ships per hour, for both the upstream and the downstream direction. For the design of a new lock in the waterway, the rehabilitation or upgrade of an existing lock the following data are required as well:

- the seasonal, monthly, weekly or even daily variation of the traffic intensity
- the types of ship (ocean going vessel, barge, recreational craft) but also how they are distributed (mixed)
- the frequency of special transport and vessels with dangerous cargoes
- the (in)balance between upstream or downstream traffic, or even the (in)balance in loaded or unloaded vessels

Besides gathering the historic data a forecast has to be prepared as well.

Traffic intensity expressed in e.g. numbers per hour could be combined with data on the dimensions or deadweight tonnage of the vessels. Subsequently the traffic intensity in numbers per hour can be worked into deadweight/hour (DWT/hour) or m³/hour. Especially expression in tonnage per time unit would be useful for economic evaluation of a lock project. For technical design of the lock a requirement for the lock capacity expressed in time, m³ and combined m³/time unit is needed. Given the required lock capacity the lock cycle of an existing lock can be evaluated and, for a new lock, the lock cycle requirements can be determined. In case of an existing lock, comparison of what is required and what is available, results either in the positive answer that the capacity is and will remain sufficient in the future or not. In the latter situation lock cycle requirements have to be determined as for the case of a new lock project.

The factors influencing the lock cycle and their effect on the lock capacity of an existing lock are the following:

• Type of the vessels and heterogeneity of the fleet:

Different types of ship have been distinguished in this report, i.e. ship, barge, recreational craft. A lock designed specifically for ocean going ships generally will be less effective if mixed with inland barges or recreational traffic. This mainly is the result of the dimensions of the indeed very different vessels. The larger the mix in vessels the less effective the lock can be operated. In many cases the type of fairway or location of the lock limits the variety of ship types and only a few governing ships are in fact considered for the practical design

• Dangerous cargoes and special transports:

The transport of dangerous cargo or special transports by ship generally reduce the lock capacity because of safety requirements. Generally other vessels are not allowed within a certain safety zone around the considered ship or transport, which effectively prohibits entrance of more than one ship in the lock chamber.

• Size of the lock chamber:

Given the lock chamber dimensions, it is quite easy to determine the number of ships that can enter the lock, which depends on the size and number of the ships that arrived to pass the lock. The larger the chamber the more ships can pass in one cycle, which increases lock capacity. Less effective filling of the lock by ships, space wise, reduces the lock capacity.

To avoid emptying or filling a lock chamber that is only partially filled with ships, in many locks an extra set of gates has been provided, say at 2/3's of the lock's length. This reduces the amount of water to be released, thus saves time and increases the lock capacity. Unfortunately such intermediate gate cannot be easily combined with water saving basins.

• The number of lock chambers:

A lock chamber can be considered as a service point for the ship traffic. The more service points the larger the amount of traffic that can be dealt with.

• The type of gates:

Depending on the type of gates the effective area of the lock chamber increases or decreases. See 'Size of the lock chamber' above for the effects on the lock capacity. If the gate reduces the air clearance of vessels some vessels require special measures to pass the lock. Although not substantial, this will negatively influence lock capacity.

Depending on the type of gates installed (e.g. mitre gates versus vertical lift gate), the lock chamber contains a larger or smaller volume of water. Consequently emptying or filling and duration of lock passage, thus lock capacity is influenced.

• Gate opening and closing time:

As a separate item within the lock cycle the time needed to open or close the gates of the lock has a direct effect on the length of the lock cycle. The longer opening and closing times, the longer the lock cycle and the smaller the lock capacity. Given the dimensions of the lock, the time needed to open or close the gates is determined by the used mechanical devices. Cautiously it could be assumed that a larger lock requires larger gates, which would require more time to open and close. This is not a significant parameter to reduce the locking time. See Table 2-4.

• Water level difference (lift height):

Given the efficiency of the locking system it takes a certain time to adjust the level of the water within the lock chamber to the required outside water level. The larger the level difference, the smaller the final lock capacity and vice versa.

In coastal areas and in tidal rivers the absolute and relative variation in water levels will be higher, which generally negatively influences lock capacity.

• The operating speed of the filling and emptying system:

Usually the locking system has more sophisticated and has a higher capacity for larger water level differences. Given a certain lift height, the faster the water level can be adjusted, the shorter the required time and the higher the lock capacity. It is a major parameter to reduce the locking time, see Table 2-1.

• Water motion in the lock chamber:

Due account should be taken of water flow or waves resulting of the filling/emptying process within the lock chamber. These hydraulic phenomena will induce ship movements, thus may result in unexpected high mooring forces. To what extend mooring forces should be limited depends on the type of traffic.

• Water management:

Water Management requirements may concern:

- Saving the upstream water,
- Prevention of upstream or downstream pollution,
- Separation of salt and fresh water.

Saving the upstream water: Due to the water management requirements, water from the lock chamber may have to be stored in saving basins, instead of being discharged directly in the downstream reach. Depending on the systems used, this may take less or more time. Regarding the lock capacity similar comments can be made as under "Water level difference". Prevention of upstream or downstream pollution: Pollution may be caused e.g. due to a ship-collision or accidental industrial spillages, resulting in the loss of dangerous cargo, such as oil, gasoline, chemical fluids. Depending on the kind of pollution the lock can be closed while the pollution in the upstream or downstream river reach is being removed. This will result in congestion for navigation. It is also possible to increase the discharge to the sea or downstream reach, in order to speed up the dissolving or dispersion process of the polluting material and get to an acceptable concentration level in the water.

<u>Separation of salt and fresh water:</u> Some technical solutions to reduce the salt water intrusion on fresh water storage basins increase the lock cycle time. In case the entire lock chamber content has to be

replaced, providing fresh water instead of salt water or vice versa, the lock cycle time increases very much.

• Manoeuvring and mooring aids in and around the lock:

The easier the vessels can perform the necessary manoeuvres to sail in and out of the lock and the more convenient the mooring facilities, the less time will be lost. Consequently the length of the lock cycle may be significantly reduced by providing sufficient manoeuvring and mooring aids.

Inspection and maintenance activities:

Insofar inspection and maintenance activities obstruct the effective use of the lock chamber, time wise or space wise, the immediate result is a decrease in lock capacity.

2.3 Manoeuvrability while navigating through the lock and approach areas

The operational analysis of a ship approaching the lock, being lifted up or down, and continuing it's journey, reveals that manoeuvrability of vessels is a key issue. Other reasons making it necessary to look into manoeuvrability are safety and the duration of the lock cycle. Manoeuvrability is influenced by:

- Visibility
- Available water depth
- Cross currents and/or cross wind
- Available water area
- Approach structures
- Tug boats or other means for manoeuvring the vessel
- Twin propellers or bow thrusters
- Visibility:

For safe and expeditious navigation a clear and unobstructed view is important. This results e.g. in requirements for the alignment of lock approaches and for positions of waiting or overnight berths at sufficient distance to the track to be navigated by ships sailing in or out of the lock. For night navigation and bad weather situations (fog and/or heavy rain) sufficient navigation aids and lighting have to be provided.

• Available water depth:

The smaller the keel clearance of a ship the slower it will sail and the more its maneuverability will be reduced. Given the draught of the governing design vessels sufficient water depth has to be provided. Water levels on either side of the lock are variable and it should be kept in mind that erosion and accretion may occur due to sediment transport. Therefore water depth is quite a dynamic variable in time.

• Cross currents and/or cross wind:

Cross currents and cross winds result in ships sailing at an angle with the axis of the approaches or the lock itself. Subsequently wider approaches or a wider lock is required to prevent collisions with the structures or running aground.

• Available water area:

Ship manoeuvring is more difficult in confined water areas. An escalation of manoeuvring problems in confined waters is caused by the fact that reducing the sailing velocity in itself decreases manoeuvrability of the ship.

Approach structures:

Besides protecting the main lock structure for ship collisions, approach walls provide physical and visual guidance to the ship respectively Master of the ship, hence; have a positive effect on the manoeuvres. Opposed to jetty type structures, which are structurally light and open, guard walls are generally closed to prevent cross currents and, to a lesser extent, also reduce the effect of cross winds.

A reason not to construct a approach wall at a lock would be the availability of tug boat assistance. This generally applies to locks in coastal areas and for locking maritime vessels. The extra space required for the tug manoeuvres pushes the approach wall into an ineffective position for situations where tug boat assistance is not required.

• Tug boats or other means for manoeuvring of the vessels:

To access approach walls, tug boats and e.g. the locomotives in the old Panama lock complexes enhance the manoeuvring capabilities of vessels. The manoeuvres of the vessel require less space or are less risky, but, in case of tug assistance extra manoeuvring space is required for the tugboat.

• Twin propellers and bow thrusters:

Although effective indeed, only a limited number of vessels has twin propellers or bow thrusters. Usually in design resort to the use of approach walls or tug assistance has to be taken into account.

Poor visibility, small water depths and cross winds and currents negatively influence manoeuvrability, therefore increase the size of the required water area. Approach walls, tug boats and extra propulsion of the ship positively influence manoeuvrability. However, compared to a ship navigating through the lock only using the main propeller, the use of these means does result in a slower process of ship passage, thus a reduced capacity of the lock.

Other requirements, primarily the result of the ship passage function and important for the lock layout, might be:

• Lay-by, waiting and overnight berths:

At the lay-by berths, close to the lock chamber, the ship(s) that will be the first to enter the lock are waiting. Ship(s) that have to wait more than one full lock cycle stay at the waiting berths. At the overnight berths ship(s) will be moored for the night or for a relatively long period with a duration unrelated to lock operation. Space for lay-by, waiting and overnight berths, has to be positioned well out of the axis of the fairway. Where applicable, separate or dedicated berths should be provided for ships carrying dangerous cargoes or for recreational craft.

• Lightering area:

A zone or area where ships with too high a draught to pass the lock can be lightered. The lightering zone includes manoeuvring areas around it and shall be at a safe distance from the sailing route in or out of the lock.

• Dedicated lock chambers:

Provided the intensity of a type of traffic is sufficient it would be warranted to construct a dedicated lock chamber fort that particular traffic. A dedicated lock chamber, e.g. for recreational navigation, has quite some impact on the overall lock layout. Not only there is an extra lock chamber to be taken into account, but the approaches to and the influence of discharging water to the other lock chamber(s) as well. A sufficiently large apron between separate lock chambers reduces the effects of letting water in, or discharging it, on navigation to or from the other lock chamber.

A quick look at the functions of a lock, other than ship passage, might result in requirements, subsequent facilities and space requirements regarding the following:

- Removal of floating objects and ice floes. (See InCom WG23, PIANC 2004)
- Prevention of sediment settlement or enabling sediment transport through the lock
- Dewatering or discharge flows due to the water management function
- Basins for salt-fresh water separation

From a Life Cycle Management point of view the following should be duly considered regarding the location of the lock and/or the layout:

• Master plan of the waterway under consideration. What is the traffic forecast, what will be the number of locks (a few high lift locks or many small lift locks), etc.?

• Expansion possibilities of separate lock chambers or the lock complex as a whole

The examples in the above are not intended to be exhaustive, but they are issues likely to be encountered. Other items or subjects can and should be added, if appropriate for the design under consideration. Requirements for the dimensions of the lock chamber can be found in Chapter 3. For outer ports and connected approaches, guidelines to establish layouts, more specific dimensions and alignments, are included there as well.

2.4 Positioning of the lock complex

Consider a project to design and construct a lock (one lock chamber) or a lock complex (multiple lock chambers, water saving basins, etc.). Depending on the area available, Greenfield or Brownfield, the footprint of the required structures or lock facilities, the characteristics of the waterway, and the interaction between the three previous factors, it has to be determined how and where the lock or lock complex will be positioned. Amongst others, the following aspects should/could be taken into consideration when determining the position of the lock or lock complex:

- The number of locks to be constructed in the waterway
- Lock approach layout
- Location of the lock and construction;

See the remainder of this section for the first item and section 3.2 for the last 2 items.

The number of locks to be constructed in the waterway

Provided there is sufficient water depth available for navigation, the lesser locks that have to be passed the better. However, construction of a lock with a large head (large water level difference), where the vessels have to be lifted or lowered over the full distance, may be more expensive than construction of a couple of smaller locks. Considering e.g. the costs of dikes upstream of the lock may or may not swing the balance. Those costs depend on the height and length of the dike. If h is the head, the height of the dike varies linearly with h, but the costs vary with h²; because the area of the cross section has to be considered. The length of the dike to be taking into account is related to h (again) and to the slope of the river or waterway. Of canals without slope the length of the dikes is known (equal to the canal 's length). Often the lock or lock complex is adjacent to weirs or barriers. In that case reasons to select the number of weirs or the location of the barrier are of importance for lock design.

Often weirs and locks are constructed in each others near vicinity. If e.g. a weir's objective is to produce hydroelectric power, it will be situated at the location with the largest difference in water level. In case the weir's primary objective is to maintain a fixed water level at a given location, it will be situated downstream as close as possible to the location of interest, so as to minimize the dimensions (especially) height of the structure. Determining the position of a weir, in a river with a small discharge in order to create navigable waterways, is more complex. The river will be divided up into a number of sections. In each section the water level will be regulated by a



downstream weir. The total number of weirs and the maximum water head over the weir determines the rough position of the weirs. Figure 2-6 provides some illustration of the above mentioned.

Intermingled as it may be, the optimum number of weirs and/or locks is determined by a number of economic factors, such as:

- The construction costs of the weirs, locks and the embankment or dikes etc.
- The ship navigation costs for the time delay having to pass the weirs or locks besides it
- The damage to the surroundings, the environment in general and the (increased) risk of flooding
- The maintenance and operation costs
- The revenue generated by lock operations (toll on ship passage)
- The revenue generated by hydroelectric power
- etcetera

In the following paragraphs we will take a closer look at some of these cost or economic factors in relation to 'a number' of locks to be designed and constructed.

Nowadays Life Cycle Management is being introduced into all sorts of civil engineering projects and into the design of the project, see section 2.1.3. In agreement with the life cycle stages the following costs would have to considered:

- Design costs
- Construction costs
- Operational costs
- Reuse and/or Removal costs

Design costs

A rough, over the thumb figure for design costs would be between 5 and 10% of the overall project costs. This indication is based on experience in the past decades, where design spanned from the idea or initiative stage to the construction stage, hence, excluding design in or for operation, maintenance and removal. Obviously within a Life Cycle Management approach of projects these stages have to be included, increasing the mentioned percentages.

For larger civil infrastructure projects it is advised to take into account the upper boundary, 10%, because the societal and/or governmental decision making process generally is very complicated, hence takes a long time and continuously invokes extra design activities. Generally lock and/or weir or barrier projects fall into this category.

Construction costs

The following costs will be included and described as construction costs:

- Construction costs per weir depending on water head
- Costs of mitigating measures

Obviously construction costs are not limited to the items mentioned.

Construction costs per weir depending on water head:

The total horizontal force acting on a weir is a function of the water level difference over the weir. The following force function can be written down:

$$F = b\left(\frac{1}{2}\rho g(H+d)^2 - \frac{1}{2}\rho gd^2\right)$$

The construction cost will be dependent on the total moment forces.

$M = b \Big($	$\left(\frac{1}{2}\rho g(H)\right)$	$(H+d)^2 \frac{1}{d}$	$\frac{1}{3}(H+d) - \frac{1}{2}\rho g d^2 \frac{1}{3}d = c_2(H^2d + \frac{1}{3}H^3 + Hd^2)$
Where:	b	=	width of the weir
	Н	=	water head (average in time)
	d	=	downstream water depth relative to the foundation
	ρ	=	water density
	C ₂	=	constant (½ ρgb)

The weir structure may be a bit smaller or larger but there will be a base sum of money that has to be paid anyway (Mobilisation of equipment, labour, temporary works). Besides the more or less fixed cost part there will be a variable part of the costs that will vary with the water head (time variation) over a weir see formula above. Thus, from a certain base sum the construction costs of the weir are proportional to the load on the structure. Written down as:

 $c_{weir} = C_1 + C_2(H^2d + \frac{1}{3}H^3 + Hd^2)$ $C_3 = costs \text{ per unit of load on structure}$ $C_1 = fixed costs$ $C_2, = constant$

Figure 2-7

The construction costs of n weirs $C_{\text{constr,n}}$ for the full length of the river amounts to:

$$C_{constr,n} = nc_{weir} = nC_1 + nC_2(H^2d + \frac{1}{3}H^3 + Hd^2)$$

Say that *nH* is roughly constant, then:

In the graph on the right there are two examples of a function representing total construction graph costs. Observe that the construction costs in the first example are minimal with the construction of 3 weirs and in the second example with the construction of 2 weirs. The difference in both cases is the ratio of the coefficients C1, C2 and C3. This ratio is important to determine the linearity of the function. If the fixed costs are relatively high and the water depth is large compared to the total water level difference, then the costs function behaves itself as a linear function. This is demonstrated in Figure 2-7. The graph shows that the least possible number of weirs always results in the lowest construction costs, which is not a very surprising conclusion, however the choice for the number of weirs is usually not based on minimum costs alone; the benefits or completely different criteria have to be considered.

$$C_{constr,n} = nC_1 + nC_2(\frac{H_{tot}^2}{n^2}d + \frac{1}{3}\frac{H_{tot}^3}{n^3} + \frac{H_{tot}}{n}d^2)$$



Influence of C1 and C2 on construction costs





Costs of mitigating measures:

Side effects upstream from the weirs include a higher level of the ground water table, discharge problems of local tributaries or discharge canals as a result of the higher water and groundwater levels, and of course local levee (dike) enforcement. In case of a fixed weir, levee enforcement is necessary after construction of the weir due to the achieved higher water levels in general and particularly during a high runoff period. A movable variable weir allows for a temporary lapse in the weir-effect during this high runoff period. However even for a movable weir, locally the water levels will be higher due to the smaller cross section at the weir. Hence for both types of weir an enforcement of the levees is needed. For the cross section of a river with forelands and summer levees, it is usually undesirable to have submerged forelands during low runoff periods. In case of more permanent submerged forelands the summer levees would have to be enforced.

Costs of mitigating measures can be approximated by a second order polynomial equation. The construction costs of the levees and the costs of the draining are directly proportional to H^2 . The costs for the levee enforcement, more specific for bed or bank protection, are usually proportional to H.

A reasonably well expression for the costs C_{miti} would be:

$$C_{miti} = nC_3 + \frac{l}{n}\frac{1}{6}\left(C_4H^2 + C_5H\right) = C_4\frac{H_{tot}^2}{n^2} + C_5\frac{H_{tot}}{n}$$

Where: C_4 , C_5 = coefficients determined by extensive research into the possible damages and measures

Besides the costs of additional civil engineering works, one must also take into consideration the full costs of changes or damage to the ecologic system. By constructing the weir, the existing food cycle of the surrounding waterway will be disturbed. Small animal species that can be counted as prey for larger fish will be hampered in their descent of the river. This would lead to a disaster for the local fishing industry, which can be translated fairly direct into financial costs. Alternatively the extra costs for provision and maintaining fish migration facilities have to be taken into account.



The graph in Figure 2-9 illustrates the development of mitigation costs in time.

The following costs will be included and described as operational costs, which is not completely correct; it would be better to consider them as costs related to the operational stage of the project:

- Operational costs for every lock and weir complex
- Maintenance costs
- Delay costs ship navigation

Obviously the operational costs are not limited to the items previously mentioned.

Operational costs

The operational costs can be assumed to be proportional to the number of weirs. One should think about the cost of operating of the weir and the locks (if required), electricity and other utilities for lighting, daily maintenance etc.. In general manpower or labour costs are the major to be considered.

Maintenance costs

The maintenance costs per annum of a hydraulic structure are dependent on the nature of the structure and the used materials. Usually the maintenance costs are a percentage of the construction costs, on average 1% per year. For a first estimate of the number of weirs to be built, this percentage can be used for the determination of the costs.

The total operational and maintenance costs can be summarised as follows:

$$C_{O\&M} = cwf\left(\gamma n + \delta C_{constr,n}\right)$$

Where:

γ	=	annual operating costs of a weir
cwf	=	discount factor
δ	=	roughly 1%
C _{constr}	<i>n</i> =	total construction costs

The nearly linear cost function for operation and maintenance is demonstrated by means of a graph in the figure below.



Figure 2-10 Operation and maintenance costs in time
Delay costs ship navigation

The cost of shipping delay is caused by the time it takes to pass the weirs or adjacent locks. Depending on the intensity of ship traffic and the capacity of the weir or lock a queue may develop, resulting in ship waiting time. Obviously, time is also consumed by the lock process; which includes the time needed to open and shut the gates of the lock, the time required for adjusting the water level in the lock chamber, the time needed for the ships to sail in and out of the lock and last not but not least, the number and size of the ships per lock cycle. Simulation techniques are used to estimate the delay time for the (overall) shipping process and in particular for the lock navigation process. Using the delay costs per unit of time, the number of ships etc. one arrives at the (total) delay costs. A considerable number of simulations will be made to vary the number of weirs with varying water level differences to estimate the delay the ships will endure. Meaningful use of the simulation is a job on itself and, unfortunately, often simulation is not possible in the early stages of a project because the necessary data is not available, or there may not be enough budget.

For the purpose of rough cost calculation, needed to decide on the number of required weirs, a simple method of approximation can be used. It is assumed that the delay time t_o for ships is divided in a waiting period t_w (the ship in the queue), and a lock period t_s (the time to pass the lock).

$$t_0 = t_w + t_s$$

Where: t_w =queue time; dependent on shipping intensity and lock capacity t_s =lock cycle; the (average) time the average ship needs to pass the lock

If there is a stretch of river in which there is more than one sluice, then the ideal situation would be that; for the first sluice there is a queue and lock cycle time and that for each sluice after that only lock cycle time. In this scenario the shipping traffic is running optimally and the shipping speed is equal for all ships. At the entrance of the first sluice the shipping traffic is optimised.

De lock cycle time can be approximated with:

$$t_s = \alpha + \beta \sqrt{H}$$

Where:

α	=	the time for opening and closing of the lock gates and the time required
		(by ships) for entering and leaving the lock chamber
β	=	coefficient, dependent on the dimensions of the lock chamber and the

capacity of the (water) filling and emptying system

The total delay for a ship:

$$t_0 = t_w + nt_s = t_w + n\alpha + n\beta\sqrt{H} = t_w + n\alpha + n\beta\sqrt{\frac{H_{tot}}{n}} = t_w + n\alpha + \beta\sqrt{nH_{tot}}$$

If per year there are N ships that pass through the weirs and the costs for the delay per ship on average is equal to P Euros per hour, see Table 2-5 below for information on inland navigation costs, then the total costs per annum:

$$C_j = PN(t_w + n\alpha + \beta(n\sqrt{H_{tot}}))$$

If a comparison is to be made between the construction costs and the total ship delay costs $C_{tot,o}$, the net present value of both costs should be used and the lifetime of the weir has to be known. Usually weirs are designed for a lifespan of 50 years.

The costs over the full lifespan n years can be transferred to their net present with the formula below, which takes inflation and interest rates into account:

$$C_{tot,o} = \sum_{j=1}^{T} \frac{100 * C_j}{(100 + r)^j}$$

Where:r=discount rate in % (financial interest rate on the market – inflation)T=lifespan

In the previous formula the costs of ship delay (may) vary from year to year. If the costs per annum remain constant then the net present value of the costs is:

$$C_{tot,o} = \sum_{j=1}^{T} \frac{100 * C_j}{(100+r)^j} = C_j \sum_{j=1}^{T} \frac{100}{(100+r)^j} = C_j cwf$$

Where: *cwf* = discount factor (*Dutch: contante waarde factor*)

With a lifespan of 50 years, 50 discount factors and their sum have to be computed, e.g. by means of a spreadsheet. To avoid this cwf can be approximated by:

$$cwf = \sum_{j=1}^{T} \frac{100}{(100+r)^j} = \frac{100}{r} \left(1 - \frac{100}{(100+r)^T} \right)$$
 for r < 8 this approximation is very inaccurate

The total costs for the delay can now be defined as:

$$C_{delay} = cwf * P * N\left(t_w + n\alpha + \beta\sqrt{nH_{tot}}\right)$$

The delay costs function increases monotonously with *n*. An example of this can be found in Figure 2-11.



Figure 2-11 Delay costs linearly increasing with no. of locks

Stating the obvious: a minimum number of weirs results in minimum ship delay costs.

In Table 2-5 the cost and relative cost of inland waterway vessels can be found. Actually this information is very hard to find if at all; so it should be used wisely.

Relative level of cost, based on price levels in 2005 in the Netherlands,						
Rhine vessel sailing loaded taken as 100% (100%= 200€/hour in 2005)						
(Ref: Rijksw	raterstaat, Inland Navigation Model System,	BVMS)				
Tankers						
CEMT	Ship types	Tonnage	Loaded	Empty	Waiting	
class		(ton)	Sailing	Sailing	at lock	
0	Others	150	24%	24%	21%	
T	Barge	350	33%	32%	27%	
П	Campine Barge	550	40%	39%	32%	
III	Gustav Koenings / Dortmund-Eems	750	49%	46%	38%	
	Length <= 74m	950	57%	54%	44%	
	Length > 74m	1150	66%	62%	51%	
IV	Johan Welker / Rhine-Herne (L <= 86m)	1550	84%	79%	64%	
	Johan Welker / Rhine-Herne (L > 86m)	1950	113%	106%	89%	
Va	Rhine Vessels	2500	157%	148%	127%	
Cargo vesse	els					
0	Others	150	18%	18%	15%	
I	Barge	350	23%	22%	17%	
П	Campine Barge	550	29%	27%	21%	
III	Gustav Koenings / Dortmund-Eems	750	35%	33%	25%	
	Length <= 74m	950	42%	39%	30%	
	Length > 74m	1150	50%	46%	35%	
IV	Johan Welker / Rhine-Herne (L <= 86m)	1550	65%	60%	46%	
	Johan Welker / Rhine-Herne (L > 86m)	1950	79%	73%	56%	
Va	Rhine Vessels	2500	100%	92%	71%	
Container ve	essels					
0	Others	150	17%	17%	15%	
1	Barge	350	23%	22%	18%	
П	Campine Barge	550	29%	28%	22%	
III	Gustav Koenings / Dortmund-Eems	750	37%	34%	28%	
	Length <= 74m	950	45%	42%	34%	
	Length > 74m	1150	55%	51%	42%	
IV	Johan Welker / Rhine-Herne (L <= 86m)	1550	74%	69%	56%	
	Johan Welker / Rhine-Herne (L > 86m)	1950	92%	86%	71%	
Va	Rhine Vessels	2500	119%	111%	92%	

 Table 2-5
 Relative level of cost for vessels, sailing and waiting

Reuse and/or removal costs:

Reuse and/or removal costs are by nature hard to predict when considering a lifespan of the structure in the order of 50 to 100 years, which is not uncommon at all for lock and weir projects. The lifespan that complicates the actual estimate, at the same time makes the cost estimating work easier because the number of years inevitably results in a low discount factor *cwf*.

Reuse costs will be similar to costs that have to be made for any Greenfield or new built project, except for having to consider costs for removal of parts of the existing structure; those costs come on top of the required budget. Off course only the relevant 'to be reused' parts of the structure will be taken into account.

Removal costs can be split in costs for demolishing the structure, or parts of the structure, and costs for disposal of the construction and demolition waste or rubble. It is not uncommon to find there is a supply and demand market for demolition material, especially (crushed) concrete and steel, resulting in negligible disposal costs. On the other hand, take good notice of the fact that the costs for disposal of polluted materials, e.g. polluted sediments or soil, can be considerable.

Total costs depending on number of weirs and water head

The total costs to be considered when one or more weirs have to be constructed and operated can be estimated by taking the sum of all the above mentioned costs. The total costs C_{tot} are:

$$C_{tot} = C_{constr,n} + C_{delay} + C_{miti} + C_{O\&M}$$

= $\left(nC_1 + C_2H_{tot} + \frac{C_3H_{tot}^2}{n}\right)(1 + \delta cwf) + cwf * P * N(t_w + n\alpha + \beta\sqrt{nH_{tot}}) + C_4\frac{H_{tot}^2}{n^2} + C_5\frac{H_{tot}}{n} + cwf\gamma n$

Using this expression for the total costs, one can calculate the cost differences for various numbers of weirs. The optimum number of weirs is the number for which the total costs are lowest.

In the following figures first a graph showing all the contributing cost items for one set of C-coefficients, see Figure 2-12. Then 4 sets of C-coefficients have been used to produce the graph in Figure 2-13; for all variations of C-coefficients an optimum can be seen for 2 weirs.



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3. Overall lock design

3.1 Lock geometry and lock configuration

3.1.1 Typical inland waterway lock lay-out and geometry

After the analysis phase and preparing a comprehensive set of design criteria, the design work gets into the phase of sketching or drawing the first overall concepts of the lock. Below a top view and longitudinal section are presented of a typical inland navigation lock in the Netherlands.



- 5. Lock chamber
- 6. Filling and emptying system
- 7. Cut-off walls and screens (sheetpiles) to prevent piping
- 8. Bottom protection

Figure 3-1 Lay-out of a navigation lock

- 5. Schutkolk
- 6. Vul- en ledigsysteem
- 7. Voorzieningen tegen onderloopsheid en achterloopsheid (kwelschermen)
- 8. Bodembescherming

The waiting berth [1] provides a safe mooring place for vessels that have to wait before being allowed to enter the lock. A guard wall [2] is a navigational aid; facilitating the entrance to the lock and protecting the structure when the ship loses rudder control. Entering locks may be difficult and potentially dangerous due to crosswinds or local currents. For navigation locks in coastal areas a different layout may be more appropriate.

When entering a lock the vessel will pass the (mitre) gates [3], which are built into the lock head [4]. The closed gates and the lock head must be able to retain the head difference over the lock, thus they should be designed to resist the forces caused by hydraulic pressure. Forces on the gates are transferred to the supports, sometimes to the sill, in the lock head and result in a spalling force (*spatkracht*) on the lock head walls and a (horizontal) line load on the sill respectively. During filling or emptying the ships remain in the lock chamber [5].

In general the lock process can be described as follows: The lock chamber is closed off from the outside water by closing the gates [3]. Subsequently the water level inside the chamber must be adjusted to the desired level; assuming the ship sails from left to right in the above figure, the water level must be raised. When locking in the opposite direction the water level in the lock chamber will be lowered. In order to achieve this, a filling and emptying system [6] is required, e.g. ducts in the lock heads [4]. After adjusting the water level to the outside level the gates are opened, the ship sails out and continues its journey. Usually the lock chamber is provided with berthing and mooring furniture, such as fenders, bollards, mooring rings and ladders in recesses *(inkassingen)*. All furniture has to be able to resist (predominantly) horizontal forces due to berthing, mooring and deberthing.

Filling and emptying the lock chamber will cause local currents. These currents, as well as the propeller wash caused by the navigating ships, may result in scour in front of the lock heads (on either side), which in time may compromise the stability of the lock structure. To prevent this bottom protection [8] can be provided.

Typically, water levels will be different at both ends of a lock. This will result in a ground water flow underneath and on both sides of the lock structure. This seepage may lead to movement of soil particles leading to cavities under and alongside the lock. To prevent this, locks must be provided with seepage or cut-off walls and/or cut-off screens (*kwelschermen*), usually made of sheetpiles [7], acting as an impermeable curtain perpendicular to the axis of the lock chamber.

Together with dikes *(US: levees)* and other structures the lock may be part of a water defense system. The upper lock head (at the side of the highest water level) [4a], the upper head, must be able to retain the highest water level. The lock head at the side with the lowest water level, the lower head [4], retains the highest water level in the lock chamber, which is selected by the designer and not necessarily the highest outer water level. In tidal areas lock heads are referred to as outer head (on the sea side) and respectively inner head (on the land side).

As part of the water defense structure the navigation lock must be able to retain water under all circumstances. If for any reason (e.g. a ship runs into a gate) the lock is damaged other measures must allowed for. E.g. locks are often equipped with spare gates, bulkheads or stop logs *(schotbalken)* are available to replace damaged gates and maintain the water retention function.

3.1.2 Special lock geometries and/or lock configurations

The lock chamber *(schutkolk)* of some historic locks is wider than the entrance to the chamber, whilst the axes of the upstream and downstream reaches run parallel; are not in line, see Figure 3-2. Besides the historic bayonet lock in France even a circular lock can be found in Lequetin, which is even worse from a manoeuvring point of view; back then ships were smaller and shipping much slower.





Figure 3-2 Bayonet lock and circular lock

In modern times the ships must enter and leave the lock as soon as possible, time seems to be money, and this has resulted in a more rational layout of the lock. The chamber width is the same as the entrance width in the lock head. A cross section of the lock often has an U-shape, vertical retaining walls and of course the horizontal floor. Together with well positioned guard walls this facilitates navigation because the situation is easily observed and assessed by the master of the ship; a clear situation enables efficient

navigation. A big help is to avoid assembling all sorts of furniture to the lock walls. If bollards, fendering and ladders protrude from the wall they are observed as bigger obstacles than they are because of their actual size; they cloud observation by the master of the ship who reduces speed accordingly. The furniture should be assembled in recesses *(uitsparingen)* in the wall, maintaining a flat face, see Figure 3-3



Figure 3-3 Lock chamber (laid open) (drooggezet)

Depending on the capacity and intensity of a waterway one could consider constructing an extra set of gates to subdivide the lock chamber in segments. In this way the lock cycle can be managed more efficiently. The extra gates are often positioned at 1/3rd of the lock length, see Figure 3-4 (not to scale). Advantages of such lock chamber partitions are:

- Time needed for filling and emptying the lock chamber is being reduced;
- In coastal areas the intrusion of salt water is being reduced;
- In case of damage due to collision the navigation lock is still partially operational.
- A. Storm gates and double sets of mitre gates for double sided water retention
- B. Double sets of mitre gates for double sided water retention
- C. Single set of mitre gates 'adapted' to double sided water retention
- D. Single rotating door 'adapted' to double sided water retention



3.2 Lock Approaches

3.2.1 Lock approach layout

For positioning the lock not only the dimensions of the lock chamber(s) is of importance, but the size and shape of the lock approach areas and the requirements to connect these navigation areas to the waterway as well.

Outer port layouts - Inland navigation

General reference is made to the RVW (2005). The Figures below show the approach area or outer of a lock. Often the adjectives upstream or downstream are added. Some times upper or lower pond is also used as a name or description. Shown in the Figures are the length and widths, if applicable, and their definitions. Subsequently Table 3-1 shows the dimensions of these lengths and widths depending on the inland navigation vessel type (according to the CEMT class).



Figure 3-5

Approach area or outer port of a lock

Table 3-1 shows the dimensions of characteristic lengths and widths for the outer ports for different CEMT classes.

CEMT	В	B _{ch}	S	Br	L _{lb} /L _{ch}
Class					
	[m]	[m]	[m]	[m]	[m]
1	5.1	6.0	3.0	5.0	1.0 – 1.2
П	6.6	7.5	3.5	6.0	1.0 – 1.2
111	8.2	9.0	4.5	7.5	1.0 – 1.2
IV	9.5	10.5	5.0	8.5	1.0 – 1.2
Va	11.4	12.5	6.0	10.5	1.0 – 1.2
Vb	11.4	12.5	7.0	11.5	1.0 – 1.2
Notes: - See figures for meaning of the symbols					
- <i>L_{ch}</i> is the length of the lock chamber					

Table 3-1Dimensions of outer ports with
berths on one side

Figure 3-6 shows the layout for the outer port of a lock wit one lock chamber. At the other side the layout would be identical, however, point mirrored.





In waterways serving traffic with an intensity warranting construction of two or more lock chambers, the outer port looks as in Figure 3-7.

Figure 3-7 Outer port for twin lock and berths on both sides

When traffic intensities are too low to construct dedicated lock chambers, not only the lock chamber but the outer port as well, has to be shared by different types of vessels. Figure 3-8 shows a layout for the outer port where some of the berthing structures can be used for both professional and recreational craft and still be kept separated from each other.



When the waterway is a river where the discharge is controlled by a weir, often the lock and weir are positioned adjacent to each other at one location. Multiple functions combined in one complex, resulting in some extra demands to the layout of the lock outer port, or, vice versa, demands to the inlet and outlet of the weir, as schematically shown in Figure 3-9. Care has to be taken that eddies or cross currents are kept away far enough from the areas where manoeuvring is complicated, e.g. close to the lock entrance areas. Approach walls can be used to mitigate the effects of adverse currents.





3.2.2 Alignment requirements with respect to the inland waterway - inland navigation

General references are available in RVW (2005), PIANC InCom WG16 (1996b), PIANC InCom WG20 (1999), and PIANC Marcom WG49 "Horizontal and vertical dimensions of fairways".

Numerous locks are or will be constructed in a curve of a waterway. Therefore the alignment of lock and approaches has to suffice to requirements that apply curves in the waterway. The following applies to a two lane waterway where the maximum current is 0.5 m/s.

The radii of curves in waterways should be chosen above a certain minimum:

- because rudder angle does not translate linearly in ship rotation
- to avoid speed reduction in the curve
- for required course corrections
- the loss of unobstructed view

Minimum radius:	R≥á	4L
Preferred radius:	R = (6L

R = radius of the curve, relative to the axis of the waterway

L = length of the governing ship

Sailing through a curve the ship needs extra space, hence a width allowance ΔB should be added to the lane width as follows:

 $\Delta B = 0.75 L^2/R$

However, the width allowance can be reduced for $\beta \le 30^{\circ}$, where β is the angle between the axes of the straight stretches adjacent to the curve.

 $\Delta B = 0.75 \text{ L}^2/\text{R *}\beta/30 \qquad \text{for } \beta \leq 30^\circ$

For R>10L, in that case β <20°, no width allowance is required, the curve is already sufficiently smooth or wide enough.

Preferably the extra width in the curve will be provided in the inside curve. If this is not possible the outside can be used or distribution over the in- and outside is possible.



Right before and after the curve a straight channel stretch of 1.5L should be available. Care has to be taken of a smooth transition from the wide curve to the narrower channel or river section. An angle α is proposed to define start and end of the transition, see Figure 3-10.

Figure 3-10 Alignment and width corrections

Taking care of a clear unobstructed view in a curve, air draught of the vessel, prevailing water levels and level of the bank, including buildings, come into play. Over a length of 5L the view has to be unobstructed, see Figure 3-11. Visibility requirements for junctions of waterways are shown in Figure 3-12.



3.2.3 Location of the lock and construction

Navigation locks are well used in the Netherlands since this is the most economic way to transfer ships from one water level to the other for small differences in water level (< 10-15m). Since navigation locks are used in case a ship has to pass a permanent water retention structure or a weir, logical locations to find a navigation lock are:

- In a river next to a weir.
- At the start and/or end of a waterway with a controlled water level.
- In coastal areas, at a passage of a primary water retention structure.



Figure 3-13 Construction of lock-weir complex Hagestein, river Lek the Netherlands

3.3 Horizontal dimensions lock

Considering ship dimensions, there is a difference in the information available between inland and maritime navigation. Generally the classification of ships in inland navigation seems to have progressed more than classification of maritime vessels.

a) Inland navigation

The dimensions of a lock chamber in Europe are shown in Table 3-2 (RVW, 2005). The dimensions are based on the CEMT table of 1992, for all waterways west of the river Elbe - Germany, and are applicable for the so called minimum lock, which is a lock operating in a traffic situation where less than 10 000 inland navigation vessels, either in the up- or downstream direction, are locked every year.

CEMT classes	Vessel size		Lock chamber dimensions (*)			
	length [m]	beam [m]	Draught [m]	length ¹ [m]	width ² [m]	sill depth ³ [m]
Ι	38.5	5.05	1.80-2.20	43	6.0	2.8
=	50-55	6.60	2.50	60	7.5	3.1
	67-80	8.20	2.50	90	9.0	3.1
IV	80-85	9.50	2.50-2.80	95	10.5	3.5
Va	95-110	11.40	2.50-4.50	125	12.5	4.2
Vb	172-185	11.40	2.50-4.50	210	12.5	4.7

1. The length between the lock heads

2. The clear width between the lock chamber walls or protrusion from the wall

3. The sill depth takes into account the draught of the vessel, squat and keel clearance. However, no allowance has been included for translation waves

4. (*) The lock chamber dimensions do not take into account that the governing ship might be entering the lock chamber with tug assistance.

 Table 3-2
 Dimensions lock chamber for inland navigation depending on CEMT classes

b) Maritime navigation

Generally the variety in ships and ship sizes passing a maritime navigation lock is quite large. This may explain why a convenient table with lock dimensions related to maritime vessels can not be found. Some guidance on the dimensions of ships can be found in EAU 2004 (Recommendations on Waterfront Structures Harbours and Waterways); PIANC MarCom WG48 Guidelines for Berthing Structures; Thoresen C. (2003); and in ROM3.1-99, 2000 (Recommendations for the design of the maritime configuration of ports, approach channels and harbour basins).

Usually the nautical conditions in coastal areas are more adverse than in inland regions and the shape of maritime vessels is prone to a larger impact of those conditions on manoeuvrability. The faster the vessels are able to manoeuvre in and out of the lock (chamber), the higher the traffic capacity. It all adds to the recommendation to use simulation to determine the size of the lock chamber for maritime locks.

To start real or fast-time simulations to find the required size of the lock chamber a set of initial values for the width and length has to be available. It is advised to do the following:

- Define the governing or design ship that should be able to pass the lock
- Assume a minimum lock chamber suffices from a traffic capacity point of view
- The length of the chamber is approximately 1.1 times the length of the design vessel.
- The width is 1.25 times the width of the design vessel.

The simulation work will be continued to check the assumption made. A variety of vessels with different transit times has to be used to find out whether or not the capacity of the lock is sufficient.

In case the lock chamber is designed to accommodate 2 vessels, either in length or in width direction, space for manoeuvring will be added only once, since the vessels enter the lock one by one. Hence, the length is:

$$L_{chamber} = L + (L \times 1.1).$$

and similarly:

$$B_{chamber} = B + (B \times 1.25)$$

An upper boundary for the width of a lock chamber, assuming the ship is navigating on its own propeller would be:

 $B_{\text{chamber}} = 2 \times B_{\text{vessel}}$.

The above initial lock chamber dimensions do not take into account that the governing ship might be entering the lock chamber with tug assistance.

When tugs or other active means, other than the vessels own rudder and propellers, are used to control the course of the vessel the required lateral clearance will be smaller than without assistance. The design vessel for the new Panama lock complexes is a 12,000 TEU containership with a beam of 48.8 m. The specified width between the lock chamber walls is 55 m, the minimum clearance on each side of the chamber is 0.3 m. Hence, the usable width is specified at 54.4 m. If the useable width were to be used completely the lock width over beam ratio would be 1.01. The useable lock width over design vessel's beam ratio is 1.15.

Obviously it is not recommended to use the ratio of 1.01 to start simulations for new lock design because it applies to exceptional events. For the more frequent event of the design vessel navigating the lock, the ratio of 1.15 can be considered as the minimum to start.

The depth of the sill should be determined using the maximum draught of the design ship with regard to the selected minimum water level in the lock chamber, and including at least 1 meter nett keel clearance. Selection of the minimum waterlevel in the lock chamber is governed by operational or availability criteria, and depends on waterlevel variations, and related frequency of occurrence on both sides of the lock. Note

that the stern of the vessels sinks deeper into the water when the engine is put to work again. Sometimes it may be necessary to construct the sill at a lower level considering the volume of water that has to be discharged and the ratio between the cross sectional area of the ship and the water area in the locks cross section, when the ship enters the lock chamber. Fresh versus salt water phenomena may add to this requirement.

c) Recreational navigation:

Depending on the traffic intensity recreational craft will have its own dedicated lock chamber or has to make use of the spare capacity of the lock chamber dedicated to commercial navigation.

Considering the construction costs of a lock, a dedicated lock for recreational craft will not be a minimum lock chamber, i.e. for use of one yacht only. To determine the size of the chamber it is advised to simulate the traffic and lock situation.

Figure 3-14 shows some configurations for locking both commercial and recreational craft in one lock chamber at the same time. Commercial vessels should get the highest priority, allowing these ships to sail into and sail out of the lock before recreational craft. A complicating factor might be the difference in allowable mooring forces on the different types of vessel due to emptying or filling the lock.



Figure 3-14 Lock chamber for simultaneous use by two types of traffic

When two different types of traffic have to be locked in the same chamber it is advised to determine the size of the chamber by means of simulation.

3.4 Vertical dimensions of the lock

In order to deal with the vertical dimensions of the lock the following subjects have to be discussed:

- 1. Top of Structure and Bottom of Structure
- 2. Ship's draught and keel clearance
- 3. Water levels

Although written in above sequence in this section the subjects are in fact completely cross connected and both in reading this text and later during design in reality an iterative mind setting or approach will be best for understanding and future use.

3.4.1 Top of structure (ToS) and Bottom of structure (BoS)

Top of Structure:

Since the lock often is a part of a flood defense system, but always part of a water retaining system, one of the components along with dikes or levees and other 'hard' structures, Top of Structure (ToS) is an important issue to be considered. Figure 3-15 shows contributors, definitions and some comments, to the construction height of structures like dikes and breakwaters. Note this figure is the same as in the 'Dutch' CT2320 lecture notes, however, translated for convenience sake. For earth structures generally there will be a significant difference between the construction height and Top of Structure level because the latter has to be maintained throughout the years in spite of e.g. settlement.



Earth Structures (dike/levee/breakwater) - Top of Structure & Construction height

For structures like locks and quays etc., apart from agreed and accepted construction tolerances, generally construction height is equal to ToS, see Figure 3-16 below. Indeed, contrary to earth structures, for this type of structures 'surplus height' as a contributor to ToS generally could be neglected, although it could be considered to take into account construction tolerances here.



Structures (lock/quay/barrier) - Top of Structure = Construction height

Moveable lock elements - the gates:

The gates of the lock are moveable, contrary to e.g. other concrete lock structures or dikes. In closed position the same ToS requirements apply to top of gate. Generally bottom of gate will be the same level as top of the sill or top of the lock head floor, when closed; detailing issues related to seals taking care of water tightness may result in some deviation to the previous. For some gate types, e.g. a lift gate or radial gate, bottom of gate will have to be determined for the gate in opened position, considering the required air draught and the design (high) water levels. Obviously, in opened position, top of gate and top of the gate support structure are 'practically' related to bottom of gate in the opened position, not to flood defense requirements.

Gates being frequently moved, and ships frequently manoeuvring in their vicinity, introduces the risk of gate failure resulting in failure of the flood defense or water retaining system. Too increase safety of the whole system, often a double gate will be provided, or at least provisions will be made for stop logs. The double gate could be positioned in the same (outer) lock head, or the gate in the other (inner) lock head could be considered as the back-up gate. In the latter situation the whole structure between inner and outer gate has to at the same required ToS level.

There are other reasons to provide the lock with more than one gate in the lock heads, or to provide intermediate gates, because a good intermediate gate arrangement has the following advantages:

- It takes less time to handle a small vessel
- Water will be saved in periods of draught
- The amount of salt water penetrating into fresh water channels is reduced
- It permits gates to be repaired or removed without shutting the lock down for traffic

See following Chapters and the Appendix for further elaboration.

In case the head difference between the upper head and the lower head exceeds 2 meters (A>2), see Figure 3-17, it is usually considered better, from a construction costs point of view, to design the upper lock head higher than the lock chamber and the lower lock head. In case the difference water between the lower canal level (LCL) and the lower lock chamber level is larger than 2 meters (A>2) it is usually better

to design a sill (*drempel*) at the lower lock head. In case the level difference is smaller than 2 meters (A<meters) usually the most economic way is to construct the lock in the same plane by constructing the lower lock head higher or by removing the sill.

For navigation lock in a conventional waterway the height of the chamber wall and the gate (4) is 1 meter above the highest water level (HLL) in the chamber. The gates positioned at (3) and (5), see Figure 3-17, have the top of the gate between 0.5 to 1 meter above Highest Canal Level (HCL), pending on the waves to be expected. For reasons of practicality these doors (3&5) are at times designed and constructed to match the height of doors at position (2) and (4). In this way only 2 different sizes of spare gates are needed.



Figure 3-17 Characteristic water levels combined with direction mitre gates; Construction level differences

Bottom of Structure:

In the above it was described how to determine ToS and it was mentioned that bottom of gate is generally at the same level as top of sill or top of lock head floor. To determine top of sill or top of floor, the ship's draught, the keel clearance and water levels have to be taken into consideration, see section 3.4.2. Only after knowing top of floor a start can be made to derive Bottom of Structure (BoS). If strength and stiffness is the only requirement to the sill or floor structural calculations will result in the required floor thickness and finally simply subtracting this thickness from top of floor will lead to BoS. Other functional requirements, e.g. an emptying and filling system using culverts in the floor of the lock, may increase the thickness of the floor, see Chapter 5.

Be aware that the deeper the level of BoS, generally, the higher the cost of lock construction.

3.4.2 Ship's draught and keel clearance

There is a minimum water depth required to enable ships to manoeuvre through the approach areas and the lock itself. Table 3-2 showed the water depth to be provided for inland navigation vessels. For maritime vessels and recreational craft a very approximate rule of thumb for the water depth D_{water} would be:

```
D_{water} = D \times 1.15 + 0.5 \text{ m} where: D = ship's loaded draught
```

When designing for recreational (sailing) craft the above formula reduces to: $D_{water} = D + 0.5 \text{ m}.$

This because the shape of the keel of recreational craft, in cross section and longitudinal section is much more curved than inland or maritime vessels, therefore, leaves more water area available for return lows that would result in sinkage of the ship.



Figure 3-18 Minimum waterdepth, draught and keel clearance

The rule of thumb for the water depth is quite an approximation; often it will be necessary to produce a better estimate for the required water depth. The minimum depth in a lock chamber is determined by the following components, see Figure 3-18 as well:

- 1. Margin for transitional waves, seiches, etcetera (wave phenomena which can cause fluctuations)
- 2. Vessel draught
- 3. Squat (*inzinken*), which includes sinkage and trim of the vessel. A moving vessel increases its draught due to return flow phenomena.
- 4. Hydraulic margin (influenced by the relation between the cross sectional area of the vessel versus the lock chamber)
- 5. Safety margin (to compensate for margins in the previous components)

Please refer to e.g. the CT4330 lecture notes for further elaboration of these minimum water depth components.

3.4.3 Water Levels

Due attention should be given to selection of the water levels in the lock. Extreme high and low water levels, i.e. water levels occurring at low frequencies, generally are or should be related to the water retention function of the lock. In design stages where strength and stability of the lock structures is checked, the extreme water levels are translated in loads used for the Ultimate Limit State (ULS). Frequently occurring water levels are important for the ship navigation function of the lock; these water levels determine to a large extend the operational capacity of the lock. From strength and stiffness point of view, frequent occurrence translates into loads and load combinations of the Serviceability Limit State. See previous sections for descriptions of water retention function and lock operation (capacity) issues.

In Figure 3-19 typical water levels situation for a lock in coastal area are shown; it is left to the reader to make a similar schematisation for a lock in an inland waterway.



Water levels in the outer ports

Looking at a lock in coastal area, attention will be focused on sea levels first because (in general) the variation in water levels is larger than those of the inland waterway, secondly there is an obvious water retention aspect, the lock is part of the flood defense system.

Flood defense will require that the lock retains water up to a defined extreme high water level, here referred to as the Storm Surge Level (SSL) but it could also be described as 'Design Water Level' (DWL). Although it could be allowed that waves, generated by a storm surge, overtop the structure, obviously, the structural integrity shall and will not be threatened. Often extra height of the structure will be added to prevent wave overtopping. This will depend on the requirements for the amount of water allowed to overtop the structure in relation to the quality of the water (salt or fresh water). Both the SSL and the wave should be related to a probability of exceedance (for instance 1:10.000 for a storm surge).

Generally High High Water Spring (HHWS) and Low Low Water Spring (LLWS) are known all over the world; often the values can be derived from tide tables for a given location. Using wind speed and fetch *(strijklengte)*, both wind set-up and wave height could be calculated to determine SSL or DWL, or at least a fair approximation, and similarly the extreme lower water levels are estimated. Both the wind speed and fetch depend on the wind direction and could be related to a probability of exceedance (for instance 1:10.000 for a storm surge). For many locations waterlevels and the related probability of exceedance are known and e.g. presented in a graph, see Figure 3-20.



When designing the lower look head the highest and lowest water lovel in the concl. (LCL)

When designing the lower lock head the highest and lowest water level in the canal (HCL, LCL) are the dominating factors.

In the absence of any wave data a minimum wave height of 1 meter should be taken into account for coastal areas and 0.5 meter for inland navigation areas for the outer lock head. Note that incident waves may be reflected, which increases the local wave height and the original Still Water Level (SWL), which was equal to SSL.

Water levels in the lock chamber

Determining the extreme water levels to be maintained in the lock chamber, a balance has to be found, between construction costs and safety, but it may be more important to consider the effect of water levels on the emptying and filling time, hence on the capacity, of the lock.

Highest Lock Level (HLL) is chosen somewhere in the middle of the water level in the outer approach area (SSL or DWL) and the water level in the inner approach area (HCL). The lowest Lock Level (LLL) is in between the lowest water level in the outer approach area and lowest mean water level of the lower canal. As a rule of thumb a navigation lock is designed to retain a water level that exceeds once a year the highest lock level and the lowest lock level. Obviously the mentioned frequency depends on the importance of the waterway and the traffic on the waterway.

4. Structural elements of a navigation lock

In this chapter sections on the following subjects have been included:

- 1. Filling and emptying systems (F/E)
- 2. Lock gates

- 4. Lock chamber
- 3. Lock head

- 5. Seepage cut-off screens
- 6. Approach structures and berthing facilities

It might seem strange to begin a chapter on structural elements with a section on F/E systems, but filling and emptying (F/E) the lock chamber is key to the efficient operation of the navigation lock. In various ways a number of the structures to be designed and constructed, if not all, are influenced by the F/E system of the lock.

4.1 Filling and Emptying Systems (F/E)

This section starts with a general description of F/E systems, including some terminology, fluid mechanics or hydraulics, water saving basins, and selection criteria. In the next two sections two most important selection criteria for F/E systems, the time required for F/E and hawser forces (*tros krachten*) will be discussed quite thoroughly. Finally some specific subjects regarding F/E, viz. water saving basins, salt water intrusion, ice control, and large water head difference phenomena, will be mentioned and provided with some references for further study.

4.1.1 Typical F/E alternatives

There are two main systems for filling and emptying the lock, viz. the "through the heads system" and the "longitudinal culverts system", the first for low lifts, the other for high lifts. Some other main features and/or characteristics of these F/E systems are mentioned in Table 4-1.

To distinguish the heads system from the longitudinal culvert system it is best to keep in mind that filling or emptying through the heads results in a very turbulent water zone near the lock head; the disturbance will spread through the whole lock chamber, see Figure 4-1. The intended use of a longitudinal culvert system is to distribute the in or outflow over the whole length of the lock chamber, therefore substantially reducing water turbulence and avoiding or reducing water level differences over the length of the lock chamber. Quiet water in the lock chamber is important for the safe mooring of ships being locked.

Through the heads Through the heads / longitudinal culverts		Through longitudinal culverts		
Lift height H<10 m	Lift height 10 <h<15 h="" height="" lift="" m="">15 m</h<15>			
Gate valves, short culverts	Short culverts, stilling chamber; or Longitudinal culverts pressure chamber)			
"Realtively" fast, simple, cheap <===> Relatively" slow, complicated, expensive				

 Table 4-1
 Main characteristics of F/E systems



Figure 4-1 Turbulent water, due to a water jet, spreading through lock chamber



In Figure 4-2, the pencil thin lines on the left are transforming into a gate with dimensions; adding more thickness or, extruding the gate and the culvert or closed conduit on the right develops.

From a hydraulic point of view, head filling, usually by means of valves in the gates, is characterized as 'open channel flow' (stroming in open waterlopen), whilst the flow of water through culverts is typically known as 'closed conduit flow' (stroming in gesloten waterlopen of leidingen). In case of flow through a gate, the available potential energy, which is a function of \sqrt{mgh} , is transformed in kinetic energy, a function of $\frac{1}{2mv}^2$, without very significant losses of energy. Regarding

closed conduit flow, the loss of energy, e.g. due to wall friction or due to elbows or bends in the culvert, will be considerable and the effect on discharge has to be taken into account.

Translation waves not only run through the lock chamber during the filling process, but a negative translation wave runs through the outer port of the lock and upstream reach as well. Vice versa, a negative wave travels through the lock chamber when it is being emptied and a positive wave moves through the downstream approach and further. In the lock chamber, generally with vertical concrete walls, the wave will be reflected (more than once), whilst in the approach area, due to spread and e.g. bottom friction the wave energy will usually dissipate quickly. Safe mooring of the vessels and safety of lock and approach structures will be matter of concern. See section 4.1.3 for further discussion.

Head filling

In these lecture notes 'head filling' will be used for systems where the lock is filled, or emptied near or at the head. Often the closing element, the gate, has valves that are used to let the water flow in or out, see Figure 4-3. The valves are positioned at the bottom of the mitre gate to prevent the water jetting through the valve from hitting the ships directly at the bow or stern. The latter generally cannot be prevented if a sill is constructed at the head. In that case short culvert systems, just going around the gate and/or a part of the lock head are useful, see Figure 4-4. Short culverts around or through the head will be considered as a head filling system.

Head filling generally is used for locks with a lift of up to 6 meter. Using it for larger lifts results in a very turbulent water near the gate and lock head, due to the large water head difference, creating a zone where ships should not be moored. This zone decreases the effective lock chamber length, or vice versa increases the total length of the lock. Of course the head filling system can be designed to minimize the turbulent water



Figure 4-3 Head filling: gate with valves and breaker plates

zone, for instance by using short culverts and dissipation or stilling chambers *(woelkelders)*, see Figure 4-4 and Figure 4-5. This may increase the use of the system even to a lift or water head of 15 m. In this

range the combination of short culvert and stilling chambers has to compete with the longitudinal culvert system.



Figure 4-4 Head filling: a pipe culvert around the lock head



If the requirements for F/E time and safe mooring are met, to make a choice between filling through the gate, or by means of a short culvert system in or around the lock head with or without stilling chamber(s), it will be best to look at the simplicity or complexity of providing valves in the gate versus culverts, possibly with stilling chambers, in the lock head and the resulting costs.

A most simple head filling system has to be mentioned: this would be lifting or opening the whole gate. Nothing extra has to be designed or constructed to the gate, which is good for costs, and the lock would be filled or emptied really fast by using the whole gate. The downside to this solution is obvious; all the water in the lock would be in turmoil and whatever ship being moored, it would be tossed around in the lock and there would be a great risk it would break its hawsers.

Stilling chambers:

The intended use of stilling chambers is to increase the lift of head or longitudinal culvert filling systems. Combined with a longitudinal culvert the stilling chamber is often positioned under the lock chamber floor, generally covering a large part of the horizontal area. In this low level position, water pressures are relatively high, therefore the name 'pressure' chamber is also used.

To enlarge the lift range the stilling chamber has to dissipate as much of turbulent water energy as possible, and has to take care of a distributed inflow of the water in the lock chamber. For that reason, often flow breaking elements, such as concrete colums or beams, are constructed into the stilling chamber.

Very turbulent water, and high water pressure, create the conditions for water hammer and/or cavitation. Both these phenomena are very sensitive to the dimensions and shape of the lock culverts and the stilling chamber in particular. It is advisable to involve hydraulic experts in the (preliminary) design and, if possible, to base the design on either existing stilling chambers or physical model investigation.

Longitudinal culverts:

For lock with lifts larger than 6 meter, going up to 25 meters or even over 25m, longitudinal culvert systems are used. The principle is to distribute the inflow or discharge of water over (many) more positions or locations in the lock chamber to avoid water zones too turbulent to moor ships. Many in or outflow positions along the whole lock chamber length are used, hence the name longitudinal culverts.



In-chamber Longitudinal Culvert System (ILCS), Marmet Lock (USA):

The ILCS consists of two parallel rectangular culverts; each culvert has two sections with manifolds at each side of the culvert. A wall baffle is mounted on the lock wall opposing these manifolds to dissipate the energy from the current. The ILCS results in an asymmetric distribution of flow along the length of the lock chamber.

The ILCS was developed to reduce the construction costs associated with large concrete gravity walls. Because the longitudinal culverts are located within the lock chamber, on top of the bottom floor, the lock walls can be thinner, which results in a less costly wall design. Assuming the culverts have to be 'outside' the navigable cross section, putting the culvert on top of the bottom has to result in a lower level of the lock chamber floor. Hence, the cost savings on the walls are reduced by deeper construction of the lock bottom. Obviously there is the risk that the exits/inlets of the culverts are damaged, e.g. by debris in the water or fallen anchors. Presumably it will take some extra precautions to provide sufficient robustness of these exits/inlets; this will further reduce the costs savings.

Lock dimensions: 360 ft long x 56 ft wide; 24 ft lift (100 m x 17 m; 7.3m)



Figure 4-8

Lifting a 6 barge push convoy





© Hydraulic Engineering Faculty of Civil Engineering Delft University of Technology In literature all sorts of similar or more specific names or descriptions for longitudinal culvert systems can be found. Look for instance in Table 2-1 for the names and descriptions of simple and complicated longitudinal culvert systems.

	Complexity	Hydraulic systems	PIANC classification [2009]
1	Simple longitudinal culvert systems	 Wall culvert and side ports system In chamber culvert system 	Low lift height: up to 10 m
2	More complex longitudinal culvert systems	 Wall culvert bottom lateral system Wall culvert bottom longitudinal system Longitudinal culverts under the lock floor 	Intermediate lift height: between 10 and 15 m
3	Very complex longitudinal culvert systems	Dynamically balanced lock filling systemPressure chamber under the floor	High lift height: above 15 m

 Table 4-2
 Names and/or descriptions for longitudinal culvert systems PIANC (2009)

Obviously longitudinal culvert systems generally will be substantially longer than 'short' ones used for head filling, thus, the literal length of the culvert could be a distinctive criterion. Remember this: rather than looking at the length of the culverts used, it is better to look at the position and/or concentration of the inflow and/or outflow ports in the lock chamber to characterize the F/E system.

The water may enter or leave the lock chamber in horizontal direction along or perpendicular to the lock axis, vertical or in any angle or orientation that is deemed necessary. The culverts can be positioned in or adjacent to the lock walls, in, below or above the lock chamber floor. For al these situations the culverts may be separated or integrated into the structure, see Figure 4-9 trough Figure 4-8.



Figure 4-9 F/E system with culverts in the lock chamber wall; culvert exits near or on the lock chamber bottom

Separate culvert systems have to constructed in any way in case the intake or discharge points of the water are at some distance from the lock chamber, see for instance the plan view in Figure 4-10. Positioning the intake or outlet a bit further away could be necessary due to the effects of the negative (inlet) or positive (outlet) translation waves in the approach areas of the lock.

Separation or integration of the culverts and either the wall or bottom of the lock chamber depends, on one hand, on the required size of culvert, on the other hand, on the available area or space in the lock wall or bottom. The structural integrity of these elements should not be compromised. Figure 4-6 and substory, see previous page, describe a culvert solution for an 'out of' the wall and 'out of' the bottom culvert structure.





A 3rd F/E system:

In the above description an underlying assumption was the use of natural water head. In some situations management of either the quantity, or the quality of water at the upstream side of the lock, results in alternative F/E systems. Gates and culverts may do part of the work, but basically pumps are used to overrule the natural water head or water level difference.

Water saving basins or side ponds

Quite frequently the design and construction of water saving basins is required. In Panama the new locks are 61 m wide and 457 m long; they have a lift of say 13 to 17 m. Water of lake Gatun, which is at an higher altitude, gets lost due to the locking process. In a lock chamber being filled or emptied, about 400 000 m³ of water flows in or out (160 Olympic swimming pools of 2m depth). It is important to use the lake's water sustainable and economically, hence, to save as much water during locking. Figure 4-11 shows a worked open



© Hydraulic Engineering Faculty of Civil Engineering Delft University of Technology illustration of the water saving basins and their emptying and filling system. Safety of mooring is not an issue in the basins but it is in the lock chamber. Thus the lock chamber has more and better distributed inlet/exit ports than the water basins. Look at the size, shape and position of the culverts between basin and chamber; it all may be described by the phrase large underground tunneling work. It is not hard to imagine that the water saving basins and underground culverts significantly increase the cost of a lock complex (in the order of 2-4 times the costs).

Selection of a F/E system

Given the importance of the F/E system for the design and construction of the whole lock it is best to carefully select the system; if need be, use a multi criteria analysis or evaluation. Looking at the description of the F/E systems in the above the following criteria for emptying and filling a lock can be defined:

- time/duration: the time required for emptying or filling the lock linearly adds to the lock cycle, thus to the capacity of the lock, see Chapter 2. In the next section formulas for computation of the F/E time will be derived. In general F/E has to take as less time as possible, under the condition of the next criterion on hawser forces (*tros krachten*);
- 2. hawser forces of the vessels moored in the lock chamber have to remain well below preset limits, depending on the traffic type and/or chosen design vessels. Safety for the ship and the lock (structure) is the background for this criterion; ships should not break loose from their mooring lines. See section 4.1.3 for calculation of the forces and limits to hawser forces. Do mind that the water discharging from the lock chamber should not create mooring problems for vessels laying by or moored in the approach area of the lock;
- 3. water management usually requires very specific measures determined by the situation at hand and it will result in special criteria. It would either be not very meaningful, or simply impossible to provide 'general' criteria in these lecture notes;
- 4. operation and maintenance robustness;
- 5. costs
- 6. etc.

The issues of point 1 and 2 in the above will be dealt with in Sections 4.1.2 and 4.1.3 respectively; they set forth the basic requirements a filling system has to suffice to in a given situation. If more than one alternative suffices and if budget permits, the issues under point 3 to 6 come into play and a multi criteria analysis, to find more selection criteria, followed by the multi criteria evaluation, will result in selection of the F/E system to be further elaborated.

In PIANC report, Innovations in navigation lock design (2009), the M factor method, from China, for selection of the type of filling system, is introduced to the greater global public. M is defined as follows:

$$M = \frac{T}{\sqrt{H}}$$

where T is the time to fill the chamber in [min] and H is the lift height in [m]

The values for M and choices for the F/E system on the right have been defined for a certain set of maximum hawser forces used in China.

(Only for conceptual design it may be assumed, there is not a significant difference between the maxima in China or elsewhere). For Chinese inland waterways:

3.5 <m 2.5<m<3.5< th=""><th>through heads system through heads system or simple longitudinal culvert system</th></m<3.5<></m 	through heads system through heads system or simple longitudinal culvert system
2.4 <m 1.8<m<2.4< td=""><td>simple longitudinal culvert system a more complex longitudinal culvert system</td></m<2.4<></m 	simple longitudinal culvert system a more complex longitudinal culvert system
M<1.8	a rather/very complex (advanced) longitudinal culvert system

4.1.2 Time required for F/E

Filling or emptying the lock chamber takes time, in fact it is the biggest contributor to the lock cycle time, both for inland and maritime navigation, see Section 2.2 and Table 2-4 in specific. Therefore the time required for F/E is of direct influence on the capacity of the lock, however, and vice versa the requirements set for the lock capacity will determine the time for F/E. Hence, it is necessary to be able to compute the time a certain F/E system needs to do the job.

Head filling

To analyse the process of filling or emptying the lock chamber and finally determining the total time T_{total} required for F/E, it is useful to obtain formulas describing, for instance the discharge and the changing water level. For that purpose several assumptions have to be made:

- Both the upstream and downstream canal are assumed to be infinitely large basins, therefore the average water level remains the same in both canals
- The lock chamber will be filled or emptied through valves in the gate or using a (short) culvert system through or around the lock head; see next assumption
- Inertia of the water mass in the lock chamber, in the (short) culvert system or stilling chamber will be neglected in the following. Friction of the water flow is neglected
- The translation wave resulting from the filling process has a relatively small wave height compared to the considered water depth

Valves in the gate completely open(ed):

From a fluid mechanics point of view, head filling through valves in the gate could be considered as a case of 'submerged' flow *(verdronken uitstroming)*, see Figure 4-12 and lecture notes 'Introduction to Hydraulic Engineering' CT2320. Formula (1) may be used to determine the discharge Q:



Figure 4-12Submerged flow under a gate. Discharge formula

Formula (1) describes a static situation, which disagrees with the 'dynamic' reality of a water level difference z that is changing in time due to filling or emptying the lock chamber. Q and z have to be considered as time dependent variables. The principle of mass conservation can be used to find a relation between the discharge Q(t) and the water level change *dz*. Multiplying the time dependent discharge Q(t) with sufficiently small time periods *dt*, mathematically spoken integrating Q(t) over time, has to equal the volume of water flowing in or out the lock chamber. The same volume is also equal to the horizontal area A of the lock chamber multiplied with the change in water level *dz*.

$$Q(t)^* dt = -A^* dz \qquad \Rightarrow \qquad Q(t) = -A^* \frac{dz}{dt} \tag{2}$$

Rewriting Formula (1) as $Q(t) = m_s f \sqrt{2gz(t)}$, substitution in (2) and rearranging results in:

$$\frac{dz}{\sqrt{z(t)}} = -\frac{m_s f}{A} \sqrt{2g} \, dt \qquad after integration: \qquad \sqrt{z(t)} = -\frac{m_s f}{2A} \sqrt{2g} * t + Const. \tag{3}$$

First the valves in the gate are closed and there is no water flow, then, at t = 0, the valves are opened and discharge starts due to the initial water level difference $z = \Delta H$. (Suppose, for the time being, opening or closing of the valves takes no time at all). Hence, the boundary condition $z=\Delta H$ at t=0 can be defined and used to solve the constant in Formula (3):

For
$$t = 0$$
; $z = \Delta H$ $\Rightarrow \sqrt{z} = -\frac{m_s f}{2A} \sqrt{2g} * t + \sqrt{\Delta H} \Rightarrow$
 $z = \left(\frac{m_s f}{A}\right)^2 2g * \frac{1}{4} t^2 - \frac{m_s f}{A} \sqrt{2g\Delta H} * t + \Delta H$ (4)

Substitute (4) in (2) and find after derivation:

$$Q = -\frac{m_s^2 f^2 g}{A} t + m_s f \sqrt{2g\Delta H}$$
⁽⁵⁾

The total filling time is easily determined, by using Formula (5) and the boundary condition t = 0; $z = \Delta H$ and Q = 0:

$$Q = -\frac{m_s^2 f^2 g}{A} t + m_s f \sqrt{2g\Delta H} = 0 \qquad \Rightarrow \qquad T_{total} = \frac{2A\sqrt{\Delta H}}{m_s f \sqrt{2g}} \qquad (6)$$

With Formulas (5) & (6), and the data below, the graph(s) on the right, for z(t) and Q(t) respectively, have been drawn.

A	=	1080	[m ⁺]
ms	=	0.85	[-]
f	=	5	[m ²]
g	=	9.81	[m²/s]
ΔH	=	4.5	[m]
T _{total}	$=\frac{2A\sqrt{4}}{m_s f_N}$	$\frac{\Delta H}{\sqrt{2g}} = 243 \ s$	

Observe that the discharge Q decreases linearly in time, whilst the water level difference z shows a parabolic decrease.

Since the water level difference does not change a lot in the later stage of the process, z slowly approaches zero,



Figure 4-13 Graph for the decrease in Q and z over time

it is more or less common practice to open the whole gate before t = T_{total} . The criterion for z being acceptably low or not is determined by the sudden water movement in the lock chamber or in the approach area of the lock, hence, by the order of magnitude of ship displacements and hawser forces.

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Including the time required to open the valves in the gate:

In reality gates or valves can not be opened or closed without the passing of some time. The time required for opening the valves will be denoted by t_h . In the previous paragraphs the formulas for Q(t) and z(t) have been derived under the condition of fully opened valves, hence, $t \ge t_h$. Now the development of the discharge and water level difference for $0 \le t \le t_h$, will be determined to find an expression for T_{total} again.

Q and z during the period 0< t \leq t_h:

Assume a linear lifting process of the valves. Therefore a term t/t_h will be added to Formula (1) for the discharge, hence:

$$Q(t) = m_s f \frac{t}{t_h} \sqrt{2gz} \tag{7}$$

Equating the Formulas (2) and (7), followed by some rearrangement makes the left and right hand side of equation (8) ready for integration:

$$m_s f \frac{t}{t_h} \sqrt{2gz} = -A \frac{dz}{dt} \qquad \Rightarrow \qquad \int \frac{m_s f \sqrt{2g}}{At_h} t \, dt = -\int \frac{dz}{\sqrt{z}}$$
(8)

Use the start or boundary condition t = 0, then the water level difference $z = \Delta H$, and find:

$$\frac{m_s f \sqrt{2g}}{At_h} \frac{1}{2} t^2 = -2\sqrt{z} + 2\sqrt{\Delta H} \qquad \Rightarrow \qquad \sqrt{z} = \sqrt{\Delta H} - \frac{m_s f \sqrt{2g}}{4At_h} t^2 \tag{9}$$

Take the square of \sqrt{z} to find z for t \leq t_h:

$$z = \Delta H - \frac{m_s f}{At_h} \sqrt{2g\Delta H} \frac{1}{2} t^2 + \left(\frac{m_s f}{At_h}\right)^2 g \frac{1}{8} t^4$$
(10)

Substitution of (9) in (7) results in Q for $t \le t_h$:

$$Q = m_s f \frac{t}{t_h} \sqrt{2g} \left(\sqrt{\Delta H} - \frac{m_s f \sqrt{2g}}{4At_h} t^2 \right) \qquad \Rightarrow \qquad Q = \frac{m_s f \sqrt{2g\Delta H}}{t_h} t - \frac{m_s^2 f^2 g}{2At_h^2} t^3 \tag{11}$$

In case filling or emptying continues after t = t_h there will be a remaining water level difference ΔH_{rest} and it will take some time to complete the process with the valve(s) completely opened. The total time will be equal to the addition of t_h to $t_{wide open}$. Formula (6) can be used to compute $t_{wide open}$ if the remaining water level difference ΔH_{rest} is known. Use Formula (10) for t = t_h to find the remaining water level difference ΔH_{rest} .

$$\begin{aligned} Formula \ (10) \ for \quad t = t_h: \qquad z(t_h) = \Delta H_{rest} = \Delta H - \frac{m_s f}{A t_h} \sqrt{2g \Delta H} \frac{1}{2} t_h^2 + \left(\frac{m_s f}{A t_h}\right)^2 g \frac{1}{8} t_h^4 \\ \Delta H_{rest} = \left(\sqrt{\Delta H} - \frac{m_s f}{A t_h} \sqrt{2g} \frac{1}{4} t_h^2\right)^2 \quad subst.in(6): \quad t_{wide open} = \frac{2A \sqrt{\Delta H_{rest}}}{m_s f \sqrt{2g}} = \frac{2A}{m_s f \sqrt{2g}} \sqrt{\Delta H_{rest}} \\ t_{wide open} = \frac{2A}{m_s f \sqrt{2g}} \left(\sqrt{\Delta H} - \frac{m_s f}{A t_h} \sqrt{2g} \frac{1}{4} t_h^2\right) = \frac{2A \sqrt{\Delta H}}{m_s f \sqrt{2g}} - \frac{1}{2} t_h \end{aligned}$$

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$$T_{total} = t_h + t_{wide \ open} = \frac{2A\sqrt{\Delta H}}{m_e f \sqrt{2g}} + \frac{1}{2}t_h \tag{12}$$

Do note the effect of more accurately including the lifting period t_h . The total time T_{total} , increases with (only) 50% of the lifting period t_h compared to the period with an immediately wide open valve.

In the above it has been assumed that the lock chamber was not completely filled or emptied when the process of lifting the valve came to a stop. In one of the following paragraphs T_{total} will be determined when filling and emptying does come to a stop before the valve is completely opened, hence $T_{total} < t_h$.

Some observations with regard to the relation between valve opening time th and discharge Q(t)

Consider the following situations:

- 1. Relatively fast opening of the valves; i.e. when the valves are fully opened there is still enough water level difference to result in an increasing discharge. See upper graph.
- Relatively slow opening of the valves; i.e. when the valves are finally fully opened the discharge is already decreasing because the remaining water level difference is small. See lower graph.

The two illustrative graphs in Figure 4-14 are drawn with the same data for A, m_s , f, g and ΔH as in the above.

Note:

The discharge curves should have been cut off beyond t_h larger than 90 and 180 s respectively, since Formula (4) is only valid for $t \leq t_h$.

Maximum discharge occurring during the lifting process:

In situation 1 the maximum discharge Q_{max} is easily found by substituting t_h into (11), which results in:

$$Q_{\max} = m_s f \sqrt{2g\Delta H} - \frac{m_s f^2 g}{2A} t_h \qquad (13)$$





Figure 4-14 Graphs showing discharge Q for different lifting periods

In situation 2 the maximum discharge has to be determined by time derivation of Formula (11) and finding the time t for which the derivative equals 0. At that moment, for that value of t, $Q=Q_{max}$. Substitution of the obtained expression for t into (11) results in Q_{max} for situation 2.

$$\frac{dQ}{dt} = \frac{m_s f \sqrt{2g\Delta H}}{t_h} - 3\frac{m_s^2 f^2 g}{2At_h^2} t^2 = 0 \qquad \Rightarrow \qquad t^2 = \frac{2A\sqrt{2g\Delta H}}{3m_s fg} t_h$$
substitute $t = \sqrt{\frac{4A\sqrt{\Delta H}}{3m_s f_s} t_h}$ in (11) and find:

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$$Q_{\max} = \frac{m_s f \sqrt{2g\Delta H}}{t_h} \cdot \sqrt{\frac{4A\sqrt{\Delta H}}{3m_s f \sqrt{2g}} t_h} - \frac{m_s^2 f^2 g}{2At_h^2} \left(\sqrt{\frac{4A\sqrt{\Delta H}}{3m_s f \sqrt{2g}} t_h}\right)^3$$
(14)

There is a third situation to be considered besides the two already mentioned in the above:

 (Very) Slow opening of the valves; discharge starts and stops before the valves are fully opened. See the graph at the right.

Notes:

- The negative values for Q have no physical meaning. The water stops flowing at t≈ 540 s before t=t_h=600 s.
- The maximum discharge for this situation can be computed using Formula (14).

For situation 3 the length of the filling or emptying period, T_{total} , can be determined using Formula (11) and realizing the discharge Q is equal to zero when the lock chamber has been filled or emptied:



Figure 4-15 Graph showing discharge for very slow lifting

$$Q = \frac{m_s f \sqrt{2g\Delta H}}{t_h} t - \frac{m_s f^2 g}{2At_h^2} t^3 = 0 \qquad 0 < T_{total} \le t_h \tag{11}$$

$$m_s f \sqrt{2g\Delta H} \qquad m_s f^2 g$$

$$\frac{m_s f \sqrt{2g\Delta H}}{t_h} t = \frac{m_s f^2 g}{2At_h^2} t^3 \qquad \Rightarrow \qquad t = \sqrt{\frac{4A\sqrt{\Delta H} t_h}{m_s f \sqrt{2g}}} \tag{14}$$

Situation 3:
$$t = T_{total} = \sqrt{\frac{4*1080\sqrt{4.5}*600}{0.85*5\sqrt{2*9.81}}} = 540.4 [s]$$

Alternatively, for situation 3, it is possible to find T_{total} using the condition that the volume of water V being discharged over this period is equal to A Δ H.

$$\int_{t=0}^{t_{total}} Q(t) dt = V \qquad t = 0, \quad V = 0 \qquad t = T_{total}, \quad V = A\Delta H$$

The boundary condition for t=0 does not result in extra information (Why?); the one for t= T_{total} does. For convenience sake notation t will be written instead of T_{total} .

$$\int_{0}^{t} Q(t) dt = \int \left(\frac{m_s f \sqrt{2g\Delta H}}{t_h} t - \frac{m_s^2 f^2 g}{2At_h^2} t^3 \right) dt \quad \Rightarrow \quad \frac{m f \sqrt{2g\Delta H}}{t_h} \frac{1}{2} t^2 - \frac{m_s^2 f^2 g}{2At_h^2} \frac{1}{4} t^4 = A\Delta H$$

Finally this results in : $t^4 - \left(\frac{8A\sqrt{\Delta H} t_h}{m_s f \sqrt{2g}} \right) t^2 + \left(\frac{16A^2\Delta H t_h^2}{m_s^2 f^2 2g} \right) = 0$ use the

ABC - formula to find :
$$t^2 = \frac{4A\sqrt{\Delta H} t_h}{m_s f \sqrt{2g}} \implies T_{total} = \sqrt{\frac{4A\sqrt{\Delta H} t_h}{m_s f \sqrt{2g}}}$$
(15)

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Notes:

For the situations 1&2, as well as for situation 3, it is possible to use the integration below to find the volume of discharged water for any t as long as t ≤ t_h, for situation 1&2, respectively as long as t ≤ T_{total} for situation 3.

$$\int_{0}^{t} Q(t) dt = V(t) \implies V(t) = \int \left(\frac{m_s f \sqrt{2g\Delta H}}{t_h} t - \frac{m_s f^2 g}{2At_h^2} t^3 \right) dt$$

- It is important to observe that the slower the valves in the gate are lifted the smaller the resulting maximum discharge, see the graphs in the above.

The development of the discharge for 0 < t < T_{total}



Figure 4-16 Graphs showing discharge during the whole lifting period

Note:

In section 4.1.3 it will be shown that hawser forces vary in agreement with the derivative (*afgeleide*) of Q with respect to time.

- Look at the graphs in Figure 4-16 for the discharge Q, and observe that going from the first branch $(t < t_h)$ to the second branch $(t > t_h)$ of the graph, there is not a smooth transition in Q for t=t_h:
 - In the right-hand graph for $t_h=180$ s, the tangents *(raaklijnen)* on either side of t_h do not have the same direction. Mathematically spoken, the derivative on the left of t_h has a different value than the derivative on the right of t_h .
 - In the left-hand graph for $t_h=90$ s, the derivative of Q not only has a different (absolute) value but changes sign as well from plus to minus. From a mathematical point of view, when increase changes into decrease, the derivative changes sign from + to -.
- Look at the graph in Figure 4-15 and note that, when comparing the three graphs, this one has the smoothest development of discharge Q and of its derivative.

Manipulating the lifting of the gate(s), selecting a smaller or larger t_h , in other words, faster or slower opening, is an effective way to influence the hawser forces in the lock.

<u>The development of the water level difference z.</u> for $0 < t < T_{total}$

Figure 4-17 shows the variation of water level difference z in time. The Formulas (10) and (4) and previously mentioned data for A, m_s , f, g and ΔH , have been used to produce the graph. The change in z is small in the later stage of the process. The whole gate could be opened, provided that the resulting longitudinal wave is small enough with regard to ship movements and hawser forces.



Longitudinal culvert filling

In contrast with the head filling system, described in the previous section, energy losses in the culvert system have to be considered, viz.:

- 1. energy loss due to:
 - wall friction in the culverts
 - local energy dissipation while entering and exiting the culvert
 - losses in bends and at bifurcation points (splitsingspunten),
 - etcetera (trash racks, valve shafts, transitions in culvert shapes and/or size).
- 2. Besides this, rectangular shaped conduits perform worse than smooth and/or rounded (pipe) conduits having the same cross sectional area. The rectangular shape increases the order of magnitude of several of the losses previously mentioned under point 1.

Looking at all the loss phenomena involved it seems hard to imagine that similar formulas for the calculation of Q, development and maximum, and the F/E time T_{total} can be used culvert filling, as the ones for heaf filling, see e.g. Formula (11) and (5) for Q, and Formula (12) for T_{total} . However, it would be very convenient to use similar expressions and be able to compare the two types of F/E systems. The solution is to use the discharge coefficient m_s as the dust bin or garbage can (*vuilnisbak*), and translate all those extra losses into a lower value for m_s . Before determining the m_s for a culvert system some comparisons will be made between "through the heads" and "through longitudinal culverts" systems.

Figure 4-18shows the differences between head filling and culvert filling of the lock chamber with regard to the discharge Q and the period required for filing T_{total} . The Q(t) line for head filling was already shown before in Figure 4-16, three culvert alternatives are added to the graph. The alternative culverts have a shape or configuration similar to the one drawn in Figure 4-21. The single culvert 100% has a cross sectional area just as big as the area used for the openings in the gate in the head filling alternative. A single wall culvert would fill the lock chamber from one side, which has big disadvantages regarding turbulence of water in the lock. To fill the lock chamber from both sides, two wall culverts have to be constructed. A 50% discharge on either side requires half the cross sectional area, which means that the width and height dimensions have to be scaled down with $\sqrt{2}$ for the two smaller culverts.



Figure 4-18 Head filling versus culvert filling with regard to discharge Q and period T_{total}

The graphs in Figure 4-18 were made for a lock chamber servicing CEMT class IV or class Va vessels; inland waterway navigation. With 5½ to 8 minutes, the systems fill or empty fast enough from a lock operation point of view, see Table 1-4. Hawser forces need to be checked for this situation, see section 4.1.3 on hawser forces.

In Figure 4-19 the discharge curve for a bigger lock and a higher lift are presented. (The horizontal area of the chamber is 7500 instead of 1080 m²; the lift 20 m i.o. 4.5 m). The lock should service a push convoy, consisting of 3*2 barges and a 40 m long push barge, which is beyond the largest CEMT class. In spite of

Locks

the (much) larger openings or cross sections of the culverts the time required for F/E increases substantially to about 12 or 18 minutes. For new lock construction the latter would have to be considered as too long. The discharge curves are smoother because the valves are opened slowly, in 10 minutes. Hawser forces need to be checked for this situation as well, however, it is easy to predict that head filling will fail the test, because in the worst case the water is jetting through the valves with a velocity of about 8 m/s (simply divide Q_{max} by f). Head filling using short culverts around or through the head will do better provided quite a large stilling chamber is being used. In case of longitudinal culverts, the distributed inflow of water should prevent potential hawser problems.





Remember that, for the same water head ΔH and the same cross sectional area f, the discharge Q will develop slower and the maximum discharge will always be lower for a culvert system than for a head filling system. Longitudinal culverts may be designed and constructed with a larger cross sectional area to obtain the same capacity. There is quite a big cost difference between filling through the gate valves or through longitudinal culverts. Construction of even larger culverts has less bearing on costs than the switch from head to culvert filling.

And again there is another hydraulic phenomenon to be considered: overtravel of the water in the culvert. At the end of the F/E period there will be no water head ΔH left; based on the formulas the water in the culverts should stop flowing, but it won't. The kinetic energy of the mass of water in the culvert will act as the new driving force. The water will overtravel up to the moment that driving force will be compensated by the newly created water level difference. This water head 'on the other side' will reverse the direction of flow and the same will happen again. When the water head has been reduced to nothing, the water will overtravel again, obviously in the other direction. An oscillatory motion in the culvert, lock chamber and part of the outer pool will be the result, see Figure 4-20. Finally, the oscillations will come to an end due to friction and other energy losses. It will take too much



time to wait for the overtravel effect to dampen out. A possibility to shorten the process is to close the valve(s) in the culvert. The closing of the valve from a certain moment close to the end of the F/E period would have to be introduced into the formulas, as was the opening of the valve. It will definitely change the discharge graph negatively. Discharge will start to decrease earlier and faster, and the time required for F/E will increase. There are some other downsides to be taken care of when a valve in a culvert is being closed, viz. cavitation and water hammer, see section 4.1.4.

Determination of discharge coefficient ms for a longitudinal culvert system:

To explain how the discharge coefficient for a longitudinal culvert has to be determined it is best to use an example. A more or less realistic example of a longitudinal culvert, constructed into the wall of a lock, is shown in Figure 4-21. Although the sketch is schematic, it does include the main energy loss contributors like entrance and exit losses, bends, and bifurcations and obviously wall friction. The flow of water is as follows:

- 1. from the upstream or upper pool more or less horizontal, depending on the shape of the entrance, but in any way sideways out of the waterway into the culvert entrance.
- 2. through a bend; here vertically down in 90⁰. The bend may be less sharp and the culvert axis may be inclined instead vertical, all depending on the actual situation. The vertical distance bridged will be in the order of the difference in bottom level between the up and downstream water pool respectively.
- 3. through a bend, here 90⁰ again, and further in horizontal direction into the (bottom) culvert that runs along the biggest part of the lock chamber and close to the lock bottom. The water passes the shaft with the lifted upstream valve, which opened up the culvert for the flow of water.
- 4. every so much meters a part of the water takes an exit turn, here in the horizontal plane, and flows into the lock chamber. The exit ports are close to the lock chamber bottom.
- 5. the closed downstream valve prevents the water from flowing through the remaining culvert into the lower pool.



Figure 4-21 3D sketch of a longitudinal culvert system

With a bit of imagination the longitudinal culvert can be considered as a part of a pipeline network; the lock chamber is the other part of the network. Regarding the culvert formulas based on closed conduit flow theory will be used, however, for flow in the lock chamber open channel flow formulations would be more appropriate. In spite of this, the lock chamber will be treated as if it was a closed conduit, a special type of course, to be able to use formulas, engineering experience and/or results for closed conduit flow and the pipeline systems world.

Below, for convenience sake, once more the basic formula to describe the discharge Q, depending on a water level difference (z or Δ H) and discharge coefficient m_s:

$$Q = m_s \cdot f \cdot \sqrt{2gz} \qquad or \qquad m_s \cdot f \cdot \sqrt{2g\Delta H} \tag{1}$$

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A formula commonly in use to calculate losses in pipeline systems is the following:

$$\Delta H_{tot} = \left(\lambda \frac{L}{D_{eq}} + \sum_{i} \xi_{i}\right) \frac{U^{2}}{2g}$$

Where:

$$\Delta H_f = \left(\lambda \; \frac{L}{D_{eq}}\right) \frac{U^2}{2g} \tag{17}$$

is the part of the losses due to wall friction, and:

$$\Delta H_i = \left(\sum_i \xi_i\right) \frac{U^2}{2g} \tag{18}$$

accounts for all the other losses, such as entrance, exit losses, elbows or bends, bifurcations (*splitsingspunt*), etc.. (16)

$$\begin{array}{rcl} \Delta H_{tot} &=& total energy loss & [m] \\ \Delta H_{f} &=& energy loss due to wall & [m] \\ friction & [m] \\ \Delta H_{i} &=& any other energy loss & [m] \\ \lambda &=& friction coefficient & [-] \\ L &=& length of the culvert & [m] \\ D_{eq} &=& equivalent diameter for the & [m] \\ rectangular profile \\ \xi_{i} &=& loss/ dissipation coefficient i & [-] \\ g &=& gravitational acceleration & [m/s^{2}] \\ U &=& flow velocity & [m/s] \end{array}$$

Rewrite Formula (16) to make it look like Formula (1) and introduce the cross sectional culvert area f in order to replace U by Q; then find an expression for discharge coefficient m_s for closed conduit flow:



Having found a formula linking m_s to all sorts of losses that occur in close conduit flow the, remaining task, in following paragraphs, is to find or determine the values to be used for the appropriate loss coefficients. The work may be laboriously, depending on the number of losses to be taken into account, but is not too complicated.

The previous statement, 'not too complicated but laboriously', is correct from a more general point of view, even for a single pipeline, however, a pipeline network is subject of investigation here. Looking a bit more detailed, a first complication is the fact that the discharge in each pipeline, or part of the pipeline of the network, is different, see Figure 4-35. There is a main pipeline, the main longitudinal culvert, and then multiple exits into the lock chamber, which are schematised as smaller branching off pipelines. The whole discharge Q flows through the first or main branch(es), then at every exit branch or chamber outlet, Q is getting smaller and smaller in the main branch. Even without the losses it needs an iteration to find Q in every part of the system. On top of it friction and other losses depend on Q, vice versa Q depends on those losses; an iterative process involving more than 2 variables is the result.

It is practical to start with determination of the losses ξ_{i} , depending on a varying Q. After that step the losses in the individual pipes, and possibly derived individual discharge coefficients $m_{s,i}$, have to combined to an overall discharge coefficient for the whole pipe network. However, first some remarks on the cross sectional shape.

General reference is made to the lecture notes of the course CT2140 or CT2320 or any other reference work in the field of hydraulic engineering or fluid mechanics. Look specifically for the sections on closed conduit flow in pipelines,

Circular versus rectangular cross section; steel pipe versus concrete culvert:

A culvert system will be used for larger volumes of water. Generally the larger the volume, the larger the required cross sectional area of the culvert will be; the dimensions and shape will be beyond what is readily available as standard prefab construction material, whether in steel or concrete. The largest or main parts of the culvert system will be constructed in-situ. Since the fast majority of lock walls is constructed using concrete, it is safe to assume that the larger parts of the culvert will be rectangular shaped.

Smaller parts, like nozzles (*spuitstukken*) may be constructed from standard pipe material. Look e.g. at the lock near Sülfeld,



Figure 4-22 Nozzles used as culvert exits, lock Sülfeld - Germany

Germany, see Figure 4-22; pipes/nozzles are being used in a specially designed configuration to break the water jets and distribute the inflow of water into the lock chamber as much as possible.

In following paragraphs, losses will be discussed for closed conduit flow in pipeline networks. Figure 4-23 shows secondary flows in a rectangular and a triangular shaped cross section, which typically do not occur in circular shaped cross sections. Travelling along the wall of a circular pipe, a water particle would not

encounter discontinuities, whilst the perimeters of e.g. rectangular and triangular shaped pipes contain pronounced discontinuities in every corner. The energy consumed by the resulting secondary flows or eddies reduces the energy available for the primary flow parallel to the axis of the pipe. The available area, and the width-height ratio of the cross section, are factors determining the number and the development into smaller or larger eddies.

The wetted part of the perimeter P and area A of a cross section of arbitrary shape are related to each other by the hydraulic radius R. Formulas (20) and (21) show how to compute R for a circular and a rectangular cross section.

Traditionally friction formulas have been developed for pipes, having circular cross sections; far less for other types of cross sections. Looking e.g. at the Jain formula, calculating the wall friction (loss), two parameters, the pipe diameter D and the Reynolds number Re, take the shape of the pipe into account implicitly. On second thought, it is the pipe diameter alone bringing the shape into the equation, because the Reynolds number includes D. Substituting the diameter D with 4R does not change the results for pipes, however, provides the

Figure 4-23 Losses due to the shape of the cross section (Idelchik)

For circular cross sections:

$$R = \frac{A}{P} = \frac{\frac{1}{4}\pi D^2}{\pi D} = \frac{1}{4}D$$
 (20)

For rectangular cross sections:

$$R = \frac{A}{P} = \frac{b.h}{2(b+h)} \tag{21}$$

$$D_{eq} = 4R = \frac{2.b.h}{(b+h)}$$
 (22)

opportunity to introduce a range of differently shaped cross sections by means of four times the appropriate hydraulic radius, see the development of Formulas (20) trough (22).

Wall friction – loss coefficient ξ_f :

Colebrook (1939), combining the work of e.g. Chézy, Von Karman, Darcy and Weisbach, proposed one formula for calculation of the friction coefficient λ for both smooth and rough pipes of circular cross sections:

$$\frac{1}{\sqrt{\lambda}} = 2\log_{10}\left(\frac{k_s/D}{3.7} + \frac{2.51}{\text{Re}\sqrt{\lambda}}\right)$$
(23)

The Colebrook formula is an implicit equation for λ , which requires quite some iteration to find a result. Alternatively the Moody diagram could be used to find the friction factor, see Figure 4-24.



The drawback of iteration using the Colebrook formula, or reading the Moody diagram numerous times, was circumvented by Jain (1976), who suggested the following explicit equation for the friction factor:

$$\frac{1}{\sqrt{f}} = -2\log_{10}\left(\frac{k_s/D}{3.7} + \frac{5.74}{\text{Re}^{0.9}}\right) \qquad or \qquad f = \frac{0.25}{\left[\log_{10}\left(\frac{k_s/D}{3.7} + \frac{5.74}{\text{Re}^{0.9}}\right)\right]^2} \tag{24}$$

for: $10^{-6} \le \frac{k_s}{D} \le 10^{-2} \qquad and \qquad 5000 \le \text{Re} \le 10^8$

The Jain formula saves quite some computational time, especially when a spreadsheet will be used to calculate through all the individual pipelines and determine the overall m_s of the pipeline network as well.

Note: Don't get confused in the following where the symbol f is used for either the area $[m^2]$ of the valve, or for the friction coefficient [-].

Next, the information to determine loss coefficients for entrance, bend elbow, bifurcation, end bend, and exit losses will be presented and discussed.

Intermezzo: Idelchik's handbook of hydraulic resistance The russian researcher I.E. Idelchik and fellow researchers did extensive research into loss coefficients for pipes, single pipes, in networks, and as flowpipe in machinery. Losses for gas and water flow, for circular and rectangular shapes, and a lot of other things water can flow through were included. Idelchik gathered all the results together and in the 1960'ies the work was first translated into French, later it became available in English. Today it is still being reprinted.

The handbook of hydraulic resistance contains many drawings explaining how water flows through pipes, bends elbows, distributors, trash racks, etc.. New editions still use the illustrations made years ago, presumably because they provide such clarity to the actual behaviour of flow in pipes, closed conduit flow.

Test results are presented in graphs and tables. For many situations, formulas to calculate the loss coefficient ξ are included. For those cases the formula has not been presented yet, nowadays availability of (e.g.) spreadsheets make it easy to find the equation describing a line or curve in a graph. Since, a set of tabular values can be plotted in a graph, it is also possible to find a formula to compute the loss coefficients based on the data presented in tables.





FIG. 6.2. - Figure de l'écoulement dans un coude de 90°

Figure 4-25 Illustration of flow and eddies in a bend or elbow



Figure 4-26 **Distributor of constant** cross section and bifurcations



Type of pipe and wall Figure 4-27 friction



F_{I}	Fr	F_r F_l	Q_l/Q_p										
$\overline{F_r}$	F_p	$\overline{F_p}$	0	0,1	0,2	0,3	0,4	0,5	0,6	0,7	0,8	0,9	1,0
0,25	1,00	0,25	+ 0.04	- 0,01	- 0,03	- 0,01	+ 0,05	0,13	0,21	0,29	0,38	0,46	0,54
0,33	0,75	0,25	0,20	0,08	0,00	-0,02	- 0,01	0,02	0,08	0,16	0,24	0,34	0,45
0,50	1,00	0,50	+ 0,05	- 0.03	- 0,06	- 0,05	0,00	+ 0,06	0,12	0,19	0,27	0,35	0,43
0.67	0.75	0,50	0,20	+0.04	- 0,02	- 0,04	- 0,03	- 0,01	+ 0,04	0,12	0,23	0,37	0,50
1,00	0.50	0,50	1,00	+ 0,72	+0,48	+0.28	+0,13	+ 0,05	0,04	0,09	0,18	0,30	0,50
1,00	1,00	1.00	+0.05	- 0,02	- 0,04	- 0,04	- 0,01	+ 0,06	0,13	0,22	0,30	0,38	0,45
1,33	0,75	1,00	0,20	+0,10	+0,01	- 0,03	- 0,03	0,01	+0,03	0,10	0,20	0,30	0,42
2,00	0,50	1,00	0,95	0,62	0,38	+0,23	+0,13	+0,08	0,05	0,06	0,10	0,20	0,40

Figure 4-28

Loss coefficients for reunion of flows (lock emptying)

D = di	etted perimeter	[m] [m]	Q ₁ =	lateral discharge – branching off	[m³/s] [m³/s]
	etted perimeter	[m]	$Q_1 =$	lateral discharge – branching off	[m³/s]
P = we					
A, F = cr	ross sectional area of flow	[m ²]	Q _p =	supply discharge	[m ³ /s]
Re = Re	eynolds number	[-]	a,h =	height of the culvert	[m]
v= kii	nematic viscosity	[m²/s]	b =	width of the culvert	[m]
k _s = ec	quivalent sand roughness	[m]	R _r =	hydraulic radius - rectangular profile	[m]
$\lambda, f = frie$	iction coefficient	[-]	R _c =	hydraulic radius - circular profile	[m]

uА

flow at the entrance

Contraction of the

Entrance loss – loss coefficient $\xi_{entrance}$:

Entering the conduit or culvert, the water flow generally separates from the wall and an eddy zone develops, see Figure 4-29. Due to the flow contraction μ the cross sectional area A reduces, which has to result in an acceleration of the flow at that point. Immediately after the contraction the available section is larger and the flow will decelerate, which results in energy loss. The loss coefficient is calculated as follows:

 $\xi_{entrance} = \left(\frac{1}{\mu} - 1\right)^2 \tag{25}$

Numerous tests showed that the contraction coefficient μ for a sudden change of the cross section shall be taken equal to 0.85.

Bend or elbow loss – loss coefficient ξ_{bend} :

Figure 4-30 shows the track a water particle follows when travelling through an elbow or bend. Observe:

- the track is quite curly and far from an averagely straight flow line
- eddies develop on either side of the elbow or bend The larger the angle *α* between the pipes, the larger the eddies that will develop.



Figure 4-29

• whether the pipe is circular or rectangular, it is split in half and in each part a whirl spins around (in opposite direction).

Obviously losses will occur in this turbulent flow; for some angles α and for circular pipes, the loss coefficients are shown in Figure 4-30.

In the culvert considered, the angle of the two pipe may not be the only property that changes, but the size may change as well; look for instance at the connection main culvert to exit port in Figure 4-21. For situations where only the width changes, from b_o to b_1 as shown at the top of Figure 4-31, the loss coefficient is calculated as follows:

$$\xi = 1.15 \left(\frac{a_o}{b_o}\right)^{-0.09} * \left[\frac{b_1}{b_0}\right]^{\left(-0.4 - 0.727 \cdot \ln\left(\frac{a_o}{b_o}\right)\right)}$$
(26)

Looking at the exit ports, not only the width b changes but the height a changes as well. In that case, it is probably better to use Formula (23) and consider the loss as a bifurcation loss instead of a bend loss.

The niche in the bottom illustration of Figure 4-31, is a sort of dead-end street and adds to the loss in the bend. This would be taken into account by a multiplication factor 1.2 over ξ . A 90° bend with a niche would result in a total loss coefficient of 1.44.



Figure 4-31 End bend (top), niche (bottom)

Bifurcation loss – loss coefficient ξ_{bif} :

At every exit pipe from the main culvert, at every bifurcation, there is a supply flow, a lateral flow branching off, and there is the remaining flow in the main culvert that continues, see Figure 4-32. As expected there is contraction with related losses in the exit or lateral pipe, however, the same occurs behind the bifurcation in the on-going pipe or main culvert. Eddy areas, reducing the effective cross section for the flows past the bifurcation, develop. The loss coefficients are calculated as follows:

$$\xi_{pl} = \zeta_{pl} = 0.85e^{\left(\frac{Q}{Q_p}\right)} * \left[\frac{F_l}{F_p}\right]^{\left(-1-0.25*\ln\left(\frac{Q_l}{Q_p}\right)\right)}$$
(27)
$$\xi_{pr} = \zeta_{pr} = \left(\frac{Qr}{Qp}\right)^3 - 0.7*\left(\frac{Qr}{Qp}\right)^2 - 0.3*\left(\frac{Qr}{Qp}\right) + 0.7$$
(28)

 $F_p =$ cross sectional area supply pipe

 $F_r =$ cross sectional area on-going pipe; generally $F_r = F_p$

- F = cross sectional area lateral pipe
- loss coefficient lateral or branch-off pipe $\zeta_{pl} =$

= loss coefficient on-going pipe $\zeta_{\rm pr}$





=

When exiting from the culvert and entering the lock chamber the flow decelerates because of being distributed over a (much) larger space. The loss coefficient is calculated as follows:

$$\xi_{exit} = \left(1 - \frac{U_2}{U_1}\right)^2 \qquad since: \ Q = A_1 U_1 = A_2 U_2$$
$$\xi_{exit} = \left(1 - \frac{A_1}{A_2}\right)^2 \qquad (29)$$





Figure 4-33 Idelchik graph bifurcation loss coefficients



Here:

 A_2 b_{exit} h_{exit}

b_{exit} h_{exit} = the lock chamber length divided by the number of exits in the wall the water depth in the lock chamber. =

Since the water depth varies during the F/E process, hexit would be dependent on z(t); this complicates computation of ξ_{exit} and the whole calculation of the overall discharge coefficient m_s. Generally A₂ is substantially larger than A₁, the cross sectional area of the exit port, and the resulting ξ_{exit} will be small. Therefore it will be justified to use the average water depth in the lock chamber during F/E.

Example: Calculation of the opened valve.	ne loss in t	he supply	/ culvei	rt / pipe	eline up to	, but not incl	uding, the		opened
k _s =0.3	[mm]	A _{main} =	5	[m ²]	P =	9 [m]			vaive
$v^{3} = 1 \times 10^{-6}$	[m ² /s]	b =	2	[m]	R _r =	5/9 [m]			
g =9.81	[m/s ²]	h =	2.5	[m]	D _{equi} =	20/9 [m]		-	
Loss		Qi	А	D _{equi}	U	Re	f	ξ	
contributions		[m ³ /s]	[m ²]	[m]	[m/s]	[-]	[-]	[-]	
$\mu_{\text{entrance}} = 0.85$	[-]	25,0	5		5,0	11111111		0,031	ξentrance
L1 = 2	[m]	25,0	5	2,222	2 5,0	11111111	0,010282	0,002	ξfriction
Bend 1 = 90	[degrees]							1,2	ξbend
L 2 = 4,5	[m]	25,0	5	2,222	2 5,0	11111111	0,010282	0,005	ξfriction
Bend 2 = 90	[degrees]							1,2	ξbend
L 3 = 8	[m]	25,0	5	2,222	2 5,0	11111111	0,010282	0,009	ξfriction
								+ =====	
$\nabla \epsilon U^2$	2 4 4 9	25	10 5	1				2,448	Σξ
$\Delta H_{loss} = \Sigma \zeta \frac{1}{2g}$	$= 2.448 - 2^{2}$	=3	.12 [<i>m</i>	J F	lence, in	the supply	culvert th	ne loss in	water head
$m_s = -\frac{Q}{\sqrt{1-\frac{1}{2}}}$	=	25	= = 0.6	а 54	mounts to	o about 3 m.			
$f \sqrt{2g\Delta H}$	5*√2*	9.81*3.1	2	т	he disch	arge coeffici	ent m co	mouted in	two different

	<i>5</i> V	0	•		
<i>m</i> _s =	$= \left(\lambda + \frac{1}{2}\right)$	$\frac{L}{D_{eq}} + \frac{1}{2}$	$\sum_{i} \xi_{i} \Big)^{-\frac{1}{2}}$	=(2.448	$)^{-\frac{1}{2}} = 0.64$

discharge coefficient m_s, computed in tw ways, see Formulas (1) and (19), is equal to 0.64.

Note: the biggest loss contributors are the two bends.

Calculating through the pipeline network / the calculation algorithm:

Some basic rules and/or start conditions to be able to calculate through a pipe network, see Figure 4-35:

- The network consists of pipes (e.g. pipe AD or pipe IJ) and nodes (here A through J)
- Each node in the network can only have one pressure
- The pressure change from A to D by a clockwise path, pipe AC then CD, must equal to the pressure change using the anti-clockwise path, pipe AB then BD
- For this specific pipe network for culvert and lock chamber:
 - There are four loops, respectively ACDB, CEFD, EGHF and GIJH
 - _ The water head between the upper pool and the lock chamber, ΔH , is known, or has to be assumed.
 - The pressure in the nodes B, D, F, H and J is assumed to be the same because the average water level of the lock chamber will be the same in all these nodes. Hence, the pressure drop over AB, is equal to, for instance, the pressure drop over ACEGIJ, irrespective of the increasing number of pipe pieces. (AB only one pipe; from A to J 5 pipe pieces)
- Flow into each node or junction must be equal to flow out of the node or junction
- For this specific pipe network for culvert and lock chamber:
 - The inflow into the pipe network will be in node A, the outflow out of the network in the nodes B, D, F, H and J
 - The sum of the outflow or discharge in the nodes B, D, F, H and J is equal to the inflow in node A
- Given a certain flow in the pipes, losses can be calculated for every pipe, hence, the pressure drop or head loss is known for each pipe.

To start the actual calculation process:

An initial discharge has to be assigned to each pipe in the network. For instance 20% of Qinflow in the pipes AB, AD, AF, AH and AJ; 80, 60, 40 and 20% in the pipes AC, CE, EG and GI respectively; whilst the discharge in pipes BD, DF, FH and HJ is zero.

 Calculate the head loss in each pipe and calculate the summed up head loss going clockwise and anti-clockwise through the first loop on the left side of the network (ACDB). Generally the clock and anti-clockwise head loss around the loop won't be the same.

 In case the clockwise head loss is the largest than the flow through the involved pipes has to be reduced. Vice versa the flow through the anticlockwise pipes has to be increased. Use e.g. the Hardy Cross correction factor.

If necessary the discharges in the pipes have to be adjusted over and over again until the head loss left around the loop is equal to the loss going right around the loop.

 Now move to the next loop (CEFD) in the network and use the previously computed results for pipe CD, which is the common pipe of these two loops. Again check the clock and anti-clockwise head loss. For this loop there will be an iteration as well; flows or discharges will have to be adjusted, head losses recalculated and checked again, until the results left and right around the loop are balanced. Then for the remaining loops EGHF and GIJH the same has to be done (EF and GH are common pipes).

Background – Cross-method:

In 1936 Hardy Cross worked around his moment distribution method, for structural analysis of large buildings, into a similar method to determine the discharge of pipelines in complex water supply networks. Until recent decades, it was the most common method for solving such problems.

For each loop:

$$\Sigma Q = 0$$

$$\Sigma \Delta H_{loss,clock} = \Sigma \Delta H_{loss,anti-clock}$$

Correction factor:

$$\Delta Q = \frac{\Sigma \Delta H_{loss,clock} - \Sigma \Delta H_{loss,anti-clock}}{2\left(\Sigma \frac{\Delta H_{loss,clock}}{Q_{clock}} + \Sigma \frac{\Delta H_{loss,anti-clock}}{Q_{loss,anti-clock}}\right)}$$
$$Q = m_s \cdot f \cdot \sqrt{2gz} \qquad (1)$$

- Having calculated through the network from left to right, now the calculations have to be done in the other direction, (from loop GIJH to loop ACDB). The whole iterative calculation process continues until head loss differences in every node and loop, for the entire network are negligable.
- Now the water level difference z or water head related to the input discharge Q, is known for the longitudinal culvert system. Knowing Q, z and f, the cross sectional area of the main culvert, the discharge coefficient m_s can be calculated with Formula (1).
- Compute other combinations of Q, z and m_s. It is recommended to calculate Q, z and m_s, say for every 0.5 m of water level difference. A slight variation in the figures for m_s may be observed, which is not strange given all the numerics. Compute and use the average m_s; this will yield sufficiently accurate results.
- With m_s, the discharge curve (Formulas (11) (5)) can be drawn and the total filling time (Formula (12)), can be determined for this longitudinal culvert system.

Clearly the computational process described in the above is best done by means of a spreadsheet or using some other network analysis software. (In Excel the iteration option and the Solver add-in are very convenient).





Notes:

 An alternative for iterative calculation trough a pipe network can be found by comparing the pipe network with an electricity network. There is an analogy between the voltage (Volt), current (Ampere) & resistance (Ohm), and the water head (△H), discharge (Q) & losses or the flow resistance K.

$$\Delta H = \Sigma \xi \frac{U^2}{2g} \implies \Delta H = \frac{\Sigma \xi}{2g} U^2 \quad or \quad \Delta H = K U^2 \quad \triangleq \qquad V = RI$$

Observe the square sign above *U*, which is not there for *I*, compare the expressions for (total) resistance, and see the extra term $2/\sqrt{(K_1K_2)}$ in the denominator for parallel resistance.

Expressions for resistances									
Serial resistance in pipe networks	Serial resistance in electricity networks	Serial scheme							
$U_1 = U_2 = U$ $\Delta H_{tot} = \Delta H_1 + \Delta H_2 = K_1 U_1^2 + K_2 U_2^2$ $K_{tot} = \frac{\Delta H_{tot}}{U^2} = K_1 + K_2$	$V_{tot} = V_1 + V_2$ $I = I$ $R_{tot} = R_1 + R_2$	$U \text{ or } I \longrightarrow K_1 \text{ or } R_1 \longrightarrow K_2 \text{ or } R_2 \qquad U \text{ or } I \longrightarrow L_1 \text{ or } \Delta V_1 \longrightarrow L_1 \text{ or } \Delta V_1$							
Parallel resistance in pipe networks	Parallel resistance in electricity networks	Parallel scheme							
$ \Delta H = K_1 U_1^2 = K_2 U_2^2 = K_{tot} U_{tot}^2 $ $ U_{tot} = U_1 + U_2 $ $ K_{tot} = \frac{1}{\frac{1}{K_1} + \frac{1}{K_2} + \frac{2}{\sqrt{K_1 K_2}}} $	$V = V$ $I_{tot} = I_1 + I_2$ $R_{tot} = \frac{1}{\frac{1}{R_1} + \frac{1}{R_2}}$	$U \text{ or } I \xrightarrow{U_1 \text{ or } I_1} K_1 \text{ or } R_1 \xrightarrow{U \text{ or } I_2} U \text{ or } I \xrightarrow{U_2 \text{ or } I_2} K_2 \text{ or } R_2$							

In a spreadsheet the total resistance for two parallel exit pipes is easily extended to the total resistance for n exit pipes. Then the parallel exit resistance has to be entered into the formula for a serial scheme to combine the supply line part of the culvert (up to the valve) with the exit part.

- In terms of hydraulics, the system should be balanced so that the hydraulic loss for the filling process is governed by the nozzles in the floor and not by the hydraulic losses in inlet, culverts or pressure chamber. As a rough estimate the cross-sectional area of the pressure chamber should be at least the same as the sum of the cross-sectional areas of the feeding culverts. As a rule of thumb, the ratio between the sums of cross-sectional areas for the nozzles and the culverts can be used for an initial design:

$$\alpha = \sum_{nozzles} A_{nozzle} \left/ \sum_{culverts} A_{culvert} \right.$$

If the parameter α is small (<1), the filling process will be slower but smoother. If it is bigger (>1.5), the filling will be faster but rougher. The indicative figures for α give some guidance for the ratio between the cross sectional areas of the nozzles and culvert or the pressure chamber. Later the ratio should be optimized on the basis of a laboratory or numerical model.

- How to incorporate the closing of the valve at the end of the filling period is a subject left to the reader. It makes sense to assume a linear closing process; linear with regard tot time.
- In the previous all the explanations and (example) calculations were in fact dedicated to the filling proces. When the emptying process has to be calculated through, a first obvious difference is that entrance and exit, and the related losses, switch; pipe lengths and bends are different, no bifurcations but joints where flows come together. Setting up the calculation (process) for emptying is left to the (student) readeer and considered good practice to get to an understanding of all the previous.

4.1.3 Hawser forces

Wind, waves or currents in the lock chamber or in the approach areas of the navigation lock will set a ship in motion or result in hawser forces *(tros krachten)* in case the ship has been tied to the structure by mooring lines. Uncontrolled motion of the ship will be prohibited, as a consequence, breaking of mooring lines must be prevented, hence hawser forces must be limited. Limiting the hawser forces imposes quite some demands on F/E.

In this section hawser forces will be considered assuming the lock has a "through the heads" system for water leveling. The F/E of a lock is accompanied by (translation) wave and flow phenomena in the lock chamber and these phenomena are generally more pronounced for a "through the head" than for a "longitudinal culvert" system. Head F/E results in a rather concentrated disturbance near the head, which irrevocable spreads throughout the whole lock chamber, or through the outer port. The reason of being of a longitudinal culvert system, which is much more expensive, is to reduce the order of magnitude of water turbulence and flow by means of distributed in or outflow in the lock chamber.

Inside the lock chamber the flow in or out at a chamber end (upper head during filling, lower head during emptying) occurs mainly in the longitudinal direction of the chamber. Generally the water area outside the lock chamber is less confined and the wave and flow phenomena will spread more and/or will be damped. Although this section will focus on ships being moored in the lock chamber it should not be forgotten to check the hawser forces of ships in the lock approach area or at the waiting berths, even though the problems to be expected are of a lesser order than in the lock chamber.

Longitudinal force components during F/E:

Figure 4-1 shows how turbulent the water mass may behave in the lock chamber. Even tough the water movements may be directed in all directions, most of the resulting forces will be in the longitudinal direction, parallel to the lock chamber axis. The forces in longitudinal are split in a number of components. Not all these components are relevant for both filling, see Figure 4-36, and emptying, see Figure 4-37. Below the 'bullet-letters' correspond to the letters in the figures:

a. Translation waves:

Because of the non-permanent character of the discharge through the openings, series of translation waves are generated at these openings, which propagate in the chamber. The waves fully reflect against the gates of both chamber ends, only partial against the bow and the stern of the vessel. The translation waves create an oscillating movement corresponding to the own frequency of oscillation of the chamber. (The oscillation period is the time needed to travel up and down through the chamber).



Figure 4-36 Forces in longitudinal direction during filling

b. Difference in impulse over the vessel length; see the frame 'Analogy' on the next page: Concentrated water jets with high flow velocity may occur behind the gate openings. These high flow velocities decrease in the longitudinal direction of the chamber due to turbulent exchange with the surrounding water. Besides and under the vessel, the flow pattern changes substantially due to



partial blocking of the cross section by the vessel. At the bow of the vessel the concentrated filling flow is less able to break down rapidly because of the limited surrounding water available, at the stern of the vessel the flow detaches and only starts spreading and breaking down further on. From the filling point at the upper head to the lower head, the average discharge decreases because a shorter part of the chamber behind the preceding cross section has to be filled. The product of ρ qu (impuls) is not the same in every section, and varies from moment to moment. On top of this, the water levels, h₁ and h₂, vary in time and from section to section as well. Use of a stilling chamber results in a substantially lesser concentrated flow near the head, hence less turbulent exchange will occur in the lock chamber, nonetheless, an impuls force will develop.

When emptying a lock the energy is not broken down in the chamber but outside the lock, thus the turbulence exchange in the lock is considerably less, which reduces impuls differences.

c. Friction (vessel, lock chamber floor and wall):

Friction between the water and the chamber floor, the chamber walls and the skin of the vessel result in a water level difference in the longitudinal direction of the chamber.

- d. Jet of water against the bow: Could be considered as a first order impuls force; consequently the 'difference in impuls over the vessel length' (bullet b above) would have to be considered as a second order impuls force.
- e. Difference in water density: If there are density differences between the water of the lock approach and the chamber, internal (density) waves are generated during F/E. These waves propagate at low speed in the chamber and reflect against the vessel and the gates.



a translatory waves



b impulse difference over the length of the vessel



These waves are accompanied by water level differences in the longitudinal direction of the chamber as well.

The F/E system should be designed such that a water jet directly hitting the ship is impossible. Of the other longitudinal force contributors, only the translation waves and the differences in impuls over the ships length are significantly contributing to hawser forces.

Since translation waves and the differences in impuls over the ships length were identified as significant contributors to hawser forces, further analysis to develop a formula for calculation is meaningful:

- For impuls differences the formulas are shown in the frame 'Analogy' on the previous page. The difficulty is to estimate the different water levels and flow velocities in two different sections at the same time. The subject will not be further elaborated in these lecture notes.
- For translation waves a formula will be derived below.

Mooring load due to a translation wave

Assume the water surface in the lock remains a straight plane, on average, in spite of al local turbulences or a running wave front. However, on one end of the lock the water level is higher than on the other. With a bit of imagination it is clear that the average water surface rotates due to the translation wave, see Figure 4-38. The rotation or slope of the water surface is equal to dz/dx. A force, the hawser force, is required to prevent the ship from sliding down, which would be initiated by the dead weight G of the ship.

The hawser force F has to be equal to:

$$F = \frac{dz}{dx}G = iG \qquad (30)$$

G, the dead weight is equal to the vessel's water displacement. An expression has to found relating dz/dx or *i* to the incoming discharge Q.



Examine the translation wave, which is between Q and *i* and start with the translation wave velocity:

$$c = \sqrt{g \cdot h}$$
 (31)

$$c = \text{wave velocity} \qquad [m/s]$$

$$g = \text{gravitation constant} \qquad [m/s^2]$$

$$h = \text{water depth (average)} \qquad [m]$$

The wave velocity is influenced by vessel presence; the vessel blocks the available water area in the cross section. This is compensated for in the following formula:

$$c = \sqrt{\frac{g(A_v - n)}{b}}$$
(32)
$$A_v = \text{ area of the cross section of the lock; } bh \qquad [m^2]$$
$$n = \text{ cross section area of the ship below water level}$$
$$[m^2]$$
$$b = \text{ width of the lock chamber}$$

The time needed for a translation wave to pass a vessel with a length L can be calculated by:

$$T = \frac{L}{c} \tag{33}$$

Discharge Q into the lock and the change of water level dz in the lock should be equal to each other. Use this to link Q, dz and the translation wave velocity c. First assume a constant discharge Q, then consider a constant increase of the discharge in time:

$$dz = \frac{Q}{b \cdot c}$$
 \Rightarrow $\frac{dz}{dt} = \frac{dQ}{dt} \cdot \frac{1}{b \cdot c}$ (34)

Now Formula (35) for the water surface slope *i* is derived using the above Formulas (30) through (34):

$$i = \frac{dz}{dx} = \frac{z}{L} = \frac{\frac{dz}{dt} \cdot T}{L} = \frac{dQ}{dt} \cdot \frac{1}{bc^2} \qquad \Rightarrow \qquad i = \frac{dQ}{dt} \cdot \frac{1}{g(A_y - n)}$$
(35)

The water slope *i* is proportional to the change in the discharge Q with regard to time, in other words, proportional to the time derivative of Q.

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In section 4.1.2 formulas for the discharge and its time derivative were found. The second derivative with respect to time yields the maximum dQ/dt. For the period the valves are being lifted, t \leq t_h:

$$Q = \frac{m_s f \sqrt{2g\Delta H}}{t_h} t - \frac{m_s^2 f^2 g}{2At_h^2} t^3 \qquad (11) \qquad \frac{dQ}{dt} = \frac{m_s f \sqrt{2g\Delta H}}{t_h} - 3\frac{m_s^2 f^2 g}{2At_h^2} t^2 \qquad (36)$$
$$d^2(Q) = 3m^2 f^2 g \qquad \qquad dQ = m f \sqrt{2g\Delta H}$$

$$\frac{d^2(Q)}{dt^2} = -\frac{3m_s^2 f^2 g}{At_h^2} t = 0 \qquad \Rightarrow t = 0 \qquad \Rightarrow \qquad Maximum \ \frac{dQ}{dt} = \frac{m_s f \sqrt{2g\Delta H}}{t_h} \tag{37}$$

For $t \ge t_h$:

$$Q = -\frac{m_s^2 f^2 g}{A} t + m_s f \sqrt{2g\Delta H}$$
(5)
$$\frac{dQ}{dt} = -\frac{m_s^2 f^2 g}{A}$$
(38)

Depending on the data for m_s , f, ΔH , t_h, and A, the maximum for dQ/dt will be calculated either by Formula (37) or Formula (38). Equating the modulus or absolute value of Formulas (37) and (38) results in:

$$\frac{m_s f \sqrt{2g\Delta H}}{t_h} = \left| -\frac{m_s^2 f^2 g}{A} \right| \qquad \Rightarrow \qquad t_h = \frac{2A\sqrt{\Delta H}}{m_s f \sqrt{2g}} \tag{39}$$

Note:

- Select a t_h smaller than calculated by Formula (39) and the dQ/dt will be the largest right at the start of the F/E process; calculate dQ/dt with Formula (37).
- Using a t_h larger than calculated by Formula (39) has the effect that the maximum of |dQ/dt| occurs when the valves are completely opened, at the end of the F/E process. The maximum |dQ/dt| has to be calculated with Formula (38).
- t_h equal to the value calculated by Formula (39): Recall Formula (6) and (12) from section 4.1.2, which compute the total F/E time T_{total} for a process with valves being lifted linearly in time and for a process with instantaneously lifting (t_h = 0) of the valves. Substitute Formula (39) in Formula (12) to find:

$$T_{total} = \frac{2A\sqrt{\Delta H}}{m_s f \sqrt{2g}} + \frac{1}{2}t_h \qquad (12) \qquad \Rightarrow \qquad T_{total} = 1\frac{1}{2}t_h \qquad (40)$$

The frame below shows a short hawser force calculation to illustrate the different results for head filling and longitudinal culvert filling, but, more important, to get a quantitative feel for the matter.

Hawser force calculation:	<u>Head filling:</u> $m_s = 0.85$
Using data from section 4.1.2: A = 1080 $[m^2]$ m _s = 0.85 [-] f = 5 $[m^2]$ g = 9.81 $[m^2/s]$ ΔH = 4.5 $[m]$ t _h = 180 [s] Some additional data required: A _v = 54 $[m^2]$ n ^{**} = 25 $[m^2]$ G ^{**} = 20200 [kN] / 2020 [ton]	$\frac{dQ}{dt} = -\frac{m_s^2 f^2 g}{A} = \frac{0.85^2 * 5^2 * 9.81}{1080} = 0.164 m^3 / s^2$ $\frac{dQ}{dt} = \frac{m_s f \sqrt{2g\Delta H}}{t_h} = \frac{0.85^* 5^* \sqrt{2^* 9.81^* 4.5}}{180} = 0.222 m^3 / s^2$ Governing $\frac{dQ}{dt}$ is the 0.222: $i = \frac{dQ}{dt} \cdot \frac{1}{g(A_v - n)} = \frac{0.222}{9.81^* (54 - 25)} = 0.8^* 10^{-3} (= 0.8 \%_{00})$ $F = iG = 0.8^* 10^{-3} * 20200 = 16 \ kN$
 Based on a 2.5 x 9.5 x 85 m³ CEMT class IV ship 	Longitudinal culvert: $m_s = 0.54$ (Figure 4-18) $F = \frac{m_{s,culvert}}{m_{s,head}} iG = \frac{0.54}{0.85} * 0.8 * 10^{-3} * 20200 = 10 \ kN$ (= 0.5%)
Mater	

Note:

Use of a longitudinal culvert system results in smaller hawser forces than use of a head filling system.

- The permillages are slightly smaller than the guideline figures mentioned in the next section. This agrees with the fact that only the hawser force due to translation waves has been computed. Other force contributions, e.g. impuls differences or friction, were not included.

Practical method to determine the mooring load

Often a vessel in a navigation lock will be stopped in its manoeuvers by putting the forward hawser around a lock bollard and the ship's bollards. By easing the hawser of one of the ship's bollards carefully, the hawser force has to be controlled by tightening or slipping of the free hawser end, the friction will slow the vessel down. It is a matter of definition, whether the hawser forces in the previously described process, should be considered as mooring load or as a berthing load because it slows down the ship. At rest, in the lock chamber or in the outer ports, emptying and filling the lock chamber results in loads on the vessel that have to be resisted by the mooring lines. Moored vessels with a mooring or hawser line configuration as shown in Figure 4-39 are not able to resist much transverse forces. The resistance to transverse forces is especially low when fixed lock bollards are used and the lines are continuously slipping to allow the ship to rise and fall with the changing water level. The preferred direction of transverse mooring forces is towards the wall (contact pressure between ship and wall, no hawser force).



Figure 4-39 Hawser configuration for different type of vessels in the lock chamber

Generally the smaller the ship (recreational craft) the smaller the number of mooring lines, and the larger the probability that only one hawser at the time is transferring the whole tensile force, or not, in case of slipping or breaking. For the lock the resulting mishap will be fairly negligable.

Inland navigation vessels are generally moored using 2 hawsers. The following is an indication for the mooring force to be expected in one hawser:

•	Ships up to	600	tonnes:	1,5 ⁰ / ₀₀ of the water displacement
•	Ships up to	2 000	tonnes:	1,0 $^{0}/_{00}$ of the water displacement
•	Push barge convoy up to	10 800	tonnes:	$0,7^{0}/_{00}$ - 1,0 $^{0}/_{00}$ of the water displacement

Mooring of seagoing vessels is different because generally four hawsers will be used, which will be held under constant tension by the winches of the ship. In conditions of transverse wind or water flow sometimes one or more extra hawsers will be used in the transverse direction (imagine the amount of time required to handle all the hawsers). An indication for mooring loads of seagoing vessels, mainly determined by the tension winch:

- $0,25^{0}/_{00}$ Vessels of over 20.000 DWT:
- $0,12^{0}/_{00}$ • Vessels of over 100.000 DWT:

During design the above indications for the mooring forces, related to the water displacement or DWT of the vessel, can be used. Obviously the governing ship has to be selected wisely, and this might prove to be difficult in case of large variations in the types of vessels using the lock.

Another approach would be to determine the governing mooring load based on the type of hawser most probably used. Instead of selecting a governing ship a hawser type has to be selected, which still requires the designer to consider one or more design ships, however, generally there is considerably less variation in the break loads of hawsers. The heaviest cables which can still be operated manually are steel cables with a diameter of 22 mm and a fracture load of circa 105 kN. If hand winches are present, like on push barges, heavier cables can be used. Nylon hawsers absorb more energy due to their larger elasticity.

Software used to determine the mooring load

The above calculations or methods are used to obtain a first estimate of the water level gradients caused by translation waves and the hawser forces. The maximum hawser force is either the result of reducing the vessel's speed when slowing down or mooring, or forces induced by filling and emptying the lock chamber. Seemingly more accurate calculation of the hawser force could be possible e.g. using the program named LOCKFILL. Figure 4-40 shows the graphical results of a calculation with the program for the forces in longitudinal direction.



Figure 4-40 Forces in longitudinal directions, calculated using LOCKFILL; during filling (upper graph), during emptying (lower graph)

Reduction of hawser forces

Different methods exist to reduce the hawser force:

a. Use of a longitudinal lock culvert:

A culvert with valves positioned in longitudinal direction will encounter the following problem. At the start the first valve will process most of the water due to the slow acceleration of the water mass in the culvert. This problem can be diminished by adjusting the valve opening. In this way the discharge will be divided more evenly thus reducing the longitudinal forces. For instance by positioning 2 valves at $\frac{1}{4}$ and $\frac{3}{4}$ of the chamber length, the chamber will be divided into four parts with a head filling of $\frac{1}{4}Q$ each. This is an optimal distribution to reduce the longitudinal forces. Additionally, the draw-off spots have to be placed as deep as possible to prevent water entering the lock to interfere with the moored vessels.

- Use of energy dissipating chamber:
 When large differences in the water level are present one can use an energy dissipating chamber to reduce the entrance speed of the water. By lifting the gate partially water will flow underneath the gate and enter a chamber. This chamber will reduce the flow velocity.
- c. Adjustment of the valve lift program: By adjusting the valve lift program the hawser force can be reduced. For instance one can start by slowly opening the valves, opening the valves serially or by temporarily interrupting the lifting. When the translation wave reflects at the opposing gate the lifting velocity can be increased.
- d. Use of many smaller fill openings instead of a few larger openings:
 - Smaller water jets dissipate their energy easier than large.
- e. Breaking the water jet:

The energy of the incoming water (jet stream) can be reduced by:

- The valve shape;
- Positioning breaker plates in front of the valve opening;
- Using a Venetian blind so one valve will open later in time.
- f. Use of a bed protection:

If water jet velocities permit a very effective solution to reduce the hawser forces is the use of a bed protection inside the lock chamber. The bed protection creates a lot of friction, which dissipates a lot of energy. However, it is quite an expensive solution as well.

4.1.4 Specific subjects regarding F/E

As announced in the beginning of this section F/E, water saving basins, salt water intrusion, ice control, and large water head difference phenomena, will either be discussed in this paragraph or provided with some references for further study.

Water saving basins / Side ponds

As explained in Section 2.1.1 every lock cycle water is lost in the upstream reach of the waterway and added to the downstream reach. The amount of water lost per leveling cycle, V_{loss} is:

$$V_{loss} = A \cdot \Delta H + W_{up} - W_{down} \tag{41}$$

where:

A = the horizontal water area of the lock chamber
 W_{up} = is the water displacement of the ship navigating in the upstream direction
 W_{down} = is the water displacement of the ship navigating in downstream direction

There are several options to reduce the water loss due to locking ships. For instance instead of constructing one large navigation lock, two smaller locks could be constructed. One large lock, leveling over ΔH , uses twice as much water as two smaller locks, leveling over $\frac{1}{2}\Delta H$, placed serially.

Another option would be to construct one or more water saving basins or side ponds *(spaarbekken)*. Part of the water will be stored in the basin(s) when the lock chamber is emptied. During filling the water from the basins will be discharged into the lock chamber. One side pond with a horizontal water surface equal to the lock chamber will save 33% water. During emptying of the chamber ¹/₃ will flow into the side pond and ²/₃ will be lost. The construction of two side ponds will result in a reduction of 50 %, see Figure 4-41.



Assume the horizontal water area A of the lock chamber and the water basin(s) are the same. Divide the water head Δ H, the water level difference between the upstream and downstream waterway, into an integer number of N steps. Therefore the lock chamber contains N water volumes with water height Δ H/N, 'above' the downstream water level. Let n be the number of water saving basins and let the water basin height be Δ H/N, hence equal to one of the N water volumes in the lock chamber. Given the condition that free flow is being used, N has to be larger than 3 because the upper water volume in the lock flows in the first basin, which is Δ H/N lower, and the last and lowest volume of water in the lock is discharged into the downstream reach, not in a basin. If this last volume was stored in a basin a pump would be needed to get the water back into the lock. The amount of water saved or lost per lock cycle is calculated as follows:

$$V_{saved} = A.\Delta H.\frac{n}{N}.100\%$$

$$V_{lost} = A.\Delta H.\frac{N-n}{N}.100\%$$

$$(42)$$

The graph in Figure 4-42 shows that there is little extra water saved using 4 or even more water saving basins.

Disadvantages of the use of water saving basins are the costs and the extended passage time.



Salt water intrusion See Appendix 2

Ice control See Appendix 3

(Both Appendices more or less correspond with text from the report PIANC, Innovations in navigation lock design, InCom-106, Brussels, 2009, (ISBN 978-2-87223-175-1).

Large water head difference phenomena

Water hammer and/or cavitation are phenomena that are closely related to larger water head differences.

Water hammer

To prevent overtravel in longitudinal culverts, the valve in the supply line will be shut when the discharge Q drops below a certain (low) limit. If a valve was immediately closed in a culvert the force, using a/the momentum equation, necessary to stop the water movement would be equal to:

$$F = m.a = m\frac{du}{dt} \tag{43}$$

Immediate closure would imply dt = 0 resulting in an infinitely large force. Fortunately the equipment used for closing the valve is not that fast. What does develop is a shock, however, the compressibility of the water and the elasticity of the culvert will reduce the impact. The shock wave (over pressure) travels back through the culvert to the upstream reach, where it is reflected by water surface of the infinitely large water volume and returns as an under pressure wave to the valve. The reflection process will continue as long as the wave energy is not dissipated by the friction along the culvert walls. The wave celerity c can be expressed as:

$$c = \frac{1}{\sqrt{\rho\left(\frac{1}{E_{bulk}} + \frac{D}{t_{wall}.E_{young}}\right)}}$$
(44)

Where:

c = wave celerity [m/s]

 ρ = density of the water [kg/m³]

E_{bulk} = bulk modulus of water; a measure for the compressibility of water [kN/m²]

D = pipe or equivalent culvert diameter [m]

t_{wall} = wall thickness of the pipe or culvert [m]

 E_{young} = Young's modulus of pipe or culvert walls [kN/m²]

The pressure increase due to sudden interruption of the flow is; Rankine (1870), Joukowski (1898):

$$\Delta p = \rho c U_0 \tag{45}$$

The pressure increase for slow closure of the valve, formula by Allievi (1929):

$$\Delta p = p_0 \left(\frac{N}{2} + \sqrt{\frac{N^2}{4}} + N \right) \tag{46}$$

$$N = \frac{\rho L U_0}{p_0 t_{valve}} \tag{47}$$

Where:

 Δp = pressure increase [kN/m²]

U₀ = water velocity when valve closes [m/s]

 p_0 = original (hydrostatic) water pressure [kN/m²]

lf:

$$\begin{split} \rho &= 1000 \text{ kg/m}^3 \\ E_{\text{bulk}} &= 2.18 * 10^6 \text{ kN/m}^2 \text{ (for } 20^\circ \\ & \text{Celcius)} \\ D &= 5 \text{ m} \\ t_{\text{wall}} &= 1 \text{ m} \\ E_{\text{young}} &= 10 * 10^6 \text{ kN/m}^2 \text{ (cracked concrete)} \end{split}$$

then:

c = 1021 m/s

Travelling back and forth in a 300 m long culvert, takes 2*300/1021≈ 0.6 s

Suppose:

$$U_0 = Q_0/A = 25/20 = 1.25 \text{ m/s}$$

then for sudden closure:

$$\Delta p = \rho c U_0 = 1000 * 1021 * 1.25 \quad \Rightarrow$$

$$\Delta p \approx 1.25 \ .10^6 \ kg \ /ms^2 = 1250 \ kN \ /m^2$$

If:

L = 300 m

$$t_{valve}$$
 = 180 s
 p_0 = 200 kN/m² ($\Delta H^* \gamma$ = 20*10)

then for slow closure:

$$N = \frac{\rho L U_0}{p_0 t_{valve}} = \frac{1000 * 300 * 5}{200 \ 000 * 180} = 0.042$$
$$\Delta p = 200 \left(\frac{0.042}{2} + \sqrt{\frac{0.042^2}{4} + 0.042} \right) \Longrightarrow$$
$$\Delta p = 45 \ kN / m^2$$

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Note:

- Given a wave shock celerity in the order of magnitude of hundreds meters per second, it is safe to assume that the return shock wave arrives back at the valve when the closing process is still continuing. As a result Formula (46) for 'slow' closure of the valve has to be used for computation of the extra pressure on the valve due to water hammer.
- In the exceptional case that the time needed for closing the gate is smaller than the time needed for the shock wave to travel back and forth, then Formula (45) for 'sudden' closure has to be used to find the extra pressure on the valve due to water hammer.
- An over pressure shock wave travels back in the opposite discharge direction, however, at the other side of the valve an under pressure wave travels in the same direction as the discharge. Once reflected by the downstream water mass, this wave returns as over pressure to the valve.

Cavitation

Some times the flow velocity can become so high that the local pressure decreases to below the vapour pressure, or the negative shock wave due to valve closure results in under pressure. Below the vapour pressure bubbles filled with vapour will develop in the fluid. When these bubbles reach places where the pressure is higher again, they implode and cause damage to structures and equipment (concrete or steel lining, valves, etc). This phenomenon is known as cavitation; some characteristics of cavitation:

- unstable flow pattern;
- less efficient discharge through the culverts (loss coefficient cavitation to be introduced);
- vibration of the valves and their lifting equipment;
- considerable wear of the valves etc., or the concrete culvert walls (cavitation erosion);
- a lot of noise.

It is possible to reduce cavitation at the inlet/outlet valves by adding air to the water. Damage can be reduced by linings made of steel or the use of high strength concrete with steel fibers. An additional measure which has a positive effect is by widening the culvert down stream. The implosion of the bubbles will occur further away from the culvert or lock wall.

References on water hammer and cavitation:

- Fluid-structure interaction in case of waterhammer with cavitation, A.S. Tijsseling, Doctoral thesis Delft University of Technology, 1993
- Elementary Hydraulics, J.F. Cruise, M.M. Sheriff, V.P. Singh, Thomas Nelson, Student edition, 2007
- Water-resources engineering, D.A. Chin, Pearson Prentice Hall, Upper Saddle River, New Jersey, 2nd Edition, 2006
- Pipeline design for water engineers, David Stephenson, Elsevier scientific publishing company, Amsterdam, 1976

4.2 Lock Gates

In history, when locks were constructed with masonry (*metselwerk*) walls, gates were constructed from wood and kept as small as possible. Good reasons to use a small gate were the limited strength of the wood, lack of the (mechanical) power that is needed to move a large gate, and costs. Frequently locks were constructed with gates having a span smaller than the width of the lock chamber, see the mentioned bayonet lock in Section 3.1.2. The historical technical limitations do not exist anymore, moreover, given nowadays economic importance of waterway transport, those limitations would not be accepted anymore. Steel, even concrete or synthetic gates, are simply designed and constructed up to the size required. The time needed to pass a navigation lock has become a dominating factor and as a result reduction of the time to operate the lock gate as well. Mechanical and electrical devices, taking care of the movement of the gate, nowadays have been developed to such an efficiency level, that it takes 3 minutes on average, to open or close gates for both inland and coastal locks, see Table 2-4.

4.2.1 Gate function

A gate *(sluisdeur)* in a navigation lock will function as a means or an element, able to separate as well as connect different bodies of water. The following functions and design requirements can be defined:

Water retention function:

The separation of bodies of water most frequently having a different water level requires:

• Prevention of water flowing from a higher water level to a lower water level. Although the gate closes off by far the biggest part of the wet cross section of the lock head, at the wet perimeter of the gate, leakage and seepage will occur.

Depending on project specific requirements limited leakage and seepage will or won't be allowed. In coastal areas strict requirements are often imposed to maintain a separation between salt and fresh water.

• Resist the resulting hydrostatic pressure caused by the difference in water level on either side of the gate and transfer this load to the supports at the lock head. The direction of the resulting hydrostatic load is always from the higher water level side to the lower water level side; hence, the direction changes when the water level on the other side gets the highest. Difference in direction can be caused e.g. by tide. If needed the gate has to be able to resist the force in both directions.

Separation or connection function:

- In view of navigation, water bodies must be separated on one side of the lock, see retention in the above, as well as connected on the other side of the lock. In general simply opening or lifting the gate results in too large and too turbulent water movements either in the lock chamber or in the lock approach areas, which means an existing water level difference has to be leveled out. Water movements, more specific the resulting mooring forces are not allowed to exceed a certain threshold. A means to take care of acceptably quiet water movements is to provide openings that can be gradually adjusted in size.
- It has to be possible to move the gate completely out of the cross sectional area required for navigation to provide safe passage for navigation.

4.2.2 Gate types

Using the most important functions and resulting requirements, first to generate, then to select gate alternatives, generally speeds up the overall design process. For navigation locks, the way to move the gate out of the required cross sectional area for navigation is quite a distinguishing criterion. Gate movement often is a translation or a rotation, hardly ever a combination of the two. Given the three dimensions there are 6 theoretical movement possibilities. Looking at the most commonly used gates, descriptions in further detail will follow, in real life only 4 of these possibilities are frequently in use, see Table 4-4.

1

	Translation	Rotation axis	Used						
	direction								
Mitre gates (puntdeuren)		vertical	frequently						
Single leaf gate (enkele draaideur)		vertical	frequently						
Lift gate (hefdeur)	vertical		frequently						
Submersible lift gate (zakdeur)	vertical								
Rolling-, sliding or caisson gate	horizontal		frequently						
(roldeur)									
Radial gate or sector gate		vertical							
Radial gate or tainter gate		horizontal							

Table 4-4	Gate	types	and	aate	movements
		.,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	~	90.00	

Mitre Gates (puntdeuren)

Mitre gates *(puntdeuren)* consist of two leaves, symmetrical with regard to the centre line of the lock, rotating around the vertical axis. The force resisting principle is very simple; in closed position both leaves point in the direction of the upper water level. On a global force level the water pressure results in normal forces in the gates and spalling forces acting on the lock head. On a local force level bending moments and shear are introduced in the doors as well.

Mitre gates generally should not be exposed to significant current or wave actions when partially open. For this reason mitre gates are unsuitable for use as a weir (*stuw*) or flood gate (*vloed deur*) but are frequently used as lock gates. In some cases, when for instance hydraulic cylinders are used to open en close the gates, the gates can be designed to resist a small reverse water head difference.

In open position the gates are turned in recesses in the walls of the lock head. In order to prevent damage when vessels navigate through the lock head, there should be enough distance between the gate and the face of the recess wall, or fenders should be attached either to the gate or the face of the recess wall.





Lock axis cross-section



Bits and parts of the mitre gate:

Stresses due to the weight of the gates are transmitted to the heel post *(achterhar)* and then via the pivots to the collar strap *(halsbeugel)* and socket *(taats)*. All the items must be designed to resist the horizontal forces; the bottom pivot and socket also carry the whole (underwater) weight of the gate. The axis of rotation of the gate should be in such a position that, on one hand, separation of the gate and the concrete

of the recess walls is ensured during the whole move from open to closed gate position and vice versa. On the other hand, to retain water in closed position the space between the gate, hence the axis, and the recess wall should be as small as possible to limit water flow around the gate. To accomplish both, a solution would be to choose an eccentric position for the gate in the gate, i.e. at some distance from the centre line.



Figure 4-44 Parts of (old) wooden mitre gates *

* Courtesy of PIANC: http://www.waterdictionary.info/

Moving the gate:

Generally speaking, the equipment for operating the gates is located above the highest upstream water level and as near as possible to the lock wall (so there is a danger of being damaged by downstream bound ships). Operation of a mitre gate, results in torsion stress which should be taken into account when designing and dimensioning a gate. Some basic information on the equipment will be presented below; the civil engineer has to keep in mind that design of the equipment, including all the necessary mechanical and electrical items, the installation and control systems, is a job on its own, best done by engineers with relevant experience.

Electro mechanical:

The so called "panama wheel" is most widely used in locks to operate the gate. The wheel is operated through a set of gears put into motion by an asynchronous motor. In spite of the constant speed of the gear wheel the rate of angular displacement of the leaf is reduced at the beginning and at the end of the operation, when stresses to be resisted may be the largest.

Hydraulically (oil):

Today almost exclusively hydraulic cylinders are used in new design. A hydraulic jack fed by a volumetric pump and attached to a frame in its centre, acts on the piston rod, the end of which is connected to the gate. The jack is positioned at 1/4 to 1/2 of the gate length from the axis of rotation.



Figure 4-45 Position of hydraulic jack moving the mitre gate

Advantages of mitre gates are:

- No air draught limitations. High superstructures such as lifting towers or pillars are not needed.
- Low cost, effective solution for smaller locks. The three-hinged mechanics scheme results in a relatively light gate structure.
- The equipment for mitre gates does not have to carry the weight of the gate; the centre of gravity remains in a horizontal plane during opening and closing. Resulting stresses during operation are reduced to those due to frictional forces at the pivots, socket and collar strap and the dynamic effect of moving the gate through the water.
- Opening and closing of the gates takes a short time.

Disadvantages are:

- The length of the lock chamber needs lengthening as well as the upper head by the recesses of the gates.
- To retain water in both directions a double set of doors is required.
- Very accurate mounting and frequent checking of the contact at points of load transmission (thrust blocks and quoin blocks) is necessary.
- Mitre gates can not be opened or closed under a water head difference
- Operation may be disturbed by debris and ice.

For increasing widths of the sluice, the type of material and the design of gates changes. Mitre gates *(puntdeuren)* which are often constructed in wood become less suitable and are replaced e.g. by steel rolling gates. The width of mitre gates is limited to approximately 20-25 meter (10-12.5 m per door). What is especially important for this is the height width ratio. When the ratio is smaller than 1 the forces on the pivot points are relatively large and may become a problem, especially the (horizontal) force on the upper one.

Advantages of mitre gates often outweigh their disadvantages as is demonstrated by their utilization throughout the world. However, since vessel size is increasing this type is less frequently applied. For reasons of construction and maintenance costs (and sometimes the status as monument) the gates of small and medium size of navigation locks are made of either steel or wood. A new development is the use of glass fiber reinforced plastics (GFRP), which have little need for maintenance.

Single leaf gate (enkele draaideur)

The single leaf gate *(enkele draaideur)* could be considered as half a mitre gate, their appearance, shape and materials used are similar, however, there are some significant differences. Where the mitre gate transfers the hydrostatic load for a considerable part by normal forces, the single leaf gate brings it all away by bending moments. The recess in the wall for the free end of the leaf gate has a different shape, see Figure 4-46, which is a bit better for streamlining water flow.

Gate recess

Single leave gate



If selected at all, the single leaf gate is most frequently used for locks having a small width, e.g. for the locking of recreational craft.

Mitre gate

Gate recess

Figure 4-46 Single leaf gate compared with a mitre gate

Advantages

- Very suitable for locks of smaller width because of simple construction and operation.
- No air draught limitations.
- It is possible to lock the door on the free end which will make it possible to retain water in two directions.
- Forces on the gates are transferred parallel to the lock wall (when the closed leaf gate is perpendicular to the lock axis).

Disadvantages

- Recesses for single leaf gates are longer than those for equivalent mitre gates, thus single leaf gates need longer structures.
- Opening and closing of the gate results in a lot of water displacement.
- More rigid and thus heavy construction needed for the pivots, socket and collar strap.
- Vulnerability for ice ad debris.
- Not suitable for wide locks because the supports will be very heavy.

Radial gates with vertical axis of motion (sectordeur)

Radial gates with a vertical axis of rotation are often referred to as sector gates. The skin plate of the gate has the curved shape of a (part of) a cylindrical circle, and has to be stiffened to resist the hydrostatic water pressures, see the Figures in this and the following paragraph. The lower and upper gate arms,

triangular shaped trusses in the horizontal plane, support the stiffened skin plate horizontally and vertically. Between the upper and lower arm there will be diagonal braces or struts, see Figure 4-47, to provide the required vertical stiffness. At their rear ends the gate arms transfer the loads to pivot points, which are attached to, better said casted into, the lock head wall. Due to the shape of the skin plate the resulting force of the hydrostatic pressure has its working line through the pivot.

Sector gates have the advantage of being statistically determinate, can resist reverse heads and can be operated whether there is a water head or not. The weight of the gates



Amagasaki City, Japan, 2003

Figure 4-47 Sector gate during construction

may exceed that of the equivalent mitre gates, making them more expensive. Another cost setback are

the large and deep recesses in the lock heads. Sector gates are more often used for guard locks or storm surge barriers where closure under free flow and unlimited air draught is needed.

Moving the gate:

Similar to mitre gates, sector gates can be pushed into movement using mechanical equipment, e.g. machinery or hydraulic jacks. The driving force can be applied to the upper gate arm. Contrary to mitre gates, those have to push away the water body in front of the gate, the sector gate cuts through the water with its edge. Therefore the installed power of the equipment used is relatively small, even though the gate itself is heavier. Again, the design of the equipment to be used, including all the necessary mechanical and electrical items, the installation and control systems, is best done by engineers with relevant experience. Note: the space for the recesses, the equipment and the 'wiring' that has to be provided, generally requires quite a lot of detailing work to be done during concrete structure design and results in quite some complications during construction due to all the required cast-in items.

An alternative for the use of mechanical devices, to move the door, is using the hydrostatic water pressure difference due to differing water levels on either side of the gate. Extra skin plate would be required at the straight face of the sector gate closest to the lock head wall, see Figure 4-48. With appropriate sealing along the edges this plate closes off the water body in the recess. If the water level in the recess is the highest, the sector gate moves out, vice versa if the water level in the recess is lower than in the lock chamber or lock head the gate will move into the recess, i.e. open. To change the water levels in the recess, water will be let in



Figure 4-48 Horizontal cross section sector gates

or out through the culverts either by means of a natural water head or using pumps.

Advantages of sector gates are:

- The possibility to close the gate in flow conditions.
- Relatively easy to move because it cuts through the water with a small surface.
- No air draught limitations, hence no lifting towers or pillars.
- The equipment for operating the gates does not have to carry the weight of the gate; the centre of gravity remains in a horizontal plane during opening and closing. Frictional forces at the pivots and the sealing result in horizontal forces.
- Opening and closing of the gates takes a short time.
- It is possible to retain water in two directions.

Disadvantages are:

- Relatively large amount of material because of the developed length of skin plate and the gate arm trusses that have to be braced.
- High support loads at the pivots resulting in heavy cast-in items in the lock head wall due to weight of the gate. To mitigate this disadvantage float elements assembled to or integrated in the gate may be used.
- The recesses required to move the gates out of the way are large and substantially increase the size of the lock head structure.
- Operation may be disturbed by debris and ice.

Note:

If the sector gate is moved using hydrostatic water pressure some of the advantages and disadvantages mentioned for sector gated in the hereinabove, become bigger, others may even be reversed.

Radial gates with horizontal axis of rotation

Radial gates with a horizontal axis of rotation are frequently known as tainter gates.

Reference is made to the descriptions of the sector gate in the above. Compared to sector gates the tainter gate:

 A single tainter gate spans the whole width of the lock head opening. Generally two sector gates are used.

Lifting to open for navigation:

- In lifted position there will be a limitation to the air draught of passing vessels.
- The equipment has to be able to carry the weight of the gate. Counter weights may be used to mitigate this disadvantage.

Lowering to open for navigation:

- Lowered there is no air draught limitation
- To lower the gate a deep recess in the lock head is required, significantly



Figure 4-50 Hydrostatic forces (left); Forces during lifting and lowering (right)

increasing the lock head structure, complicating construction because of the much deeper foundation, hence considerably increasing costs.

Again, the equipment has to carry the weight of the gate. Besides counter weights, floating elements
assembled to the gate may be used as well to mitigate this disadvantage.

It is possible to use this type of gate for guard gates, flood defense or storm surge barriers, provided the gate body is designed high and strong enough to resist underflow or/and overflow.



Figure 4-51 Radial or tainter gate during construction (left & middle); hydraulic jack (right)

SIDE-SEAL FRICTION

RAISING

MASS OF GATE

Lift gate in upward direction (hefdeur)

Lift gates spanning the entire width of the lock chamber and operating in vertical recesses enable hydrostatic loads to be transmitted to the lock head. Lifting the gate opens the lock. The deadweight of the gate is balanced by counterweights in order to reduce the operating forces of the equipment. High gantries or guide towers (*heftorens*) are required in most cases to guide the gate when moved into open position. This is illustrated in Figure 4-52.



Figure 4-52 Lock head for a lift gate

The advantages of lift gates are:

- Little space is needed for the lock head, thus enabling the total lock length to be reduced to a minimum.
- Considering the driving gear, it is possible to open the gate under a water head because the force directions are in different planes.
- Not seriously affected by debris and ice in closed or opened position.
- Easy to control and repair.
- The support system of the gate is statically determinate and therefore largely insensitive to differential settlement.
- It is possible to retain water in two directions.

Operational disadvantages are:

- Air draught limitation of vessels.
- The superstructure, lift towers or columns, is large and heavy.
- Lifting the whole gate results in very strong underflow.
- Spillage of water on vessels passing the gate.
- Difficulties with the roller tracks because of the deflection of the bottom of the gates when they are under full head.
- Balancing the gate may be complicated and expensive. For opening, floating ice and debris, as well as ice frozen to the gate may increase the weight of the gate and negatively influence the balance with the counterweights.

Submersible lift gate (zakdeur)

The submersible lift gate *(zakdeur)* is lowered for the passage of ships. The first phase of lowering can be used for filling and emptying the chamber. For purposes of repair the gate is lifted above the water level. This type of gates takes little horizontal space and has the advantage that vessel height is not limited.

However, it requires a large recess deep into the ground to accommodate the gate and its construction and maintenance costs are high.





Figure 4-53 Cross-section of a submersible gate

Most of the horizontal loads on the gate are diverted to the side recesses; a small amount is transferred to the bottom support (onderaanslag). The submersible gate is able to retain water in 2 directions.

Caisson, rolling or sliding lock gate (roldeur)

The caisson, rolling or sliding lock gate (roldeuren) is preferred where large spans have to be bridged without limiting the air draught. This type of gate can only be used when sufficient space exists besides the lock, because the width of the lock head structure is about two times the doors length, see illustrations in Figure 4-54.

The rolling gate moves on underwater rails or sliding tracks, which bears the risk of the gate getting jammed due to accumulation of sediments or debris on those tracks. Especially if the gate frequently remains in the open position it should be considered to wipe the rail or track clean before or during gate operation. Moving the gate, reducing weight and frictional forces, thus reducing wear and tear as well, is easier when buoyancy tanks are used. When the gate is in closed position the tanks can be filled, now they serve as ballast tanks, to ensure the necessary contact pressure.



Figure 4-54 Rolling lock gate and its lock head; cross sections and 3D impression

A combination of factors results in the generally considerable thickness of rolling gates. Obviously one factor is the large width of the lock to be spanned by the gate. The need for stability results in wheels or slides on both sides of the gate and two rails or sliding tracks under one door, not just one in the center of the door, which adds to the thickness. Ever so often the thickness of rolling gates can be used to provide passage to light traffic.

In closed position the rolling gate can be considered as a simply supported beam; in fully opened position there are hardly any horizontal forces on the door. In those positions stability of the gate is not a problem. During the opening or closing process the 'upper' corner of the gate is completely unsupported. A force on this door area, e.g. due to a remaining water level difference or large flow, may result in significant stability problems. Keeping the resulting vertical force within a distance of 1/6 of the gate's thickness to the centre line of the gate is a good measure for providing stability, but requires a certain thickness of the gate.

The advantages of rolling lock gates are:

- Suitable for locks of large width.
- No air draught limitation.
- It is possible to retain water in two directions.
- Easy to maintain.

Disadvantages:

- Requires a lot of space besides the lock (head).
- Requires a large recess or lock head.
- Can not be opened or closed under water head conditions.

Innovative gate types

The following has been included to show some developments and/or possible gate innovations; it is not proven technology what is shown, in fact there is a good chance the idea will never leave the drawing board or design stage. And of course gate innovations are not limited to what is shown here.







a) Curved rolling gate (plan)

Figure 4-56 Innovative gate types



c) Inflatable valve



d) composite crash barrier

4.2.3 Transfer of forces

As an example the transfer of forces through a mitre gate will be discussed. The following items have to be analysed:

- a. Identify possible (hydraulic) loads and load combinations and calculate their magnitude
- b. Schematisation and selection of supports (opleggingen)
- c. Determine the load transfer through the gate or door

a. Identify possible loads and load combinations and calculate their magnitude

To identify loads and the combinations it helps to look at the gate in opened and closed position, or consider the situation when the gate is being moved.

- 1. Load on a mitre gate in closed position:
- 2. Load on a mitre gate in opened position:
- 3. Loads on the doors due to debris when trying to close
- 4. Loads on the mitre gate subjected to ice
- 5. Loads on mitre gates due to ship impact
- 6. Loads on a mitre gate under wave attack
- 7. etc.

These situations and loads combine to a number of load combinations. As example combinations 1 and 2 will be elaborated in the hereinafter.

b. Schematisation and selection of supports (opleggingen).

When schematising and selecting the type of supports for the gate, not only the transfer of forces has to be taken into consideration, but preventing leakages or flow under or around the closed gate as well.

In opened position the gate is simply supported, see left-hand side illustration in Figure 4-57. The bottom pivot provides both the vertical reaction and a horizontal reaction force. The upper pivot provides the horizontal reaction in the opposite direction. Rotation in the horizontal plane around the axis through the supports is possible. Looking at deformations globally, the gate will get out of square *(schranken)* if it is not stiffened sufficiently in its plane. This is a problem if there is too little clearance *(speling)* between the bottom of the gate and top of the lock head floor. In the act of opening or closing the gate, the bottom of the mitre post *(voorhar)* would touch and scratch the floor.

Obviously leakage of water or sealing is not an issue at al when the gate is open (or being opened).



Figure 4-57 Supports, loads and resulting deformations

When closed and retaining water, there are two (main) alternative systems to support a mitre gate. In the first alternative, in the middle in Figure 4-57, the mitre gate has a vertical support at the bottom pivot point *(taats)*. Horizontally there will be a line support at the mitre post *(voorhar)*, and either one or two simple supports at the heelpost *(achterhar)*, the last depending on the clearance in the bottom pivot shoe.

To be able to install the gates, there has to be a certain clearance or construction tolerance available in the bottom pivot point. After installation of the door into the lock head, little can be done about the bottom pivot or the available clearance. Often the upper pivot is readjustable to facilitate installation, which is also convenient for later operational stages.

It is possible to design and construct the pivot shoe and socket (bottom) with such a large clearance that the heel post is being pushed against the concrete wall of the recess by the water pressure. Suppose enough (horizontal) clearance is available in upper pivot as well, than the heel post touches the wall over its entire length. This is the second support alternative in closed position. In that case the gate has line

- e.g. hydrostatic water pressure
- e.g. dead weight

supports on three sides for horizontal loads, the fourth side, the top girder, is free or unsupported, see Figure 4-57 on the right.

For both alternatives supports and loads have been drawn at the right-hand side door, the deformations have been indicated at the left-hand side door. The deformations of the gate are important for analysing and preventing the leakage of water, usually done by providing sealing between the gate's edges and the 'surrounding' concrete. Hand in hand with larger shear and torsion stresses, that are bigger for simply supported plates than for line supported plates, go larger deformations of the gate that result in more leakage. Larger deformations and leakages require larger seals, see Section 4.2.4, which increases initial and maintenance costs.

c. Determine the load transfer through the gate or door

Sketch how forces acting on a structure are being transferred. In case of a mitre gate draw up how the loads are transferred by beams, girders, the axis and hinges to the lock head.

This time consider a closed mitre gate first: the governing load to be considered is the maximum water level difference.

Global force transfer

From construction mechanics point of view, the V-shape of the two leaves is a statistically determinate three-hinged truss, which transfers the water pressures by normal forces to the supports in the gate recesses; this is the global or overall force action. Locally the water pressure results in bending moments in the truss.

Figure 4-58 shows a top view of forces acting on a closed mitre gate, only one of the doors has been included, as well as the force vector diagrams for decomposition of the main forces. The resulting hydrostatic load W is resisted by the force H, the force between the 2 doors of the mitre gate, and spalling force S (*spatkracht*), the force acting on the lock head and vice versa. For reasons of symmetry the forces H between the 2 doors, in the point(s) of contact of the mitre posts (*voorharren*) are in equilibrium. They are of the same magnitude but their direction is exactly opposite. Both H and S can be split up in a direction parallel (N) and a direction perpendicular (F) to the door.



Figure 4-58 Forces and reactions on a closed mitre gate

Since the reaction force S can be divided in a force parallel and a force perpendicular to the gate these forces are a function of the hydrostatic pressure W and the angle α . Expressed in formulas:

$$S = H = \frac{W}{2\sin\alpha} \qquad \qquad F = \frac{1}{2} W \qquad \qquad N = \frac{W}{2\tan\alpha}$$

From the above formulas it can be derived that for small angles α the spalling force is high. This would suggest using a larger deflection angle α . However, when increasing this angle the gates will become larger, therefore requiring a heavier structure, but the soliciting load will be larger as well, hence the gate will be heavier. For reasons of economics usually an angle of tan $\alpha = 1:3$ has been determined as the most optimum one.

As shown in the previous section, the spalling force S at the heelpost, or F and N, may work as a concentrated load *(puntlast)* or as a distributed load on the lock head structure and gate, action is reaction, depending on the support conditions. The same goes for the mitre post; depending on the detailing of the gate the force H will either be more concentrated or distributed.

Local or internal force transfer

The flow of forces through the gate, the local or internal force transfer, is strongly influenced by the type, the arrangement or composition the girders and posts of the gate. The 'local' arrangement has to be in agreement with the 'global' situation, ie. the support conditions of the gate, see previous paragraphs. For the girder and post arrangement the following two main options exist:

1. Load transfer equally distributed along the vertical posts, illustrated in Figure 4-59. The gate will be constructed using a relatively large number of horizontal girders (*regeldeur*), more or less equally spaced, which results in an equally distributed load on the heel and mitre post. The horizontal water pressure is predominantly carried away by horizontal internal forces. In agreement with the equal distribution the pivots should have quite some clearance to allow contact, either direct or via sealing, of the heel post along the whole height with the lock head wall. As such, load transfer along the post of the lock head is via distributed line loads N and F.



Figure 4-59 Flow of forces in a gate with (many) horizontal girders

2. Load transfer concentrated in a view points, see Figure 4-60. Instead of a predominant horizontal force flow, a larger part of the load is flowing to the supports via vertical routes. The gate will be constructed using two heavy girders (*regels*) between the heel and mitre post. The water pressure is transferred via plates and subsequently posts (*stijlen*) to the main girders. Positioning of the 2 heavy girders should be such that they carry away about the same load and such that there is an optimum between the bending moments in the heelpost and the other posts.

Both the heel and mitre post are simply supported, the heel post by the upper and bottom pivot, the mitre post will be provided with two contact points or surfaces that can be less or more pronounced. No, or only very little clearance will be allowed in the pivots at the heel post. In view of nowadays abundant availability of high precision (with very little clearance) ball bearings (*kogellagers*), being able to resist large loads, this is not a problem anymore.



Figure 4-60 Concentrated flow of forces in a gate – simply supported

Given the availability of appropriate ball bearing supports, it is more straightforward and cost effective to use the principle of global and local force transfer through concentrated flow of forces and simple supports.

Now consider the open gate again.

The governing load in, when the gate is in an open position, will be the dead weight of the gate. Unless a very special upper pivot point is designed and constructed (this was done for the gates of the Oranjesluizen in Amsterdam), the downward dead weight will be resisted by an upward reaction force in the lower pivot point. The destabilizing moment to be resisted will result in 2 equally large horizontal forces in opposite directions; tension on the upper support and compression on the lower support, if the lock head is taken as reference. This is illustrated in Figure 4-61.



Figure 4-61 Balance of forces for an open gate

When the size of the gate increases, the gate will tend to get out of square *(schranken)* due to its dead weight. This is a problem especially for gates made of wood, because wood elements are usually limited to a certain size and need a lot of connections. From a stiffness or deformation point of view the welding of steel is much more convenient. The shear deflection of wooden mitre gates can be limited by using beams or girders in a diagonal position, i.e. by using (wooden) struts. A steel tension bar is a better solution because it adds considerably less weight. This is illustrated in Figure 4-62.





Steel gates with appropriate stiffener arrangements are much stiffer than wooden gates, and getting out of square is rather easily solved, if it would be a problem at all.

4.2.4 Sealing

Retention of water is one of the main functions of the lock and the gate is the means to this end. To retain all the water the gates must be connected completely watertight to the surrounding lock head structure, and the doors of e.g. sector or mitre gates should not be left ajar (*op een kier*). However, to move the gate there must be some clearance between the gate and the lock head and to leave a small gap between two doors could be better than running the risk of damaging then during closure. To prevent the flow of water through these clearances and to allow movement of the gate as well, sealing (*waterafdichting*), generally (soft) wood or rubber profiles attached to either the gate or the lock head, is being used.

For mitre gates the positions to use a seal are:

- Between the heel post (*achterhar*) of the mitre gate and the wall of the lock head; this seal runs in vertical direction.
- Between the lower girder of the gate (*onderregel*) and the lock head floor, or lock bottom; this is a horizontal seal.
- Between the mitre posts (voorharren) of the 2 doors of the mitre gate; another vertical seal.

Sealing between the lower gate girder (steel) and the chamber floor:

On the bottom girder of the gate two steel strips are fillet welded *(hoeklas)*, see Figure 4-63. Between the strips a hollow rubber profile reinforced with a steel U-profile is assembled to the gate by means of a long bolt running through the welded strips, the U-profile and the rubber of the seal. A nut on the other side, fixes the whole seal to the gate. The U-profile keeps the rubber material in shape and position in case of gate displacements perpendicular to this cross section. When the gate is closed the rubber profile makes contact with a metal strip casted into the mitre sill, and only needs a little compression to prevent water seeping through.





Sealing between the heel post of a wooden mitre gate and the lock head:

In Figure 4-64 on the left-hand side, the heelpost of a wooden mitre gate door and the posts on the wall of the lock head are shown. If there is sufficient clearance in the pivots, the door is firmly pushed against the posts on the wall. The usually hard wood heel post is not equally stiff in both directions, as a result sealing is not perfect at the spot where line support N works. Using a compressible seal is in conflict with transfer of the line force N. Use of a soft wood seal, pine or oak (grenen of eik), at the spot where line force F works solves the leakage problem. On the right-hand side of the Figure a sketch to illustrate what is the result, considering the displacement of points of the heel post, of the eccentric position of the axis; eccentric to the heel post centre. Especially when there is little or no clearance in the pivots such a sketch is better used to determine the best position for one or two seals. In this situation the advise is to use rubber sealing which is more easily compressed.





Figure 4-64 Top view on the heel post of a wooden mitre gate and the post on the lock head

In both figures in the hereinabove the steel strips (*aanslagstrippen*) on/at the surface of the concrete, have steel anchors at the back to cast them into the wall. The usually stainless steel strips have to be positioned with more than the usual accuracy. It is not uncommon to cast the biggest part of the concrete walls first using block outs, which keeps hollow spaces free in the wall. In a later stage anchors and stips are positioned in the block outs and on the wall and carefully casted into the wall.

4.2.5 Gate selection

When selecting a gate not only the gate has to be taken into consideration, but the surrounding lock head structure as well. Although costs will be used as a criterion in a gate selection procedure, the first to be considered are the criteria derived from functional requirements. The following could be taken into account when selecting the most suitable gate:

a. Type of navigation lock and lock width:

For a first 'rough' selection of gate alternatives, i.e. on a general level without considering any specific local conditions or a special ship traffic situation, Table 4-5 could be used.

Type of lock	Lock width*		Mitre gate		Single I	eaf gate	Rolling gate	Vertical	lift gate
		Single sided	Double sided	Double sided	Single sided	Double sided	Double sided	Single sided	Double sided
		water retention	retention	water retention	water retention	water retention	water retention	water retention	water retention
		<	$\langle \rangle$	\langle					
Single	Very small				Х				
sided water	Small	Х			Х				
retaining inland	Medium	X						Х	
lock	Large	Х						Х	
Double	Very small					Х			
sided water	Small		X	Х		Х			
retaining inland	Medium		X	Х		Х			
lock	Large		Х	Х			Х		Х
Daubla	Small		Х	Х		Х			
sided	Medium		Х	Х		Х			
retaining	Large		Х	Х			Х		
sea lock	Very large						Х		

Table 4-5 Gate selection per type of navigation lock and depending on lock width	Table 4.5 Gate selection per type of payingtion lock and depending on lock with
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* Lock width explanation:

٠	Very sma	ll 4-6 m	٠	Medium	10-16 m
•	Small	6-10 m	٠	Large	16-24 m

Very large >24 m

b. Retention of water in one or both directions:

Mitre gates are only able to retain water in one direction. When water retention is required in two directions a double set of mitre gates must be installed, or special measures must be taken to equip a single set for this task. A double set of mitre gates results in an increased length of the lock heads. Rolling or sliding gates, or lift gates are possibly a more suitable solution.

- c. Opening and closing of gates under head difference:
 Unless special precautions are made, mitre gates and rolling gates should not be opened or closed under a water head difference for reasons of strength and stability.
- d. The area available for lock heads:

Often the lock head for a lift gate requires the least of area and is relatively easy to construct, however, the big disadvantage of the lift gate will always be the limitation of the air draught of the passing

vessels. Due to their height lift gates often are a landmark, which is difficult to fit in harmonically with the surrounding environment. Except the submersible gate, all the other gate types have a larger horizontal footprint, relatively spoken, than the lift gate. The construction and maintenance cost disadvantage of submersible gates generally overrides the horizontal space advantage.

e. Width of the lock chamber:

Lift and rolling gates are able to span the full width of a lock chamber. However, the bigger lock width and larger gate go hand in hand with a larger and heavier lock head structure. Since the 2 doors of mitre gates have about half the size of the chamber width this will results in lighter lock head structures.

f. Equipment to operate or move the gate:

The centre of gravity of lift gates is moved vertically, this requires a large amount of power, thus considerable machinery. Although this can be reduced significantly by the use of counter weights, a certain price has to be paid with regard to machine power on one hand and the required strength and stiffness of the concrete structure on the other hand. The same will apply for a submersible gates and sector gates with a horizontal axis.

Mitre, rolling gates and sector gates with a vertical rotation axis move in horizontal direction, which is less demanding for equipment. The installed power and the equipment will be smaller.

g. Vulnerability for ship impact

Supports of mitre gates are vulnerable for ship impact. Usually both the gates as well as the supports are damaged severely. Even in opened condition the door might suffer from ship impact. By their more sturdy structure, rolling or sliding gates are better equipped to resist vessel impact. As an additional advantage a rolling or sliding gate cannot be damaged when it is opened because at that time the gate is completely protected in its recess.

h. Inspection, maintenance and repair:

Moving or sliding parts of a gate are subjected to quite some wear and tear, therefore regular inspection, maintenance work and repair are necessary. Lift gates are lifted out of the water frequently, which offers good opportunities for inspection, small maintenance and minor repairs. Rolling or sliding gates can temporarily be set in dry in their recesses; the recess functions as a temporary dry-dock in that case. However, to keep the navigation lock operational it will be necessary to provide two gates per lock head. For inspection and repair on the rails or sliding track the navigation lock is locally dewatered by using a steel watertight structure which is open on the bottom and functions as a kind of diving bell. Mitre gates are usually lifted out of the water for maintenance and repair; this requires a double set of doors if the lock has to remain operational. When inspecting or repairing the lock head a maintenance limpet *(inspectie kolk)* is used to inspect the pivot, pindle, quoins, pivot shoe or supporting frame. A limpet can have a box shape and is put in position using a mobile crane and is ready to operate after the water is pumped out. Before positioning the limpet the canal bed must be cleared of objects that might obstruct the sealing of the limpet.

4.3 Lock Head

4.3.1 Functions and requirements

Design of a lock head goes hand in hand with design of the gate. Full reference is made to section 4.2, especially to the illustrations showing lock heads, for a better understanding of this section.

The lock head has three main functions:

 Accommodate the gate and whatever is necessary for opening and closing the gate: The lock head needs recesses for the gate in open position. The recess must be designed in such a way that collision with the gate is prevented as much as possible when ships are passing. Furthermore all the equipment and other items to operate the gate need space or a place in the lock head structure.
2. Retention of water, sealing, prevention of seepage (groundwater):

On a general level, being part of the lock implies the lock heads have to retain water. On a detailed level this results in design of sealing 'between' the gate and the lock head structure, see section 4.2.4. Besides the previous, ground water flow underneath and alongside the lock head structure must be avoided, not only for quantitative or qualitative water management reasons, but to prevent erosion and piping of the structures as well. Therefore, lock head and lock chamber have to be connected watertight and under and besides the lock head cut-off screens may have to be constructed.

3. Load transfer:

When the lock gates are closed the lock heads have to transmit the resulting hydrostatic load via the walls and floor to the foundation, without too much of deformation.

Below, some of the principal design requirements for the lock heads are derived from the functions, others are the result of operational considerations or boundary conditions.

Choice of filling and emptying system

The lock will be filled and emptied either by a head filling system, or by a longitudinal filling system. The choice of a filling and emptying system may pose strict constraints on the lock head design and has been described in section 4.1.

Safety of the flood or storm defense system

The lock may be part of the coastal defense or the flood protection system. This will definitely determine the Top of Structure level of the lock head(s) and will also contribute to a positive decision on providing the lock head with a second or back-up gate. The word contribute has been chosen deliberately for two reasons. First, there is the much more economical alternative of using stop-logs, see next paragraph, second, other reasons, e.g. operational security or the maintenance of gates, will have to be used as well to justify the costs of a spare gate.

Need of Stop-log recesses

These are vertical slots in the lock head walls, positioned in front and behind the gate. In these recesses horizontal beams, so-called stop-logs (*schotbalken*), can be lowered; piled on top of each other a retaining wall is the result. Traditionally stop-logs are wooden beams, nowadays steel stop-logs are often used for larger spans and for larger water head differences. Stop-logs serve the following purposes:

- 1. Back-up of the primary gate:
- See previous paragraph.
- 2. Emptying the lock chamber:

When the whole lock chamber has to be emptied and the gates can not be used for this purpose, because they are in maintenance as well, the stop-logs can be used to create a temporary retaining wall. The most common solution is to apply two rows of stop-logs at a distance of a few decimeters and fill up the gap with clay in order to create a watertight structure.

 Protection of the closing structures: This may be needed, for instance, to protect the closing structure against floating ice.

Equipment to move the gate:

Obviously the equipment, and installations, for gates will be completely different depending on the gate type, but it will be installed in or onto the lock heads. This will govern the design of the lock heads in varying extent, since the forces in a heavy lift gate and the provisions to be made; will be completely different from those needed for a wooden mitre gate.

4.3.2 Type of lock heads

Due to the large structural stiffness that is required and the large forces, e.g. hydrostatic water pressure, horizontal soil pressure and concentrated loads caused by operating the gate and the equipment or machinery, the design is in most cases based on an open concrete box or U-profile *(bakprofiel)*. Usually a monolithic structure is chosen in which the floor is rigidly connected with the walls as can be seen in Figure 4-65. By far the most lock heads have a U-shaped structure to enclose the required cross sectional water area with as little as concrete as possible.



Figure 4-65 Cross section of a monolithic U-shaped structure for a lock head As mentioned before the standard U-shape will transform in more complicated 3D shapes because recesses for the gates have to be provided and enclosed spaces for machinery and equipment.

Depending on the gate type, the construction method and local conditions variations to this theme are designed and constructed. In Table 4-6 a number of alternatives are shown with description of the situations where or when to apply them, for a lock head with mitre gates. It may help to look at the Figures in the previous section on gates to develop similar ideas for lock heads for other gate types.

	Description	Application area
1	Standard U- shaped lock head build in-situ	This type of lock head is still used frequently, because it is simple to construct and it is a very rigid structure. All the other alternatives are based on this U-shaped structure.
2	Pneumatically immersed lock head	The lock head is constructed at ground level and pneumatically immersed to its operational depth. The construction alternative is used when only little horizontal area is available (no building pit needed).
3	Stripped U- shaped head, floated in	The stripped lock head is built in a dry dock, then transported and immersed on a prepared gravel bed at its final destination. After ballasting with concrete the head will be finished. This alternative will be applied when building activity at the lock location is not desired and a short construction period is required.
4	U-shape lock head with a construction joint in the middle	This lock head is only applied for very large sea locks, thus only when the width of the floor is relatively large compared to the wall height. Apart from lower internal forces the hinge in the floor (only) has disadvantages regarding construction, different rotations at the joint, leakages etc
5	U-shape lock head with two construction joints	This alternative is a variation of alternative 4, the difference is the use of two joints instead of one. The same (dis)advantages apply.

Table 4-6Lock head alternatives

Locks

Besides the normal lock head alternatives in Table 4-6, Table 4-7 represents two flexible lock head alternatives. The purpose of a flexible lock head is to be relative easy extendable. This way, with minor structural adjustments, the lock will be capable of serving the shipping traffic for its whole structural lifespan. In general locks become already too small in 25-50 years, while a normal lock structure as a lifespan of about 100 years. Thus, this type of locks is from a Life Cycle Management (LCM) point of view a good alternative for lock (re)construction.

	Description	Application area
1	Floatable lock head that is floated in and can be replaced quite easy See 3 in	The stripped lock head is built in a dry dock, then transported and immersed on a prepared gravel bed at its final destination. After ballasting with 'sand' the head will be finished. When the head becomes too small it is floated out and replaced by a larger lock head of any width and/or depth that is required.
2	Extendable lock head; to be widened by demolishing the temporary wall	For this flexible head a standard U-shaped lock head is constructed. However in this head a demolishable wall is placed, which initially reduces the width of the head. When the width of the original U-shaped structure is required the wall can be demolished and the gates replaced.



4.3.3 Structural engineering aspects

The most important structural engineering aspect in the design of the lock head is the check on the stability strength and stiffness of the structure. For stability the lock head, or any other structure, is considered as a monolythical thing on a foundation. Checking the stability of the structure translates into checking the strength of the soil under the structure. For the check on the strength and stiffness of the structure as a whole, are taken into consideration. Of course the properties of the construction materials, often concrete and steel, come into play as well.

Note that stability and strength of the structure have to be checked for ULS conditions, using load factors to obtain the required safety, whilst in SLS deformations are checked without the use of load factors. In the remainder of this Section, stability, strength and stiffness will be further elaborated without making distinction of ULS and SLS, without the use of load factors.

Stability

For overall stability the following failure mechanisms must be checked:

Macro stability:

- 1. Bearing capacity (vertical) $\Sigma V = 0$
- 2. Lateral bearing capacity
- 3. Overturning No tension
- 4. Overturning Redistribution & σ_{max} ΣM = 0

In the listing above 'Overturning' has been mentioned twice. The check on overturning using the criterion of no tensile stresses under bottom of structure is more conservative, i.e. results in bigger foundations, than the criterion for overturning where redistribution of stresses is allowed, provided the maximum bearing capacity of the subsoil is not exceeded.

 $\Sigma H = 0$

 $\Sigma M = 0$

Micro stability:

5. Piping (onder- en achterloopsheid)

1. Bearing capacity (vertical)

When a lock is constructed besides the waterway, on average more weight is removed, excavating the soil above the anticipated level of bottom of Structure, than added due to the weight of the concrete structure. The exception to this general rule is the lock head structure, where bigger volumes of concrete, for instance for lift towers or the walls around the recess for a rolling gate may increase the downward load on foundation level. Constructed in the waterway, usually there is a surcharge on the bottom foundation level all over the structure, but this may be quite limited. Most frequently even the soil underneath a lock head has enough vertical bearing capacity to support the head with a shallow foundation. A first, rough estimate to be used for the bearing capacity of a shallow foundation would be about 300 kN/m². Of course a Brinch Hansen calculation to determine a more accurate figure for the bearing capacity has to be made.

If a pile foundation is necessary an initial pile bearing capacity of 1000 kN per \neq 450 pile (square 450 x 450 mm² pile), compression, is a safe figure to work with, provided the pile toe is in a layer where the cone value is around 10 N/mm² (CPT-value). Generally the use of the Koppenjan formulas for pile bearing capacity results in higher figures.

The upward groundwater pressure is a very important parameter because it reduces Σ V_s, the resulting vertical downward force. In a first, strongly simplified, approximation the decrease in piezometric level can be assumed to vary linearly with the position along the longitudinal axis of the construction. When vertical seepage cut-off screens (*schermen tegen onderloopsheid*) are applied, the seepage length can be approximated as sketched in Figure 4-66; a 2D-approach is used here. With total seepage length the formula, shown in the Figure, for the upward pressure at a given *x*-coordinate can be found.



Figure 4-66 Upward ground water pressure

The true pressures will have to be determined by drawing a square net or by using a finite element flow model; see the Manual 'Ground water flow' for an example. From the above it will be clear that the locations of the seepage cut-off screens are an important factor in the determination of the upward pressure under the lock floor, hence, in the stability of the lock head.

Keep in mind that short time intervals of high or low water levels will hardly have any noticeable effect on the upward ground water pressure.

Note:

- The soil pressure under the structure remains the same from left (h₁) to right (h₂) whether the piezometric level, read the water pressure, varies or not, assuming the dead weight of the structure is nicely distributed. The latter also depends on the stiffness of the whole structure, lock heads and lock chamber. Since the soil pressure is equal to the sum of the effective stress plus the water pressure ($\sigma_{soil} = \sigma'_{eff} + \sigma_w$), an increase in water pressure reduces the effective stress.
- A reduced effective stress requires less vertical bearing capacity of the soil. Water carries a part of the load. However, less effective stress reduces the lateral or horizontal bearing capacity, see next paragraph.

2. Horizontal or lateral bearing capacity

In Figure 4-67 the main loads on a lock head are sketched. Two resultant forces can be distinguished:

- Σ *H*: The resultant force of the horizontal water pressure on the gates and the lock head. In the direction perpendicular to this Figure e.g. soil pressures have to be added.
- Σ *V*: The resultant force of the vertical forces, being: the dead weight of the lock head and gate(s), the weight machinery or equipment, the weight of the water above the lock floor; reduced by the upward (groundwater) pressure.

The stability requirement against sliding is:

When the forces ΣH and ΣV are divided by the surface area A and f is replaced by tan(δ) the following formula results:

In this expression, σ'_n is the effective soil stress (*korrelspanning*), ϕ is the angle of internal friction of the subsoil. In Ultimate Limit State appropriate load factors and material factors have to be taken into account.

Overturning in general

After checking the vertical and horizontal equilibrium, the structure has to be checked on overturning. Considering shallow foundations, soil pressures, generally linearly varying, have to provide the resisting or stabilizing moment. Similar to the checks on vertical and horizontal stability, and using the already computed results of those checks, the resultants of the vertical and horizontal forces are calculated. The overturning moment will be computed using the arm between Bottom of Structure (BoS) and the working line of the horizontal resultant, and the arm or eccentricities of the vertical force(s). Instead of continuing the analysis with an overturning moment M, the calculated moment is translated into the resultant vertical force and an eccentricity e of this force. The multiplication ΣV .e has to equal ΣM .



Figure 4-68 Resulting ΣM , ΣH , ΣV and e_{tot}

Note:

In construction mechanics courses the point of origin used to do calculations is often chosen at the utmost right hand side and in the centre line of the structure or centre line of a part of it. From a mathematical or systematical point of view this is a wise decision because it avoids most of the confusion about plus-minus signs to be used for forces and their contributions to the moments, which depends on direction and position.

In hydraulic engineering it is more convenient to select the centre and bottom of the structure as the point of origin to check overturning stability, see Figure 4-68 and further explanations hereafter. Often the use of the centre bottom point of the structure has the advantage that the large dead weight force does not contribute to the moment.

In Figure 4-69 the effect of an increasing eccentricity, from left to right, on the soil pressures under the bottom slab is shown, assuming a homogeneous soil body of constant stiffness under the structure (the stiffness may vary from layer to layer though). Alternatively the force V_s could be larger, or both the



Figure 4-67 Main (hydrostatic) loads on a lock head

 $\tau < \tan(\delta).\sigma'_n$ with: $\delta = \frac{2}{3}\phi$

 $\Sigma H < f.\Sigma V$

eccentricity and force could increase. As explained a larger force V_s is the result of a larger overturning moment due to either larger forces or larger eccentricities (in order not to cloud the Figure, M and H have not been drawn).

In the middle and right-hand side illustration of Figure 4-69, the largest effective stress σ_{eff} under the foundation is calculated as follows:

- 1. the stabilizing or resisting force V_r is working opposite and in the same working line as the soliciting force V_s
- 2. knowing the eccentricity e of the force V_s , the distance a can be determined
- 3. V_r is the resultant force of the soil pressures, which are assumed to vary linearly. Given the linear variation of soil pressures, there has to be a triangular pressure distribution. The work line of V_r has to run through the centre of gravity of the triangle, hence, a is equal to ¼ of the base of triangle. As a result V_r equals ½ * σ_{eff} * 3a; with some rearranging σ_{eff} is ⅔V_r/a.



Figure 4-69 Effect of increasing eccentricity on soil pressure distribution

3. Overturning - No tension

When V_r is the equal opposite in the same working line as V_s the structure is stable; there is no danger for overturning. However, often the requirement that the whole base has to remain in contact with the soil beneath it will be put forward. This demand can be presented in formulas as follows for foundations with a rectangular footprint:

$$\sigma_{\text{eff}} = -\frac{\Sigma V_{s,tot}}{b} - \frac{\Sigma M_{s,tot}}{\frac{\gamma_6}{b^2}} = -\frac{\Sigma V_{s,tot}}{b} - \frac{\Sigma V_{s,tot}}{\frac{\gamma_6}{b^2}} \ge 0 \qquad \text{hence:} \quad e \le \frac{\gamma_6}{b}$$

The above mentioned requirement could be popularly phrased as: 'No tension allowed' under the base slab of the structure'. To suffice to this requirement the vertical component of the resulting force has to stay within the core of the foundation, e has to smaller than 1/6 of the base.

Obviously it has to checked that the effective stress σ'_{eff} is not exceeding the maximum bearing capacity of the (sub)soil, which can be determined using the Brinch Hansen.

4. Overturning – Redistribution

As illustrated in Figure 4-68 the overturning moment, thus the eccentricity and/or vertical force, may be very large. In steel material this would result in compression 'and' tension stresses, but the only possible force interaction between the bottom of the structure and the soil is a compression pressure, tension is not possible due to the nature of soil. The tension pressures that would be needed to resist the overturning moment are redistributed. As a matter of fact there are no 'tension pressures', but there will be increased compressive stresses under the part of the base that remains in contact with the soil.

Again a check has to be made on the effective stress σ'_{eff} , which should not exceed the maximum bearing capacity of the (sub)soil. Compared to the similar check when the 'No tension' requirement is used, there

is a higher chance on failure, i.e. the bearing capacity being too small for the required effective stress, because the stresses under the effectively working part of the base are definitely higher.

Some observations on the above stability checks:

- The higher the resulting effective stress under the structure in ULS checks, the higher the stresses that have to be used doing the SLS checks on (differential) displacements, although without load factors. Especially the situation with redistribution of stresses for overturning may become critical in SLS as well.
- The lock head length or width, do not forget to check the other direction, required for stability can be found by iteration through the above stability checks. By the way, the final length of the lock heads will be the result of either the stability requirements, or the required length for gate recesses, or other functional or construction requirements.
- It is important to make a distinction between the upper lock head and the lower lock head, taking the generally higher water levels 'outside' as a starting point. Most probably, horizontal water pressure on the upper lock head is partly absorbed by the lock chamber behind the head. The lower lock head has no other structure behind its back, when the water is high in the lock chamber, so this situation is less favourable.

For sea locks the situation is a little different, because in this case the design level for the upper head is storm surge level, and the design level for the lower head is only HLL. So in this case the loads on the lower head are less. Of course this situation changes when the lower head also functions as a secondary sea defence.

5. Piping (onder- en achterloopsheid)

Piping is the process of (ground)water flow, in the soil under and/or around the structure, having enough transport capacity to erode soil particles. As soon as the first particles are carried away, more water will be able to flow along the same route because of the reduced flow resistance and more soil particles will be washed away. The process strengthens itself and a pipe will develop. This will undermine the structure and, in the end, result in instability of the whole structure. To evaluate piping, the formulas of Bligh and Lane (ref Manual § 20.3 "piping") can be used.

Piping is influenced by the type of lock chamber that is applied: is it a closed basin or has a permeable chamber floor, for instance a filter and rubble stone protection, been used? In the first case the piping length (*kwellengte*) is related to the full length of the lock, in the second case the piping length is related to the length of the lock head only. The second case results in a larger hydraulic gradient and a greater risk of piping.

Strength and stiffness

For the check on the strength and stiffness of the structure a more detailed look at the structure itself, and the constituting elements is required. Nevertheless, the work starts with the same forces used for the stability analysis or checks; now the flow of forces within the structure, not the force transfer into the subsoil has to be determined and checked. To do so, the properties of the construction materials, often concrete and steel, have to be considered.

From a structural engineering point of view lock heads are structures that should be designed using 3D models. 2D schematisation is less meaningful because of the truly 3D shape of the lock head, see e.g. Figure 4-54. Also forces are acting along all 3 axes, for instance 'free' water pressures acting along the lock axis, soil and ground water pressures perpendicular to the same axis. The forces and the resulting load combinations to be analysed for the head and the spreading of these forces through the 3D structure is quite laborious if not complex. Nonetheless, starting from scratch 2D approximations can and will be used to arrive at a first conceptual design for the 3D lock head, but in following design loops the 3D FEM models will be used to do the final checks on strength and stiffness.

4.3.4 Construction aspects

There are several construction methods for lock heads; this paragraph will deal with some of them, taking in-situ construction of a standard U-shaped lock head as a starting point.

A large part of the lock head has to be constructed below the existing ground water level. Consequently a construction pit or braced excavation is needed, to construct the lock head. Design of the construction pit or braced excavation depends on several conditions at the lock location:

- Available space for construction (horizontal)
- Strength of the soil
- Permeability of the soil in combination with (differences in) piezometric levels or groundwater head(s)

Based on these conditions some construction methods and the temporary structures required are mentioned in Table 4-8. All the options need dewatering to a certain extend to keep the building pit dry.

	Slope combined with a seepage screen	Sheet pile wall	
Deep impermeable layer			
	This option is selected when there is enough building space and an impermeable layer is available	This option is selected, when there is an impermeable layer, but not so much space to build the lock head	
Underwater concrete with tensions piles	Not possible	This type of building pit is constructed when not much space is available and a thin layer of underwater concrete is desired.	
Under water concrete without tensions piles	Not possible		
		The same as above, but no tension piles are required, because the underwater concrete is made thick enough to resist the upward water pressure.	

 Table 4-8
 Characteristics of some construction methods for a U-shaped lock head

4.4 Lock Chamber

Although a 3D structure, lock chamber design generally is based on analysis of the 2D cross section. Therefore, designing a lock chamber is less complex than designing the lock heads. The two main components are the lock chamber walls and the lock chamber floor. In the next two sections the different types of walls and floors are described and in the third section the combination will be discussed. Finally, lock chamber types and selection criteria are listed.

4.4.1 Lock walls

The lock chamber wall has several functions:

- soil and water retention;
- guiding the vessel through the navigation lock;
- mooring the vessel during the F/E process.

Important for the preliminary design are the forces due to horizontal earth pressure, generally the governing load, water pressure, deadweight, surcharge load, hawser forces and ship's impacts. The wall has to be designed such that overall stability is guaranteed ($\sum M = \sum V = \sum H = 0$) and that it is able to resist all the forces (material strength > load). Stability and strength are generally considered in the Ultimate Limit State (ULS).

In addition, in Serviceability Limit State (SLS), to these forces the lock walls are subjected to (tensile) stresses induced by temperature. These stresses result in cracks. A way to prevent this problem is the use of expansion joints *(dilatatievoeg)*. Depending on the wall thickness these joints are positioned at a distances of approximately 25 meter.

The horizontal forces can be resisted in several ways, e.g. by means of:

- a gravity structure
- a retaining wall structure
- a wall combined with a deep foundation

The vertical forces will be taken up by the soil, either more uniformly distributed by a shallow foundation or more concentrated by using a deep or pile foundation. In following paragraphs several wall structures will be discussed in more detail.

Gravity structure (uniformly distributed bearing):

The worklines of the resulting horizontal and vertical force usually do not go through the centre of gravity of the structure, hence, there is an overturning moment. Stability is obtained by a linearly varying distributed soil pressure at the foot of the structure, see Figure 4-70 for the resulting soil pressures. A safe, conservative condition for stability would be to demand that the workline of the resulting external force lies within the core of the footing. In that case the whole footing remains in contact with the soil and no major redistribution of soil pressure is necessary to find equilibrium.





Figure 4-70 Forces on a gravity structure

The vertical dead weight forces *G* and *V* are reduced by the upward water pressure (not shown in Figure 4-70). The resulting vertical downward force has to be resisted by the soil pressure *O* beneath the wall.

The horizontal force H must be resisted by soil friction W at the bottom of the structure. The following must be valid:

W > H	Where:	f	=	friction factor
W = f(G+V)		G	=	dead weight of the structure
		V	=	weight of the soil

To provide a large enough soil friction force W, care has to be taken of:

- a. Sufficient friction between structure and soil. Make sure that the bottom surface of the wall is rougher than the soil. The formula $f = tan(\delta)$ can be applied. For neutral soil pressures $\delta \approx \frac{1}{2}\phi$, whilst for sheet pile walls $\delta \approx \frac{2}{3}\phi$, where ϕ is the angle of internal friction. For a smooth wall f is considerably smaller; use a $\delta < \frac{2}{3}\phi$.
- b. Enough deadweight *G*. This may lead to larger dimensions of the structure; alternatively heavier materials may be used, e.g. natural stone. (Rock obtained during excavation can be used in the concrete mix).
- c. A large external force V, if the above measures were not sufficient. The reinforced concrete wall structure may be designed and constructed as an L-shaped retaining wall; the bottom slab large enough to be loaded with sufficient deadweight force V.

Note:

- Horizontal support for the wall may be partially or fully produced by the lock floor structure, see section 4.4.3;
- Typically, gravity structures are structures on or with a shallow foundation.

Retaining wall structure:

For this type of structure, sheet pile walls, combi-walls and diaphragm walls can be used. Sheet pile walls and combi-wall are made of steel H profiles, U-shaped or Z-shaped sheet piles or steel piles (circular cross section), whilst diaphragm walls are made of reinforced concrete. Figure 4-71 presents an anchored steel sheet pile retaining wall.

The wall is schematised as a simply supported beam; a girder on two







supports. The horizontal force H, the resultant of active soil pressure, will be resisted by the anchor force A and the passive soil pressures B. Most of the vertical force will be carried away by soil, part of it increases the horizontal earth pressure and 'creates' a vertical friction force along the wall. Toe resistance of the sheet pile will make up the difference between active (down) and passive friction (up).

To reduce the bending moments in the wall, several measures are possible:

- a. Put the anchor at an efficient position. By moving the anchor down, the span reduces and the span moment decreases as well. An optimum level of the anchor results in a span moment has the same order as the support moments.
- b. Use more than one anchor.
- c. If B is not the result of passive soil pressure, but the reaction force of another structure, B should be positioned as high as possible, thus reducing the span. B is obviously often supplied by the bottom of

the lock chamber. The lock bottom slab and opposite lock wall form a system, which may be optimised, see section 4.4.3.

d. Drive the sheet pile deeper into the soil; instead of a simply supported beam, a beam with a simple and a fixed support, is a better schematisation for this situation. The anchor is still a simple support, however, deeper down there is a fixed support. The sheet pile will rotate around point *S*. Due to the rotation a passive soil force *C* is being introduced. Combined with force *B* the sheet pile is fixed in the soil; the wall structure is statically indeterminate. The anchor force could be considered obsolete, not necessary for the stability anymore, however, the anchor generally is not omitted in order to limit the bending moments and to limit the deformations of the wall.

Note:

- When steel sheet piles are used, they will be covered with wooden posts, or concrete slabs, and fender beams (*regels*), to prevent corrosion and damage to wall and ships.
- For steel structures, considering maintenance, preventing corrosion is the key issue. Usually steel sheet piles have a shorter life span than concrete diaphragm walls.

Wall combined with a deep foundation

If the bearing layer in the subsoil is located at a very deep level and the lock chamber has a considerable depth, which results in a large retaining height, the solution shown in Figure 4-72 can be used. This usually applies to sea or coastal locks. In the past the solution was used to by-pass the lessr strength of the materials of that time. Nowadays, this type of structure is mainly used for quay-walls. The horizontal and vertical forces of the top



Figure 4-72 Retaining wall with a deep foundation

structure are transferred by the piles into the subsoil. The sheetpile wall may contribute to vertical resistance, but its main function is to retain soil and water, hence it resists horizontal loads as well

4.4.2 Lock chamber floor

For design of the lock chamber floor (sluisvloer) the following functions have to be considered:

1. Water retention function – or not:

Retaining water implies resisting water pressure. A concrete floor is impermeable and will be loaded by hydrostatic water loads. If the floor of the lock chamber is permeable, for instance because of the natural soil bottom or use of a (rock) filter, there is a lesser chance on water pressure built up. The type of floor influences the stability of the lock heads and the chamber walls. The permeable type of floor has to be checked on:

- Ground-water flow towards the lock chamber, so during a high ground-water level and LSP inside the chamber;
- Currents and turbulences caused by the filling and emptying system;
- Currents and turbulences caused by ship movement's en ship propellers (wash load).
- 2. Load transfer function:

A concrete floor, due to its stiffness, generally has considerable influence on the stability, hence the flow of forces in the structures next to it. Either it has to be specifically designed not to attract these forces, e.g. by using expansion joints in a concrete floor or use of a filter, or it has to be designed and constructed to resist the forces.

A distinction can be made in a water-tight floor, a permeable floor without support function and a permeable floor that supports the lock walls. In general impermeable structures are rigid. Therefore scour is not a threat. The three types mentioned will be discussed a bit further, focusing on loads and load combinations.

Impermeable floor:

The impermeable floor has to be able to resist the two extreme load situations listed below. Usually the second situation, and the resulting load combination, is governing structural design.

- 1. Low ground-water level outside and HLD (high locking datum, *HSP*) inside the lock chamber: This results in a force in a vertical downward direction which, together with the dead weight of the floor, has to be resisted by the soil underneath the structure. In case the soil has insufficient bearing capacity for a shallow foundation piles have to be used.
- High ground-water level outside and LLD (low locking datum, *LSP*) inside the lock chamber, or even no water in the lock for maintenance reasons: The resulting vertical load is equal to *G* times the difference between the piezometric level of the ground-water under the floor and the height of the column of water above the floor, reduced with the dead weight of the floor.

If the resulting vertical load on the floor is positive, upwards, then the following structural solutions could be considered:

- a. Increase the dead weight of the floor structure by applying a thicker floor or using heavier material.
- b. Supply the floor with anchors or piles. Duly consider the fact that the load on the piles may change from tension to compression, and vice versa, caused by varying ground-water levels or the filling/emptying of the lock chamber.
- c. Connect the floor with the chamber walls. The walls provide support to the floor if there is a resulting upward force on the floor; in the other case they transfer vertical downward to the floor.
- d. A combination of the solutions a, b and c.

With a high load in upward direction (risk of floatation!) solution b is financially more attractive than solution a, especially when tension piles are used. Solution c generally is a good option. Nowadays the requirement that a lock has to be set dry is demanded less frequent than in the past.

Permeable floor, without supporting the lock chamber walls:

In a permeable floor the relation between the permeability of the floor k_{fl} and of the underground k_{gr} is of importance. Consider the following situations:

1. $k_{\rm fl} \ge k_{\rm gr}$

By using a flow net or a finite element program the ground-water flow can be calculated. The hydraulic load due to ground water flow has to be combined with flows and turbulences, caused by ships or the filling and emptying system. Next step is to investigate the stability of the upper layer of the floor structure; the soil or the top stones of the filter have to be cohesive, heavy or large enough to resist the hydraulic loads.

2. $k_{\rm fl} < k_{\rm gr}$

Especially when $k_{fl} \ll k_{gr}$ (in that case the floor is almost impermeable) the total upward pressure has to be resisted by the floor structure, the bottom soil or filter layers. In fact the situation agrees with a clay layer above permeable sand layers: there is great risk of hydraulic burst-up. To prevent this, the clay layer has to be thick enough (to provide a counter load) and has to be of homogeneous constitution and thickness.

If a filter is used as lock bottom, groundwater will flow into the lock chamber. If the filter is not working properly water pressure will built up in the filter and in combination with other hydraulic loads scour will occur, which might endanger the stability. To reduce groundwater flow or piping, short sheet pile walls may be used to increase the seepage length, which results in a smaller exit velocity of the groundwater flow.

Permeable floor, which supports the lock chamber walls:

This type of lock chamber floor is almost the same as the impermeable floor option, however, in this option, holes are made in the floor and filled with gravel, so the water pressures under and above the floor are equal. Consequently, the lock chamber cannot be dewatered, but it is protected against scour; piping is an issue. If the holes in the floor are large, it might be better to describe the floor structure as a number of beams, acting as struts *stempels*) between the walls, with a gravel filling besides or between them. In any way, this floor type maintains the wall support function, so the lock chamber wall can be more slender compared to a normal permeable floor.

4.4.3 Combination of lock chamber walls and floor

The natural soil or filter bottom is not considered here, because there is little to be combined from a structural point of view; the walls will be designed as stand alone structures. What remains to be discussed in this paragraph is the concrete lock chamber floor, either permeable or impermeable, in combination with the lock walls.

A stand alone concrete floor of certain minimum thickness is impermeable, but may not be heavy enough to resist upward water pressures. Increasing the thickness of the floor will solve this problem, usually at the cost of a large amount of extra concrete and extra excavation with all the related problems. Very thick floors are generally made of mass concrete (*stamp beton*) and structural reinforcement is 'not' used. (The thinner the concrete floor, the more need there is to use reinforcement). As a result of the horizontal earth pressures behind the walls a normal force (compression) will is introduced in the floor, vice versa the floor provides a support force to the walls. Although stand alone, there is structural interaction.

Quite some (design) effort would be necessary to provide a watertight joint between the floor and the wall structures. Even in this stand alone version, due to the normal load and floor-wall interaction, bending moments may be introduced, which result in vertical forces on the wall. To prevent this, a watertight joint could be designed, which transfers the normal forces but not the shear forces and moments. In a following alternative the thick gravity floor is replaced by a reinforced concrete floor able to resist bending moments. However, the bending moment is not transferred into the walls, normal and shear forces are, see Figure 4-73. Water tightness can be obtained by positioning rubber slabs at the contact surfaces between floor and wall.



Figure 4-73 Lock floor as a reinforced concrete floor

Since the bending moment in the floor increases quadratically with increasing lock width, a possibility to reduce this is the introduction of support moments on the floor. Hence, the joint connection develops into a full connection stiff concrete joint between the walls and the floor. The continuous concrete cross section, fully integrated or monolithic structure is the result, see Figure 4-74.



Figure 4-74 Upward forces with a reduced field moment

An additional advantage of this option is that the large weight of the wall is transferred via the floor to the soil resulting in a more equal distributed load on the subsoil. There will be less problems with peak soil pressures and settlements and displacements in the soil will be less.

In navigation locks in coastal areas the lock is not only very wide but also very deep, due to the vessel size.. A continuous floor with such a wide lock is not very sensible due to the size of the bending moment. Therefore, joints are often applied which work as hinges and still are capable to transfer normal and shear forces. This is illustrated in Figure 4-75.



Figure 4-75 Cross section of a navigation lock with a hinge halfway

In the above the focus was on the cross section of the lock chamber; now some attention will be paid to the longitudinal section.

Usually locks are built with expansion joints which separate the chamber into sections with a typical length of 25-30 meter, see Figure 4-76. The expansion joints allow each block to move independently and reduce stresses that otherwise might occur due to differential settlement or expansions due to concrete hardening or other changes in temperature. The expansion joint needs to be water tight, to keep the water in the lock and to prevent groundwater and soil from coming into the lock.



elements and water stops



If the calculated differential settlements are above the limit, the only solution may be to construct the lock chamber without expansion joints. This joint less monolithic structure results in a very robust structure that can resist not only differential settlement in general but also local unexpected settlements (e.g. due to scour), see Figure 4-77. It is not necessary to build the whole structure at once. Construction joints can be used to separate different segments. Reinforcement can be carried through the construction joints.

A monolithic lock design results in increased internal stresses. Joints normally reduce these stresses by a small translation or rotation, compressing the (rubber) expansion joint material. The extra internal stresses in monolithic design have to be combined with the other loads (water pressure, earth pressure, deadweight, etc.) and the structure has to be designed for the Ultimate Limit State and the Serviceability Limit State.

For the future, a properly designed and constructed monolith structure without joints can reduce maintenance and eliminate major problems with leaking expansion joints that appeared on a regular basis in the past. [PIANC 2009, Innovations in Navigation lock design]

4.4.4 Chamber selection

By making a choice of chamber several factors play a part. The most important ones are:

- a. Initial costs, life cycle costs;
- b. Width of the lock chamber, height of the walls and their mutual relation;
- c. Water levels, including groundwater level;
- d. Condition of the soil; such as bearing capacity and the permeability of the bottom;
- e. Construction requirements: such as the dimensions of the available construction site and the permissibility of groundwater lowering during construction;
- f. Availability of material and equipment.

Floor Wall	A. Impermeable floor, gravity floor (C*)	A. Impermeable floor, with tension piles (C*)	B. Permeable floor		
Gravity structure		No additional value			
Retaining wall structure					
Retaining wall with a deep foundation					
Continuous concrete cross section		No additional value	No additional value		
One joint concrete cross section		No additional value	No additional value		
C* : Floor option C has the same layout as floor option A, but the lock floor contains holes, so no large water pressures differences will occur.					

Table 4-9 Wall-floor combinations in a lock chamber

4.5 Seepage cut-off screens

The hydraulic gradient over the lock will initiate a groundwater flow underneath and alongside the lock structure *(onder- en achterloopsheid)*. If the groundwater flow or seepage becomes strong enough to transport soil particles, erosion endangers the stability of the lock. This failure mechanism, referred to as piping, because the eroded voids are pipe shaped, is illustrated in Figure 4-78.



Undisturbed situation





Beginning of the piping process



Growing pipe

Failure (not stable)

Figure 4-78 Failure mechanism due to piping

To prevent piping the following could be done:

- Construction of a geometrically closed filter on the bottom and on the slopes at the downstream side of the structure.
- Reducing the permeability of the soil. This would imply the use of injection techniques, which is usually expensive.
- Extending the seepage length. This can be done by:
 - Enlarging the lock itself, which would be quite expensive;
 - Applying vertical seepage screens under and beside the lock, perpendicular to the longitudinal axis. The screens cut off the groundwater flow, thus, the name cut-off screens. This is the most common solution.

The cut-off screens under and besides the lock must be positioned in the same vertical plane; in this way they form a continuous flange or collar around the structure, in the ground. The top of the screens besides the lock will b slightly above the local groundwater level. Cut-off screens are constructed from sheet piles and their length is determined by a hydraulic study. The minimum width of the screens must be equal to the width of the construction pit used for construction of the lock, because backfilled soil is generally more loosely packed than the undisturbed soil. For this reason the permeability will be higher and piping a greater risk.

Leakage through the sheet pile wall must be avoided, so during the pile driving attention must be paid to the correct interlocking of the sheets.

The connection between the sheet pile wall and the lock floor must be and must remain impermeable. This may be problematic under the lock bottom. When the lock design is based on a pile foundation, the soil underneath the floor may settle a little with respect to the lock itself. The embedded cut–off screen will settle along with it, which results in a gap between the screen and the lock floor, the pipe that has to be prevented. The solution is to embed or anchor the top of the sheet piling into the floor. An additional number of small cut-off screens in between the main screens may be necessary to close the gap under a pile foundation lock. These screens can be relatively short, about one meter, since their only function is to seal the gap due to settlement. When the lock has a shallow foundation a different problem occurs. In this case, settlement of the subsoil may cause the lock floor to settle more than the sheet piles, so the lock floor will effectively be resting on the sheet pile wall, introducing undesirable bending moments in the lock floor. This situation can be avoided by installing a compressible material, e.g. cork or foam plastic, at the top of the sheet piling, see the sketch for the detail in Figure 4-79



Figure 4-79 Connection cut-off screen to the lock floor

Figure 4-80 Position of the seepage screens in case of an permeable lock chamber floor

The connection between the side screens and the lock wall is usually obtained by casting part of a single sheet pile into the concrete wall itself. The lock of this sheet pile then sticks out from the wall and forms the starting point for the rest of the sheet pile screen.

It must be noted that all the above considerations only apply to locks with impermeable floors. When floors with filter structures are applied, the seepage screens will have to be positioned under the lock heads as can be seen in Figure 4-80.

The position of cut-off screens influences the upward groundwater pressure. The best location for a cutoff screen is at the upstream side of the upper lock head. When the hydraulic gradients are large the seepage length must be extended further and a second screen must be applied. The best position for this second screen is at the upstream side of the lower lock head. Especially the lower head benefits from a reduction in upward pressure because it is not supported by an adjacent lock chamber. When the lock is constructed in a tidal area and the hydraulic gradient changes direction with the tide it is common to use two seepage screens, one on each side of the lock, even when the gradients are small.

4.6 Lock approach structures

Berthing facilities have to be provided for ships having to wait for the next locking opportunity. Approach or guidance works are structures that guide the ships quickly and safely into the lock chamber upon entering; safe for the ship and the lock. One possible layout of berthing facilities and guidance works, typical for North-Western Europe, is depicted in Figure 4-81.

For proper guidance to entering ships, the angle α must be small. However, a small α means a large distance between the berthing facilities and the lock itself, which leads to a longer lock cycle and smaller lock capacities. A good compromise can only be found from practical experience and model studies. In the Netherlands this has lead to a value of tan(α) = 1:6 for inland waterway locks that are mainly used by self-propelled cargo ships, not by push convoys combinations.

Ships that leave the lock must not be hindered by waiting ships, so the berthing facilities must be constructed away from the line through the lock walls (see dashed line in Figure 4-81). It is recommended to allow for some clearance between this line and the sides of the waiting ships. This will increase the sailing velocities of the outgoing ships, reduce the locking times and improve the capacity of the lock. The dimensions of the berthing area ($W \times L$) are usually equal to the dimensions of the lock chamber ($w \times l$).

In this way all ships that will enter for the next locking are at the shortest possible distance from the lock. When a large supply of ships is expected the berthing area can be extended.



W_c= Width of the lock chamber

and the fairway

Figure 4-81 Design of the lock approach

Locks that are also used by push convoys are usually only marginally wider than the convoy (e.g. lock chamber width 24 m against a convoy width of 22.80 m). In this case a slightly bent and slowly narrowing guidance wall is applied to improve safety. It must be understood that a push convoy is difficult to manoeuvre because of its large mass (a four-unit push convoy weighs 8800 tons, compared to 1350 tons for a common Europe class vessel).

Berthing facilities near sea locks are usually not constructed as a single continuous area, but as a series of detached berths. In this way the tugboats, that may be needed to assist large sea-going vessels with entering or leaving the lock, have more manoeuvring space. In this situation the guidance wall will be a continuous structure because of its function.

The main design requirements can be deducted from the definitions that were given in the introduction:

- The structure must be able to absorb the impact of a ship without damage to the ship or the structure. (both berthing facilities and guidance works)
- The structure must provide a sheltered waiting place for ships, hence, must resist the mooring forces caused by waiting ships (berthing facilities)
- The structure must be designed in such a way that ships can sail alongside smoothly. (both berthing facilities and guidance works)
- When needed, there must be a connection between the ship and the river bank to enable the ship's crew to go ashore (berthing facilities)

The first requirement determines the magnitude of the forces that must be absorbed. The third requirement almost inevitably leads to the design of a



Figure 4-82 Bird's eye view of a lock approach

continuous structure. The last requirement shows that additional facilities (e.g. gangways, ladders etc.) may be needed.

Loads:

Ships can cause two types of loads on the structure:

- Berthing or collision forces (Aanvaarbelasting)
- Mooring forces (Afmeerbelasting)

For berthing or collision forces a flexible structure would be best because larger displacements dissipate more kinetic energy. For mooring forces a stiff structure would be the best solution.

So, in a way, the impact forces and the mooring forces require contrary solutions. For large ships this has lead to the development of separated structures: berthing dolphins to absorb impact forces and mooring dolphins.

Structural alternatives

The above analysis leads to three observations:

- the main design problem is the absorption of a horizontal force
- flexible structures are attractive
- continuous structures are preferred

A common solution in Europe that satisfies these three observations is depicted in Figure 4-83 below.



Figure 4-83 Steel guard wall



Figure 4-84 Guidance walls with floating girders

This solution consists of a number of vertical steel piles that have been driven into the subsoil to provide the desired degree of resistance. These piles are connected by one or more horizontal steel girders. The number of girders depends on the possible water level variations. A hardwood fender is constructed in front of this girder because wood provides a better material to guide ships than steel, because it is more flexible. When the water level variations are large the fixed steel girders can be replaced by floating structures, see Figure 4-84

Sometimes sheet pile walls are used as guidance works. These are very stiff constructions, so to avoid damage to ships a more flexible structure must be placed in front of the sheet piles, for instance a framework made of azobé (type of wood).

In the USA approach walls are used quite differently compared to Europe. The vessels, in the USA generally large push convoys, keep in touch with the approach wall, 'slide or slip' along the wall, while they

are manoeuvring into the lock chamber. In Europe the vessels avoid every contact. The different procedure is reflected in the layout of lock and approach structures. In Europe the guard wall has an angle with lock axis, see Figure 4-81, in the USA there is no such angle and the face of the approach wall is flush with the face of the lock chamber wall.

There is an explanation for the typical manoeuvring procedure in the USA. Often the navigation lock is part of a weir complex in the river and constructed close to one of the river banks. In the approach zone of the lock the flow pattern of the water is in the direction of the weir, where water discharges, adjacent to the lock, see Figure 4-85.

The vessels are 'pulled' into this flow and to maintain course, into the navigation lock, quite some rudder and propeller action is required. However, the vessels can not use their engine power because the push convoy has to be slowed down to avoid collision with the lock. The approach wall is a real guidance structure; it pushes the convoys in the right direction and prevents them from ending up in front of the weir.

Figure 4-86 and Figure 4-87 show some pictures of the floating approach wall for the navigation lock in Olmsted – USA.



Figure 4-85 Birds-eye view Olmsted weir & lock - USA



Figure 4-86

Floating approach wall; Olmsted lock USA



Figure 4-87 Nose pier of approach wall during construction

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Appendices

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A1 Inland waterway vessels, CEMT classes & transport capacity

Spits lengte 38,50 m. breedte 5,05 m. diepgang 2,20 m. laadvermogen 350 ton		CEMT Class I
Kempenaar lengte 50 m. breedte 6,60 m. diepgang 2,50 m. laadvermogen 550 ton	22x	CEMT Class II
Dortmunder lengte 67 m. breedte 8,20 m. diepgang 2,50 m. laadvermogen 900 ton	36x	CEMT Class III
Vierbaksduwstel lengte 193 m. breedte 22,80 m. diepgang 2,50/3,70 m. laadvermogen 11000 t	on 440x	CEMT Class VIb
Containerschip lengte 50 m. breedte 8,60 m. diepgang 2,50 m. laadvermogen 24 teu	24x	≈ CEMT Class II
Containerschip lengte 110 m. breedte 11,40 m. diepgang 3 m. laadvermogen 200 teu	200x	CEMT Class Va
Containerschip Jowi-klasse lengte 135 m. breedte 17 m. diepgang 3 m. laadvermogen 470 teu	470x	CEMT Class Vb
Tankschip lengte 110 m. breedte 11,40 m. diepgang 3,50 m. laadvermogen 3000 ton		CEMT Class Va
Autoschip lengte 110 m. breedte 11,40 m. diepgang 2,50 m. laadvermogen 600 ton	600x	CEMT Class Va
RO-RO schip lengte 110 m. breedte 11,40 m. diepgang 2,50 m.	72 x	≈ CEMT Class IV

1 teu = 1 20-voets container

A2 Water Management

Water shortage and salt water intrusion are two important issues concerning water 'resource' management. These issues will interfere with lock design to an extend that depends on local conditions.

Impact of climate changes has to be considered as an important source of uncertainty for the long term water resource planning.

A2.1 Water shortage - Quantity

In arid regions, areas with a small water supply or in canals, the water consumption of locks can be an important optimisation goal. It should be considered that obviously a climate change is taking place, which might be a relevant factor during the lifetime of the lock.

In canals the water consumption of a lock must be compensated by pumping stations if no natural water supplies for the canal are available. This results in high costs for building the pumping stations, maintenance and energy supply which depend on the amount of consumed water.

If the water on the upper side of the lock is also needed for other purposes (e.g. drinking water, irrigation), water saving can be very economic, as less water has to be transported / cleaned / desalinated. It must be considered on which side of the lock the water quality is better and if exchange must be controlled. This must be carefully judged, as the implications are not obvious.

Especially the water resource management of large locks (lengths > 200 m and/or lift height > 15 m) is not an easy question to solve. European directives (EEC, 1997) require that the friendliest solution from an environmental point of view must always be studied, even without considering economical questions. So when we have to consider a large water head, even if the most feasible technical and economical solution is to build a standard lock (without saving system), it is mandatory to study other solutions (several locks with reduced water head or the use of water saving basins), which might increase significantly the building and operational costs.

It must be considered that locks with water saving basins, lock ladders or twin locks (with water exchange between the locks) show a slower operational speed than the equivalent standard type of locks, if no further constraints are applied. In special situations, where the impact of the filling/emptying process on the upstream/downstream waterways is relevant (i.e. waves, surges), locks with water saving basins can be operated faster than standard locks, as the amount of water which is exchanged with the waterway is smaller.

From water resource management point of view, the optimisation goal is to find a good balance between water consumption, pumping costs and the possible transport of contaminants through the lock.

A2.2 Salt water intrusion - Quality

Salt water intrusion in inland waterways by locking ships in a navigation lock induces large problems when this water is used for drinking water or to sprinkle agriculture areas. Another problem is that the salt water may penetrate in the soil and replace the fresh ground water, which would induce problems for the vegetation.

Salt water intrusion is always accompanied by loss of fresh water. Because salt water has larger density then fresh water, it naturally enters in the lock chamber and replaces partially the fresh water. The loss of fresh water is mainly a problem during the summer, when the need of fresh water is the highest.

Salt water intrusion problems not only play a role in the design of navigation locks between the sea and an inland waterway provides salt water intrusion, but also for locks between a salted inland navigation way and a fresh water storage basin.

The amount of salt water intruding on a fresh water storage basin by a navigation lock, when no preventive measures are taken to separate salt and fresh water, depends on the number of navigation cycles, the size of the lock and the difference in water levels (lift height).

Due to the difference in density of salt and fresh water, currents in the lock chamber will occur during the navigation cycle. These currents cause forces on the ships, which have to be considered.

To reduce the salt water intrusion and the loss of fresh water few solutions can be mentioned:

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- Air-bubble-barrier in the lock head to avoid exchange of salt and fresh water as the gates are opened.
- Movable sill on the locks floor to reduce the water depth to a minimum for the ships to be locked.
- Systems to remove the salt layer during locking by:
 - i) Leveling and transversal exchange (Duinkerken, NL),
 - ii) Leveling and, transversal and longitudinally exchanges,
- Use of multiple lifts,
- Salt water sump (see P R Cardiff [P.R. 10-02])

For the description about these systems see PIANC'1986 1986 on Locks (Section 12), see attached CD-ROM Directory A3.

Besides technical solutions also management solutions can be mentioned, such as:

- reducing the opening time of the gates,
- reducing the number of lock cycles,
- in case a lock is equipped with two lock chambers, a bigger one and a smaller one, use the lock chamber which fits the best to the ships to be locked.

The above mentioned technical solutions are very expensive and technically complicated. The management solutions are not complicated and do not increase the building costs or maintenance costs.

One of the best references about Water quality control at ship locks, Prevention of salt and water exchange is Kerstma et al (1994) in which a number of projects are described.

Obviously, for the Panama Canal lock project salt water intrusion studies, including the required modelling work, were conducted. These studies were critical in discarding the use recycling pumped water, of one and two lift locks, and the adoption of three lift locks with water saving basins. Executive summaries study can be found in/at:

http://www.pancanal.com/esp/plan/estudios/0260exec.pdf

http://www.pancanal.com/esp/plan/estudios/0276-01.pdf

A2.2.1 Salt water intrusion and loss of fresh water during the lock navigation cycle

This section presents the lock navigation cycle and the calculation of salt water intrusion and loss of fresh

water for navigation locks without separation of salt and fresh water.

1. Lock Navigation cycle

A navigation lock between the sea or a tidal river with salt water and a canal with fresh water will be considered in the following. The amount of chlorine in the salt water on the seaside of the navigation lock will be much higher than in the canal-side (fresh waterside).

Consider the navigation cycle at the moment that the gate at the salt-water side of the lock has been opened for quit a long time, so the lock-chamber will be completely filled with salt water having the same density as the water at the seaside.

The gate at the seaside will close and the emptying of the lock-chamber starts, discharging salt water to the canal when the water-level of the canal is lower, or filling with fresh water when the water-level of the canal is higher.

Empting the lock chamber will bring an amount of salt water in the canal, which is mixed with the fresh water. The density of the salt water in the lockchamber will not change.

Filling the lock chamber with fresh water, in case of higher water level on the canal, the average density of the salt water in the lock chamber will decrease.

Opening the gate at the canal-side, the salt water in the lock-chamber will exchange with the fresh water in the canal by internal flow, with the final result that the total lock-chamber-content enters the canal and lands as a rather thin layer on the bottom of the canal. The water in the lock-chamber has turned fresh.

The exchange of salt water will be stopped when the gate at the canal-side closes.

After closing the gate at the canal-side the leveling of the lock-chamber starts again, empting fresh water on the sea or tidal river when the water level on the sea is lower, or filling with salt water when the water level on the sea is higher.

Empting the lock chamber will bring an amount of fresh water to the sea. This is a large loss of fresh water. The density of the fresh water in the lockchamber will not change. Filling the lock chamber with salt water, in case of higher water level on the sea or tidal river, the average density of the fresh water in the lock chamber will increase.

After filling the lock-chamber the gate at the sea-side opens and the total amount of fresh water in the lockchamber will be pushed away by salt water. The exchange of fresh water will be stopped of course when the gate at the sea-side closes.

Ships in the lock-chamber will decrease the amount of exchanged salt or fresh water with their waterdisplacement tonnage.

2. <u>Calculation of salt water intrusion and loss of</u> <u>fresh water</u>

The formula to determine the amount of salt water entering in the canal at each lock cycle is:

$$V_{salt} = (h_s - z_d) b_{lc} l_{lc} - V_s$$
 (maximum amount)

and the mass of CL⁻ - ions is:

$$C = V_{salt} \frac{(\rho_1 - \rho_2)}{1.4} \quad \text{(maximum amount)}$$

The formula for the amount of fresh water entering in the sea at each navigation cycle is:

$$V_{fresh} = (h_{fr} - z_{lc})b_{lc} l_{lc} - V_s \quad \text{(maximum amount)}$$

with:

$\begin{array}{rcl} V_{fresh} &= \text{volume fresh water} & m^3 \\ V_s &= \text{volume displacement of} & m^3 \\ ships \\ C &= \text{mass of CL}^- \text{-ions} & \text{kg} \\ h_s &= \text{water-level of salt water side} & m \\ h_{fr} &= \text{water-level of fresh water} & m \\ side \\ z_d &= \text{threshold level of fresh} & m \\ water side \\ z_{lc} &= \text{bottom-level of lock-} & m \\ chamber \\ b_{lc} &= \text{width of lock-chamber} & m \\ l_{lc} &= \text{length of lock-chamber} & m \\ \rho_1 &= \text{density fresh water} & \text{kg/m} \\ \rho_2 &= \text{density salt water} & \text{kg/m} \end{array}$	V _{salt}	= volume salt water	m°		
$V_s = \text{volume displacement of } m^3$ $C = \text{mass of CL}^- \text{- ions} \text{kg}$ $h_s = \text{water-level of salt water side} \text{m}$ $h_{fr} = \text{water-level of fresh water} \text{m}$ $side$ $Z_d = \text{threshold level of fresh} \text{m}$ $water side$ $Z_{lc} = \text{bottom-level of lock-} \text{m}$ $chamber$ $b_{lc} = \text{width of lock-chamber} \text{m}$ $l_{lc} = \text{length of lock-chamber} \text{m}$ $\rho_1 = \text{density fresh water} \text{kg/m}$	V _{fresh}	= volume fresh water	m ³		
$\begin{array}{llllllllllllllllllllllllllllllllllll$	Vs	 volume displacement of ships 	m ³		
$\begin{array}{llllllllllllllllllllllllllllllllllll$	С	= mass of CL ⁻ - ions	kg		
$\begin{array}{llllllllllllllllllllllllllllllllllll$	hs	= water-level of salt water side	m		
$\begin{array}{llllllllllllllllllllllllllllllllllll$	h _{fr}	= water-level of fresh water side	m		
$ \begin{array}{llllllllllllllllllllllllllllllllllll$	Zd	 threshold level of fresh water side 	m		
$\begin{array}{ll} b_{lc} & = \mbox{ width of lock-chamber } & \mbox{m} \\ l_{lc} & = \mbox{ length of lock-chamber } & \mbox{m} \\ \rho_1 & = \mbox{ density fresh water } & \mbox{kg/m} \\ \rho_2 & = \mbox{ density salt water } & \mbox{kg/m} \end{array}$	Z _{lc}	= bottom-level of lock- chamber	m		
I_{lc} = length of lock-chamber m ρ_1 = density fresh water kg/m ρ_2 = density salt water kg/m	b _{lc}	= width of lock-chamber	m		
$ \rho_1 = \text{density fresh water} kg/m \\ \rho_2 = \text{density salt water} kg/m \\ kg/m \\ $	I _{Ic}	= length of lock-chamber m			
$ \rho_2 $ = density salt water kg/n	ρ_1	= density fresh water kg/m ³			
	$ ho_2$	= density salt water kg/m ³			

The factor 1.4 (conversion from V_{salt} to C) depends on the temperature of the water. For a water temperature of 10 °C the factor is 1.4. For 20 °C the factor is approximately 1.3.

A2.2.2 Induced flow in the lock chamber

A flow is created in the lock chamber due to difference in density of salt and fresh water during the navigation cycle.

1. Filling and emptying the lock chamber

When a fresh-water-lock chamber is filled up with salt water, the salt water sinks to the bottom of the chamber and the fresh water layer stays above the salt water (Fig. A1)

Filling up a salt-water-lock chamber with fresh water, the fresh water flows into as a layer above the salt water layer.

The water flowing into the lock chamber reflects against the gates and particularly against the bow and stern of the moored ships. Due to this flow and its reflection, an internal wave is generated with a large wavelength.

When emptying the lock chamber there is no wave generated inside the lock chamber, but only in the approach channel.



Fig. A1: Lock filling/emptying - Density flow during filling the fresh-water-lock chamber with salt water (above) or the salt-water-lock chamber with fresh water (below)

2. Exchange of salt and fresh water by opening the gates

As soon as the gates are opened the exchange of salt and fresh water starts due to the density

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difference (Fig. A2). In case of a fresh-water-lock chamber and a salt-water-approach channel a salt layer will enter over the bottom of the lock chamber and the fresh water will flow at the surface of the approach channel. When the gates of a salt-waterlock chamber are opened the fresh water of the approach channel will flow into the lock chamber at the water surface and the salt water will flow into the approach channel along the bottom.

As long as the ships are not sailing there will be a minimum of mix of the salt and fresh water.

The incoming water layer reflects against the gates and particularly against the bow and stern of the moored ships, and generates an internal wave with a large wavelength.

Salt-water-approach channel Lock			Fresh-water channel
	Gates	chamber	Gates open
	closed		
		Į	

Salt-water-a channel	ipproac	h Lock	Fresh-water channel
	Gates closed	chamber	Gates open

Salt-water-a channel	approac	h Lock	Fresh-water channel
	Gates closed	chamber	Gates open
		4	<u> </u>

Salt-water-a channel	approac	h Lock	Fresh-water channel
	Gates closed	chamber	Gates open

Fig A2: Gate Opening - Density flow during exchange of salt-water-lock chamber with fresh water

A2.2.3 Exchange of salt and fresh water as function of time

Opening the gates between a salt-water-lock chamber and a fresh-water-approach channel, the fresh water will flow into the lock chamber at the water surface with a speed, calculated with the formula:

$$c_i = 0.53\sqrt{\varepsilon.g.(h_{bi} - z_k)}$$

with c_i the velocity of propagation of fresh water (m/sec).

For a lock with a fresh-water-lock chamber and a salt-water-approach channel the factor 0.53 changes to 0.43.

The exchange of fresh and salt water at the location of the opened gates is stable during a long time and depends of the dimensions of the open gates. The discharge is calculated by:

$$Q_u = b_d . (h_{bi} - z_d) . C_u \sqrt{\mathcal{E} . g . (h_{bi} - z_k)}$$

with:

Q_u	= discharge of exchanged water	m³/s
b _d	= width of lock	m
h _{bi}	= water-level of fresh water side	m
Z _d	= threshold level of fresh water side	m
Z_k	= bottom level of lock chamber	m
C _u	= coefficient of discharge of exchanged water (= 0.2)	-
3	= $(\rho_1 - \rho_2)/\rho_2$ = relative difference in density	-
ρ_1	= density fresh water	kg/m ³
ρ_2	= density salt water	kg/m ³
g	= gravitational acceleration (about 9.81)	m/sec ²

After a certain period, the fresh water front reaches the closed gates, reflects against these gates and travels back in the direction of the opened gates. The fresh water front reaches the open gates after a period t_1 :

$$t_1 = \frac{2l_k}{c_i}$$

© Hydraulic Engineering Faculty of Civil Engineering Delft University of Technology On this moment (t_1) the volume of salt water in the lock chamber replaced by fresh water is:

$$V_u = Q_u t_1$$

with V_u , the exchanged volume of water (m³)

In these calculations the volume of the ships in the lock has also to be taken into account. When the ships leave the lock their volume will be replaced by fresh or salt water water.

A2.2.4 Example

As example for the calculations of salt water intrusion on a fresh water storage basin we look at the design of the renovation of the "Grote Sluis" (Fig. A3), Spaarndam, The Netherlands (2007).

This lock is situated in the western part of the country, between Zijkanaal C, a side branch of the North Sea canal, and the Spaarne, a large fresh water basin with a lot of flower-plantations (tulips) in it's surroundings. Zijkanaal C (side branch of the Noordzeekanaal) is salted through the large locks at IJmuiden, the entrance to the port of Amsterdam.

The renovation of the "Grote Sluis" is necessary because of the bad condition of the lock, the gates, the mooring places and so on, and also to reduce the large amount of salt water, which enters in the freshwater storage basin at each navigation cycle and which is very harmful for the flower-plantations.

The lock chamber of the "Grote Sluis" has a length of 80 m, a width of 25 m and a depth of 5.1 m. The gateopenings have a width of 12 m.

The water level difference is normally 0.40 m, at witch the salt water side (Zijkanaal C) is the highest side.

The lock is used for commercial navigation (approx. 4700 ships/year) and a lot for pleasure navigation (approx. 20.000 pleasure boats/year).

The number of navigation cycles is approximately 5000 a year from the Spaarne to the Zijkanaal C (from fresh to salty water) and approximately 5400 a year the other way (from salty water to fresh water).

The average mass of CL^- - ions on the side of Zijkanaal C is 1565 mg/ltr and the maximum mass is 3500 mg/ltr. In the near future this average mass of CL^- ions will increase to 2250 mg/ltr, due to the proposed construction of a new large lock (70 x 500 m) at IJmuiden.

With the above mentioned data, calculations are made for the total mass of CL⁻ - ions each navigation cycle, taken in account that the gates stay open after the ships have left the lock chamber, which is up to now the usual lock operation mode at the "Grote Sluis".

At each navigation cycle a mass of approximately 18.000 kg CL⁻ - ions are added to the fresh-waterstorage basin, the "Spaarne". This means in one year an amount of approximately 100.000.000 kg (100.000 ton) CL⁻ - ions. It is worth to take some actions to reduce this amount of salt!

To reduce this amount of CL⁻ - ions the lock chamber will be divided in two parts, a small and a larger lock chamber to have a better link with the number of ships to be locked. Also the lock method will change, witch means, among other things, that the gates will be closed immediately after the ships have left the lock chamber and the pleasure boats have to wait until the total lock chamber is filled.



Fig. A3: Overview of the "Grote Sluis" te Spaarndam in The Netherlands

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A2.2.5 Additional Forces on ships

Filling and exchanging the lock chamber causes additional forces on the moored ships, due to the difference in density between salt and fresh water. The largest forces are generated when the gates are open and the lock chamber exchanges salt to fresh water, or opposite.

The maximum force on a ship caused by the difference in density is approximately:

$$F'_{r} = \frac{\varepsilon}{2.l_{s}.C_{b}}.d_{s} - \frac{(d_{k}^{2}.b_{k} - d_{s}^{2}.b_{s})}{d_{k}.b_{k} - d_{s}.b_{s}}$$

with:

F'r	= relative longitudinal force	-
	(relative to ship tonnage or	
	displacement)	
ls	= length of the ship	m
C_b	= block coefficient (*)	-
b_k	= width of the lock chamber	m
bs	= width of the ship	m
ds	= depth of the ship	m
d_k	= water level depth	m
3	= $(\rho_1 - \rho_2)/\rho_2$ = relative	-
	difference in density	

(*) The block coefficient (C_b) is the ratio between the real ship volume under water and the volume of a block equals to length x width x depth of the ship.

A2.2.6 Situation on the approach channel

By exchanging of water of the lock with water in the approach channels, the average density of the water in the approach channels will change. The salt-water-approach channel will become little less salty and the fresh-water-approach channel will become more salty.

The fresh water on the salt-water-approach channel will be as a thin fresh water layer upon the salt water. This thin fresh water layer flows to the sea and will be mixed with the salt seawater by wind, ship propellers and so on.

The salt water on the fresh-water-approach channel will take the shape of a thin salt water layer positioned on the bottom of the approach channel. This salt water layer extends over a long distance from the lock into the approach channel.

The salt water on the fresh-water-approach channel is harmful for the agriculture, environment, fishery, and so on, and should be removed.

To remove this salt water layer the best way is to collect it in a deep sink in the bottom of the channel and to flush it away with a discharge sluice or pumping station.

Another way is to avoid salt water on the freshwater-approach channel, as much as possible. This requires special lock systems (see PIANC'1986 Report, section 12)

A3 Ice control in locks

The following focuses on methods to avoid ice formation in locks and in the locks entrance channels (emptying and filling). For more information on ice formation in channels see PIANC InCom Working Group 23 "*Technical and Economic Problems of Channel Icing*" (published in 2004). Some of the results of this report, the ones most relevant for ice control in locks, are presented in Table 5.1. The report points out the fact that the methods for ice control at locks are largely dependent on the water temperature in the lock. Experience has shown that the water temperature should be approx. $0.3 - 0.5^{\circ}$ C before the use of bubbling systems or the continuous use of current inducers in ice control is possible. Otherwise the use of these methods leads to an increased ice formation, which worsens the ice situation. In this case it is advisable to stop the traffic for the cold period instead of trying to continue it.

Methods	Means	Results	
Thermal	Conducting additional heat in	Efficient and considerably cheap if waste heat is	
engineering	to the water mass	available. Otherwise expensive.	
	Steam	Quite inefficient	
	Hot water	Efficient	
	Bubbling systems	Efficient if there is heat capacity in the water mass,	
		otherwise may increase ice formation	
	Solar heat	Inefficient	
	Wind power	Inefficient	
	Heat of the ground water	Could help in locks	
	Dusting of ice sheet (mixing	Works if performed at the right moment, otherwise	
	with ashes)	inefficient	
Electrical methods	Resistors in the lock wall	ck wall Works, but the changing of resistors is difficult	
	Heating panels and mats on	Efficient, but damage resistance is poor. Quite	
	the lock walls	expensive and reduce the usable width of the lock.	
	Resistor panels on gates	Efficient	
	Radiant heaters	Inefficient	
Chemical methods	Low-adhesive coatings on the	Poor duration and require regular renewal. Reduce	
	lock walls	the ice adhesion strength of ice, but do not eliminate	
		the need for manual ice removal.	
	Lowering of the freezing point	Calcium chloride and sodium tested with poor results.	
	of water		
Mechanical	Bubbling systems, high flow	Efficient in controlling ice movements in the locks,	
methods	air screening and surface	e especially in the gate recesses and the upper funnel	
	current inducers	area of the lock.	
	Use of manpower	Requires a lot of labour, reliable, used together with	
		other methods	
	Chipping	Widely used in removal of ice from structures,	
		reliable, but time consuming	
	Excavator	Used during the most severe winters to relieve the ice	
		situation in the lower funnel and waiting areas of the	
		locks, may damage the canal structures	
	Chainsaw	Relatively inefficient and vulnerable. May cause	
		damage to the canal structures.	
	High pressure water jet	Expensive	

Table 5.1 : Methods for ice control in locks (InCom-WG23, PIANC 2004)

To estimate the ice formation in locks and canals with ongoing traffic and the need for additional heat

in the locks and canal, a series of field test were carried out in Finland. The tests included full scale

tests on the Saimaa Canal with warm water conduction to the canal. The purpose of the tests was to find out the needed heat input to the canal in order to control the ice situation so that the brash ice layer thickness would not exceed 1.0 m.

The results from the canal sections can be generalized to the locks. The theoretical calculations were checked and calibrated with the field study measurements.

The amount of brash ice forming during the ice breaking is approximately:

 $V_L = (q^{t*B_r})/(L^{*}(1-e)) (m^3/m)$ where

 B_r = width of the trough (lock)

t = time between the ice breakings

L = fusion heat of ice (307 MJ/m^3)

e = porosity of brash ice (approx. 0.33)

q = the heat loss in ice formation, $12*T^{0.9}*n^{0.3}$, where

T = mean temperature between the ice breakings (°C)

n = frequency of ice breaking (times a day)

The thermal energy needed to melt the ice is approximately:

 $E_1 = T_w^* v^{1.5*} k_1^* A^* b$ (W/m) where

 T_w = temperature difference between the flowing water and ice melting point (°C)

v = flow velocity (m/s)

 k_1 = trough roughness factor

A = constant 4,800 (W*s^{1.5})/(m^{3.5*o}C)

b = width of the bottom of the brash ice layer (m) $k_1 = (1+4*h_1)*C=1$

 h_1 = mean thickness of the brash ice layer (m)

C = equalizing factor (h_1/h_{1max})

 h_{1max} = mean maximum thickness or local maximum thickness of the brash ice layer

The vessel passages also consume the thermal energy when it rotates the warm water melting the ice. This energy can be estimated with formula:

 $E_2 = k_2 T_w Q (W/m)$ where

 k_2 = vessel factor (0.5 – 2.0) Q = constant (50 MJ/°C*m a vessel passage)

The amount of ice melted between the ice breakings is approximately:

 $V_s = (E_1 + E_2)/(L^*(1-e))$

In order to get a stabilised ice situation in the lock the heat input must be at the same level as the heat loss from the lock. The results indicated that the heat loss from the canal is between 1 - 4kW/m, when the brash ice layer is approx 1.0 m thick. Most of the heat loss takes place through the brash ice layer. When the trough in the canal is approx. 20 m wide, the heat loss is approx. 50 -200 W/m². In lock the heat loss is somewhat bigger, if the brash ice layer is kept thinner and if the bubbling system and current inducer are in use. Anyhow, the heat loss of 200 W/m² corresponds to the heat loss from open water at -10°C temperature, which means that the required heat input very seldom exceeds this despite the use of bubbling and current inducers except in severe winters. These results indicate that for instance the maximum heat input to control the ice situation in a 110 m long and 12 m wide lock would be approx. 66 - 264 kW. This heat input can theoretically be done for instance by conducting 3 litres/s of +30°C water to the lock.

With regards to ice formation, the Finnish Maritime Administration carried out a major study on the Saimaa Canal during 2004 - 2005. As a result of this study this administration created an ice formation simulation model for the Saimaa Canal.

The study consisted of field measurements in different canal ice conditions with different canal heat capacity. These measurements were the basis in creation of an ice formation simulation model, which was used to verify the previous theoretical calculations of the required heat input needed in controlling the ice situation in the canal.

The simulation consisted of three traffic situations (one, two or four ship passages per day). The results pointed out that the designed 120 MW heat outlet (discharge of warm water with 1 km intervals) is sufficient in controlling the ice situation. During mild and normal winter vessels with 1A-ice class could operate almost without icebreaking assistance. During severe winters icebreaking assistance is needed and extreme winters the use of convoys is needed. The simulation model also pointed out that short traffic stops effectively reduce the formation of ice. The model also pointed out that the melting effect in between the warm water discharge points varies more than predicted in the theoretical calculation.

New vessel and propulsion systems (such as double acting ships and rudder propulsion systems) ease the ice navigation problems as they become more common.

A4 Case study: Naviduct in Enkhuizen

A4.1 Introduction

The navigation lock in *Enkhuizen* (the Netherlands) has been constructed between 1969 and 1971; it is the Northern connection between the *IJsselmeer* and the *Markermeer*. The navigation lock is suitable for vessels of class CEMT Va. Adjacent to the navigation lock there is a stop lock (*spuisluis*). Across the navigation lock a road is situated. The bridge can be opened when sailing ships have to pass.

The existing navigation lock is causing delays for vessels and road traffic during the (busy) summer season. The capacity of the navigation lock is insufficient for



Figure App3: 1 Naviduct Enkhuizen

recreational ships. It is expected that the number of ships will increase within the next decade causing more and more delays for the vessels. In addition, the bridge crossing the lock is causing traffic delays when opened.

To resolve this problem a Naviduct will be built on the intersection between ship- and road traffic. These functions have to be combined. The Naviduct is an aquaduct which is able to function as navigation lock. In this way the capacity of the road and waterway will be increased.



Figure App3: 2 Artist impression of the Naviduct



Figure App3: 3 Situation near Enkhuizen and the Naviduct

A4.2 Terms of Reference (Programma van Eisen)

A4.2.1 General

The registered water levels between 1976 and 1990 have been analysed. Based on this data the high water levels have been determined. In addition to this wave action caused by wind has been determined. The increase in water level in the IJsselmeer over the operational life span is estimated at 0.20 meters.

	IJsselmeer	Markermeer
Design level based on	NAP+1,90 m	
1/10.000 yrs		NAP -1,05
Significant wave height	0,69 m	
Wave period	2,2 s	
Wave length	7,70 m	
Design level based on		NAP +1,70
1/1.000 yrs	NAP -1,50	0,90 m
Significant wave height		2,5 s
Wave period		10,0 m
Wave length		
The soil conditions can be summarized as:

- Weak soil layers with low permeability consisting of clay and peat till NAP-13.0 m

- Sand layer, till NAP -25.0 m
- Clay layer, till -31.0 m
- Capillary height (stijghoogte) water in weak soil layer NAP -0.40 / -0.20 m
- Capillary height Sand Layer NAP -2.00 m

Based on prognoses the number of vessels passing the locks is

Vessel type/number of passages	1989	2010
Inland navigation	4.500	4.650
Other commercial traffic	5.800	5.000
recreational vessels > 20 m	5.000	10.000
recreational vessels < 20 m	55.000	80.000

A4.2.2 Functional requirements

The global dimensions are determined by 2 parallel navigation locks which have an efficient length of 125 m and width of 12.5 m. The sill depth is determined at NAP -4.50 m. The maximum water level in the lock is NAP + 0.45, the minimum is NAP -0.5 m. Locking is possible at water level differences less than 1.00 m.

A risk analysis has been performed which states that a protection of the lock gates against collision is not necessary. During periods of frost and ice, it should be able to operate the locks. The design life of the construction is 100 yrs, the gates and other replaceable elements must be designed cost efficiently.

The navigation lock has to be designed to retain:

- High water level on the IJsselmeer (probaility of failure is 1/10.000)
- High water on the Markermeer (probability of failure is 1/1000)

The level of the gates is at NAP +2.00, the level of lock at NAP +2.65 m. The height of the existing *Houtribdijk* is NAP + 6.40 m. A new dike will be constructed at NAP + 4.00 m. At a water level exceeding NAP+ 0.70 m or at water level differences larger than 1.00 meter all gates will be closed.

A4.3 Design

Excavation works

Costs associated with excavation work (*grondverzet*) account for a large factor in the budget. For this reason the lock will be located as much to the North as possible, against the *Houtribdijk*. The orientation in East-West direction depends on the chosen steepness of the road (5%). The orientation in North-South direction is chosen in a way as to limit the length of the breakwater (*leidammen*) and ensure sufficient manoeuvring space for the vessels in the waiting and berthing area. This resulted in an orientation which makes an angle of 10⁰ to the north as shown in Figure App3: 4.



Figure App3: 4 Sheetpiles (damwand) around the artificial polder to construct the Naviduct

The design is made in such a way that the road and navigation lock is located in an artificially created polder. The lowest level, consists of the road, is at an elevation of NAP -10.50 m.

The weak soil layer is excavated to a depth of NAP -11.50 and replaced by sand. The remaining soil layer with a thickness of 1.50 meter remains to function as a vertical barrier between the different water levels (capillary height). The artificial polder has sheet piles which act as a barrier in horizontal direction. The sheet piles are founded in the clay layer. The weak layers near the *Houtribdijk* are dredged away.

Lock gates and lock head

The gates should operate independently. To minimize the width the island between the 2 lock chambers should be as small as possible. Hence, the hydraulic gate driving machinery has been positioned behind each other, parallel to the lock axis, as is illustrated in Figure App3: 5.



Figure App3: 5 Hydraulic system of the lock gate

The lock head has been designed around the selected gates. A cross section is shown in Figure App3: 6.



Figure App3: 6 Cross section of the lock head

Lock chamber

The lock chamber differs from conventional navigation locks because the lock chamber is constructed above a road. Demands on the water retention of chamber floor and bottom are therefore high. In addition to these demands the lock chamber retains soil in the artificial polder.



Figure App3: 7 Cross sections of the 2 lock chamber

The slice in the middle of the lock chamber is one big monolithic construction consisting of the lock chamber, pump room, and road intersection. The pumps are installed for the drainage of the artificial polder.



Figure App3: 8 Cross section Naviduct at waterway-roadway intersection

A4.4 Construction

The following construction methods were taken into consideration:

- a) construction pit with dewatering (open bouwput en bemaling).
- b) cofferdam with sheet piles and underwater concrete (bouwkuip)
- c) cofferdam with sheet piles and underwater concrete and tension piles (bouwkuip met trekpalen)
- d) pneumatic caisson (pneumatisch afzinken)

The use of a construction pit was selected since this proved to be the most cost efficient.



Figure App3: 9 Construction methods

A4.5 Waiting- and berthing place

The lock is enclosed by a waiting and berthing place in the North as well as the South. The function is to:

- Provide an area to wait
- Protect the navigation lock against wave attack
- Create a calm area which facilitates manoeuvring



Figure App3: 10

Waiting and berthing locations Naviduct

A4.6 As-built

The construction of the Naviduct started in 1999 and was finished in 2003.



Figure App3: 11

Naviduct Enkhuizen