Course CT4460

Polders

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These lecture notes are part of the course entitled ‘Polders’ given in the academic year 2014-2015 by the Water Resources Section of the faculty of Civil Engineering, Delft University of Technology. These lecture notes may contain some mistakes. If you have any comments or suggestions that would improve a reprinted version, please send an email to o.a.c.hoes@tudelft.nl.

When writing these notes, reference was made to the lecture notes ‘Polders’ by Prof. ir. J.L. Klein (1966) and ‘Polders and flood control’ by Prof. ir. R. Brouwer (1998), and to the books ‘Polders en Dijken’ by J. van de Kley and H.J. Zuidweg (1969), ‘Water management in Dutch polder areas’ by Prof. dr. ir. B. Schulz (1992), and ‘Man-made Lowlands’ by G.P. van der Ven (2003).

Moreover, many figures, photos and tables collected over the years from different reports by various water boards have been included. For several of these it was impossible to track down the original sources. Therefore, the references for these figures are missing and we apologise for this.

We hope that with these lecture notes we have succeeded in producing an orderly and accessible overview about the genesis and management of polders. These notes will not be discussed page by page during the lectures, but will form part of the examination.

March 2015

Olivier Hoes

Contents

1 Introduction

Water has always been a big issue in the Netherlands. Sometimes there is too much, sometimes too little, and now and then it leads to dangerous situations. On the one hand excess water has to be discharged as quickly as possible to prevent flooding; on the other hand freshwater has to be conserved. Freshwater is used to control water levels, as drinking water, for irrigation, and in preventing salinization (by flushing or dilution). In recent years a water shortage has led to an increased number of botulism and blue-green algae cases and to problems with cooling water capacity for electricity production.

Water can also be a threat because of river discharge, sea or extreme rainfall events, for example:

- RIVER DISCHARGE On 31 January 1995 the river Rhine discharged 12,000 m^3/s, forcing 200,000 people to leave their homes;
- SEA Large parts of the provinces of Zealand and South Holland were flooded due to a spring tide during the night of 31 January 1953. The storm caused over 90 dike breaches and approximately 1,835 people were drowned;
- EXTREME RAINFALL The total precipitation in 1998 measured 1,240 mm, while the average annual precipitation over the period 1971-2000 was only 793 mm. Extreme rainfall events on 13 and 14 September and on 27 and 28 October 1998 flooded large parts of the Netherlands. The national government paid a total of € 290 million in compensation for the damage caused.

These threats, combined with the fact that a quarter of the Netherlands is below mean sea level (Figure 1-1), make measures necessary to protect the country. Examples of measures are the construction of dikes, ditches and pumping stations.

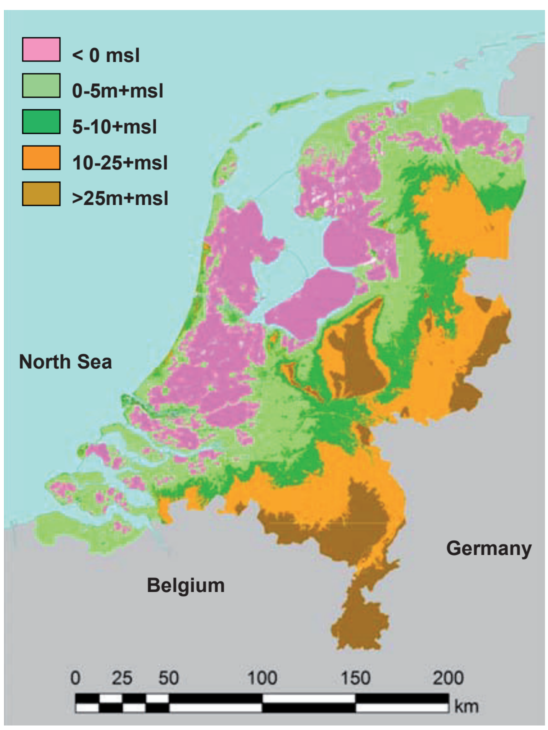


Figure 1-1 Elevation map of the Netherlands; roughly 25% is located below Mean Sea Level (MSL) (source: AGI-RWS)

On the one hand, excess precipitation needs to be drained. On the other hand, water levels also need to be controlled, in order to secure the growth of agricultural crops. To do so, excess water is drained by ditches or drains into canals. These canals discharge into larger canals, which discharge their water into rivers, lakes or the sea by means of pumping stations. A belt canal system can be used as an intermediary.

Water levels are controlled not only in polders and belt canal systems but also in urban areas, brooks and even whole river basins. Since river basins usually have a relatively large slope, the levels are controlled by weirs. Embankments and dikes also protect the land from large discharges into brooks and rivers.

Ditches and canals in polders have practically no slope. Therefore the water level can be completely controlled, independent of the adjacent area. Pumping stations and drainage sluices discharge excess water and water can be let in during dry periods. This taking in of water can be problematic because of both the quality and quantity of the water.

Dutch catchments with larger slopes can be found in the more elevated areas in the east and the inner dune area. Polders are catchments with hardly any slope and can be found in the low-lying areas and near the large rivers. Between the two is a transitional area with slopes deviating from relatively large to non-existent.

In the Netherlands polder and dike management is one of the oldest forms of engineering. It is probable that dikes already existed in peat areas more than two thousand years ago. Though the Romans occasionally constructed a dike, the serious construction of dikes along the large rivers started around 800 AD. Dikes along the sea have an even later origin. Urban areas were first protected from sea flooding; rural areas did not follow until much later.

In the Middle Ages people became more involved in flood prevention and control. More dikes were constructed, draining the area by drainage sluices, Dutch scoops and Archimedean screws. In the 14^{th} century wind mills improved the drainage and made it possible to reclaim marshes and lakes.

With the invention of steam-driven pumping stations at the end of the 18^{th} century more and deeper lakes could be reclaimed. Later the diesel and electrical pumping stations followed. Nowadays almost all Dutch stations are driven by electricity, but in other areas of the world diesel is still used, especially in areas where electricity is insufficiently available.

Dutch polder and dike engineering have led to some prestigious projects such as the Delta Works in the Province of Zealand and the Zuiderzee Works forming the current Lake IJssel. Both these projects were not only developed for flood protection, but also for freshwater retention, recreation, and nature conservancy and urban development.

2 Geology and soils of the Netherlands

2.1 Geological sequence of soils

When thinking of geology one of the first questions that arises is how old the earth is. A few centuries ago man assumed that it was 6,000 years old. However, throughout history understanding of the earth has continued to improve; by 1800 its estimated age had increased to 80,000 years and nowadays it is assumed to be 5 billion years old with a 3 billion year-old crust. The thickness of the crust is generally thought to be 40 km; drillings have given us information about the outer 4 to 5km.

Table 2-1 Sequence of geological eras

Era		Million of years ago (mya)	
Precambrian		4500 -	542
Paleozoic	Primary	542 -	245
Mesozoic	Secondary	245 -	65
Cenozoic	Tertiary	65 -	2.4
	Quarternary	2.4 -	now

However, the solid crust has not been static since its origin. The plates were periodically pressed together and folded into mountain ranges; in the relatively stable periods that followed, chemical and physical degradation caused erosion and sedimentation. This process of mountain formation is called orogenesis (or orogeny). Because of these periods the earth now consists of land and ocean, instead of just oceans with thick sediment beds.

2.2 The Paleozoic, Mesozoic and Cenozoic (tertiary) eras

The Paleozoic era, also known as the Primary era, began roughly 542 million years ago. Originally it was assumed that the oldest life forms had their origin in this era. Later on it was discovered that life forms did exist even earlier but that it was very primitive; therefore fossils from the Paleozoic era are the oldest complex life forms with a skeleton or shell. In the Paleozoic era the continents drifted towards each other, creating one large continent named Pangea. During this era the Netherlands was sited on the equator where tropical coral reefs grew. During the so-called Variscan orogeny the crust was tilted, the coral reefs died, and the Netherlands became a coastal swamp/lagoon where transgression (a sea level rise that causes the shoreline to retreat upwards) and regression (a sea level fall that causes the former sea bed to be exposed) created a sequence of organic layers and sand/silt layers. During this era the Netherlands moved northwards, which caused a sequence of soils and rocks of great economic importance: brown coal, pit coal, salt layers and natural gas.

The Mesozoic era started roughly 245 million years ago and is subdivided into the Triassic, Jurassic and Cretaceous eras. This era is probably the best known era in history because of its dinosaurs. During this time the Netherlands moved further north and the climate changed from dry subtropical (desert) to Mediterranean. The prevailing erosion reduced the relief. Salt layers and petroleum are the main benefits from this era.

The Cenozoic era started approximately 65 million years ago. Pangea was broken and continents drifted apart. Mammals came into prominence and reptiles became less important. However, tectonic influences were not yet over: horizontal pressure caused soil to fold and horsts and gorges were formed. Coal fields and sand layers were broken up and oil was formed in the anticlines. In the Netherlands marine sediments are the main soils from this period. Due to sea level rises and falls, the sea expanded (transgressed) or shrunk (regressed). The sediments from this period are heavy marine clay that has low hydraulic conductivity.

2.3 The Cenozoic (Quarternary) era

There are several subdivisions and time boundaries available for the Quaternary. Here we will use the North European subdivision and time scale. The Quaternary is subdivided into two time spans: the Pleistocene (2.4-0.01 million years ago) is the period with large ice sheets and the Holocene (10,000 before present - present). The Pleistocene is also called the Diluvium and the Holocene Alluvium.

The lower boundary of Quaternary soils is approximately 600m below ground level in the province of North-Holland. The Quaternary is of great importance to the Netherlands because it formed the country's top soils; during this period the human race became the most important species on earth.

Table 2-2 Subdivision of the Quaternary (mya million years ago, BP before present)

Era	Sub-era*	Period
Pleistocene	Pretiglian*	2.2-2.4 mya
	Tiglian	1.7-2.2 mya
	Eburonian*	1.3-1.7 mya
	Waalian	1.1-1.3 mya
	Menapian*	1.0-1.1 mya
	Bavelian*	0.8-1.0 mya
	Cromerian*	0.4-0.8 mya
	Elsterian*	0.32-0.4 mya
	Holsteinerian	0.30-0.32 mya
	Saalian*	0.13-0.30 mya
	Eemian	0.10-0.13 mya
	Weichselian*	0.01-0.10 mya
	*with cold period/glacial	
Holocene	Preboreal	10,000-9,000 BP
	Boreal	9,000-8,000 BP
	Atlantic	8,000-5,000 BP
	Subboreal	5,000-2,900 BP
	Subatlantic	2,900 BP - present

Pleistocene

The Pleistocene period is the period of ice ages. The ice age phenomenon is quite rare in the earth's history; other period with ice ages were the Carboniferous (300 million years ago) and Silurian (435 million years ago). Not all countries use the same standard names for the ice ages because the glacial depositions from the Alps did not make physical contact with those from Scandinavia, Scotland or North America.

Table 2-3 Names of glaciers from different origins

Alpine	NW Europe	Great Britain	North America
Günz	Eburonian	Beestonian	Nebraskan
Mindel	Elsterian	Anglian	Kansan
Riss	Saalian	Wolstonian	Illinoian
Würm	Weischelian	Devensian	Wisconsinan

During the ice ages, land ice and glaciers expanded and covered large parts of the land. In the intermediate periods, known as inter-glacials, the temperature returned to 'normal'. However, all ice ages from the past million years or so last much longer (100,000 years) than inter-glacials (10,000 to 20,000 years). So, you might argue that current temperatures are abnormally warm compared to the 'normal' temperature during ice ages!

Not everything is known yet about the causes of ice ages. There are both terrestrial and extraterrestrial causes for ice ages and the main extraterrestrial cause is irradiation on the northern hemisphere by the sun. This irradiation varies according to the Milankovitch cycles (a Serbian mathematician who lived between1879 -1958):

1. The earth has an elliptical path around the sun. The eccentricity of this ellipse changes every 96,630 years. This cycle has an influence on the distance between the earth and the sun;
2. The tilt of the earth's axis with respect to the orbit varies from 21.8° to 24.4° over a period of 41,000 years. This cycle influences the angle of the solar radiation reaching the earth. Currently the earth's tilt is 23.5°. In 10,000 years' time the tilt will reach a minimum of 22.6°, which will cause a smaller difference between summer and winter than now;
3. The position (precession) of the earth's axis shows an oscillation over a period of 25,800 years; this cycle determines on which day of the year the earth is closest to the sun. A good example of this precession is the movement of a child's spinning top. When the spinning top is not completely upright, gravity will pull it around its rotating axis. However it will not fall over, because it will rotate around the vertical axis instead.

Ice ages are characterized by large areas with land ice. Therefore there must be land where snow and ice can accumulate; this can either be land at high latitudes or on highly elevated land at lower latitudes. Because between the Carboniferous and the Tertiary one large continent was formed on the equator, no ice ages were formed in this period.

What triggers ice ages is still unknown. One theory is the braking down of the Gulf Stream. The Gulf Stream transports warm, salt water from the tropics to the north. Here the water cools down and therefore becomes heavier. It then flows back over the ocean bed to the south. This phenomenon causes temperatures in Northwest Europe that are possible to live in. The theory is that when the ice at the North Pole melts it disrupts the density balance of the Gulf Stream and therefore the Gulf Stream stops. This will cause the temperature to drop and the ocean to cool down, which allows the glaciers to grow. When snow and ice accumulates, the albedo of the earth (its reflectivity) increases and this will intensify the process.

The beginning of an ice age is characterized by an increase in precipitation; this falls in vast quantities in the form of snow and accumulates in the northern regions. In other regions the precipitation will be in the form of rain. The accumulation of snow causes a drop in the sea level, which increases the slope of rivers. This increased slope and increase in discharge from precipitation will cause extensive erosion. The accumulated snow will become ice under its own pressure of weight and will start flowing downwards. The moving ice transports debris, which is known as moraine material.

When the temperature starts to rise, precipitation decreases and glaciers regress. The sea level rises and the erosive force of rivers decreases and with that the glacier's transport capacity. Sedimentation starts with the rough, heavy materials and is followed by finer materials, until the river bed is almost completely filled up. During the next ice age the river will cut into its own sediments. When this happens quickly, the old sediments will remain in place and river terraces will form.

During the first two ice ages mentioned in Table 2-3 as well as in the last, ice did not reach the Netherlands. During the Waalian interglacial era (between the Eburonian and the Elsterian eras) mainly clay was deposited (i.e. the Tiglian formation). During the Holsteinian interglacial era (between the Elsterian and Saalian eras) depositions were mainly fine to very coarse sands and only locally some clay or peat, which occurred mainly in the provinces of Drenthe, Groningen and Friesland.

For the Netherlands the most important glacial era is the Saalian glacial era. In this era ice covered most of the Netherlands, roughly the area above the river Rhine. Fluvial sand deposits are found south of this line.

Before this ice age, the rivers of Germany and the Netherlands mainly flowed to the north. However, the ice obstructed this flow and the rivers were forced to flow west. These rivers (of which the Rhine is the most important) deposited very heavy clay (in Dutch*: potklei*). In the northern parts of this area the boulder clay (in Dutch*: keileem*) is well known. During this ice age the sea level was about 100m lower than it is today. Most sediments that were deposited during the Holsteinian interglacial were subsequently eroded.

The ice put great force on the ground it flowed over. The soil was frozen, which caused it to break under the pressure. The frozen soil plates were pushed to the side out of the glacier's way. The hills that were formed are called lateral moraines (in Dutch: *stuwwal*); examples are the national parks at Veluwe and Heuvelrug near Utrecht.

After the regression of the glacier during the Eemian interglacial, fluvial erosion occurred; the sea level rose to 4-6m above its current level and a large inland lake was formed (Lake Eemian, where Lake IJssel is now). Some sand layers were deposited, and in other parts thick peat layers developed. During the warmest period in the Eemian interglacial, forests reached up to the North Cape, where there is now tundra.

During the last ice age, the Weichsel glacial, ice did not reach the Netherlands. The temperatures were very low, permafrost was common across the whole country and the sea regressed to form a polar desert. Sediments on the sea bed were exposed due to the regression and wind transported sand over the land. The two most common deposits were wind-borne sand deposits (in Dutch*: dekzand*) and loess. Loess particles are finer and lighter than sand deposits; therefore the loess travelled farther and was deposited in Limburg, while most of the sand was deposited on hills in the Veluwe and North Brabant.

At the end of this glacial the climate changed, temperature rose and the top soil melted. The melt water and river water flowed over the soil through the deep riverbeds. Swamps and peat were formed. Slowly the soil became more permeable, the sea level rose and the North Sea gained its current shape.

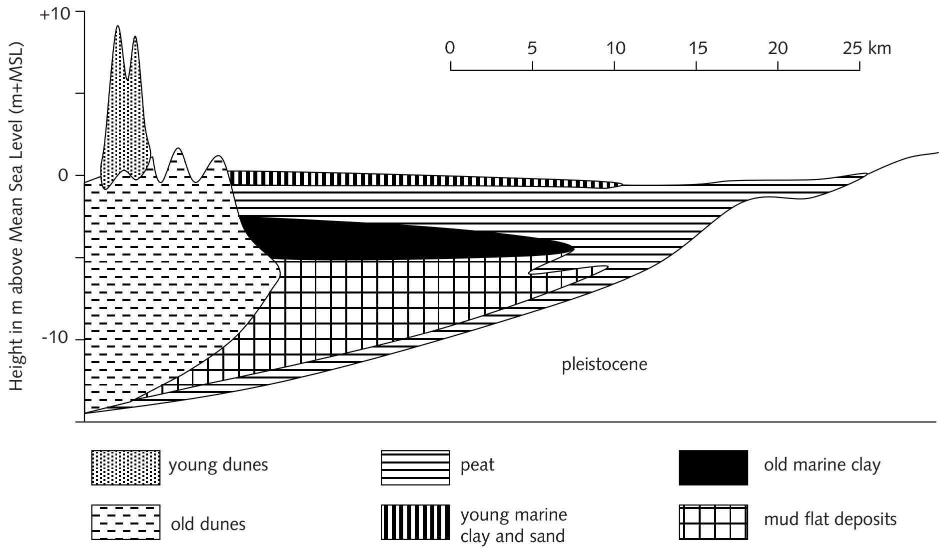


Figure 2-1 Holocene in the west of the Netherlands

Holocene

Generally speaking, the Holocene is an interglacial. Interglacials have a time span of approximately 20,000 years, which means that according to the Milankovitch cycle a new ice age will occur in 10,000 years' time. However, this assumption must be treated cautiously because we do not know what the effect of human life will be on the processes. The increased greenhouse effect and the melting of polar and glacial ice make it theoretically possible for an ice age to be created before that time. However, the terrestrial conditions for an ice age will be fulfilled (land on one of the poles or high mountains on lower latitudes) because the continents will not move more than 1km in the coming 10,000 years.

The boundary of the Holocene and its deposits is not consistent. Generally the start of peat formation is taken as the beginning of the Holocene (10,000 years BP). At this point in time the climate became more stable and suitable for human life. In the beginning of the Holocene, the sea level was approximately 20m lower than it is today. In the western and northern part of the country vast peat areas were formed until the beginning of the Atlantic. During the Boreal the weather was dry and warm with prevailing easterly winds. During the Atlantic, westerly winds prevailed, providing warm and moist conditions. The peat layer that developed was on average 10m thick.

The sea level started to rise approximately 7,000 year ago, preventing more peat from forming. The peat was flooded from the west and the deposits which were laid on top of the peat layers compressed the peat. This layer, which was lower at the west, mostly lies approximately 20m below mean sea level (MSL) and forms an impermeable layer that is broken in places by flushing.

The sea level rise was partly absolute due to the melting of ice and glacio-eustatic deformation and partly relative due to the tilting of the tectonic plate that the Netherlands is placed on. The sea flooded the provinces of North and South Holland and the northern parts of Friesland and Groningen. Sand deposits covered the peat with layers of more than 10m. Waves caused by the westerly winds deposited new sandbanks along the coast that were the base for new dunes. Behind those new dunes lagoons were formed where finer deposits were placed. In the western parts of the Netherlands 4-6m deep old marine clay deposits were formed. The new dunes closed together, therefore the lakes behind the dunes became freshwater lakes and new peat was formed.

Around 5,000 years ago, the dunes stopped growing, because winds changed and fresh sand was no longer provided. Peat growth began again in the Subatlantic. The border between the two peat formations is called the Weber border. Along the coast some dunes were broken through during the regression periods and peat was flushed out. The Zuiderzee (later Lake IJssel) was formed approximately 1,000 years BP by one of the breaks in the dune.

In many coastal areas and also around Delft, floods occurred. The channels of those floods deposited coarser material than the surrounding floodplains. Due to settlement, the floodplains are now lower than the channels. Later, between 1,100-1,600 years ago, fresh sea floods deposited marine clay. Rivers also flooded local peat and the peat was eroded or covered with fluvial clay.

Large volumes of sand were released during the erosion of the dunes. This sand was flushed to the shore where it was picked up by the wind and transported onto the land. New dunes were formed approximately 2,850 year ago. However, due to a large rabbit population, the vegetation was affected and the sand was blown away. The dunes now have a tendency to move inland, but this is prevented by man.

Knowledge of the layers of the soil is important for polder management. In particular the properties of the deeper layers (mainly their permeability and conductivity) are of great importance. The dependency on the sea, rivers, canals and lakes are determined by those properties. Groundwater quality is determined by the soil layers it flows through and the properties of the soil are determined by its geological history. In addition, marine deposits can contain salt. Also, the thickness of layers is important, both for crop cultivation and the distance to aquitards.

2.4 Human influence on soil formation

Soil formation is now hampered by human processes, such as dike implementation. When man first came to the Netherlands, around 12,000 BC, they settled on the high areas. Later, when the lower areas began to be populated, they built villages on the higher locations in the low-lying areas. In the provinces of Friesland and Groningen artificial mounds were built to place the houses on. These mounds were the first human interference with natural processes.

These types of mounds can also be found in the province of Zeeland. Here the degree of rise in the sea level can be established from looking at the raising of the mounds. Up to 1200 AD the mounds were raised frequently or even periodically even though they were already built on the higher locations.

Although the Romans probably built some dikes for the protection of villages, the first real dikes were built later in approximately 800 AD. By building dikes, first along the rivers and later on along the shore, the water was shut out, preventing it from depositing sediments, through which the natural growth of these areas no longer continued. Outside the dikes these sediments continued. Dikes prevent flooding and erosion, which means that they are a measure of land conservation.

Shutting out water not only shut out natural growth but also caused settlement of the soil, which was also increased by drainage for agriculture. Settlement continues even today, but it has decreased. Most settlement occurs where the soft layers are thickest, which is in the western and northern parts of the country. Very recently land conservation has led to a shortening of the shoreline in the form of the construction of the Lake Ijssel Dam and the Delta Works.

Human influence has also caused some geological depositions to disappear; this influence can be considered destructive. For example, large-scale peat mining activity created lakes with marine clay beds. The lakes caused erosion due to waves in the prevailing wind direction. In order to prevent this erosion the lakes were reclaimed. Lakes that still remind us of the peat mining period are the lakes close to Loosdrecht, Vinkeveen and Reeuwijk. In other parts (e.g. Zeeland) salt mining was common. In particular peat mining outside the primary dikes can be dangerous and has led to large-scale erosion.

In the east, north and south peat mining was also a large industry. However here peat was formed on diluvial sands and mining exposed fertile soils, because the top layers were mixed with the underlying sands. In river forelands subject to flooding (in Dutch*: uiterwaarden*) clay was mined for use in brick and roof tile production.

River sand was extracted for the raising of building sites, road construction and lime-sandstone production. Deep sand soils were also a source for extraction. Sand and gravel extraction from the Meuse caused new vast water bodies.

In contrast to this loss of land, there has also been a gain in land. Most lakes that were created in peat mining have since been reclaimed. This has happened with large parts of the current Lake Ijssel. Creating dikes outside the primary dike area is still taking place. Therefore measures are being taken in order to gain land.

2.5 Sea level rise

Causes

During the last ice age (Weichsel glacial) the water level of the North Sea was approximately 60m below the current mean sea level. This means the North Sea was almost completely dry; this was because all the water had accumulated in the glaciers and snow. Since this glacial period, temperatures have risen, causing the ice to melt and sea levels to rise. The temperature rise has not been gradual; fluctuations are commonly accompanied with sea level rises (transgression) and falls (regression). An important transgression caused the Westland formation: mud flat deposits on top of the peat layers. The sea level is still rising, albeit at a slower rate.

It is generally assumed that the total volume of water on earth is a constant, whether it is in fluid, solid or gas form. Currently an estimated 22 million km^3 water is accumulated in ice in the polar regions and glaciers. If all this ice melted, the sea level would rise 55m above the current mean sea level; fortunately this has never occurred in geological history. Nevertheless the prevailing temperature rise will cause the sea level to rise further due to the melting of ice and the lower density of water at higher temperatures.

Relative sea level rise

The elevation of the Netherlands is determined with respect to NAP, the Normal Amsterdam Level (Dutch: *Normaal Amsterdams Peil*), which is almost equal to the mean sea level (MSL). This refers to the Pleistocene subsoil, which means that the NAP is linked to the Pleistocene subsoil and will therefore also decrease. In the southern part of the Netherlands this decrease is negligible; in the north it is 2 to 4cm per century. A possible cause is the rise of Scandinavia due to the melting of the ice caps and the subsequent reduction in the force on the soil.

The rise in the sea level with respect to NAP is called the relative sea level rise; it consists of the actual sea level rise and the reduction in the NAP. In this relative sea level rise an actual sea level rise has the same effect as a reduction in the NAP reference level.

The relative rise in the sea level over the past hundred years can be derived from measurements taken at a number of reliable staff gauges in the sea. A rise of 17cm per century has therefore been calculated. Lately it has also been possible to determine the relative sea level rise with respect to the Pleistocene subsoil. In order to do this the age of the peat just above the Pleistocene level is used together with the help of C 14 research. Carbon that the plants took in whilst they were alive is composed of the normal carbon atom with a weight of 12 and an isotope with an atom weight of 14. After the plant dies, the C 14 slowly deteriorates and the radioactivity reduces over a known half-life of 5,570 years. In this way the age of a large number of peat samples lying on the Pleistocene can be determined. The results of the western part of the Netherlands are given in Figure 2-2. The curve does not show any measurement from the past 2,000 years, because insufficient samples are available. The sea level rise with respect to the Pleistocene subsoil since the last ice age, 75 cm per century, appears to have been very high but it has subsequently decreased. It is currently approximately 20 cm per century (Figure 2-3). For hydrological calculations a value between 35 and 85 cm per century should be taken for reasons of climate change.

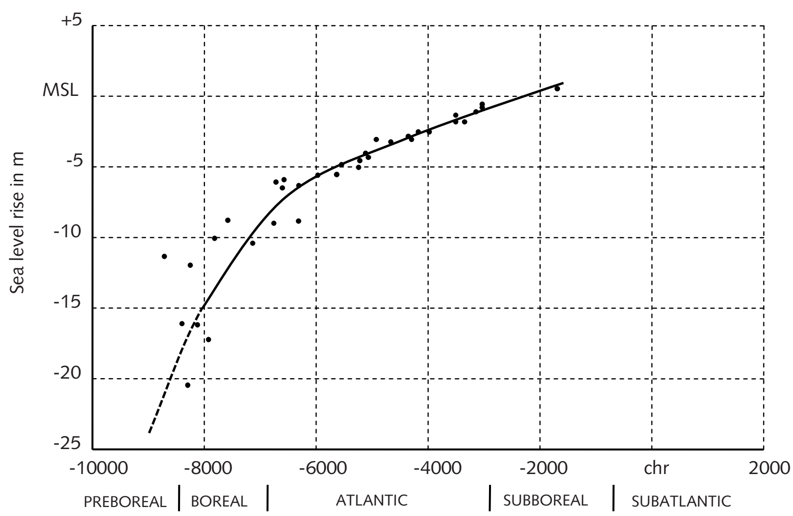


Figure 2-2 Relative sea level rise with respect to the Pleistocene subsoil (Klein, 1966)

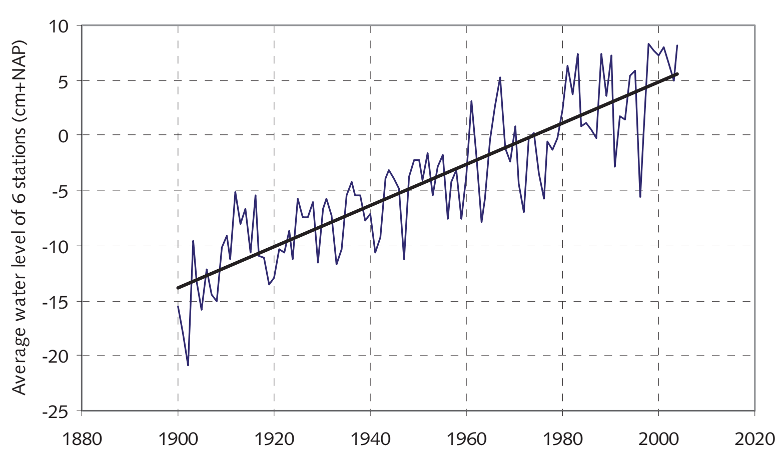


Figure 2-3 Relative sea level rise between 1900 and 2004 in the Netherlands (www.milieuennatuurcompendium.nl)

Settlement

As well as the relative sea level rise, the drop in the ground level compared to the reference level NAP is important. This drop is caused by human interference and by settlement. The ground level drops due to the extraction of clay, and sand and peat mining.

Settlement is a big issue. Newly-formed peat layers can consists of up to 80% water; clay layers in polders can contain 40 to 50% water. When pressure is put on these soils, water is pressed out and settlement occurs. In the southern islands of Zeeland peat layers in the subsoil have caused 1 to 2m of settlement in the past 2,000 years. In the surrounding marine clay polders, settlement has caused a descent of 20 to 60cm over the same period.

Pressure increase also occurs when land is reclaimed, where the water level is brought down to below ground level. When constructing new polders on clay ground it is important to always consider settlement as this can be very important where there are layers of peat in the subsoil. Peat that is deep in the subsoil can also be subject to settlement, as research with C14 measurements has showed. These peat layers are now only a few decimetres thick, where they once were over 5m thick. However, this settlement has now ceased.

When water levels in existing polders are reduced, settlement can also occur, cancelling out part of the level lowering. This phenomenon is also known as 'pumping down a polder'.

Peat soils not only experience settlement due to the lowering of the polder water level, but settlement also allows oxygen to enter the soil, causing deterioration of the organic matter. This deterioration alone lowers the ground level approximately 10cm per century.

The settlement in many polders, which forces the polder water level to be lowered, and the continuous rise of the sea level have had a large influence on the discharge methods of various polders. Older polders have had to replace discharge structures with gravity flow pumping stations, because outer water levels have kept rising with respect to the polder water levels. The older polders are often much lower than newer polders and the old ones now often discharge into the newer ones. On many islands typical saucer-shaped spots were formed, such as on the island of Tholen.

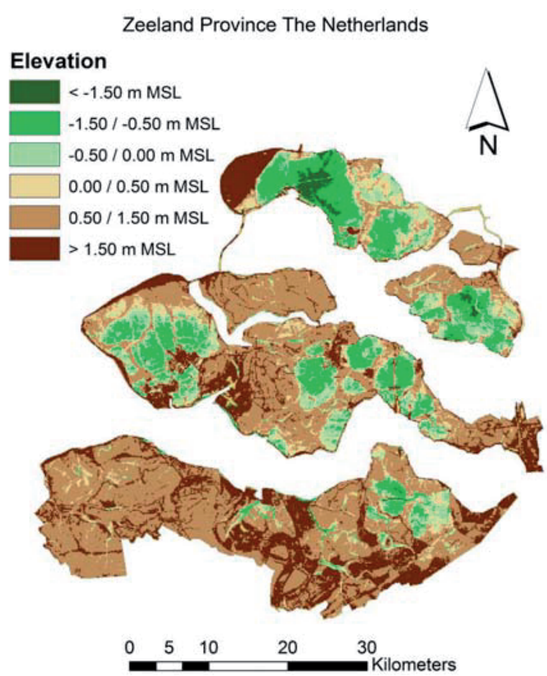


Figure 2-1 Saucer shapes in the islands of Zeeland (Source: AGI-RWS)

3 Polders and belt canals

3.1 The creation of the Dutch polders

Natural levees, mounds and dikes

The first inhabitants of the country now called the Netherlands lived on high diluvial ground. Lower-lying parts of the Netherlands were also occupied later on, on naturally elevated areas such as natural levees and Pleistocene outcrops. Later still, in a period of rising sea levels, people in the northern part of the country were forced to build mounds as a consequence of rising sea levels.

The first dikes were built at the beginning of the ninth century AD to protect inland villages from flooding. In this way large areas in the provinces of North and South Holland, the west of Utrecht, parts of Friesland and Overijssel and later also of Groningen were gradually embanked and cut off from external water. The task of these dikes was to keep out the external water: the sea, inlets and tidal channels, creeks, lakes and large areas of water that connected with rivers and the open sea. At first, it was not possible to discharge excess water, but very quickly valves were installed that were opened at ebb (low) tide.

Dams and artificial discharges

In the second half of the 13th century many creeks, tidal channels and rivers in the West of the Netherlands were closed off at the mouth. Place names ending in 'dam' such as Edam, Monnickendam, Amsterdam and Rotterdam etc. are reminders of this damming. However although these dams removed the threat of flooding but also the loss of ebb tides which allowed to discharge water under free flow. The in those days available means to discharge artificially, such as Archimedean screws or Dutch scoops, were only partially effective. Therefore surplus water could no longer be discharged. However the invention of the windmill at the beginning of the 15th century improved this situation. The combination of dams and windmills turned brackish water systems into non-tidal freshwater belt canal systems *(*in Dutch *boezemstelsel)*. Adjacent land also discharged into the belt canal, initially by gravity. As time passed the soil in some of these areas began to settle and eventually needed to be embanked and sometimes drained.

Large parts of the belt system land still discharge today by gravity. Examples are in Rijnland, Delfland and Fryslan. The surface water levels in these belt canals are usually kept a few decimetres below [1]NAP (Normal Amsterdam Water Level).

Many of the current belt canals have their origin in the 16th century, excavated as ring canals surrounding reclaimed lands. However the North Sea Canal belt system was built in the second half of the nineteenth century when the IJ bay was closed off. It connects the port of Amsterdam to the North Sea.

[1] Known in Dutch as *Normalnull*, nowadays almost equal to MSL.

Figure 3-1 A belt canal near Delft (photo: Water Resources Section, TU-Delft)

The origin of parcelled landscapes ('slagenlandschap')

The vast peat areas in the provinces of North and South Holland and the west of Utrecht were drained starting from the higher clayey areas along the rivers. The urban areas were originally established on the natural levees of the river and on the sandy ridges. This is also where the fortified towns of Naarden, Woerden, Oudewater and others lay.

Strips of ground (in Dutch *slag)* parallel to the rivers were drained. The excess water was transferred into the river, initially using an Archimedean screw or Dutch scoop. Later windmills replaced this manual labour. Often more than one strip was reclaimed and the strips were separated from each other by embankments and watercourses (Figure 3-2).

Figure 3-2 Parcelled landscape -'slagenlandschap'.

Reclaiming lakes

The invention of the windmill in the 16th century made it possible to reclaim lakes. In the 17th century, the Dutch Golden Age, reclamation boomed, because many rich merchants considered this new land to be a good investment. Later on it also became possible to drain deeper lakes too by siting several mills together (in Dutch a *molengang*) to increase the potential lift. They were referred to as two man mills or three man mills according to how many mills were placed together. Afterwards the use of Archimedean screw mills meant that the same lift could be achieved with just one mill.

The shape of the reclaimed land

The reclaimed land in the north western part of the Netherlands was created out of lakes, which became elongated as a result of erosion caused by the prevailing wind. The reclaimed land usually has the deepest point – and the mill –at the north-east point of the length, which runs from south-west to north-east.

Erosion was often the main reason to reclaim these lakes. For example, reclamation of Lake Haarlem (the *Haarlemmermeer*) in the middle of the 19th century was mainly driven by its growing threat to the cities of Amsterdam and Leiden. This lake was the last to be reclaimed.

The Lake IJssel polders were among the last to be reclaimed; East Flevoland (*Oosterlijk Flevoland*) was drained between 1950 and 1957 and South Flevoland (*Zuidelijk Flevoland*) was drained between 1959 and 1968. These polders can easily be identified by their regular grid of rectangles defined by roads and watercourses.

The reclaimed land in the provinces of South Holland and Utrecht consists mainly of drained peat lakes that originated from the creation of the old peat polders. Villages lay along the canals which used to be used to transport peat by boat. During the process of collecting peat, narrow ridges of peat were left in place alongside the canals to maintain groundwater levels and to protect the urban areas from flooding. These ridges still stand a few meters higher than the surrounding reclaimed land today.

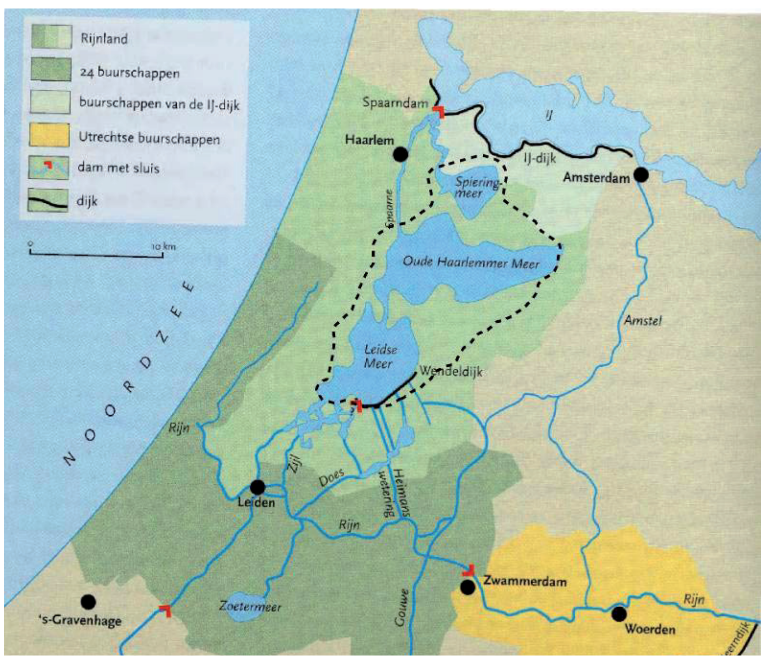


Figure 3-3 Spaarne was dammed (solid black line) in 1253. Three watercourses were constructed in the South for better drainage (Source: G.P. van de Ven, Leefbaar Laagland, 2003.) The dotted line shows the current Lake Haarlem.

The reclamation of estuaries

In the northern provinces of the Netherlands the gradual draining of the estuaries formed curved polders around the old *Middelzee* and the Dollard. Along the coast the continual silting and sedimentation made a progressive embanking possible in strips, parallel to the coast. This was also carried out at the top of North Holland, West Brabant and along the northern embankment of the river Meuse (*Maas*). The gradually increasing sea level, which caused the later polders to be embanked at a higher level and increased settlement of the older polders, caused a rising ground level towards the coast.

Circular shapes formed by embanking urban areas

On the islands in the southwest of the Netherlands embanking started from the urban centres, creating more or less circular dikes. The rise in sea level and settlement gave these islands a typical circular shape, as can be seen at Tholen. Sometimes the embanking started from two centres, which eventually grew together, such as on Goeree-Overflakee.

3.2 Types of polders

A polder is defined as a level area which has originally been subject to a high groundwater or surface water, and is separated from the surrounding hydrological regime, to be able to control the water levels in the polder.

Thus, a polder shows the following characteristics:

- A polder does not receive any foreign water from a water course, but only from rain, seepage, or by irrigation intake;
- A polder has an outlet structure (sluice or pump) that controls the discharge;
- The ground water and surface water level are independent from the water level in the adjacent land. These water levels are artificially maintained in order to optimize the objectives of the polder.

The names for the different types of polder are based on their location. ‘River polders’ discharge into rivers. ‘Outer polders’ are located outside the primary flood control system and have their own dikes. These polders discharge directly into open water or to an embanked polder. ‘Inner polders’ are located inside the primary flood control system; their excess water is discharged directly into open water or other polders. ‘Summer polders’ are situated outside the primary flood control system; although they are surrounded by a dike or embankment, they are usually only dry in the summer. In the winter they are flooded when there is a high river level or storm. There are also summer polders, mainly in Friesland, that are only drained in the summer.

Polders can also be divided according to their origin. Reclaimed land (*droogmakerijen)* are formed by the draining of lakes and sea belt canals. These occur mainly in the provinces of South Holland, North Holland and Utrecht. They also occur in Friesland and Groningen but these are usually shallower. The IJsselmeer polders are among the newest of this type of polder.

Endikement has occurred because of the embanking of land outside the dikes. This land can be made of mud flats or salt marshes, which are dry at high water, and of silt, which is only dry at low water. Silting (*aanwas)* is the term for the growth of land laying outside the dikes in a horizontal direction; sedimentation (*opwas)* is the growth in a vertical direction.

The new land consists of both drained and poldered land, that in general have regular divisions and grids. In contrast, the old land often has irregular water ways and paths.

The polders can be further divided by their ground type and use. There are peat polders, clay polders and sand polders. When dividing by ground use there are arable polders and grassland polders, which can also be divided into those used for the production of hay and those used for grazing cattle.

Supporting functions

In water management the central issue is to support the user functions, e.g. wildlife, agriculture, residential, recreation or shipping. In the Dutch system these functions are defined by the provinces. The water board translates them into quality and quantity objectives, resulting in the necessary everyday management of required water levels, rinsing regimes, etc.

Most functions mainly depend on groundwater. However the groundwater levels are dependent on the surface water levels. Therefore an understanding of the principles behind the system is crucial. The systems can be very complex, because often compromises have to be made between diverging interests. A lowering of the surface water level may be needed to increase the drainage of a section for agriculture butt the same time this drainage can be harmful to the countryside because it depends on local seepage. Therefore an assessment has to be made.

Water level decrees

The whole process of water system analysis and balancing of the stakes involved is carried out in preparation for the water level decree *(*in Dutch *peilbesluit)*. All the areas under the supervision of a water board are evaluated in a long term cycle. The targets that are established for the surface water level (or the regime) are set down in water level decrees (see textbox) that have legal status. Provincial regulations state precisely when a water level decree must be established for a particular body of surface water. The water level decree contains a ruling on the desired water levels in the water courses for the various seasons. Water boards are obliged to maintain these target levels as much as possible.

Excerpt from the Dutch water management law (*Wet op de Waterhuishouding)*, Section 2 Water level decrees, Article 16. Source www.wetten.overheid.nl

'It is statutorily required for a quality controller to make one or more water level decrees for water bodies under his control. The quality controller is responsible for ensuring that the target levels laid out in the water level decree are maintained as much as possible for the periods that are set. By determining the water level decree, articles 5 and 9 of the management plans have to be taken into account, since these apply to the surface water bodies that the water level decree is laid down for.'

The function support in polders can be easily carried out by water management because polders are hydraulically-independent areas, in which the water level can be managed, largely independently of the surrounding area.

Water level departments

Usually a polder has more than one target level. The polder can be divided into polder departments or sections, where deviating levels are dealt with. In fact these level sections are also polders, divided from the other polders by dikes and constructions. The handling of the level in a level department is carried out by siphons, discharge sluices, pumping stations, dams or closable culverts.

Water discharge

Drainage can be either of groundwater (*ontwateren*) or surface water (*afwateren*). Excess water is discharged into small channels through drains and trenches. These channels then discharge into larger channels which lead to discharge sluices or pumping stations that lead to open water. Sometimes water is discharged by gravity flow, when the receiving water level (sea, river of Ijsselmeer) is lower than that in the polder. When this occurs, a drainage sluice or dam is placed at the border of the polder and the receiving waters so that the water level can be managed. With drained polders the receiving water has a water level that is higher than the polder and a pump is necessary to overcome this height difference. Discharge through gravity flow is hardly ever possible nowadays. This is due to the settling of the soil and the rising of the sea level which on many polders has caused the external water level to gradually become higher than in the polder.

The general assembly of the district water board De Oude Rijnstromen:

Having read the relevant proposal by the board of chairman and members;

Having considered the advice of the water management committees;

Having considered the decisions in the Regulations Water Management Rijnland;

DECREES THAT:

1. Eight water level departments are established for the polder known as 'Het Langeveld'. The boundaries are indicated on the certified map, which is enclosed as a supplement to this decree;
2. For the water level departments mentioned under 1 the following target levels are appointed:

Area	Summer level	Winter level
1.24.1.1	NAP+2.15 m	NAP +2.05 m
1.24.1.2	NAP+1.70 m	NAP+1.60 m
1.24.1.3	NAP+1.60 m	NAP+1.50 m
1.24.1.4	NAP+1.15 m	NAP+1.05 m
1.24.1.5	NAP+1.10 m	NAP+1.00 m
1.24.1.7	NAP+0.75 m	NAP +0.65 m
1.24 1.8	NAP +0.45 m	NAP +0.35 m
1.24.1.9	NAP +3.90 m	NAP+3.80 m

3. The summer level will be set around April/May and the winter level will be set around September/October.

As decided in the public meeting of the general assembly dated 8 December 2004.

{signature}, chairman

3.3 The belt canal system

As already discussed, excess water from polders can be discharged into open water bodies, i.e. the sea, rivers, and lakes. However, another option is to discharge into a belt canal system, which will discharge into one of the water bodies above. Almost all polders along the Dutch coast discharge into a belt canal system.

Belt canal system levels

As well as a storage and drainage function, the belt canal often functions as a waterway for the transport of cargo or for wildlife and arable farming in the belt canal area (the area draining directly into the belt canal area).

The target level is set, taking all its functions into account. However it is usual to set upper and lower boundaries rather than one target level.

Closed belt canal systems

When a belt canal system is declared closed, further pumping is banned *(*in Dutch*: maalstop)*. As soon as the upper boundary level is reached, polders are no longer permitted to discharge into the belt canal system. This is done to prevent the breaching of dikes and flooding of the areas surrounding the belt canals. When drainage is by gravity flow the polders are allowed to drain into the belt canal system at all times. In general, this happens with belt canal systems that have a large storage capacity.

Some polders have a storage basin that is used for the storage of water when discharging into the belt canal system is not permitted. This system is also applied along the coast line, which enables water to be stored at high tide which can then be discharged by gravity flow at low tide by using the access channel.

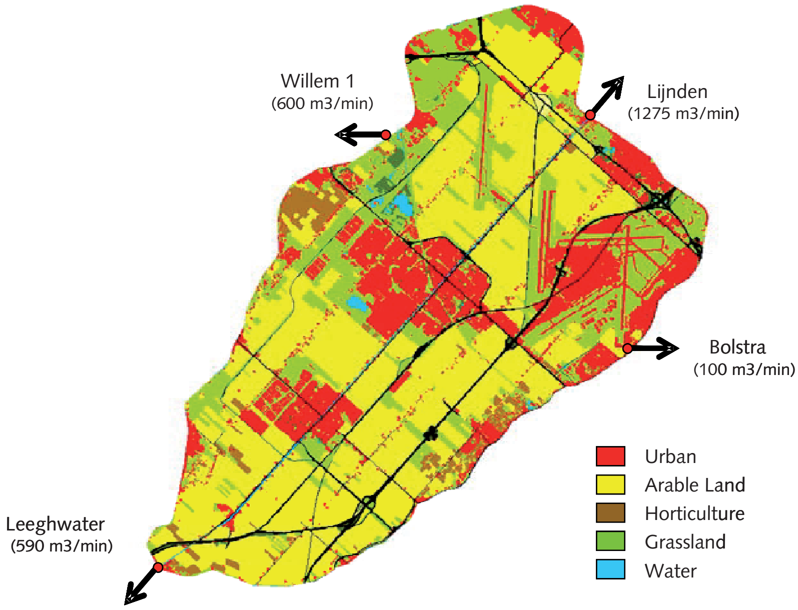
3.4 Water boards today

In the Netherlands water boards are responsible for the water systems. Water boards are public bodies and all stakeholders are represented in the water boards, i.e. landowners, residents, environmental organizations, and industries. Water boards are responsible for dikes, domestic waterways and roads, water discharge and intake, and water quality.

The boundaries of the water board area, number of board members and their salaries, and the task of the water board are defined by the Provincial States. Usually a general regulation is set for all water boards within the province. The elections, voting rights, financial administration and management are also covered by this regulation.

Figure 3-1 The new (1,275 m^3/min) and old (819 m^3/min) pumping stations at Lijnden in the north of the Haarlemmermeer polder.

The *Haarlemmermeer* polder



The Lake Haarlem (*Haarlemmermeer*) polder is located between Amsterdam and Leiden and has a surface area of 18,230 ha. The national airport Schiphol and residential areas of Hoofddorp, Nieuw Vennep, Zwanenburg and a part of Badhoevedorp are situated in this polder.

The land was reclaimed between 1849 and 1852 using three pumping stations: Leeghwater, Cruquius and Lijnden. These three stations pumped out more than 830 million m^3 to drain Lake Haarlem (*Haarlemmermeer*).

Nowadays the precipitation surplus and seepage water is discharged by four pumping stations: Lijnden (1,275m^3/min), Leeghwater (590 m^3/min), Bolstra (100 m^3/min) and Koning Willem I (600 m^3/min). With all pumping stations running the total pumping capacity is 20 mm/day.

In day-to-day quantity management the pumping station at Lijnden in the north of the polder is used. The Bolstra pumping station is used to keep Schiphol Airport dry. The pumping stations at Leeghwater and Koning Willem I are only used under very wet conditions when the Lijnden pumping station is not sufficient.

The reason that Lijnden pumping station is used is that a significant amount of brackish seepage water enters the polder. By using Lijnden the distance that this 'chloride cloud' has to travel through the belt canal to the North Sea is minimized. If the Leeghwater station in the south was to be used, the water would pass important (salt sensitive) horticultural areas. In order to minimize the chloride concentration in the polder, water from the belt canal is let into the polder through siphons; about 30 million m^3 per year. How many mm is this per m^2?

The Haarlemmermeer polder is subdivided into 83 water level departments. The largest has an area of 4,267 ha and it is located along the main canal that runs from the pumping station at Leeghwater straight to Lijnden. The ground level in the polder varies from -6.0m NAP near Nieuw Vennep to -4.0m NAP at Schiphol. The land use is currently divided as follows: 400 ha surface water, 300 ha wildlife, 2,700 ha grassland, 7,900 ha arable land, 300 ha horticulture and 6,700 ha built-up areas (buildings, industry, roads, and Schiphol Airport).

Not all polders are regulated. For many of the smaller polders that are owned by one or only a few people, regulations seem unnecessary. The management of a water board is carried out by a chairman, secretary, treasurer and board members, usually elected by the landowners in the area. Their vote is pro rata according to the amount of land they own but for large areas of land this is usually reduced. Landowners with small amounts of land usually have a combined vote and in some water boards even tenants have a right to vote. Where electing the management by all the landholders is not possible for practical reasons, for example, where a water board has a large number of landholders, only the primary landholders will take part in the election.

The landowners are charged with the annual costs incurred by the water board, proportional to the amount of land owned and this is decided on the basis of a register which lists the landholders and the amount of taxable land that they hold. In some situations the costs of a certain measure are so high that the interests of all landowners are taken into account when allocating the costs.

The joint structures of a polder are sometimes listed in a register (in dutch: *legger*), together with a description of the location, size, ownership and person responsible for maintenance. The maintenance is mainly taken care of by the polder itself, except for that of the channels and the main canal. These are cleaned and dredge by the landholders of the adjacent land. When a piece of land is leased, the maintenance is the responsibility of the leaseholder. The board performs periodic inspections (in Dutch*: schouw*) to check this maintenance.

The water board has the authority to lay down regulations, which must be followed, if necessary under police force. These local regulations are necessary for the correct management of the water board's interests. For example, the landholders are obliged to maintain certain waterways at levels that are laid out in the register. However it can also state what is not permitted, e.g. to dig a hole in a dike.

Figure 3-4 View of the main canal from the Leeghwater pumping station facing North.

The task of the water board is to make sure that the water system functions properly under changing circumstances such as environmental planning, climate and policies, meaning that the water system has to comply with regulations concerning safety and function support. This specifically means:

- Measuring and keeping databases up to date. Fixed obligations to bodies such as provinces and the State make it necessary to continually monitor the condition of the water system. Measurements necessary for the operational management and the carrying out of the other tasks are specific for each water board;

- Meeting the targets set concerning safety: i.e. taking care of the dikes, signalling and raising an alarm and handling crisis situations;
- Operational water management: determining, maintaining and evaluating target levels or required surface water regimes in order to give maximum support to the functions of an area (wildlife, recreation, water transport) and, for example, responding to crises;
- Realizing and maintaining water quality standards: not only normal duties but also in connection with applications for permits or for improving the operational management;
- Guarding ecological targets established for example in connection with fish migration and the EU directive on birds and natural habitats.

Table 3-1 Water boards in the Netherlands

	Name	Seat	Area [ha]	Website
1	Aa en Maas	Den Bosch	161 000	www.aaenmaas.nl
2	Waternet	Amsterdam	70 000	www.agv.nl
3	Brabantse Delta	Breda	171 000	www.brabantsedelta.nl
4	Hoogheemraadschap van Delfland	Delft	41 000	www.hhdelfland.nl
5	De Dommel	Boxtel	151 000	www.dommel.nl
6	Wetterskip Fryslân	Leeuwarden	346 000	www.wetterskipfryslan.nl
7	Groot Salland	Zwolle	118 000	www.wgs.nl
8	Hoogheemraadschap Hollands Noorderkwartier	Purmerend	196 000	www.hhnk.nl
9	Hollandse Delta	Dordrecht	100 000	www.wshd.nl
10	Hunze en Aa's	Veendam	207 000	www.hunzeenaas.nl
11	Noorderzijlvest	Groningen	144 000	www.noorderzijlvest.nl
12	Peel en Maasvallei	Venlo	129 000	www.wpm.nl
13	Reest en Wieden	Meppel	137 000	www.reest-wieden.nl
14	Regge en Dinkel	Almelo	135 000	www.wrd.nl
15	Rijn en IJssel	Doetichem	195 000	www.wrij.nl
16	Hoogheemraadschap van Rijnland	Leiden	112 000	www.rijnland.net
17	Rivierenland	Tiel	200 000	www.wsrl.nl
18	Roer en Overmaas	Sittard	92 000	www.overmaas.nl
19	Hoogheemraadschap van Schieland en de Krimpenerwaard	Rotterdam	35 000	www.hhsk.nl
20	Hoogheemraadschap De Stichtse Rijnlanden	Houten	83 000	www.hdsr.nl
21	Vallei & Veluwe	Apeldoorn	245 000	www.wve.nl
22	Velt en Vecht	Coevorden	91 000	www.veltenvecht.nl
23	Scheldestromen	Middelburg	190 000	www.wszv.nl
24	Zuiderzeeland	Lelystad	150 000	www.zuiderzeeland.nl
25	Blija Buitendijks	Blija	123	

Figure 3-2 Map of all 25 water board areas in the Netherlands

Blija Buitendijks is the smallest water board area in the Netherlands with 123 ha and it has no structures at all. Its area is located outside the primary dikes and consists of mudflats. The area is inspected once a year by three volunteers. This mudflat is used as pasture for sheep.

4 Land acquisition, preparation and improvement

4.1 Introduction

Large parts of land in the Netherlands have become open water due to peat mining, salt mining, erosion, and flooding. However since the mid-13th century the Netherlands has also reclaimed roughly 400,000 hectares of land. A general summery of that reclamation is given in Figure 4-1. The surface area of land reclaimed over the last 500 years is approximately 310,000 hectares (ha), including the Lake IJssel polders. The majority of this reclamation took place in the 20th century (see Table 4-1 and Table 4-2).

Table 4-1 The polders of Lake IJssel in the 20th century

Polder	Surface area [ha]	Year reclaimed
Zuid-Flevoland	43,000	1968
Oost-Flevoland	54,000	1957
Noordoost Polder	46,000	1942
Wieringermeer Polder	20,000	1930
Andijk Poler (pilot-project)	40	1927
Total	163,000	

Figure 4-1 Land reclamation in the Netherlands since the Middle Ages

However not all of the land lost due to mining and erosion has been reclaimed. Some of the lakes that are a result of peat mining still exist (e.g. at Nieuwkoop, Reeuwijk, and Loosdrecht). The greater part of Lake IJssel also used to be land; a large part, but not all, of this lake has been reclaimed. In the province of Zeeland there are still areas of land that were flooded but that up until now have not been reclaimed; for example the *Land van Saaftinge* and the *Verdronken Land* (Drowned Land) in Zuid-Beveland.

Table 4-2 Polders reclaimed in the 17th, 18th and 19th century with a surface larger than 1,000 ha

Polder	Surface area [ha]	Year reclaimed
Groot Mijdrecht	2,020	1877
Noorder Legmeer Polder	1,400	1877
Groote IJ Polder	1,870	1875
Prins Alexander Polder	2,668	1874
Houtrak Polder	1,240	1873
Haarlemmermeer	18, 300	1852
Nootdorp Polder	1 ,138	1844
Zuidplas Polder	4 ,100	1840
Nieuwkoop Polder	2 ,710	1809
Zevenhoven Polder	1 ,778	1809
Bleiswijk Polder	3 ,750	1778
Bovenkerker Polder	1 ,345	1770
Hazerswoudsche droogmakerij	1 ,776	1765
Vierambacht Polder	1 ,785	1744
Binnenwegse Polder	1 ,106	1706
Driemans Polder	1 ,015	1671
Wassenaarsche Polder	1 ,031	1666
Schermer	4 ,770	1635
Heerhugowaard	3 ,500	1630
Wijde Wormer	1 ,620	1626
Purmer	2 ,750	1622
Beemster	7 ,100	1612
Total	69, 000	

Change of land use

Land use has changed drastically since the industrial revolution. New uses such as urban development, industry, shipping, sports facilities, and rail, road and air transportation all require land. These changes in use affected farmers and market gardeners, who were either given compensation or relocated to new polders.

Land improvement

Nowadays there is a major move towards land consolidation or improvement, because a lot of the original reclamation was inadequately carried out. This, because of the increased demands by agriculture, threatened to affect the soil profile (a vertical cross-section of the ground that shows the horizontal patterns; in practice usually up to 1.5m below ground level), elevation and drainage. Land consolidation, currently usually carried out as part of land exchange (the process in which the plots of lands, waterways and infrastructure in an area are reorganised according to their function), reduces these threats and achieves a higher yield.

4.2 Peat bogs and peat moors

No water no peat
3 to 5 thousand years ago, about 50% of what is today The Netherlands consisted of peat bogs; so-called peat bogs (laag veen) had risen between the dunes along the North Sea coast and higher inland sand soils, whereas on the inland sand soils peat moors (hoog veen) had evolved. Original impassable peat bogs - too wet to drive and to dry to sail - are nowadays very rare. Most of the peat bogs have disappeared by erosion and subsidence; as well as excavated by those who needed turf as fuel.

In Dutch the word for peat (*veen*) refers both to the soil material and the landscape. These landscapes are shaped when the production of plant remains exceeds the dismantling and removal of the same remains. Most often these processes are in equilibrium, but the presence of water can considerably reduce the degradability of organic material. When soils are fully saturated, both the temperature and amount of oxygen drop. The micro fauna - which should reduce the plant remains - becomes less active. Meanwhile bog mosses (e.g. sphagnum) start to grow which disgorge toxic compounds making the water acidic. This in turn leads to even less active moulds and bacteria. The plant remains degrade slower than that they are produced, and pile up as peat. Peat will never be formed without water.

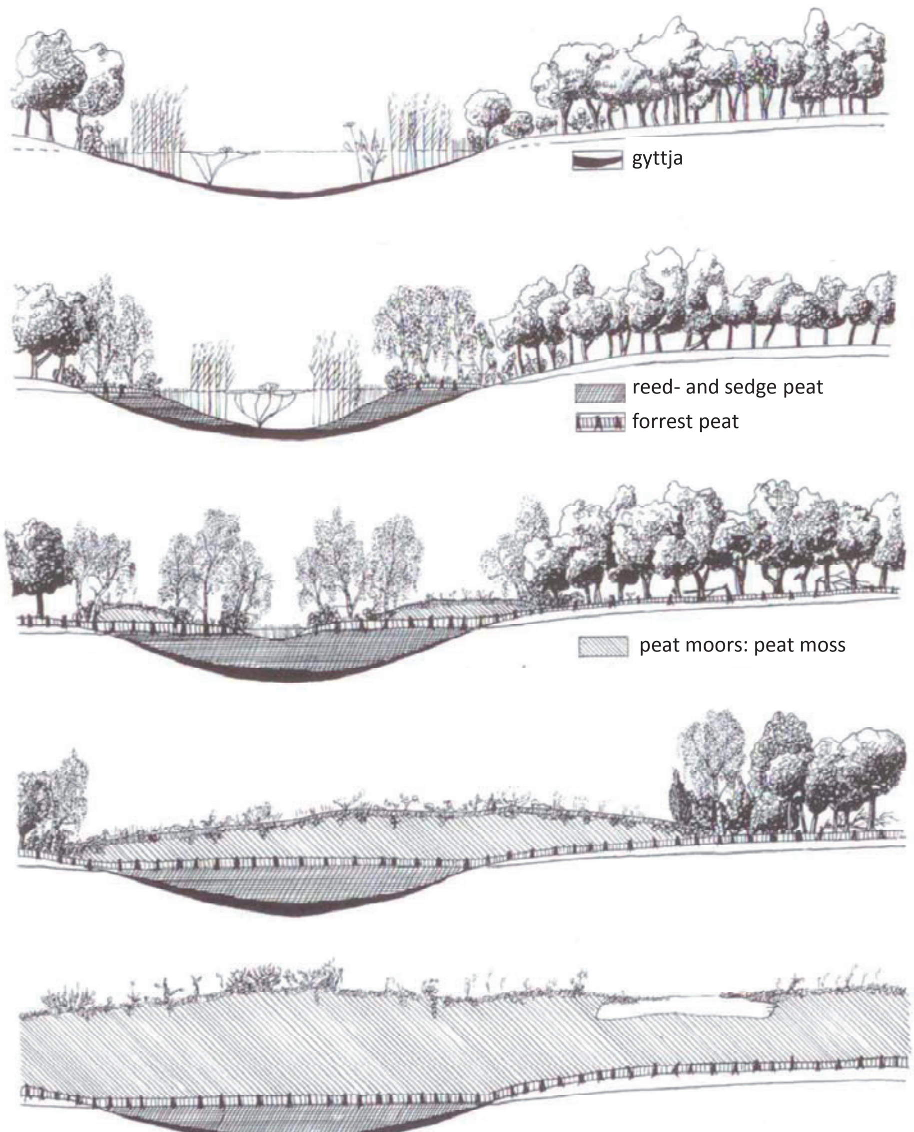
Just a few places have the right amount of water for the formation of peat. At tropical locations peat is formed in the monsoon season, but oxidizes in the dry season. On the other hand, it might rain so much that plant remains are flushed away. Currently, around 5 million km2 of the globe is covered with a peat layer of 30 cm or more. The water needed for peat is eventually coming from precipitation, but this does not necessarily fall on the same location where peat is formed. The necessary water can be supplied by a river or by seepage. This water absorbed compounds, influencing the formation of peat.

Peat bogs
Peats formed in eutrophic groundwater are called peat bogs. Most often these are located at lower elevations in a landscape. The types of vegetation that grow in peat bogs depend on different factors.

1) Water depth. Higher order plants cannot grow well if water is very deep, the peat is than formed by micro-organisms. This peat is called gyttja, which means mud in Swedish. At a depth of around one meter reed, cat's tail and at shallower depths sedge plants are able to grow. The peat formed here is called sedge peat (zeggeveen). If the water level is just below the surface than trees and bushes are able to grow and provide the plant remains. Alders (elzen), willows (wilgen), and birches (berken) form forest peat (broek- or bosveen).
2) Stagnant water. Whether water is moving or stagnant influences peat formation, as some plants do like and others do not grow in flowing water. Moreover, flowing water is also richer in oxygen, slowing down the formation of peat.
3) The chemical composition of the water determines partly which plants can grow in peat soil, and how fast the remains degrade to peat. Many high productive plan species grow well under eutrophic, neutral and base circumstances. However, bacterial activity is also high. The result is that it becomes difficult to recognize the plant remains in these peat types.
4) The type of plants is also of importance. During the last ice age all alders (elzen) disappeared from the Netherlands, and it took several thousand years for them to re-migrate. Birches however were present during this period, resulting in birches forest peat at places where you might expect alders forest peat.
5) Time. The circumstances, under which peat is formed, change by the formation of peat! A lake for example becomes shallower by the formation of peat, allowing different plants to grow. This makes the lake even shallower, resulting in other plants,

other peat, etcetera. A cross section typically shows a sequence of peat types. A growing peat bog often blocks the drainage possibilities in an area, which raises the water level and allows peat to grow at locations where it could not grow before.

As the permeability of peat is very low, peat bogs that get their water from groundwater only make their own growth close to impossible, as the layers are getting thicker and the ground water cannot reach the upper layers anymore. The result is that oligotrophic precipitation will determine the plant species that grow on top. This will be mainly peat moss. Under favorable circumstances this peat will grow above the surrounding areas and become peat moors.



Figuur 4-1Peat bogs and peat moors growing thicker and expanding horizontally

Peat moors

Peat moors solely depend on precipitation and are always above the surrounding area. The low permeability of peat prevents percolation from precipitation to the groundwater, causing that the water level is continuously high. This means that dying plants get submerged and increase the peat layer in thickness and in width. The result is a wet bubble on top of the original dry landscape, solidified by peat moss vegetation, some higher species and their remains. These higher species are specialized in order to withstand the nutrient scarcity, the acidity, and abundance of water in peat moors. These are small, fragile and thrifty with nutrients.

One of the vital requirements for peat moors is a more or less equally distributed precipitation surplus, which creates a high and stable or constant groundwater level so that the peat remains will be wet without interruptions. When water levels drop, oxygen enters and peat starts decaying. Many peat moors already exist for thousands of years, and are apparently capable of accommodating climate fluctuations. This is caused by several mechanisms through which peat is able to maintain itself:

A) The Acrotelm is the upper layer of several decimeters with a high permeability compared to the bottom layers. When the water level rises up to this permeable layer, the excess water will easily drain away. But the more the water level drops, the more difficult it gets to flow away. This feature is caused by the weight of the newest layers on top, which press upon the deeper layers and reduce the permeability. The Acrotelm flattens the daily and weekly meteorological fluctuations.
B) Mooratmung means that peat layers swell up under wet conditions and shrink when the water level drops. In swollen state, the plant remains are further apart from each other allowing water to flow away and vice versa. The result from Mooratmung is that the water level coincides pretty well with the lower side of the vegetation. This reduces variations in terms of weeks and months.
C) Peat moss compaction. Peat moss adapts to the actual circumstances. With less water, their inner tubes become small with more branches, improving their capability of capillary absorption. A side effect of this thick-set composition is a denser peat structure that can retain water. The reverse occurs with too much water. Peat moss compaction regulates monthly and yearly variations.
D) Changes in peat moss micro patterns. Some peat moss species are better able to cope with relatively dry and some others with wet circumstances. Thus under periods with a water shortage, the 'dry' peat moss species with many branches and a better capillary absorption capacity prevail. If the water level is continuously high, then the shallower 'wet' peat moss species proliferate. This damping mechanism works over terms of years and decades.
E) Changes in peat moss macro patterns. The peat production on the edges of a bog or moor does not differ from the production in the center. However in the center, where peat formation started, the layers are thicker and older. Although the degradability is slow, peat is always disintegrating and this makes this reduction is bigger in the center than on the edges. This eventually causes complete lakes emerging in the center of peat moors. This damping mechanism works over terms of centuries.

Peat bog reclamations

The Western part of the Netherlands used to be covered with a thick layer of peat pillows of several meters. Up until the 14th century excess water was drained by small peat streamlets towards the rivers. Reclamations of these peat bogs started in the 10th century and commenced so extensive, that the period is known as the 'Big Reclamation' (*De Grote Ontginning*). The objective was of course creating arable land. In the beginning the peat surface was still high, and digging several ditches was sufficient to create an accessible plot.

The reclamations started from a naturally dry basis: the more sandy levees along rivers and peat streamlets. More inland these natural streamlets needed to discharge the drainage water of the ditches were lacking, and had to be dug first. These artificial streamlets are called '*Weteringen*'. Farmhouses were built along these artificial drainage bases. Before starting digging, the peat bog colonists agreed on a direction of the ditches – usually perpendicular to the basis – and the distance between the ditches. The length of the ditches was not always decided upon and resulted in competition - every man for himself (*vrije opstrek*).

From the beginning of the 11th century the Bishop of Utrecht and Landgrave of Holland started to regulate reclamations. The contracts for reclaiming uncultivated land were called '*copen*', nowadays recognizable in village names like Nieuwkoop, Boskoop, Hoenkoop. Although the word '*copen*' is nearly similar to the Dutch word 'kopen' (to buy), these contracts were not purchase-agreements, but contained the duties and rights for future residents towards the authorities, and also the length of the reclamations. This immediately caused ribbon developments still present west and south-west of the city of Utrecht.

The drainage of peat bogs caused a gradual subsidence, as peat starts to oxidize when it gets into contact with air. At first, the well-drained areas were suitable for cereals, whereas cattle could graze on the parts not yet reclaimed. After some centuries, the surface had subsided that much, that agriculture was impossible. The surface level approached ground water level, causing that the reclaimed land could only be used as meadow-land. Cattle-farming became the most dominant agricultural activity in the western part of the Netherlands.

Meanwhile subsidence continued, and embankments were raised, to force back the water from outside. Polders came into being, draining on canals or rivers with still an open connection to the sea. These polders used ebb-tide to discharge excess water. Despite these efforts, agriculture did not return. Farmers suffered more and more from swampy grass-lands and farmyards. Raising the farmyard was often easier than deeper drainage of water. These dwelling terps were built between the 12th and 14th century and are around one meter above the surrounding surface level. Starting from the 15th century wind mills were also used to drain excess water to areas with higher water levels, but the problems with wet grass-lands remained.

Later, mainly in the 16th and 17th, the Dutch Cities were expanding rapidly and needed fuel! For many farmers it became economically interesting to excavate their land and sell it as turf. During the late middle ages, only peat above ground water level was sold. However, starting from 1530, submerged peat was also dredged. The local water authorities and provincial governments tried to stop this destruction, but without much success. Many areas became lakes, which were later drained again, turning them into polders like the Beemster and Schermer. Emptying these lakes in the 17th century were investment projects for new agricultural land. The layout was systematic; perpendicular angles, large plots, straight roads and canals. The geometric patterns used corresponded to the idea of beauty during these days.

4.3 Subsidence in peat polders

One of the major problems of peat polders is subsidence: buildings on pile foundations increasingly stick out of the surrounding surface, household connections to the gas and water network come under tension, saline seepage increases, ground water in confined Pleistocene layers may burst up the Holocene. This subsidence takes place due to three causes: oxidation of organic material, consolidation and shrinkage.

Oxidation of organic material start when people excavate small trenches to improve drainage from peat pillows. By doing so, peat loses water irreversibly which makes the surface accessible for farming, and also allows air to penetrate the soil, which starts the oxidation. The organic matter of peat originates from plant remains and contains carbohydrates, lignin (substance that binds wood cells), proteins , organic phosphorus, sulfur compounds, fats, waxes and resins. From these, the carbohydrates are the easily degradable compounds. 'Fresh' peat can consist of 80% carbohydrates. However, the percentage drops sharply with increasing degree of degradation. Peat with a high degree of decomposition is made up of around 20 % carbohydrates. In addition to plant remains, the organic matter of peat also consists of humus molecules. The proportion of humus molecules increases with the degradation. Humus molecules are acid substances with a high molecular mass. In general, humus molecules are more resistant to degradation than the plant materials from which they are formed. Thus, the composition of peat is a measure of the degree of degradation: peat with a relatively low carbohydrate content and a relatively high content of humus molecules has a higher degree of degradation.

The fresh organic material in the soil is broken down by micro-organisms. The main micro-organisms are bacteria, actinomycetes and fungi. The actinomycetes are strictly speaking also bacteria, but they - just like fungi - develop hyphae. The actinomycetes are good at breaking down cellulose and the more resistant components. The fungi are especially important for the start of the degradation process and are suitable for the degradation of lignin. The driving forces behind the peat degradation process are assimilation and dissimilation. Assimilation is the use of the converted organic components as building blocks for the cellular material of the micro-organisms. Dissimilation is the oxidation of the organic matter in order to obtain energy for the vital function of the micro- organisms.

Consolidation is the subsidence of the soil due to compaction by its own weight. Fresh peat consists of approximately 90% water. Due to drainage, peat that used to float, appears above the ground water level. The weight of this layer now has to be carried by the underlying layer. This lower layer is compacted, and at the same time water is squeezed out. Consolidation is a soil-mechanical process that decreases logarithmically with time. 60-80% of the consolidation takes place in the first 3 years following a water level adjustment.

The extra load after a water level adjustment produces primary consolidation and secondary consolidation. The primary consolidation occurs as a result of the reduction of the pores and starts when the load is applied and ends when all the excess groundwater is drained. Due to the secondary settlement the soil deforms very gradually due to creep of the grains. In the Netherlands, the formula of Koppejan is widely used for the calculation of consolidation.

Shrinkage is a decrease in volume due to drying of the ground. This shrinkage is seasonal and is undone in the winter by swell . However, a small portion is irreversible. In practice, this pulsating image of surface heights can reach an amplitude of 10 cm. In addition to land subsidence, shrinkage leads to extreme cracks in drier periods.

The Netherlands Agricultural Research Service (Dienst Landbouwkundig Onderzoek DLO) used to have several experimental farms, including one in Zegveld at which research on oxidation, consolidation and shrinkage was conducted for a long time. In 1966 the surface level was accurately measured and the plots divided into blocks with a high surface water level (30 cm minus surface level) and low surface water level (60 cm minus surface level). In 2003 the surface level was accurately measured again. The subsidence over this period in the high and low surface water level blocks was 6 mm and 12 mm per year respectively. What really matters is not the surface water level, but the groundwater level at the end of summer and the soil temperature. The higher the temperature and deeper the groundwater level, the faster the subsidence. To really limit subsidence, one needs to apply small distances between ditches or submerged drainage.

4.4 Reclaiming land and embanking

Reclaiming land

Reclaiming land is the process of pumping a lake dry. In doing this, one has to consider the adjacent areas and their interests in water discharge, intake and shipping. Due to the low water level of the reclaimed land, the groundwater levels in surrounding areas are often effected, as for example at Kuinre after the draining of the *Noordoostpolder* (North East Polder). In the later polders in Lake IJssel the lakes surrounding the polders were made much larger, in order to prevent this lowering of the water table.

Planning-wise, reclaimed land has to fit in the surrounding area when it comes to adjacent land use and infrastructure. Originally many polders were meant for agricultural purposes, but nowadays polders have other land uses such as urban development, industry, recreation and nature.

Embanking

Embanking is the process of damming up new land that has washed up outside the old embankments. Land used to be considered ready for embanking when the soil was washed up to above average high water. The soil that washed up consisted of heavy clay. The advantage of this method was that the new embankment was built up under dry conditions.

In 1904 a law was passed that ensured the proper implementation of a polder and the protection of third parties' interests. This law allowed reclamation either by or on behalf of the government, or by virtue of a water board regulation. Third parties, such as the inhabitants and landowners of adjacent land, could object to the Provincial Executive about the reclamation plans using a set procedure. This law is still operational today.

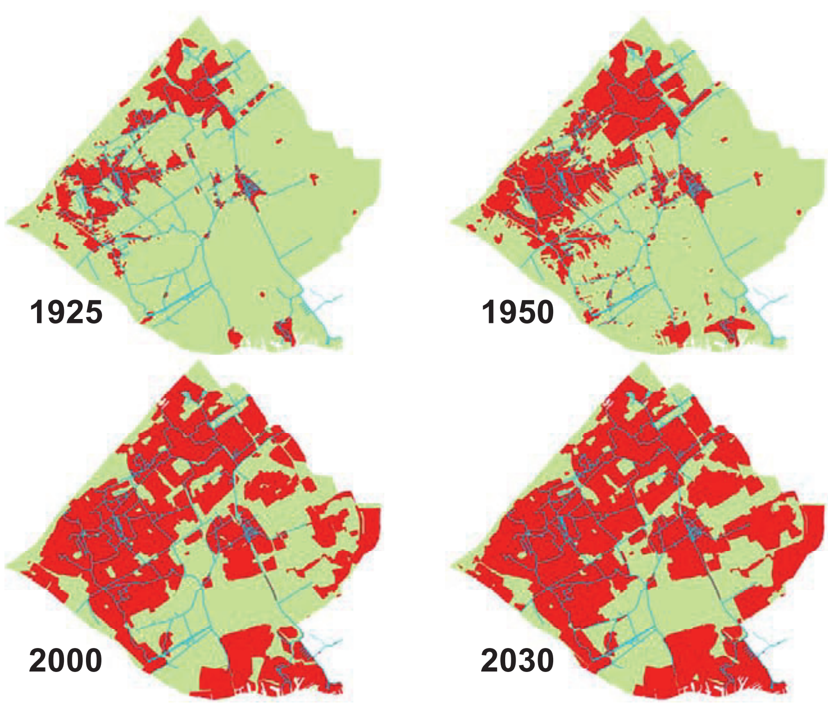


Figure 4-2 The increase in paved surfaces within the area of the Delfland Water Board. The blue lines are the belt canal system. The city of Delft is in the centre

Law passed on 14th of July 1904, containing regulations concerning the undertaking of land reclamation and enclosures. (Still in force on 25th of July 2008)

WE, WILHELMINA, by the grace of God, Queen of the Netherlands, Princess of Orange-Nassau, and so on and so on,

All those who will hear or read this, salutations!

Because We have decided that it is desirable to make decisions concerning the undertaking of land reclamation and enclosure;

We, having consulted the Council of State, and in joint discussion with Parliament, have therefore approved and understood the following:

Article 1

[1] Land reclamation and enclosures that have not been carried out by the State may not be carried out unless We, having consulted the Provincial Executive of the province or provinces in which the land is situated, have given approval.

[2] This regulation does not apply to land reclamation when a permit for land reclamation has already been issued together with a permit for peat extraction or when it is undertaken according to the regulations of a water board or a peat polder.

[3] The first clause is not applicable to enclosures when it is undertaken in accordance with a water board's regulations.

[4] Land reclamation and enclosures will only be undertaken by the State, excepting provisions under particular laws, after the Provincial Executive, referred to in the first paragraph, has been consulted, and the provisions that are included in articles 3, 4 and 5 have been satisfied and provisions have been made that are considered necessary for the national interest by Our Minister charged with the affairs of water.

Article 2

When making a request for a permit, maps of the particular land reclamation or enclosure must be made in duplicate, on which are shown the dike, the public roads, the water pipes, the drainage sluices or drainage equipment and which shows the position of the work in relation to the surrounding area, expounded by a description with necessary drawings and accompanied by an estimate of costs.

[…]

Article 6

[1] Any necessary conditions will be imposed when issuing the permit to protect those interests that could be damaged by the activity and an order may be made to deposit a sum of money in order to cover the costs that might arise from a deficient execution.

[2] The deposit will be returned, as soon and as far as it is no longer needed.

[…]

Article 9

The law dated 16th September 1807, relating to drainage of the areas and other public works, and the Imperial decree of 11th January 1811, containing regulations concerning the administration and the maintenance of polders, is revoked.

The decrees and orders will be placed in the Gazette and will be carefully executed by all the Ministerial Departments, Authorities, Local Authorities and Public Servants that are concerned with them.

Issued at Het Loo, 14th of July 1904.

WILHELMINA

4.5 Accretion methods

The methods that encourage accretion, both horizontally and vertically, are called accretion methods.

According to Dutch civil law, accreted land is the property of the landowner of the adjacent land. New islands that appear in streams and rivers are the property of the Dutch State. In the 18th and 19th century many farmers were able to extend their property by encouraging accretion. In particular some farmers in the province of Groningen created elongated plots of land this way; they became known as ribbon farmers. This process was very labour intensive and changes in working practices meant that it became less attractive over time to private individuals. However in the 1930s the State needed to provide labour intensive work and therefore took the accretion over. Because of this the newly reclaimed land became the property of the State and ownership boundary lines were established.

Accretion is only possible in areas where sand or silt are present in the sea or river water, as these can cause settlement. The accretion can be encouraged by the use of appropriate plant cover and the construction of settlement basins. The use of shellfish also helps settlement, because the shellfish take in silt that cannot settle and excrete it in a form that can.

Figure 4-2 Spartina Townsendii

Plant cover used to consist of rush, and reed, but later on grass species such as Spartina Townsendii (otherwise known as cord grass or false spike) began to be used. This grass species retains sludge very well and it can be used up to 80cm below average high tide. When there is more silt it is also possible to use marsh samphire (a type of seaweed) up to 50cm below average high tide, sea aster up to just below average high tide and also thrift up to slightly above average high tide. Sedimentation basins are filled as quickly as possible in order to prevent the sediment from sinking too quickly. Once in the sedimentation fields the water has to stand still for as long as possible to ensure maximum sedimentation and that the new sediment attaches to the existing sediment. In the final stage, the water has to be discharged as quickly as possible. Sedimentation basins use groynes, gullies and the adapted *Sleeswijk-Holstein method*.

Groynes are only used as longitudinal dams where there are large tidal variations, because transverse dams, especially at the head, are affected too much by strong tidal flow. Ditches and gullies are used behind the groynes.

Gullies and small embankments were originally used in a grid shape to divide the land into 100m x 100m plots. The same measurements are still used; basins of approximately ¾ ha, surrounded by low embankments, that were erected using the settlement from the ditches or groynes (Figure 4-3). Gullies are also used in the Sleeswijk-Holstein method.

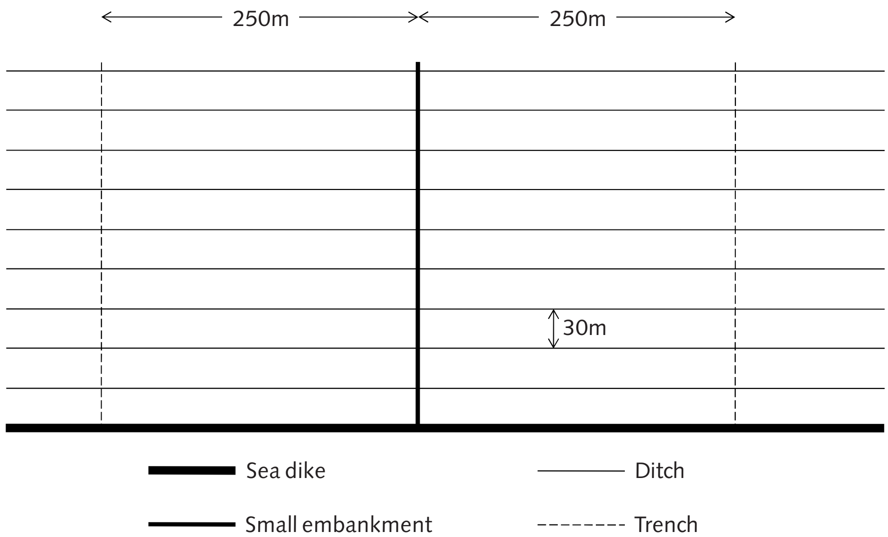


Figure 4-3 Boosting accretion by the use of ditches in sedimentation basins

The (adapted) *Sleeswijk-Holstein method* is applied when water levels of 1.0-1.2 meters below the mean high tide water depth make the use of plant cover impossible (Figure 4-4). Squares of 400 x 400 m are edged by wooden dams between poles (Figure 4-4). The dams are banked up on both sides at an angle of 1:5 with earth that has been extracted from the trenches on both sides. The squares on the sea side have two openings that give access to the two main ditches. Clay dams subdivide the large squares into sections of 100 x 100 m. Ditches in these smaller sections are 1- 2 meters wide and 20-25 cm deep, lying 5-10 m apart. The two main ditches are 2-3 m wide and 25-45 cm deep. The perpendicular ditches are 2 m wide and 25 cm deep.

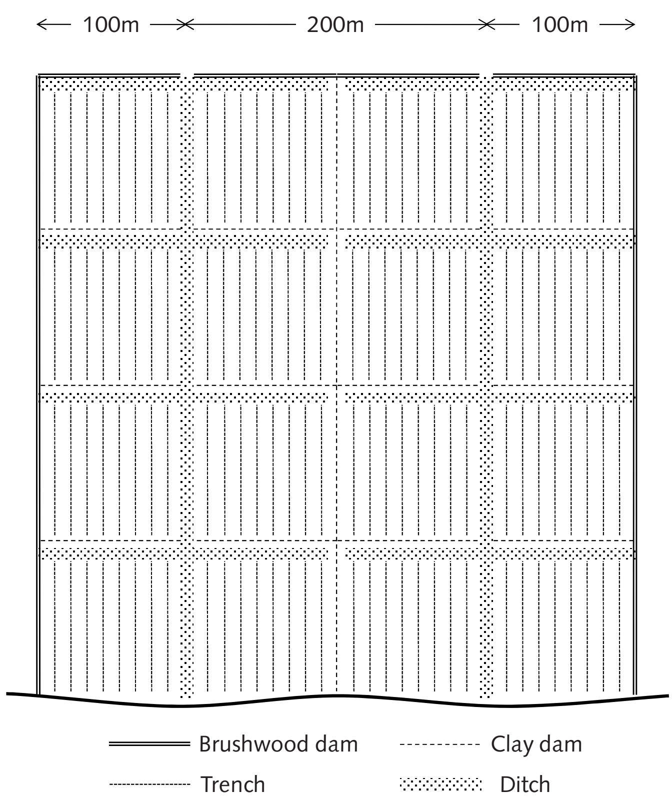


Figure 4-4 Accretion using the Sleeswjjk-Holsteinse method

4.6 Land improvement

Compared with current standards, many clearance projects in the past were inadequately implemented. Today's increased agricultural demands have revealed problems in the allocation, accessibility, soil profile, surface elevation, drainage, and water supply. Land improvement can largely overcome these problems, resulting in increased production and an enlarged business economy. Occasionally land improvements are carried out privately but they are usually part of a large-scale land consolidation (i.e. re-allocating of the land).

Land allocation and accessibility

Many polders have old, long and narrow plots with lots of water courses, many of which are too wide, and with too few roads. The farms are often located randomly and each farmer's plots are scattered over the whole polder. Land consolidation can solve these issues, with better shaped plots and better situated farms. Problems with drainage and water supply can also be tackled thanks to this new network.

Soil profile

The soil profile in many older polders is often poor. This can be because of the presence of hard layers in the sub-soil, which do not enable the roots of crops to fully develop. Often these layers (usually heavy clay layers) have low porosity, creating stagnant water conditions.

The layers in the soil profile are often in the wrong order, such as in some polders in South Holland, where a layer of acid-sulphate clay (extremely acidic clay that typically has rust spots and jarosite – yellow flecks) is on top of a good loam or sand layer. Sometimes a loam layer is covered with sand. The opposite also happens: sand is covered by a loam layer. In this last instance the structure is good, if the loam layer is sufficiently thick.

A way of solving these problems is to trenching the soil. Trenching breaks up the hardpan and inverts the soil layers, exposing soil with better properties. This has happened in the Noordoost Polder.

Figure 4-3 Trenching plough (ploughs up to 1.2 meters)

Surface elevation

Irregular elevation is common in sandy soils and river and marine clay soils. These irregularities can cause large variation in crop growth. Low-lying areas often have an excessively high groundwater table and are therefore extremely wet after rainfall. Locally this can lead to 'cold soils' and it can make it difficult to access the land or to work on it in spring time.

Irregular elevation can be improved by levelling the land. This is very expensive, because it involves a lot of earth moving. Levelling does not always mean the land surface has to be horizontal. In sloping areas the slope of the groundwater table is used as a guide.

Levelling is often combined with the implementation of new waterways and is mostly carried out during land consolidation. Generally the top layer of the soil, which is good for agricultural use, is removed separately and replaced on top after levelling has been carried out. This is true above all for humus-poor sandy soil and heavy clay soil. However this process makes the levelling even more expensive, because as well as moving the sub-soil the good top soil also has to be moved. Therefore the choice of earth moving is determined by the price of the movement, the quality of the replaced soil and the demand that the soil profile may not become worse and preferably becomes better. On the other hand, soil improvement generally leads to rising crop yields and an improvement in the soil workability and management.

Figure 4-4 Levelling the soil

5 The required groundwater level

5.1 Groundwater level

The water level in a recently installed groundwater piezometer, observation well or borehole becomes (semi) constant over time. This level is called the groundwater level. The pressure at this point is equal to the atmospheric pressure. The plane of the groundwater level is called the groundwater table or phreatic surface. Below the phreatic surface the pressure is higher and pores are filled with water, except for those that are completely closed off from others. Above the phreatic surface the pressure is lower than the atmospheric pressure (10.3 mwc). Here mainly vertical flow occurs due to precipitation, withdrawal of water by plants and evaporation, and the capillary properties of the soil.

The area above the phreatic surface, the *unsaturated zone*, can be divided into three zones:

1. The *full capillary zone* is situated directly above the groundwater table in which the pores are completely filled with groundwater. The capillary zone is the saturated portion of the capillary rise. The water head in this zone is inversely proportional to the diameter of the pores; the smaller the pores, the higher the head, but also the smaller the velocity. In every soil type there is a height possible up to which the velocity of capillary rise is sufficient to cover the plants' moisture demands. For each soil the capillary rise is dependent on its content: in general, sandy soils have a capillary rise of 20-80 cm, clayey soils of 60-100 cm and peat soils 30-60 cm;
2. Above the capillary fringe lies the *funicular zone*. In this zone only the small pores are filled with water which is connected to the groundwater. The larger pores are filled with air;
3. The top layer is called the *pendular zone*, where no connection is made to the groundwater. The water present is retained water, which clings to the soil particles or stays in closed pores.

In soils with small particles (i.e. clay, loam and peat) the transitions between the zones is very smooth. The capillary fringe and funicular zone can be quite large. In coarse soils, like sandy soils, the zones are small to non-existent and when present, the transitions are very sudden.

5.2 Water withdrawal by plant roots

For their growth, crop plants withdraw water using the active part of their root hair at approximately 1 cm from the tip of the root. This water is taken in at the roots, flows through the plant and is transpired through the leaves. A transpiration coefficient gives an indication of the amount of water needed (in kilograms) to produce 1 kg of dry matter. Generally this coefficient is somewhere between 200 and 600.

For example, a good potato harvest gives 30,000 kg/ha and 25,000 kg/ha leaves, with a water content of respectively 75% and 80%. Taking 400 as the transpiration coefficient this gives $400*[0.25*30{,}000+0.20*25{,}000] = 5{,}000{,}000$ kg/ha = 5,000 m^3/ha. This is equal to 500 mm. The amount of water needed for the potatoes and leaves themselves can be disregarded (this is approximately 4 mm). Table 5-1 gives the transpiration coefficient of eight different crops. Maize has the smallest coefficient; grass the largest.

Table 5-1Transpiration coefficient of various crops

Crop	Coefficient	Crop	Coefficient
Grass	540	Early potatoes	200
Maize	160	Lucerne/alfalfa	400
Winter wheat	380	Late potatoes	370
Summer barley	300	Beet	450

Plant roots need water and nutrients, but also air. The ideal conditions in volume units of soil : water : air are 1 :1 : 1. In reality this condition is never reached, because the pore volume of sand is around 40% and of clay 50%, so smaller then two thirds. An equal balance between the volume percentage water and air of 1 : 1 can be reached, which is achieved in the funicular zone. This zone is therefore the ideal zone for water withdrawal. Moreover in this zone water replenishment is possible, due to the connection with the capillary fringe. The lower boundary of the funicular zone is the boundary for the water-withdrawing roots. The necessary air is lacking below the transition zone. Above, in the pendular zone, only retained water is present. This can be withdrawn, but because its replenishment is dependent on precipitation, the amount of water present is less constant. Water that is bound to soil particles on a molecular or colloidal level, such as imbibition water and hygroscopic water, cannot be withdrawn.

The water content in a soil can be expressed by the pF value, which gives a relation between the pressure head (h) and the water content (-). This value is the logarithmic of the pressure head (in cm). [pF=log(-h)]. Figure 5-1 gives the pF curves for light marine clay and humus sand.

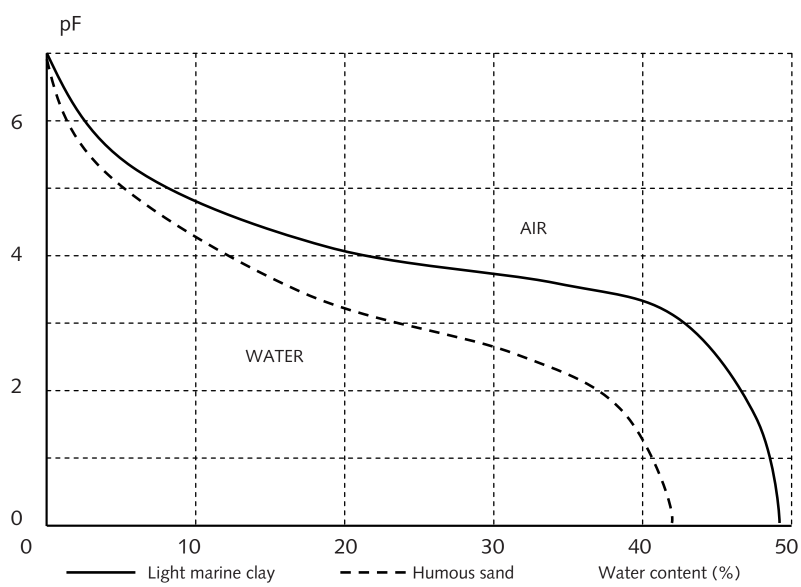


Figure 5-1 Relation between the water content and the pressure head

At a pressure head of 16,000 cm (pF = 4.2) plants are unable to withdraw water. This boundary is called the *wilting point*. The other boundary is given by the *field capacity*, at pF = 2 (pressure head of 100 cm). This pressure appears when after precipitation the water in the soil has had the time to infiltrate to the groundwater. This water can be retained within the soil against gravity (see also Table 5-2).

Table 5-2 pF scale

pF	Suction pressure [cm]	Soil property
-∞	0	Saturated
0	1	Extremely wet
1	10	Wet
2	100	Field capacity
2.3	200	
2.7	500	
3	1 000	Germination slows down
4	10 000	Growth slows down
4.2	15 850	Wilting point
5	$1 \cdot 10^5$	Air dried
6	$1 \cdot 10^6$	
7	$1 \cdot 10^7$	Oven dried

The amount of water that is available depends on the soil type and properties. Plants can withdraw water from the pendular zone between the field capacity (pF = 2) and wilting point (pF = 4.2). Figure 5-1 shows that for clay this is 48-17=31% and for sand this is 36-11=25%. The humus in the sand of Figure 5-1 has a large impact on the amount of water available. Plant growth can even be impossible; for example, with a sandy soil the groundwater table is deep, because there is a precipitation deficit in the Netherlands during the growing season.

Besides sand's poor retaining capacity, the water/air balance is not good for plant growth compared to clay and the funicular zone is almost non-existent. The structure of sandy soil can be improved by mixing with compost. Farmers knew this several centuries ago; in large parts of the Netherlands the top layer of the peat was saved in the peat mining process. When the mining was done, the top layer was put on top of the sand layer, which made the sandy soils humus and fertile. Other sandy soils were made fertile by applying manure containing straw from stables. Sandy soils are only fertile when they contains enough humus and a high and constant water table is maintained, allowing plants to withdraw water at the boundary with the capillary fringe. In clay soils poor growing conditions occur when groundwater tables are too high, forcing all the air out of the pores in the root zone.

As explained earlier, the most favourable conditions for water withdrawal by plant roots are found in the funicular zone. The corresponding most favourable depth of the groundwater table depends on the soil type/properties and the rooting depth of the crop. Table 5-3 shows the rooting depth of several crops.

Table 5-3 Overview of ideal rooting depths of various crops (in cm)

150 Very deep	100 Deep	75 Average	40 Shallow	Very shallow
Lucerne	Grain	Potatoes	Vegetables	Grass
Fruit (trees)	Beet	Maize	Bulbs	
		Fruit (bushes)		

This table gives the ideal root depths. In reality these depths can be much less, due to hardpans, rocks, infertile layers, or too high or too low groundwater tables. Grass usually roots within 20 cm of the surface. This is partially due to mowing and grazing, which hinder root growth. However, grass roots can grow deeper, as can be seen in a dry period, but this reduces the yield.

5.3 Groundwater level categories

(This paragraph is our summary of chapter 13 in 'Bodemkunde', by P. van der Sluis)

The groundwater level follows the meteorological conditions, with some delay. Therefore there is not just one groundwater level, as it is not a static concept. When we speak of a groundwater regime, we mean the margins between which the groundwater level fluctuates. These groundwater regimes can be characterised by the *Average Highest Groundwater depth* (AHG; in Dutch *GHG*) and the *Average Lowest Groundwater depth* (ALG; in Dutch *GLG)*. These AHG and ALG mark the summer and winter groundwater levels in a year with average precipitation and evaporation.

The AHG and ALG differ spatially due to differences in surface elevation, hydrological conditions and soil properties. In order to give an accurate representation of the groundwater conditions, the AHG and ALG together give a *groundwater level category* (in Dutch: *grondwatertrap* (GT)). These categories have boundary conditions based on the AHG and ALG, as given in Table 5-4.

The boundaries are based on practical and agricultural demands:

- Variations in groundwater level occur at small depths;
- With basic field measurement equipment it is possible to describe the soil conditions and properties up to 120cm depth;
- The '40cm depth' used to be the groundwater level of well-drained fields. Nowadays it is agreed that this should be deeper.

Table 5-4 Groundwater level categories (source: Alterra, rapport429.pdf)

Groundwater level category (GT)	AHG (cm ground level)	ALG (cm ground level)
I	<20	<50
II	<40	50-80
II*	25-40	50-80
III	<40	80-120
III*	25-40	80-120
IV	40-80	80-120
V	<40	>120
V*	25-40	>120
VI	40-80	>120
VII	80-140	>120
VII*	>140	>140

GT I en II are wet soils. GT III and IV are soils with very limited fluctuations in groundwater level during the year. Seepage (in Dutch*: kwel)* or infiltration from surface water result in shallow ALGs. Percolation (downward seepage, in Dutch*: wegzijging)* often occurs with deeper ALGs (GT V and VI). GT VII soils have very deep drainage.

Now the question is: how to determine the AHG and ALG. When all groundwater level measurements in winter are averaged to the AHG and all from the summer are averaged to the ALG, too few fluctuations are found to determine a GT. More useful values are found when the highest measurements in winter and lowest measurements in summer are averaged. The standard procedure to determine the AHG and ALG is:

Take a database of 8 years' measurements with an interval between the measurements of 14 days. Determine the average of the three highest measurements per hydrological year (HG3), as well as the three lowest measurements (LG3). Note: a hydrological year starts on 1st April. The AHG (or ALG) is defined as the statistically expected value of the HG3 (or LG3) over the period in which the groundwater regime has no drastic changes.

5.4 The most favourable groundwater level

Each crop has its own demands concerning water availability, and each soil has its own properties. The most favourable groundwater level for a crop is therefore not the same for all soils. The earliest tests for favourable groundwater levels were carried out in tanks where the groundwater level could be varied with considerable precision and precipitation could be simulated. These tests showed that precipitation had little influence on the groundwater level. With shallow groundwater levels the few millimetres of precipitation did not make a difference and with deep levels it was too little to raise the yield. This led to the disregarding of precipitation in later tests. Table 5-5 gives the results of such tests to determine favourable groundwater levels for bulb growth.

The largest yields for both hyacinths and tulips were found at groundwater levels of 50 cm below ground level. The exception was for hyacinths growing on fine dune sands, where the favourable groundwater level was 10cm lower. The root system became very shallow when the groundwater level was raised, reducing the yield. When the groundwater level was lowered, the root system would also become shallower, with only a few long roots shooting down. Here yields were reduced too.

Table 5-5 Influence of groundwater levels on the bulb yield on coarse and fine dune sand

Groundwater level (in cm ground level)	Hyacinths		Tulips	
	Weight increase in %		Absolute production	
	Coarse	Fine	Coarse	Fine
50	110	85	2120	1500
60	90	100	1990	1360
70	65	75	1720	1230
80	45	68	1570	1210

With similar tests the hay yield and protein content was determined for a clover-grass mix on several soil types. This test is difficult to conduct properly, because the hay is cut in several sequences and not just the yield, but also the nutritional value, are important. Results are shown in Table 5-6.

Table 5-6 Influence of the groundwater level on the hay yield and protein content of a clover-grass mix conducted on several soil types. The hay yield is given in 100kg/ha and the protein content in percentage of dry matter.

Groundwater level (in cm ground level)	Coarse sand		Fine sand		Clay		Peat	
	Hay	Protein	Hay	Protein	Hay	Protein	Hay	Protein
40	118	12.6	174	12.1	180	13.3	173	13.4
70	50	6.0	152	10.8	162	16.3	162	14.5
100	43	5.3	113	8.9	142	14.0	124	11.2
130	33	4.1	72	5.8	141	13.1	123	10.9

The largest yield is given by a groundwater level of 40 cm for all soil types. The nutritional value is for some soil types larger with lower groundwater levels. The results of Table 5-6 are given in -2.

The largest yield on a clay soil is set at 100% and the other yields are given as percentages of this yield. The largest yields on all soil types are produced at groundwater levels of 40cm below ground level, due to the shallow root system of grass; 70 to 80% of the root system is situated within the first 5cm, and 3-6% is deeper than 20cm. Large yields at shallow groundwater levels are due to the water supply by the funicular zone, which is connected to the groundwater. This zone is very thin to non-existing in sandy soils, which results in a smaller yield.

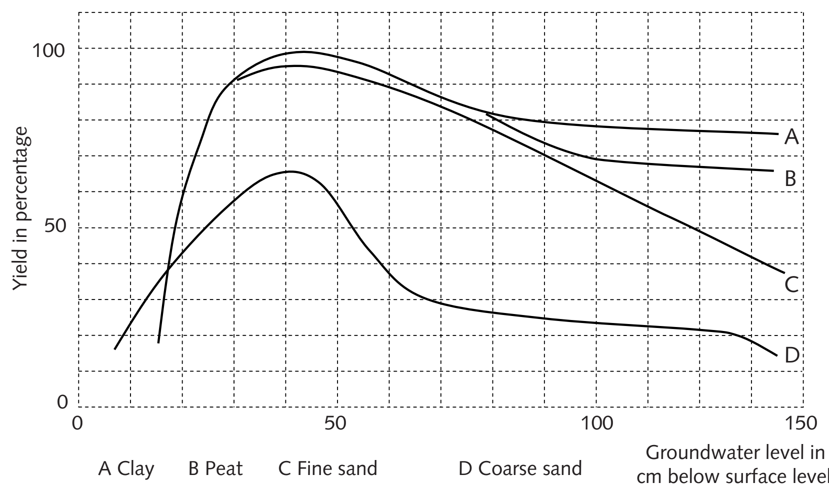


Figure 5-2 Hay yield of a clover-grass mix at several groundwater levels

With high groundwater levels, yields drop drastically due to suffocation of the roots. Clay, peat and fine sands show this phenomenon the strongest. Theoretically no yields are found when the capillary fringe reaches ground level: pores are completely filled with water and air is absent.

Deep groundwater levels cause the yields to decrease gradually. Coarse materials show a faster decrease than fine materials. Coarse sand has a small amount of water present between the field capacity (pF = 2) and wilting point (pF = 4.2). This amount is larger for fine sand, so the clover grass is able to grow better at deep ground water levels.

For some soil types the speed at which the yield drops to zero strongly depends on the amount of precipitation. When precipitation is abundant, the water in the funicular and pendular zone can be sufficient to ensure a good yield, even though groundwater levels are deep.

When precipitation is too low, the wilting point is quickly reached; first ly with coarse sand, later fine sand, peat and eventually clay. However plant growth is better than would be expected from the limited root depths, because all plants extend their rooting depth when in need of water. Yields will be lower than with a shallow groundwater table, because extending rooting depth costs energy.

Deep rooting crops show totally different yield patterns than this clover-grass mix. In Figure 5-3 the curves of wheat growing in heavy clay and average-size sand are given. In sand, wheat shows the largest yield with a groundwater level of 70cm below ground level. Heavy clay shows a favourable groundwater level of 130cm below ground level. The total yield here is larger than on sand. Loam (sandy clay) shows a lower favourable groundwater level (-150cm) and the yield is even larger.

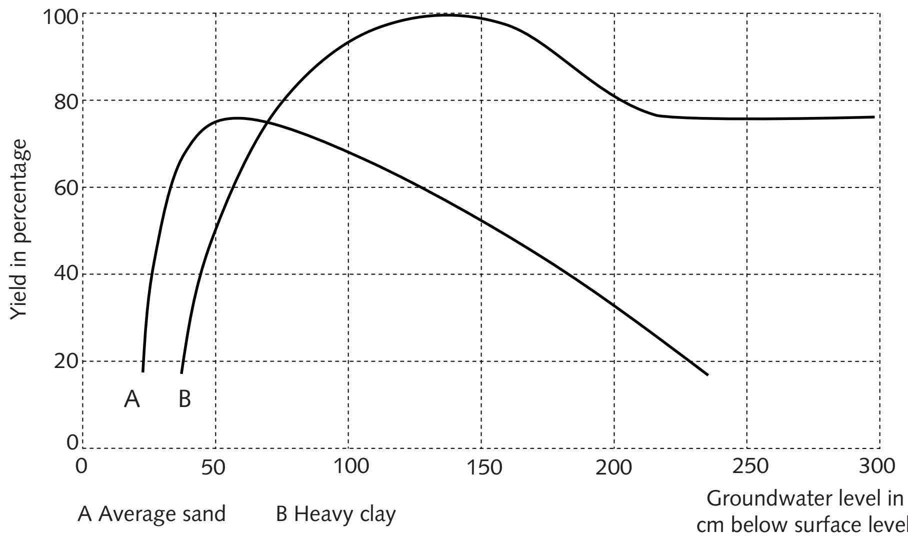


Figure 5-3 Yields of wheat at different groundwater levels

Raising or lowering of the groundwater level by 30cm will show just a minimal decrease of the yield. Usually, water levels in polders with clay or loam soils are maintained higher (at -120cm). This way the depth of ditches can be decreased, which leads to less (brackish) seepage and a smaller discharge head at the pumping station. Also less earth excavation and space is needed for watercourses.

A shallow groundwater level would lead to suffocation of the roots and a decrease in yield. Clay soils are more susceptible to this raising than sandy soils. Lowering the groundwater table causes a decrease in yield. For averaged size sand the amount of water available between the field capacity and wilting point is not sufficient to compensate for the precipitation deficit during the growing season. When the groundwater level is lowered, in clay soils the yield initially decreases, but then remains constant. The amount of water available between the field capacity and wilting point in clay soils is sufficient to compensate for the precipitation deficit during the growing season.

The tests shown above are all conducted with a constant groundwater level. However in reality the groundwater level is not constant. The influence of fluctuating groundwater levels on the yield of a clover-grass mix has also been studied. This research is more realistic; however it is very difficult to conduct properly. Groundwater levels fluctuate due to precipitation, evaporation and water withdrawal by plants and humans. The main conclusions from this research are:

1. The favourable average groundwater level is equal to the optimal constant groundwater level as found in the previous discussed tests;
2. Crop yield is very susceptible to fluctuations in the groundwater level, when the highest yields are harvested at shallow groundwater levels, so for crops with shallow root systems like grass and vegetables (on all soil types) and wheat when cultivated on sandy soils;
3. When groundwater levels are situated lower than the optimal groundwater level, crop yields are less influenced by fluctuations;
4. An increase in fluctuation generally leads to a decrease in yield. The reason for this is that roots need to adjust to the groundwater level. When roots are extended to low groundwater levels and this level rises, long roots are suffocated and die and new roots and root hairs need to be grown. When the water level drops again, the root system needs to be extended again. All of this costs the plant a lot of energy.
5. The largest yields are found when groundwater levels are low in winter and high in summer. Low groundwater levels in winter allow the soil to ventilate and the soil will increase its temperature faster in spring. Surface water levels are kept high in summer in an attempt to create high groundwater levels.

For agricultural purposes, controlling groundwater levels (especially shallow) is crucial.

6 Drainage

6.1 Groundwater flow

In the Netherlands it takes precipitation between one and one and a half hours to infiltrate into the ground. Surface drainage or the formation of puddles is not common. The precipitated water infiltrates downwards with a clean front. When the water content of the soil is below field capacity, the soil retains water. Only after that can the remaining water permeate and replenish the groundwater. When the soil is saturated, all the precipitated water will percolate (within one to two days). In this situation the groundwater table will rise ten to twenty times the height of the precipitated water, depending on the soil porosity, when the soil is not drained. Groundwater tables that are too high can cause suffocation of the plant roots, and destroy crops. Also soil can become too cold and the soil structure can change due to too high groundwater tables. All these situations can cause damage. In order to prevent this, excess water is drained.

Darcy's Law describes groundwater flow. This law assumes a constant pressure difference, discharge and flow profile:

$$Q = kA\frac{\Delta h}{L}$$

With:

Q	= discharge	[m^3/s]
k	= hydraulic conductivity	[m/s]
A	= flow area perpendicular to L	[m^2]
Δh	= pressure difference	[m]
L	= flow path length	[m]

Darcy's Law can be applied to groundwater flow towards ditches, assuming a homogeneous soil down as far as the bottom of the ditch and a hydraulic conductivity k.

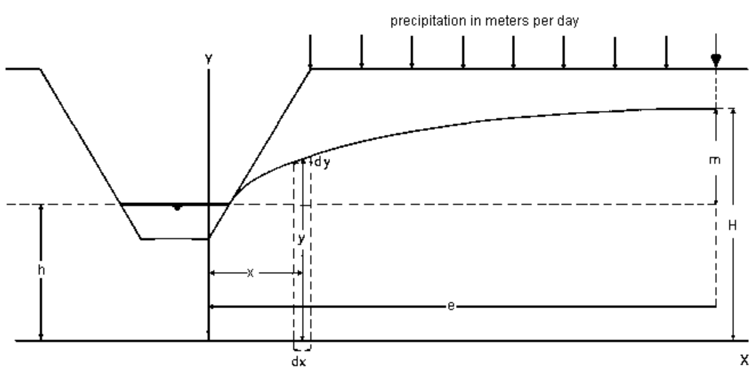


Figure 6-1 Stationary state of groundwater table with precipitation N m/d

This calculation assumes a constant precipitation intensity q (m/d), resulting in a stationary state where the discharge becomes equal to the precipitation. In the centre of the plot the discharge is zero. At a distance x from the ditch the discharge Q = q (½L-x) per unit width, perpendicular to the flow plane. L is equal to the distance between two parallel ditches.

According to Darcy's Law the discharge is: k·y·(dy/dx), with dy/dx as the slope of the groundwater table at place x, and y the height of the aquifer per unit width, perpendicular to the flow plane. This results in:

$$Q = q\left(\tfrac{1}{2}L - x\right) = k\frac{dy}{dx}y$$

Or: $$q\left(\tfrac{1}{2}L - x\right)dx = kydy$$

Or: $$q\int_{0}^{\frac{1}{2}L}\left(\tfrac{1}{2}L - x\right)dx = k\int_{D}^{H} ydy$$

Or: $$\left| q\left(\tfrac{1}{2}Lx - \tfrac{1}{2}x^2\right)\right|_{0}^{\frac{1}{2}L} = \left|\tfrac{1}{2}ky^2\right|_{D}^{H}$$

Or: $$q\left(\tfrac{1}{4}L^2 - \tfrac{1}{8}L^2\right) = \tfrac{1}{2}k\left(H^2 - D^2\right)$$

Concluding: $$q = \frac{4k\left(H^2 - D^2\right)}{L^2}$$

If we assume: H = D+h, this leads to:

$$q = \frac{8kDh + 4kh^2}{L^2}$$

This is known as Hooghoudt's (drainage) Equation.

With:

q	= constant precipitation	[m/d]
k	= horizontal hydraulic conductivity	[m/d]
D	= height of the surface water above an aquitard	[m]
L	= plot width between two parallel ditches	[m]
h	= height of the groundwater table above surface water table at ½L	[m]

When the ditches are located just on top of above an aquitard, D becomes zero and the discharge will be equal to:

$$q = \frac{4kh^2}{L^2}$$

When the confined layer lies very deep under the ditches, then h^2 can be ignored with respect to D. This leads to:

$$q = \frac{8kDh}{L^2}$$

In reality the conductivity above the drains is often different from that below the drains, because of cracks and roots. Hooghoudt's extended formula takes this into account:

$$q = \frac{8k_1Dh + 4k_2h^2}{L^2}$$

with:

k_1	= hydraulic conductivity above the drains	[m/d]
k_2	= hydraulic conductivity below the drains	[m/d]

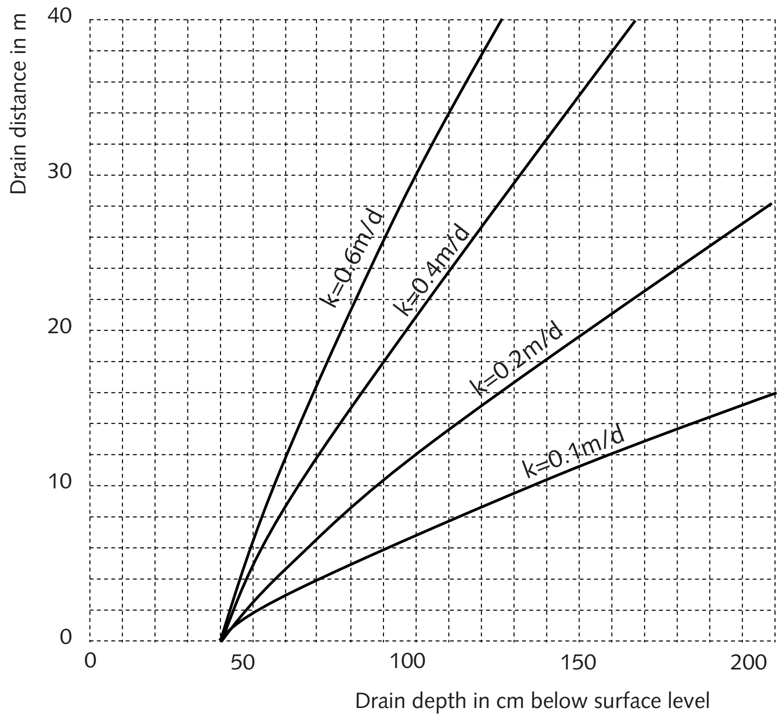


Figure 6-2 Relation between the drain depth and drain distance with a discharge of 7mm per 24 hours and a drainage depth in the middle of the plot of 40cm. The aquitard is assumed to be 10m below ground level.

Hooghoudt's Equation can also be used to calculate the distance between the drains. Nevertheless, one must take into account that part of the potential energy will be necessary for radial flow towards the drains. For this Hooghoudt decreased the depth D to a smaller equivalent layer with a depth d. The depth d is a function of the radius of the drain r, the drain distance L, and the depth D. To determine d Hooghoudt developed different tables and graphs. Nowadays it is more practical to use formulas, such as Dagan's Formula:

$$d = \frac{\frac{\pi L}{8}}{\ln \frac{L}{\pi r_0} + F(x)} \qquad \text{and} \qquad x = \frac{2\pi D}{L} \qquad \text{and} \qquad F(x) = \frac{\pi^2}{4x} + \ln\left(\frac{x}{2\pi}\right)$$

In which:

d	= equivalent depth	[m]
r_o	= radius of the drain `	[m]

The general criteria for the Netherlands are a minimal groundwater depth of 50cm, at a precipitation intensity of 7 mm/day.

Example of Hooghoudt's Equation

Determine the distance between two drains, such that the groundwater level remains 40cm below surface level at a discharge of 7mm/day. The conductivity is 1m/day, the layer D is 5m, and the drain radius is 0.04m. The drains are located at 1m below the surface level.

$$L = \sqrt{\frac{8kDh + 4kh}{q}} = \sqrt{\frac{8 \cdot 1 \cdot d \cdot (1-0.4) + 4 \cdot 1 \cdot (1-0.4)}{7/1000}} = \sqrt{686d + 343}$$

This equation can only be solved iteratively as the equivalent depth d depends on the distance L. First assumption L = 50 m

$$x = \frac{2\pi D}{L} = \frac{10 \cdot \pi}{50} = 0.63$$

$$F(x) = \frac{\pi^2}{4x} + \ln\left(\frac{x}{2\pi}\right) = \frac{\pi^2}{4 \cdot 0.63} + \ln\left(\frac{0.63}{2\pi}\right) = 1.62$$

$$d = \frac{\frac{\pi L}{8}}{\ln\frac{L}{\pi r_0} + F(x)} = \frac{\frac{50 \cdot \pi}{8}}{\ln\frac{50}{0.04 \cdot \pi} + 1.62} = 2.58m$$

$$L = \sqrt{685 \cdot 2.58 + 343} = 46m$$

Second assumption L=46 m

$$x = \frac{2\pi D}{L} = \frac{10 \cdot \pi}{46} = 0.68$$

$$F(x) = \frac{\pi^2}{4x} + \ln\left(\frac{x}{2\pi}\right) = \frac{\pi^2}{4 \cdot 0.68} + \ln\left(\frac{0.68}{2\pi}\right) = 1.39$$

$$d = \frac{\frac{\pi L}{8}}{\ln\frac{L}{\pi r_0} + F(x)} = \frac{\frac{46 \cdot \pi}{8}}{\ln\frac{46}{0.04 \cdot \pi} + 1.39} = 2.47m$$

$$L = \sqrt{685 \cdot 2.47 + 343} = 45m$$

Third assumption L=45 m

$$x = \frac{2\pi D}{L} = \frac{10 \cdot \pi}{45} = 0.70$$

$$F(x) = \frac{\pi^2}{4x} + \ln\left(\frac{x}{2\pi}\right) = \frac{\pi^2}{4 \cdot 0.70} + \ln\left(\frac{0.70}{2\pi}\right) = 1.35$$

$$d = \frac{\frac{\pi L}{8}}{\ln\frac{L}{\pi r_0} + F(x)} = \frac{\frac{45 \cdot \pi}{8}}{\ln\frac{45}{0.04 \cdot \pi} + 1.35} = 2.45m$$

$$L = \sqrt{685 \cdot 2.45 + 343} = 45m$$

This is close enough to be acceptable!

Experiment:

Redo these calculations in Excel with the function 'goal seek'.

The exact solution is 44.953m. However there are many uncertainties in the calculation of k and D, and hardly any serious drivers on drainage machines who will move the last 47 mm.

Ernst's Formula

As well as Hooghoudt in 1940, Ernst also developed a steady state drainage formula around 1960 which can be used to determine the spacing between drains. According to Ernst the height of the groundwater table above surface water is the head loss for vertical flow, horizontal flow, and radial flow:

$$h = h_{vert} + h_{hor} + h_{rad} = \frac{qD_{vert}}{k_{vert}} + \frac{qL^2}{8\sum(kD)_{hor}} + \frac{qL}{\pi k_{rad}} \ln \frac{aD_{rad}}{u}$$

D_{vert}	= thickness of the layer in which vertical flow is considered	(m)
k_{vert}	= vertical hydraulic conductivity	(m/day)
$\sum(kD)_{hor}$	= transmissivity of the layers through which water flows horizontally	(m^2/d)
k_{rad}	= radial hydraulic conductivity	(m/day)
a	= geometry factor of the radial resistance	(-)
D_{rad}	= thickness of the layer in which radial flow is considered	(m)
u	= wet entry parameter of the drain	(m)

The head loss h_{vert} may be determined as the difference between the readings of the piezometers (1) and (2). As the vertical hydraulic conductivity is difficult to measure under field conditions, it is often replaced by the horizontal conductivity, which is relatively easy to measure with the auger hole method (see also §6.2). In principle this assumed isotropy is not correct, but often neglected as the vertical head loss is generally small; when q = 7 mm/day, k = 1 m/d and D_v = 0.6 m, h_{vert} is only 4 mm. The head loss by vertical flow becomes significant with poorly permeable soils (heavy clay).

The head loss due to horizontal flow equals the difference between the readings of the piezometers (2) and (3). If the impervious layer is very deep, the values of $\sum(kD)_{hor}$ increase to infinity and consequently the horizontal head loss decreases to zero. To prevent this, the maximum thickness of the soil layer below the drain level through which flow is considered is restricted to ¼ L.

The head loss h_{rad} may be measured as the difference between the reading in the piezometer (3) and the water level in the drain. This term has the same restriction for the depth of the impervious layer as the equation for horizontal flow (i.e. D_{rad} < ¼ L).

The geometry factor (a) depends on the soil profile and the position of the drain. In a homogeneous profile, the geometry factor equals 1. In a layered soil the geometry factor depends on whether the drains are in the top or bottom soil layer. If the drains are in the bottom layer then again a=1, as the radial flow is assumed to be restricted to this layer. If the drains are in the top layer, then the value of a depends on the ratio of the conductivity of the bottom layer and top layer (k_{bottom}/k_{top}). Ernst distinguished the following situations:

- $\frac{k_{bottom}}{k_{top}} < 0.1$: the bottom layer is considered impermeable and a = 1;
- $0.1 < \frac{k_{bottom}}{k_{top}} < 50$: a depends on the ratio k_{bottom}/k_{top} and D_{bottom}/D_{top} (see Table 6-1);
- $\frac{k_{bottom}}{k_{top}} > 50$: a=4

Table 6-1 Ernst's geometry factor a

K_{bottom}/k_{top}	D_{bottom}/D_{top} 1	2	4	8	16	32
1	2.0	3.0	5.0	9.0	15.0	30.0
2	2.4	3.2	4.6	6.2	8.0	10.0
3	2.6	3.3	4.5	5.5	6.8	8.0
5	2.8	3.5	4.4	4.8	5.6	6.2
10	3.2	3.6	4.2	4.5	4.8	5.0
20	3.6	3.7	4.0	4.2	4.4	4.6
50	3.8	4.0	4.0	4.0	4.2	4.6

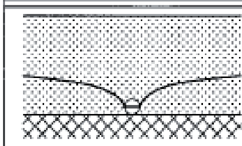
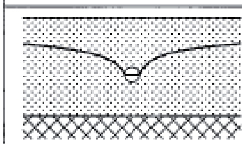
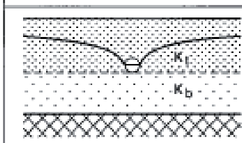
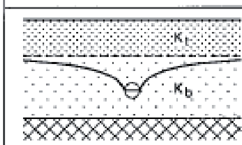
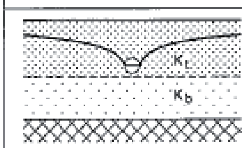
SCHEMATIZATION	SOIL PROFILE	POSITION OF DRAIN	THEORY	EQUATION
	homogeneous	on top of impervious layer	Hooghoudt/ Donnan	$q = \frac{4K(H^2 - D^2)}{L^2}$
	homogeneous	above impervious layer	Hooghoudt with equivalent depth	$q = \frac{8Kdh + 4Kh^2}{L^2}$
	two layers	at interface of the two soil layers	Hooghoudt	$q = \frac{8K_b dh + 4K_t h^2}{L^2}$
	two layers ($K_t < K_b$)	in bottom layer	Ernst	$h = q(\frac{D_v}{K_t} + \frac{L^2}{8K_bD_b} + \frac{L}{\pi K_b} \ln \frac{D_r}{u})$
	two layers ($K_t < K_b$)	in top layer	Ernst	$h = q(\frac{D_v}{K_t} + \frac{L^2}{8(K_bD_b + K_tD_t)} + \frac{L}{\pi K_t} \ln \frac{aD_r}{u})$

Figure 6-3 Overview of profiles for which Hooghoudt or Ernst should be used

Figure 6-4 Low hydraulic conductivity causes small ditch distances, here approximately 15m

Example of Ernst's Formula
Assume a two-layer profile with the drain in the top layer. Determine the distance between two drains, such that the ground water level remains 40cm below surface level at a discharge of 7mm/day. The drains are located at 1.5m below the surface level. The conductivity k_{top} is 0.5 m/day, the top layer D is 2m. The conductivity k_{bottom} is 2m/day, the bottom layer D is 4m. The wet perimeter is 0.3.

$k_{bottom}/k_{top} = 2/0.5 = 4$; $D_{bottom}/D_{top} = 4/2 = 2 \rightarrow a = 3.4$
$D_1 = D_0 + 0.5\ m = (2-1.5) + 0.5\cdot(1.5-0.4) = 1.05\ m$
$\sum(kD)_{hor} = k_1D_1 + k_2D_2 = 0.5\cdot 1.05 + 2\cdot 4 = 8.52\ m^2/d$

$$h = h_{vert} + h_{hor} + h_{rad} = \frac{qD_{vert}}{k_{vert}} + \frac{qL^2}{8\sum(kD)_{hor}} + \frac{qL}{\pi k_{rad}} \ln \frac{aD_{rad}}{u}$$

$$\frac{h}{q} = \frac{1.1}{0.007} = \frac{1.1}{0.5} + \frac{L^2}{8\cdot 8.5} + \frac{L}{\pi\cdot 0.5} \ln \frac{3.4\cdot 0.5}{0.3}$$

$$157 = 2.2 + 0.015\cdot L^2 + 1.1\cdot L \qquad \rightarrow \qquad L = \frac{-1.1 + \sqrt{1.1^2 - 4\cdot 0.015\cdot -155}}{2\cdot 0.015} = 72\ m$$

6.2 Hydraulic conductivity

Conductivity is the ability of soil to let liquid or gas through. The conductivity coefficient is a measure for the permeability of water and can be defined as the proportion factor in Darcy's Law ($v=-k\cdot i$). Particularly in fluvial deposits the conductivity coefficient is different in horizontal and vertical direction (anisotropy).

Hydraulic conductivity measured from a soil sample in the lab is not equal to the hydraulic conductivity of the same soil sample *in situ.* The hydraulic conductivity is not just dependent on pore size, but also on root canals, worm passages, cracks and the amount of coarse pores (Figure 6-5).

Figure 6-5 Soil profile in eastern Flevoland (source: Pesticide leaching in polders, Field and model studies on cracked clays and loamy sand, Klaas P. Groen (1997))

The best way to measure the hydraulic conductivity is *in situ* where soil conditions are not disturbed. These measurements can be done using the auger bore hole method. For this method a bore hole is made up to 60 to 70cm below groundwater table with a diameter of 6 to 9cm. When the groundwater table has stabilised, it is measured and then lowered over a distance y_0. The rising water level is measured several times. A rapidly rising water level indicates a high hydraulic conductivity *k*. A graph of the water level measurements over time can help determine the hydraulic conductivity. The measurements must be stopped before 25% of the water pumped out has flowed back into the borehole;
i.e. before $y < \frac{3}{4} y_0$.

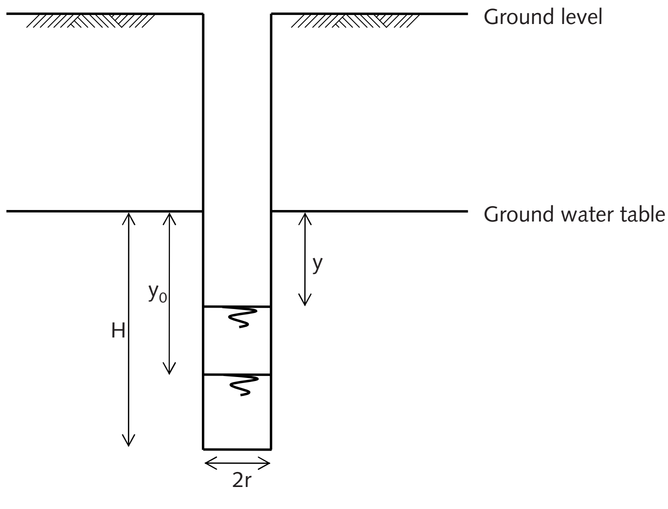


Figure 6-6 Determining the hydraulic conductivity using a bore hole

The inflow of the water into the bore hole goes through both the bottom and the sides of the hole. At time t with a water level of y below groundwater table the inflow is:

$$\frac{(k \cdot 2\pi r \cdot H) \cdot y}{a} + \frac{(k \cdot \pi r^2) \cdot y}{a}$$

The rate of climb is:

$$-\frac{dy}{dt} = \frac{(k \cdot 2\pi r \cdot H) \cdot y}{a \cdot \pi r^2} + \frac{(k \cdot \pi r^2) \cdot y}{a \cdot \pi r^2} = \frac{ky(2H + r)}{ar}$$

Integrating this, with boundary conditions:

When $t = 0$; $y = y_0$ and at $t = t$; $y = y$

$$\int_{y=y_0}^{y=y} -\frac{dy}{y} = k \cdot \frac{2H + r}{a\,r} \int_{t=0}^{t=t} dt$$

Or: $$k = \frac{ar \ln \frac{y_0}{y}}{t(2H + r)} = \frac{2.3ar \log \frac{y_0}{y}}{t(2H + r)}$$

With:

r	= bore hole radius	[m]
H	= height of the groundwater table below the bottom of the bore hole	[m]
a	= factor indicating the density and shape of the flow paths	
k	= hydraulic conductivity	[m/d]

In practice other - sufficiently accurate - empirical formula are prefered, which do not have some to be determined factor a. If the impermeable layer is at least ½H below the bottom of the hole:

$$k = \frac{4000 \cdot r^2}{(H + 20r) \cdot \left(2 - \frac{y_{av}}{H}\right) \cdot y_{av}} \cdot \frac{\Delta y}{\Delta t}$$

Or if the impermeable layer is at the bottom of the hole:

$$k = \frac{3600 \cdot r^2}{(H + 10r) \cdot \left(2 - \frac{y_{av}}{H}\right) \cdot y_{av}} \cdot \frac{\Delta y}{\Delta t}$$

With:

k	= hydraulic conductivity	[m/d]
H	= bore hole depth below the groundwater table	[cm]
y_{av}	= average drawdown	[cm]
r	= radius of the bore hole	[cm]
Δy	= head difference during the measurement	[cm]
Δt	= time between the measurements	[s]

Example:

t(s)	y(cm)
0	47.2
20	43.5
40	39.9
60	36.5

S> ½ H

r = 4 cm

H = 89 cm

Δy= 47.2-36.5 = 10.7 cm

y_{av} = ½ * (47.2+36.5) = 41.9 cm

$$k = \frac{4000 \cdot 4^2}{(89 + 20 \cdot 4) \cdot \left(2 - \frac{41.9}{89}\right) \cdot 41.9} \cdot \frac{10.7}{60} = 1.05\ m/d$$

(Note that the measurements are in cm and s, but the answer in m/d!)

The hydraulic conductivity is determined for several soil types.
The values are given in Table 6-2.

Table 6-2 Typical values of horizontal hydraulic conductivity k

Fraction	k in m/d
16-43μ	0.2
43-74	0.9
74-104	2.2
104-147	4.1
147-208	8.0
208-295	15.6
295-417	28.5

The actual value in a plot can be determined using several bore holes. Table 6-2 shows that for small soil particles (fractions) the hydraulic conductivity is small. Clay could improve its hydraulic conductivity by coagulation but in reality, the hydraulic conductivity for newly reclaimed land will improve over time, due to improvement in the soil structure and the creation of root canals, worm passages and cracks.

It is important to realize that the hydraulic conductivity is not the same as the infiltration capacity. The infiltration capacity can be determined using an infiltration test. In this test two rings of different diameters are placed on the soil. The height of the rings is the same. Both of the rings are gently pushed into the soil and then filled with water, which will then infiltrate. Due to sideways infiltration, the infiltration from the outer ring will be faster than that of the inner ring. The infiltration of the inner ring can be considered to flow straight downwards. Infiltration speed can be measured by the lowering of the water level in the inner ring as a function of time. Typical values are between 0.5 and 20mm per hour.

Table 6-3 Indication of infiltration capacities of several soil types

Soil type	Infiltration capacity in mm/hr
Coarse sand	500
Fine sand	20
Clayey fine sand	11
Light loam	10
Loss	6
Peat	2.2
Loam	2.1
Light clay	1.5
Heavy clay	0.5
Clayey loam	0.4

6.3 Drainage methods

Drainage can be done using ditches (small canals), trenches or drains.

Ditches (small canals) are useful in soils with high conductivity, such as sandy soils. Lower hydraulic conductivity demands smaller distances between drains, which results in very narrow and small plots. Combinations of ditches and trenches or drains are often used, but t he land surface is significantly reduced by using ditches, accessibility is reduced and maintenance is expensive.

Trenches are often used, especially in grassland where drains are not useful, due to too low drainage depths. Usually trenches are placed at a distance of 10 to 15m and the bottom of the trench is at polder water level. The slope of trenches is often small and at the end of the trenches drainage pipes transport the water to the ditches. This is done to make it possible to drive over the ends with tractors without damaging the system. Using trenches also has disadvantages; trenches are the ideal environment for many weeds; cattle destroy trenches by walking on them; and the whole plot becomes less accessible for machines. Like ditches, trenches require a lot of maintenance and the land surface is reduced. However unlike ditches, trenches are easy and cheap to construct.

Mole drainage is a special type of drainage, which is not commonly used in the Netherlands. With this type of drainage a bullet-shaped cylinder (of approximately 8cm) is pulled through the ground at 60 to 75cm below ground level to form a course. For many agricultural uses, this drainage is too shallow. However, in peat and heavy clay soils this type of drainage is frequently used. The ends of the course usually drain into ditches using drain pipes. Mole drainage is very cheap and relatively easy to apply. However, the drains can easily collapse and have a short life span. The pulling of the ball through the ground can also cause the walls of the tunnel to become compressed and silted up. This 'mole effect' decreases the drainage.

Figure 6-7 Mole plough which can be attached to a tractor. The disk in the front cuts through the grass and roots. The ball is ploughed through the ground after which the roll on the back of the frame closes the ground (source: www.wifo.nl).

Pipe drainage has been used for many years. In the early days earthenware pipes were placed in trenches. Nowadays drain pipes are made of synthetic materials, which are placed in the ground by special drainage machines. Water enters the pipes through small perforated holes or slits. Drains are usually placed 8 to 20m apart; closer than this would be too expensive. As with mole drainage, pipe drainage can cause the soil to become compressed and silted up.

Drains are placed perpendicular to the long stretches of a plot, starting in the middle sloping to the sides (with a slope of 0.1 to 0.25%; see Figure 6-8). The slope in the drains is needed to make sure the velocity in the drains is enough to prevent sedimentation. The drain ends are placed 10 to 20cm above polder water level. This way the velocity at the end of the tube is not reduced when the discharges are small. In the middle of the plot the drains are generally placed at 80cm below ground level to prevent the tubes from freezing in winter or braking when the land is worked on.

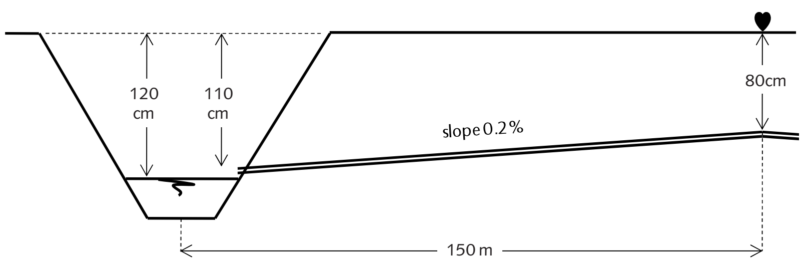


Figure 6-8 Placement of drains series

The length of the drains is determined by the slope and the difference between polder water level and the groundwater table at the middle of the plot. In the example of Figure 6-8 the plot width is 300m. Larger drain lengths are applied when the slope is reduced and the diameter of the drains are increased. It is also possible to place a web of drains; a main drain has several branches. Cleaning these systems is very difficult and they can only be placed in a sloping area.

The functioning of the drains is checked at the end of the drains, which is usually placed above polder water level. When drains are blocked, water is put in under high pressure to flush the system.

6.4 Determining the polder water level

In order to maintain an optimal groundwater level, excess precipitation is drained to ditches. The water level in these ditches has an influence on the groundwater table in the surrounding soils when the soils have a higher hydraulic conductivity. When conductivities are low, the influence is significantly reduced.

Soils are said to be ‘deep’ or ‘shallow’ depending on the depth of the aquifer. When soils are shallow, the aquifer is thinner than the optimal drainage depth. On these shallow soils (i.e. clay on top of sand), high groundwater tables need to maintained, in order to support the top layer. Surface water level will be high, restricting drainage.

The surface water target level set in order to influence the groundwater table is called the polder water level or target level, generally expressed in meters with respect to MSL (Mean Sea Level). When the polder water level is set, one has to take into account the soil type, crops grown, surface elevation and crop yield. General guide lines are:

Grassland:

- light soils: 40-60cm below ground level
- clay and loam: 70cm below ground level

Agricultural use:

- light soils: 70cm below ground level
- clay and loam: 120-150cm below ground level

Soil types are known for large parts of the Western world. For the Netherlands these are indicated on the ‘Soil map of the Netherlands’ (Alterra, www.bodemdata.nl; Figure 6-9). Due to land consolidation soil types in polders are often very well known, but in new polders or areas where the soil types are not as well known, drillings are carried out to map the soil types.

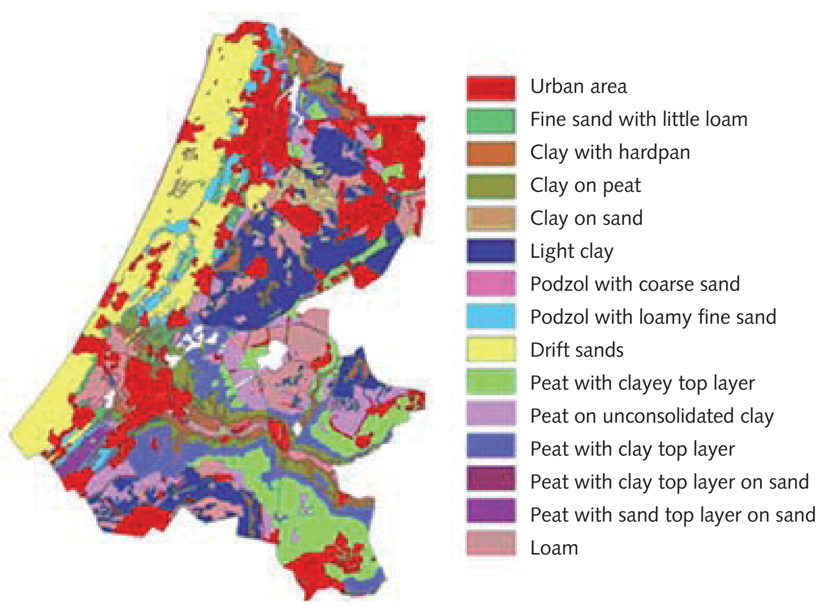


Figure 6-9 Example of the digital soil map (source: Rijnland Water Board)

The choice of which crop to grow depends to a large extend on the soil properties. Some guidelines can be set here as well:

1. Soils with a clay or loam layer of more than 80cm on top of a sand layer are very well suited for agricultural use. Low groundwater tables can be maintained, resulting in maximal yields;
2. Soils with a clay or loam layer of less than 80cm on top of a sand layer are most suitable for grassland. Groundwater tables cannot be kept low enough for agricultural use. When instead of sand a peat layer lies below the clay or loam, deep groundwater tables can cause settlement and there are structural change in the peat layer;
3. Sandy soils without humus are suitable for grassland and agricultural use where very shallow groundwater tables are needed. Here crops can take their water from the capillary fringe;
4. Sandy soils with humus are suitable for all uses, if groundwater tables can be adjusted to the crops grown;
5. Peat soils are suitable for grassland and vegetable cultivation. Crops with deeper roots will not grow well, due to the shallow groundwater tables. When groundwater tables are lowered, settlement and structure changes set in.

The land uses and cultivated crops of most polders are known. The cultivated crops of the Netherlands are given in a national land use map (in Dutch: *Landelijk Grondgebruikkaart Nederland*, www.lgn.nl; Figure 6-10). In general, low-lying areas are suitable for grassland and horticulture and higher areas are suitable for arable land. Low-lying areas are the most vulnerable to higher groundwater levels. In these areas the boundaries of the polder water levels will be chosen carefully. In high-lying areas deviations from the target groundwater level are less harmful, due to the flat slopes in the yield curves.

Figure 6-10 Example of a land use map (LGN4) for the TU area

Surface elevation is usually given in the Digital Elevation Map (DEM) of a country. The Netherlands have this DEM given in the AHN (an acronym for *Actuele Hoogtebestand Nederland*, www.ahn.nl; Figure 6-11). The data from this map is assembled from laser altimeter measurements for the period 1996-2003. The 'No data' pixels are water surfaces and green houses (glass). The resolution of the AHN-map is 5m. The future AHN2 map will have a 0.5 m resolution.

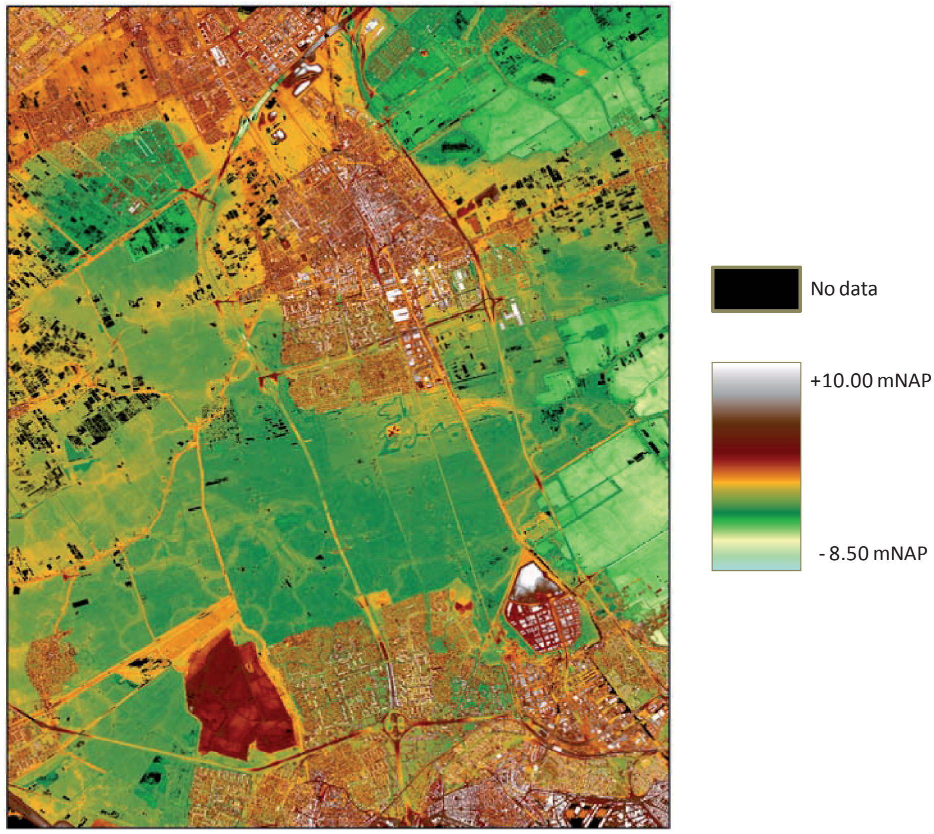


Figure 6-11 Example of the DEM / AHN for the TU area

For polders it is important to keep in mind that the soils are subject to settlement, depending on the soil material. Relatively new polders consisting of clay are known to settle 30 to 40% of the depth of the clay layer; loam 10 to 30%. Settlement can be calculated using the Koppejan Equation.

Simple polder water level calculation

Calculating the polder water level is carried out using one map with both the soil properties and elevation of the area on it. An elevation classification is given for each soil type. The surface area of each part with respect to the total area is calculated. It is assumed that the polder water level is equal to the actual groundwater table. For each field the yield percentage for a certain polder water level is calculated, assuming certain crop cultivation. These percentages are multiplied by the part area of the total surface area and totalled, which gives an average yield percentage of the polder for a chosen polder water level. An example is given in Table 6-4 for a polder water level of 590cm below MSL (Mean Sea Level)

Table 6-4 Example of a polder water level calculation

Soil profile and land use	Elevation in m - MSL	Part of total polder area	Average drainage depth at polder water level -5.90m + MSL	Yield in %	Yield times area
Clay (deep)					
Arable land	4.00-4.20	0.03	1.80	96	2.88
Arable land	4.20-4.40	0.04	1.60	98	3.92
Arable land	4.40-4.60	0.08	1.40	100	8.00
Arable land	4.60-4.80	0.05	1.20	96	4.80
Arable land	4.80-5.00	0.03	1.00	90	2.70
Arable land	5.00-5.20	0.04	0.80	80	3.20
Grassland	5.20-5.40	0.06	0.60	90	5.40
Grassland	5.40-5.60	0.06	0.40	70	4.20
Clay on sand (1m)					
Arable land	4.40-4.60	0.05	1.40	80	4.00
Arable land	4.60-4.80	0.05	1.20	80	4.00
Arable land	4.80-5.00	0.08	1.00	90	7.20
Arable land	5.00-5.20	0.08	0.80	80	6.40
Clay on sand (1/2 m)					
Grassland	5.20-5.40	0.11	0.60	90	9.90
Grassland	5.40-5.60	0.10	0.40	70	7.00
Clay on sand with silt (1/2 m)					
Grassland	5.20-5.40	0.09	0.60	80	7.20
Grassland	5.40-5.60	0.05	0.40	90	4.50
		Average	yield percentage		85.3

The difference in value of yields of grassland and arable land is not taken into account. In the example yield percentages are calculated at polder water levels of 590, 610, 620 and 630cm below MSL. These values are given in Figure 6-12.

It therefore follows that at a water level of 620cm below MSL yields are maximal (94%). When these type of calculations show a yield percentage above 90%, that polder water level is chosen. Lower percentages need to be raised, for example by levelling of the surface elevation, which can be very expensive.

Usually cultivation of a different crop is a less expensive solution. A possible decrease in value needs to be taken into account. Increasing the yield can also be achieved by subdividing the polder water level divisions. Costs of this subdivision need to be included in the cost-benefit analysis.

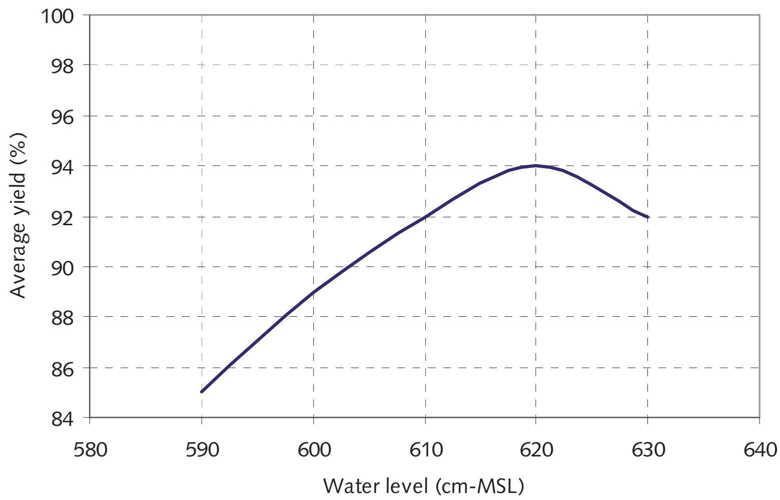


Figure 6-12 Calculation of the polder water level

In the table it is not possible to see in which adjacent divisions a different polder water level can be maintained in order to increase the yield percentage. GIS is used in order to get an idea of the possibilities.

Special conditions often influence the determination of the polder water level. When a clay or loam layer is situated on top of a sand layer, the polder water level cannot be too low, because the ditches will require a lot of maintenance. In low polders (sometimes brackish) seepage can occur. Often higher water levels are chosen.

When clay or loam is situated on top of a peat layer, the lower boundary of the polder water level will be at the top of that peat layer. If not, settlement and structure change will cause problems. Sometimes the polder water level needs to be higher to protect the foundations of buildings or to prevent water seepage to adjacent polders. Costs are also important in the determination of the polder water level; i.e. the costs of earth moving or an increased discharge head and capacity.

Deviation from the polder water level

In many polders a higher water level is maintained in summer, in an attempt to compensate for the groundwater table falling in summer due to the precipitation deficit. In plots where drains are installed, the polder water level is set to above the drains. Soils with higher hydraulic conductivities respond well to this higher water level. Where the hydraulic conductivity is low, the influence of the polder water level is minimal. Therefore the groundwater table will respond only slightly to the elevated polder water level. Here the summer water level is applied to use the water as drinking water for cattle and for watering by sprinkler installations.

In winter a lower polder water level is maintained, in order to improve the drainage. Only soils with peat in are not drained more in winter than in summer.

Polders are divided into several sections, where different polder water levels are maintained. Management is carried out at pumping stations, drainage sluices and weirs. Fixed weirs can cause a problem in situations when precipitation rates are high. Increasing drainage causes water levels to rise, which increases the plunging height. The receiving ditch will then receive more water than the pumping station can discharge. In order to prevent this from happening, the fixed weir needs to be adjusted so that the discharge is equal to the discharge of the pumping station. Therefore fixed weirs are only applied when the upstream section is less than 10% of the polder surface area.

Figure 6-1 Fixed weir (Rijn en IJssel Water Board).

The inlet of water from more elevated areas is in reality not as simple as illustrated here. Often larger discharge heads have to be overcome or water has to be let in from belt canals. Sometimes individuals are allowed to maintain a lower water level than the official polder water level. Only the individuals farming on the low lying areas are allowed to do this, so that it does not cause problems for others and it is only allowed when yields are significantly increased. This individual lowering of the water level is advised against, because too low water levels can cause settlement. If this is the case, lower water levels have to be maintained permanently. Moreover, for farmers who have lowered the water level the situation does not improve when the water level of the whole polder needs to be lowered. Precipitation can also cause large problems when this individual lowering is applied. In these situations excess water is usually discharged faster than the pumping stations can handle.

7 Allotment layout

7.1 Allotment

Roads, ditches, canals and other waterways subdivide a polder into sections. This division is referred to as allotment and one section is called a plot: an undivided piece of land usually owned or rented by one farmer. When several crops are grown on one plot the sections are called parcels. The long sides of a plot are bordered by ditches. The short side of a plot is bordered by a canal on one side into which the ditches discharge, and a road on the other side (see Figure 7-1).

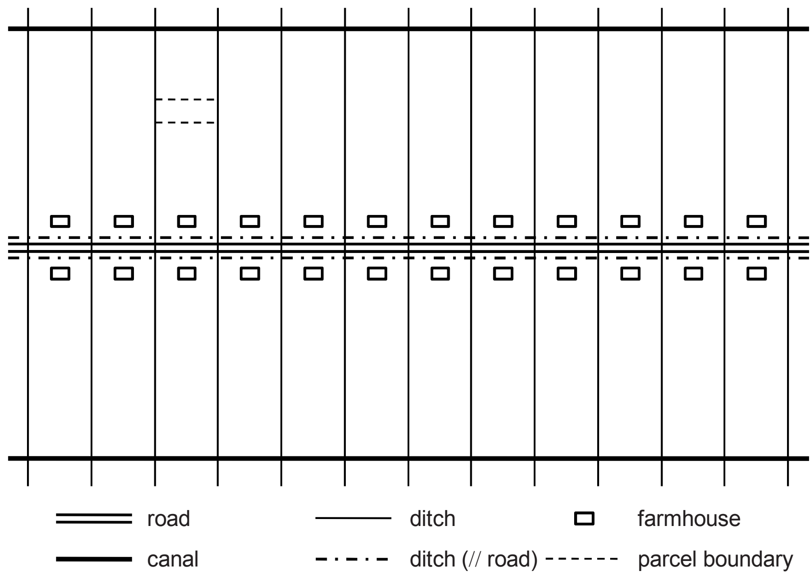


Figure 7-1 Allotment with a road on the short side of the plots

A farmer can own one plot, but it is also possible to own half a plot or more than one plot. The allotment is planned in the allotment layout. This layout is a complete infrastructural design with plots, but it also includes the zoning scheme: where farms, towns, (rail) roads, waterways, industry and recreational areas are located. If a farmer could decide for himself on the layout of his property, he would prefer a square piece of land with his farm in the middle, as he would then have the shortest distances to work on his land. In reality this would be very expensive, due to the large costs of roads and waterways. A layout with the long sides of the plot bordering the roads would also give the same problems (see Figure 7-2).

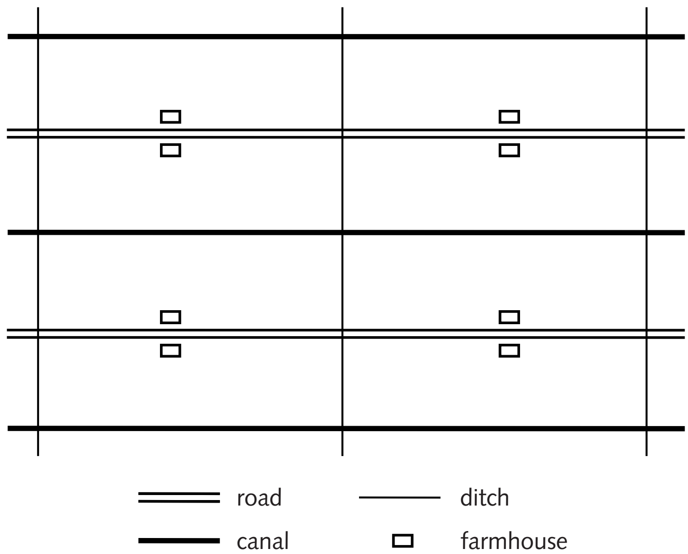


Figure 7-2 Allotment with a road on the long side of the plots

Modern plot design is based on minimizing investment costs and annual costs per hectare. The shape of these modern plots is shown in Figure 7-1. Longer plots would decrease the costs of roads and waterways, but it would increase costs for farmers.

The canals discharge into a larger secondary canal, leading to the pumping station (Figure 7-3). When the ground is level, the pumping station is located in the middle of the long side, in order to minimize earth moving costs and surface loss.

Canals are not located next to roads if possible because this would mean that each farm would need a bridge. Sometimes, if there are no other possible locations for the canal, a second parallel road is built in order to avoid the necessity for a lot of bridges. A disadvantage of this solution is that the ditch for each plot needs a culvert to cross the road. If no other solution is possible, the effect of this problem can be minimized by constructing a second ditch parallel to the road. This ditch receives water from a few perpendicular ditches between the plots and discharges into the canal through a culvert.

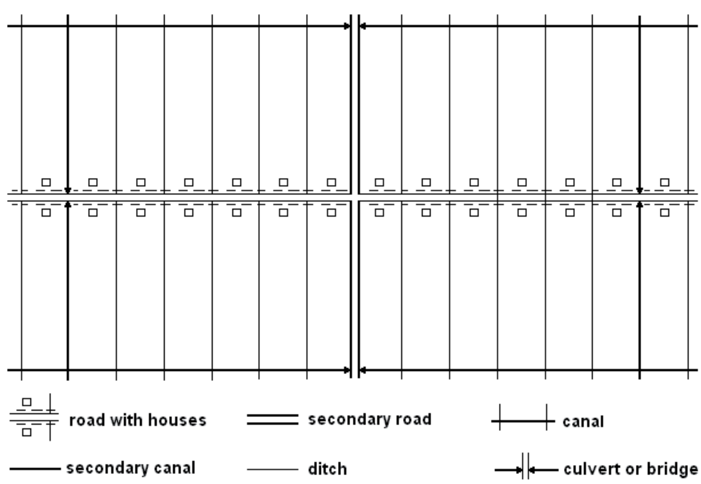


Figure 7-3 Example of modern allotment design

In the design of the allotment it is, for economic reasons, desirable to minimize the number of farms that are directly adjacent to the dikes. Roads next to a dike can only have houses on one side of the road, which makes them expensive. Roads on top of dikes - which can be found around several former lakes - require costly ramps to the farms. Sharp-edged corners are usually avoided but if this is not possible, the farm is placed in this corner.

Figure 7-4 Regular allotment in a new polder (Beemster) and irregular allotment in the older Mijzenpolder

7.2 Plot size in polders

The sizes of plots in older polders are usually determined by developments in water management. The oldest polders in provinces of Zeeland and Friesland have irregular plots.

Sandy soils were used as arable land and were mostly located on higher ground, close to the villages. The low lying land (in polders) was used as grassland. The distances to the villages meant that travelling times were long and costs of production were high. All over the country villages were build on elevated areas: mounds, old flow paths of rivers, and other sandy areas.

The polders in former lakes were better structured. The farms were placed on the plot that the farmer owned. The plot sizes were increased and the accessibility was improved. Generally a Dutch farm was 9.5 hectares on average when intended for arable farming and 11.5 hectares when intended for stock breeding. Horticultural farms were usually not larger than 2 hectares. The most profitable farm size is nowadays much larger, because of bigger investment costs and the costs of materials (see Table 7-1).

Table 7-1 Average size of agricultural farms in the Netherlands in 2003 and 2006 (source: LEI bedrijveninformatienet)

Year	2003		2006	
Farm type	Number [-]	area [hectare]	Number [-]	area [hectare]
Dairy	22 260	39	19 910	42
Pig breeding	3 980	8	3 890	8
Calf breeding	1 080	10	1 070	11
Chicken (eggs)	430	5	540	6
Chicken (meat)	380	5	340	6
Arable land	8 700	50	8 230	52
Greenhouses	6 420	2	5 200	2
Mushroom cultivation	380	1	260	1
Vegetables in open soil	950	16	870	18
Bulb cultivation	1 060	22	920	23
Fruit	1 490	12	1 450	13
Tree cultivation	2 270	6	2 260	7
Combined farms	6 120	31	5 550	33
…	…	…	…	…
Total agriculture	65 660	27	60 500	29

In modern polders arable farms have an average area of 50 hectares, but individual farms can deviate from that standard (Table 7-2). Sometimes one farm owns one plot, but usually a farmer owns several plots or portions of a plot. In the table below the extremely small and large farms are disregarded. In the Noordoost Polder (in the northern part of Flevoland) plots are relatively small. The reason for this is that a lot of farmers who had been displaced elsewhere were relocated here.

Table 7-2 Plot and farm sizes of modern polders (arable farming)

Polder	Plot size [hectare]	Farm size [hectare]			Average number of land plots per farm
		Small	Average	Large	
Lake Wieringer (*Wieringermeer*)	20	10	42	72	2.1
Noordoost Polder	24	12	27	48	1.4
Oostelijk Flevoland	30	18	40	60	1.7

Bearing in mind the costs of roads and waterways, it is preferable to choose plots that are as long as possible. However, very long plots also have drawbacks for farmers, who then have longer travelling times and higher transport costs. The optimal maximum plot length is therefore 1,200m. When plots are longer than this, a paved path is placed along the long side of the plot, in order to prevent damage in wet periods. Bearing in mind annual and investment costs, the most efficient plot sizes can be calculated for both layout as well as land use. By layout the construction of roads, canals, ditches and drainage system and the maintenance of roads and canals is meant. Costs of land use include employees' wages, costs of loss of time for both employees and machines, maintenance of ditches, costs of transport and loss of yield from turning places.

Figure 7-5 gives all the costs per hectare for several plot sizes and lengths of the Noordoost Polder. The most efficient plot size given here is 30 hectares with a length of 1,000m. It is generally found that smaller plots with shorter lengths and large plots with long lengths are most efficient; the costs per hectare all give a minimum plot width of approximately 300m.

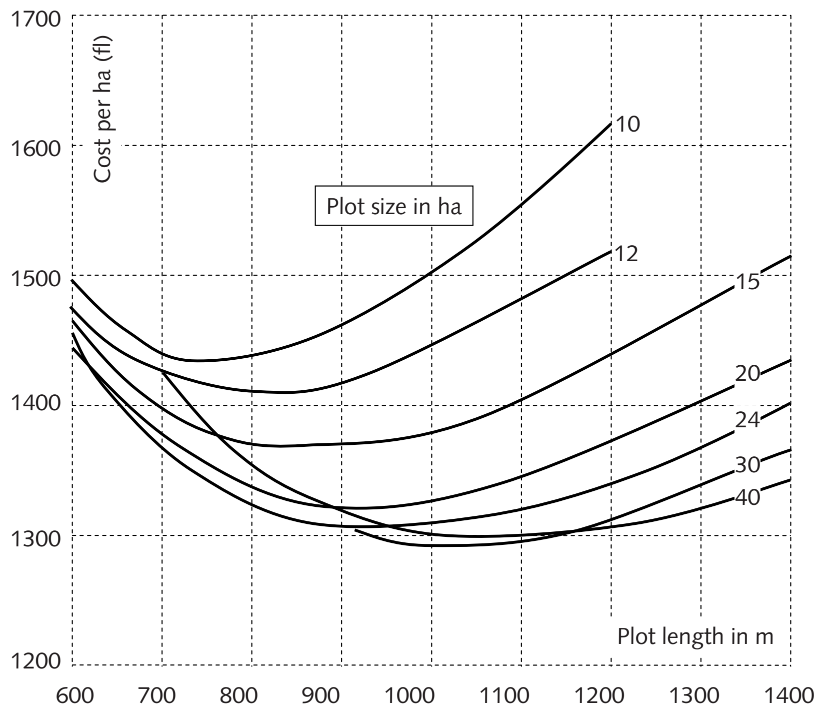


Figure 7-5 Investment costs and annual costs per hectare of several plot sizes in the Noordoost Polder (1950 prices, eight years after reclamation)

For the later parts of Flevoland the selected plot size was 30 hectares with a length of 1,000m. However for a few plots that were larger than 30 hectares a plot length of 1,000m was still considered to be the most efficient. The Noordoost Polder and Lake Wieringer (*Wieringermeer*) have standard plot sizes of respectively 24 and 20 hectares; both polders have a standard plot length of 800m.

7.3 Polders in the province of North Holland

In the province of North Holland the polders were created when lakes were reclaimed. Most of the lakes had an elongated shape from southwest to northeast, due to the prevailing south westerly winds causing erosion in the north eastern part (Figure 7-6). Examples of these polders are Purmer, Beemster, Schermer and Wormer. All of these polders were reclaimed using windmills discharging water into a ring canal surrounding the whole polder. Besides being used as a receiving water body, the ring canal also functioned as a shipping route.

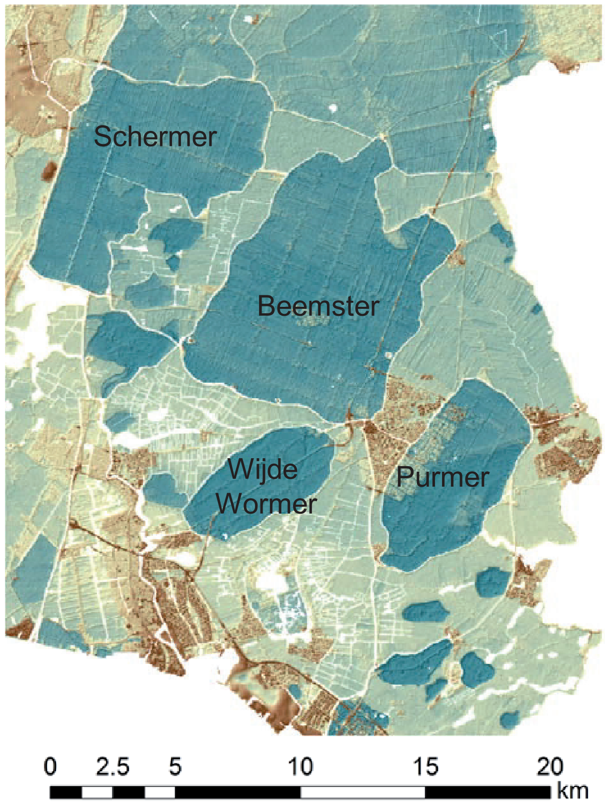


Figure 7-6 Digital Elevation Model (DEM) of a part of the province of North Holland with the four polders mentioned. The southwest-northeast orientation of the polders is clearly visible.

The Purmer Polder was reclaimed in 1622. In this polder one main canal leads to the pumping station in the north eastern corner. The ring canal is connected to Lake IJssel at Edam and Monnikendam for both water discharge and shipping. Two main roads were constructed along the length of the polder, with farms on both sides of the road. Farms were placed randomly next to the dike. Some side roads improved accessibility from the west and east. Originally only one village, Purmer, was planned in the polder, but nowadays Purmer has been fused with the municipality of Purmerend, of which the largest part is located outside the polder. Plots were placed perpendicular to the roads (see Figure 7-7).

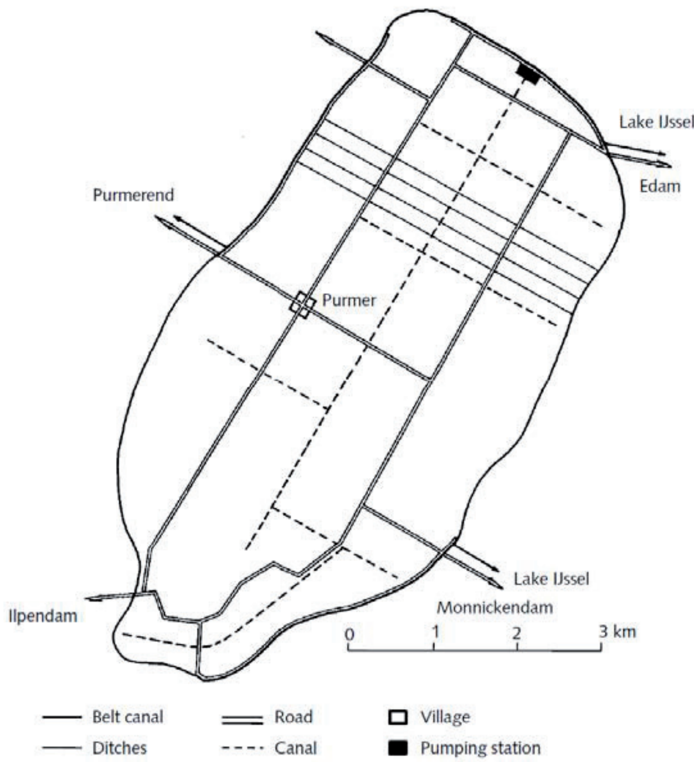


Figure 7-7 Allotment design of the Purmer

Lake Beemster was reclaimed in 1612. The polder was laid out with a net of canals and roads, dividing the polder into squares of 900x900m. The plots were placed randomly perpendicular to a road. The dike surrounding the polder has a road and farms placed next to it. Two pumping stations, one in the northeast and one in the west, discharge excess water.

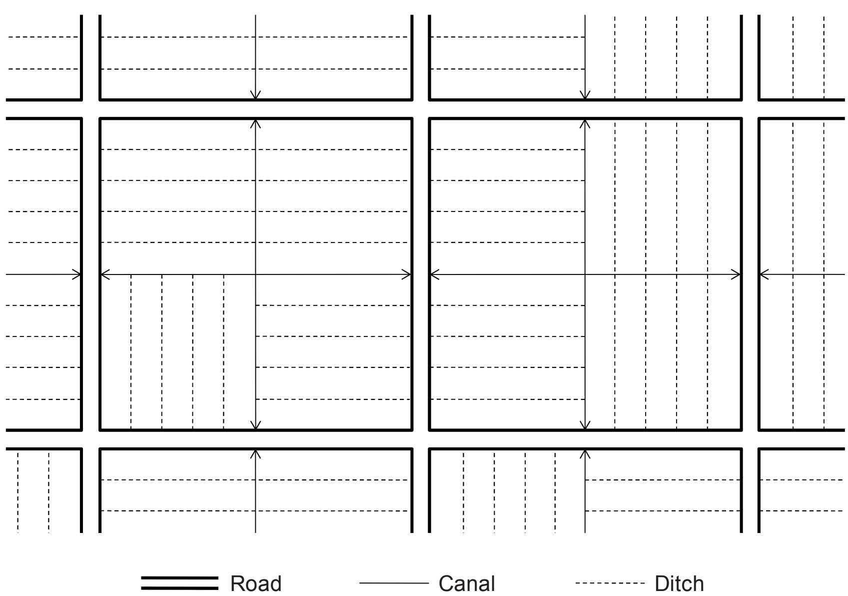


Figure 7-8 Allotment design of the Beemster

7.4 Polders of Lake IJssel

The *Zuiderzee* works were a large project that included dykes, land reclamation and drainage systems. The first plans were made as long ago as the 17th century to reclaim land in the Zuiderzee (now known as Lake IJssel). In 1891 Lely made an elaborated plan for this reclamation, but it was not until a large flood in 1916 that these plans were actually set in motion. Three polders were formed: Lake Wieringer Polder, the Noordoost Polder and in two phases the Flevoland Polder (see Figure 7-9).

Figure 7-9 Zuiderzee works: closure dikes and land reclamation

Lake Wieringer Polder

The first land reclaimed was Lake Wieringer (the *Wieringermeer)* which consists of 20,000 hectares. The reclamation started in 1927 and in 1930 it was declared land. The polder is divided into three sections; two pumping stations, Lely and Leemans, discharge the excess water. A large waterway system with sluices has been realized for shipping. The plots are a standard 800x250m. In the design six villages were planned in the middle of the polder and seven villages on the outer ring; each village would have a surface area of 1,500 to 1,600 hectares.

During realization this was decreased to only three villages in the polder each with a surface area of 6,000 to 7,000 hectares per village, from which *Wieringerwerf* was planned as the centre of the polder. However, the three villages were located too far from the outer ring of the polder. In practice this made Middenmeer, which is located more to the south, the most important village of the polder (see Figure 7-10).

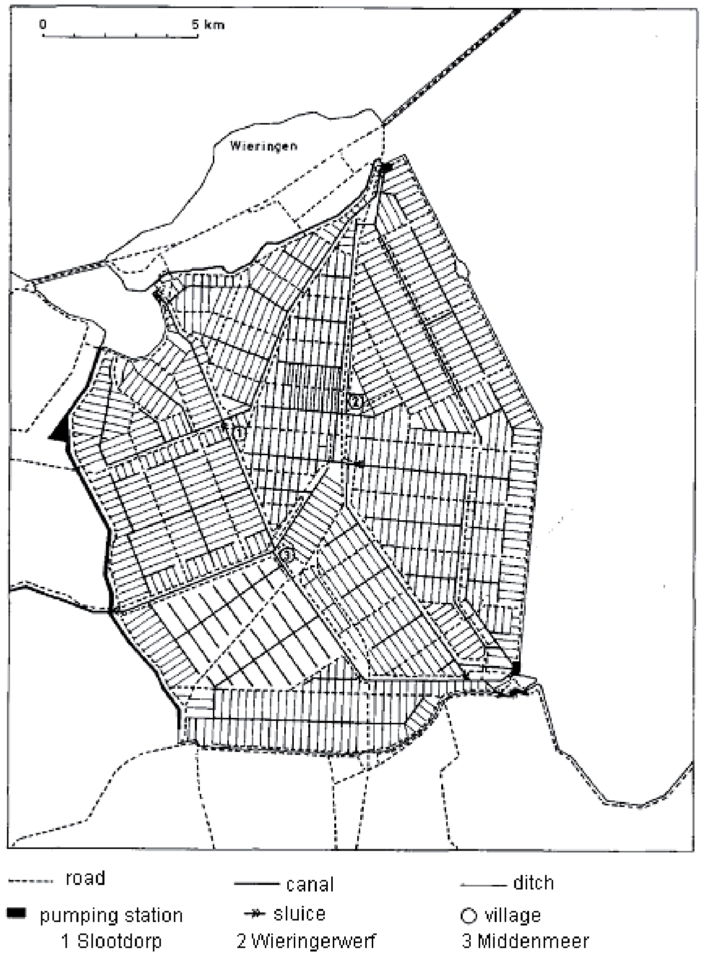


Figure 7-10 Polder Wieringermeer

On 17 April 1945 the polder was attacked by the Germans, who blew up the dike at two sites near Den Oever. About 8,000 people had to leave the polder immediately, and it took about 48 hours before the whole polder was flooded. Five hundred farmhouses, 1,150 houses, 7 schools, and 9 churches were destroyed or heavily damaged. On 5 August 1945 the dike was rebuilt and on 9 August the pumping stations started to discharge the water from the Wieringermeer Polder; on 11 December the polder was once again dry.

The Noordoost Polder

The reclamation of the Noordoost Polder (48,000 hectares) started in 1936 with preparatory soil replacement and in 1942 the polder was officially dry. The polder is divided into two sections; the first has a polder water level of NAP-4.50m and discharges through the pumping station Smeenge at Kraggenburg/De Voorst. The second section has a polder water level of NAP-5.70m and has two pumping stations: Vissering at Urk and Buma at Lemmer. The two sections are divided with a sluice, the *Marknessersluis*, which is opened occasionally to discharge water to the lower section. The canals are suitable for shipping, and connected to the surrounding water bodies by sluices.

Originally six villages were planned, but this was increased to ten, with Emmeloord in the centre. Plot sizes are standardized to 800x300m. In the north eastern corner of the polder a supply-discharge system has been constructed using aqueducts (see Figure 7-11).

Figure 7-11 Aqueduct in the Noordoost Polder; a concrete water supply channel crosses a water discharge channel (source: Zuiderzeeland Water Board).

Southern and Eastern Flevoland

The division in Southern and Eastern Flevoland is due to the fact that the polder was reclaimed in two sections. Eastern Flevoland was reclaimed between 1950 and 1957 and two years later the reclamation of Southern Flevoland started, which was finished in 1968. However, nowadays the two sluices in between are no longer used and the two sections operate as one.

The high polder section along the Veluwezoom has a polder water level of MSL-5.20m. Discharge water is pumped by the pumping stations De Blocq van Kuffeler in Almere, Lovink close to Harderwijk and Colijn close to Ketelhaven, as shown in Figure 7-12.

The low-lying section has a polder water level of MSL-6.20m. Here water is discharged by De Blocq van Kuffeler, Wortman close to Lelystad and Colijn. Colijn and Blocq are located at the convergence of the high and low-lying sections and therefore can be used for discharging water from both sections. In emergencies, a lock at Colijn can be used to discharge water from the higher into the lower section. A culvert in the south western part of the polder can also be used for this.

The first allotment designs were published in 1957. The design included 10 villages, of which Lelystad would be the main city (Lelystad was named after the designer of the Zuiderzee works, Cornelis Lely.) In 1960 the original designs were revised, reducing the number of villages to five. Lelystad would have an important regional function for Eastern and Southern Flevoland, as well as for the Markerwaard Polder, which at that time had not yet been built.

Between 1963 and 1975 the Houtrib Dike (*Houtribdijk*) was constructed between Lelystad and Enkhuizen; this would form the northern boundary of the Markerwaard. From1970 onwards the Markerwaard led to debate: first the method of implementing the Markerwaard was subject to discussion and later on the necessity for the whole polder. In 1986 the Government decided not to reclaim the land (for more information see: www.markerwaardpolder.nl).

Figure 7-12 Pumping stations in Southern and Eastern Flevoland

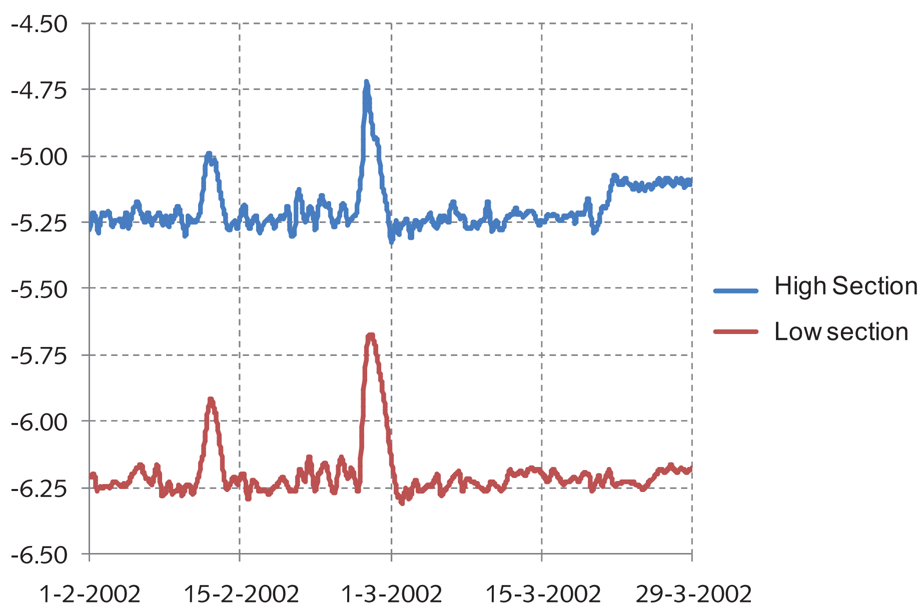


Figure 7-13 Water levels in the high and low-lying sections between 1 February and 29 March 2002

7.5 Land consolidation

A law on land use, development and planning passed in 1985 is the standard according to which land consolidation is implemented in the Netherlands. The main goal of land consolidation is to improve the layout of a polder in order to improve the current agricultural demands, and to increase both yields and farm management efficiency. Water management can be upgraded, allotment design can be upgraded to modern standards and the soil conditions can be improved. These improvements are of course only carried out when there are a lot of complaints. The scattering of properties is often due to the division of properties upon the death of a farmer.

Another possible method for changing the allotment design is the reallocation of land. The principle is the same as land consolidation, but the difference is that the decision for the change is not made by the landowners, but by the government and it is enforced by law.

When land consolidation is requested, preparatory research is carried out by a department of the Ministry of Agriculture and Nature (*Dienst Landelijk Gebied,* or DLG). A preliminary design is made, including the roads, waterways and landscape plan. Sometimes a new allotment layout is also included, with new locations for farms, utility works and public works. The goal is to design a modern and rational allotment plan and improve the value of the land in an area.

A vote amongst the owners decides on the implementation of the plans. The plan is implemented when either the majority of owners or the owners of the majority of land vote for it. If a vote is not cast, it is assumed to be *pro* the land consolidation. Since landowners finance the land consolidation, leaseholders are not allowed to cast a vote. Changes in leases have to be approved by the Land Tenure Board (*Grondkamer*), which protects the interests of the leaseholder. After land consolidation a leaseholder must be allowed to lease a piece of land equal in size and value.

A committee estimates the increase in the value of the land due to the land consolidation and the increase in value is billed to the landowner. The owner can object to the estimated land value after consolidation. After the whole consolidation process, the actual increase in value is determined, including the increase due to the consolidation of parcels and improved accessibility.

The new plots are assigned to the landowners according to the land contributed to the consolidation. A standard percentage of land is subtracted for waterways and roads. Differences of up to 5% are deducted. In the assigning of land to the owners, their wishes and demands are taken into account as much as possible. Land can be used for public use (i.e. a State highway). The total value of this land may not be more than 5% of the total and is only allowed when it complements the goal of the land consolidation. Extra costs of the project which are not covered by the landowners' contribution are paid for by the State. Costs can be very high, especially when land is levelled or canals and farms need to be relocated. Also poor soil conditions can raise the costs (i.e. the construction of roads on unstable subsoil). However land consolidation can also be very cheap, when land only changes ownership; this type of land consolidation is called an administrative land consolidation.

Usually the demands for land improvement are lower for grassland than for arable farming; horticulture has the highest demand. Ditches and canals that have become superfluous are filled with the soil that has been produced by the digging of the new ditches and canals. When farmhouses and farm buildings are not on or near the plots the farmer owns, it is sometimes decided to relocate the farmhouse or to build a new farm. This increases the costs drastically.

In land consolidation projects the distance from the farm to the farmer's plot is reduced, which increases the accessibility of the polder in general. Without having to build new farmhouses, the average distance to the plots is reduced to 75-90% of the original distance by re-allocating the plots. When farms are rearranged in these ways this number can increase even further. In a land consolidation project in the province of Zeeland the average distance from farm to plot was reduced from 1,600m to 1,200m. After redistribution of land the distance was further reduced to 520m.

8 Watercourses and roads

8.1 Watercourses

Watercourses in a polder are mainly used to drain excess water from the soil and discharge it to a pumping station or sluice. When precipitation deficits cause a water shortage, the watercourses can function as transport for the inlet water. The dimensions of the watercourses are calculated using hydraulic equations. In order to do so, the volume of water that needs to be discharged (supplied by precipitation) is the defining parameter.

The discharge (in mm/24h) at which a pumping station or sluice is dimensioned is called the specific discharge or design discharge. This discharge is the dimensioning parameter for the whole system. The slopes and dimensions are chosen in such a way that the water levels will not become too high and the velocities in the watercourses will not exceed limits. Watercourses can be divided into tertiary, secondary and primary watercourses:

Primary watercourses or canals receive water from mainly secondary watercourses, and transport it to pumping stations or sluices. In larger polders these watercourses can also function as shipping routes for both professional and recreational shipping. All functions need to be taken into account when dimensioning the watercourses. Dimensions can be calculated, assuming a slope and a discharge (the design discharge multiplied by the surface area). Earth moving and flow velocities are also boundary conditions. Wide canals need less depth, so less earth moving. Large slopes lead to large velocities and small profiles. It used to be common to take a maximum flow velocity of 10 to 20 cm/s, but nowadays the maximum flow velocities depend on the soil properties, which allow larger velocities in some soil types (Table 8-1). Soil types also influence the side slope, as the table shows. The high velocities shown are only applicable when the waterways are deep. When seepage makes the side slopes unstable, low velocities are applied. Roads close to the waterway also make the slopes less stable. Steep side slopes are usually covered with materials or plants. In very deep waterways two-step slopes are sometimes used.

Secondary watercourses are the larger ditches along the plots, dikes and roads. Like the tertiary watercourses their discharge can be considered minimal. The slope of the water table ensures that the water flows, and therefore no bottom slope is needed. The secondary watercourses only have to be dimensioned when they discharge water from an area larger than 30 hectares. Usually this is not the case, and the dimensions can be chosen: 0.45m depth below polder water level, 0.50m bottom width and a side slope of 1:1 or 1:2, depending on the soil type.

Tertiary watercourses are the drains, trenches and small ditches, directly within the plots. Drains and trenches are used for draining the soil; the excess water flows through the drains and trenches into the ditches. Usually the bottom level of the trenches is levelled with the polder water level. The trenches are not dimensioned to the system, since their discharge function is practically non-existent. The dimensions of trenches are usually 0.40m for the bottom width, 0.35m for the depth and a side slope of 1:1 or 1:¾, depending on the soil type. The small ditches have the same dimensions, only these have their bottom 0.35m below polder water level. Sometimes these small ditches function as boundaries for cattle, in which case they are wider. These ditches are also used for drinking water for cattle and in arable farming as water for sprinklers.

Sometimes the depth of watercourses is limited due to sandy layers in the subsoil. When a clayey or loamy layer overlies a sandy layer and the top layer is broken away, the sand becomes unstable. In the polders in the west of the Netherlands the clay layer on the bottom of the ditches is very thin; this causes seepage to pass through, which is usually brackish or even salty.

Table 8-1 Applicable velocities and side slopes for waterways

Soil type	Velocity [m/s]	Side slope		
Clay and loss	0.60-0.80	1:1	to	1:2
Loam	0.30-0.60	1:1½	to	1:2½
Coarse sand	0.20-0.50	1:1½	to	1:3
Fine sand	0.15-0.30	1:2	to	1:4
Solid peat	0.30-0.60	1:2		
Soft peat	0.15-0.30	1:2	to	1:4

The polder water level should be below the drain ends, and is only allowed to overflow the drains in extreme rainfall events lasting a short period of time. Sometimes this leads to shallow drain systems or the application of a very small slope in the drain system. Another option is to reduce the plot width.

Watercourses are kept clean to prevent harmful plant growth. Twice a year the watercourses are cleaned, removing plants and mowing the side slopes. The watercourses are also dredged every now and then, removing sand, parts of plants and sludge in order to restore the hydraulic properties of the system. The secondary and tertiary watercourses have to be maintained by the owners, whereas the primary watercourses are maintained by the water board, since this maintenance is in the interest of all. The water boards have special machines built for this maintenance.

8.2 Calculation of canal profiles

The bases for hydraulic calculations of flow through watercourses are:

The conservation of mass:
$$B\frac{\partial h}{\partial t}+\frac{\partial Q}{\partial s}=0 \qquad (1)$$

The conservation of momentum:
$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial s}\left(\frac{Q^2}{A}\right)+gA\,\frac{\partial h}{\partial s}+c_f\,\frac{Q|Q|}{A\,R}=0 \qquad (2)$$

In the second equation the first term shows the impulse supply, the second term is the net input of impulse, the third term is the gravitational force and the fourth is the friction.

At nearly stationary and uniform flow - as in polders - the water level and discharge do not change over time ($\partial h/\partial t=0$ en $\partial Q/\partial t=0$). This means that in (1) discharge does not change in space ($\partial Q/\partial s=0$). Also the slope of the water level can be considered equal to the slope of the canal bottom ($\partial h/\partial s=-i$). Putting all of this together leaves the Chezy equation:

$$-gA\,i+c_f\,\frac{Q|Q|}{A\;R}=0$$

Or, put more simply:

$$Q=A\sqrt{\frac{Ri}{c_f}}=CA\sqrt{Ri}$$

with:

Q	= discharge	[m^3/s]
A	= cross sectional area of flow	[m]
C	= Chezy coefficient	[m^{½}/s]
O	= wetted perimeter	[m]
R	= hydraulic radius (A/O)	[m]
i	= slope	[-]

The equation consists of three variables: profile, slope and discharge. Therefore when two variables are assumed, the third can be calculated. A drawback of the Chezy equation is the dependency of the Chezy coefficient on the canal depth. This coefficient can be determined using Nikuradse's, Kutter's or Bazin's (amongst others) formulas. For Dutch polder water courses the determination is carried out by using Strickler's formula. This formula state:

$$C = K_S \cdot R^{1/6}$$

In which K_S is a coefficient for the roughness with the dimensions [$m^{1/3}/s$]. The roughness is dependent on the cover of the bottom and side slopes, which is influenced by the time of year, soil type and depth. The most commonly used form of the formula is:

$$Q = K_S \cdot A \cdot R^{2/3} \cdot i^{1/2}$$

with:

Q	= discharge	[m^3/s]
K_s	= Strickler's roughness coefficient	[$m^{1/3}/s$]
A	= cross sectional area of flow	[m]
R	= hydraulic radius (A/O)	[m]
O	= wetted perimeter	[m]
i	= slope	[-]

Sometimes Manning's formula is used rather than Strickler's. The formulas are very similar; the only difference is the location of the coefficient:

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot i^{1/2}$$ Here n [$s/m^{1/3}$] is the Manning roughness coefficient.

The Strickler formula was originally developed for plant-covered waterways. Larger waterways experience relatively little influence from plant coverage, which makes the other formulas more appropriate.

Calculations are quite simple to carry out with a spreadsheet. The discharge for each section should be set as the product of the design discharge and the surface area which drains into the section, multiplied by the discharge of 'upstream' sections. A slope has to be assumed, keeping in mind that the water level should not exceed the level of the drains (usually 10 to 20cm above polder water level). At the end of the canal a pumping station will cause a drawdown of 20 to 50cm.

The slope of the canals is determined by the acceptable velocity in the canals and the amount of earth moving needed. For polders the slope in the upstream area is chosen at 2 to 3 cm per km and this gradually increases to 5 cm per km or more downstream. The values are variable and need to be considered for each polder separately, depending on the possibilities of gravity flow.

Table 8-2 Some values for the coefficient Ks in the Strickler formula. A small waterway is defined as a waterway with a maximum depth of 80cm and hydraulic radius of 50cm.

Watercourses	Summer	Winter
Small waterway with light soils	35	35
Small waterway with heavy soils	25	25
Medium-sized waterway with light soils	40	30
Medium-sized waterway with heavy soils	30	20

Figure 8-1 Excavator with a 'slope tray'

8.3 Head loss over culverts, bridges and weirs

A large part of head loss in polders is caused by structures, such as culverts, bridges and weirs. The loss in energy head is the sum of entrance loss, friction resistance and exit loss. The total loss can be described in a formula.

Culverts and bridges:

$$\Delta H_{tot} = c_{in}\frac{v^2}{2g} + \frac{v^2}{k^2 R^{4/3}}L + c_{uit}\frac{v^2}{2g}$$

where:

ΔH_{tot}	= energy head loss over the culvert	[m]
c_{in}	= coefficient for entrance loss ≈ 0.5	[-]
v	= flow velocity	[m/s]
g	= gravitational acceleration	[m/s^2]
k	= Strickler roughness coefficient for concrete≈ 70	[m$^{1/3}$/s]
R	= hydraulic radius	[m]
L	= length of the culvert	[m]
c_{uit}	= coefficient for exit loss ≈ 1	[-]

Bridges and broad culverts are usually placed within the cross sectional area of flow of the waterways. Broader structures are only implemented when the head difference or velocities become too large. In these cases the waterway is gradually broadened towards the structure.

For small discharges the culverts are round, elliptic or rectangular. Sometimes a shutter or flap is installed to close the culvert. If possible culverts with a diameter smaller than 40cm are avoided and under roads the diameter should be at least 50cm. In order to prevent the culvert from being blocked by floating rubbish, the culvert has 10cm or more free space above the polder water level. In this way the friction decreases and rubbish can pass through the culvert. Exceptionally, when the culvert's diameter is less than the water depth, the culvert is placed completely under water.

The head difference available in a polder department has to be divided between that needed in the waterways and that needed for the local losses in structures. Generally it is stated that half of the available head difference should be used for the passing of structures. In polders many culverts are situated in earth dams, constructed to provide access to farms or land. All the culverts might lead together to a large building up of hydraulic head in the end of sections during extreme rainfall events. Therefore, the local losses per culvert need to be minimized to a few millimetres.

Weirs

Weirs increase the water level as the water flows over the weir. For simple weirs with free flow the following formula applies:

$Q = 1.7\ m\ b\ h^{3/2}$.

when:

Q	= discharge over the weir	[m];
m	= coefficient dependent on the crest shape	[-];
b	= width of the weir	[m]
h	= water level upstream with respect to the crest	[m]

The coefficient *m* incorporates the shape of the crest into the formula. This coefficient is found somewhere between 1.0 for a broad crested weir to 1.35 for a short crested weir with a well-rounded crest. For less simple crest shapes, information can be found in the book 'Discharge Measurement Structures' by M.G. Bos (1989).
(see http://www.alterra.wur.nl/UK/publications/ILRI-publications/Downloadable/)

Bernouilli: $H = h_1 + \frac{v_1^2}{2g} = h_2 + \frac{v_2^2}{2g}$	→	$v_2^2 = (H - h_2) \cdot 2g$
$Q = v \cdot A = v \cdot B \cdot h$	→	$Q = Bh_2 \cdot \sqrt{(H - h_2) \cdot 2g}$
$Q = \max, \mathit{when} \frac{dQ}{dh_2} = 0$	→	$\frac{dQ}{dh_2} = \sqrt{2g}B\left((H - h_2)^{1/2} - \frac{h_2}{2}(H - h_2)^{-1/2}\right) = 0$
$(H - h_2)^{1/2} = \frac{h_2}{2}(H - h_2)^{-1/2}$	→	Multiply both sides with $(H - h_2)^{1/2}$
$(H - h_2) = \frac{h_2}{2}$	→	$h_2 = \frac{2}{3}H$
$Q = B \cdot \frac{2}{3}H \cdot \sqrt{\left(H - \frac{2}{3}H\right) \cdot 2g}$	→	$Q = \frac{2}{3} \cdot B \cdot H \cdot \sqrt{\frac{1}{3}H \cdot 2g}$
$Q = 1.7 \cdot B \cdot H^{3/2}$		

8.4 Backwater curves

A change in cross sectional area or a structure might create a backwater curve in a watercourse. This backwater curve can be either positive or a negative. When the energy head is larger than $H_{uniform}$ a positive backwater curve occurs. This curve is negative when the energy head is smaller than $H_{uniform}$. Calculating the surface profile manually is quite a lot of work, but it can be done in Excel or Matlab using the 'standard-step method'. If the deviations at the structure are small compared to the depth (<10%) one can use the following simpler formula to estimate the backwater curve:

$$h_x = h_u + z_o \cdot e^{\frac{-3i \cdot x}{h_u}}$$

where:

h_x	= water depth at location x	[m]
z_0	= backwater curve at the structure	[m]
i	= bed slope / gradient	[-]
x	= distance to the cross section or structure	[m]
h_u	= uniform depth	[m]

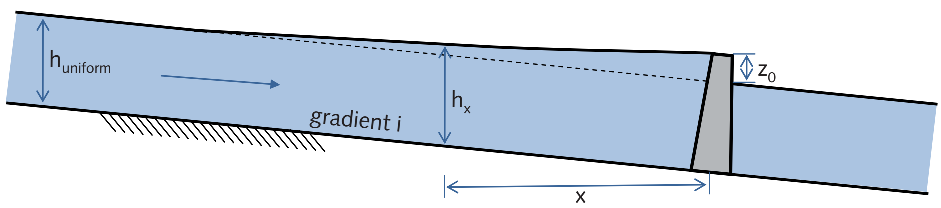


Figure 8-2 Positive backwater curve caused by a weir

Example

A canal has the following dimensions: b = 8.0m, side slope 1:1, bed slope of 5 cm per km and the Strickler roughness coefficient is 30 $m^{1/3}/s$. The design discharge is 2.98 m^3/s, using a depth h = 1.40m and velocity v = 0.23m/s. A control weir keeps the water level exactly at this 1.40m. The canal bottom at the weir is at +1.20m MSL. A discharge of 2.50m^3/s in the same canal makes $H_{uniform}$ = 1.26 m. The table below was made using Excel. Note that the backwater curve is still 4 cm even at 10 km upstream of the weir.

Table 8-3 Example of a calculation (Assignment: check the table with the given formula in Excel)

"x" [m]	*Water depth [m]*	*"A" [m]*	*Velocity [m/s]*	*Bed level [m+MSL]*	*Water level [m+MSL]*
-	1.40	13.16	0.19	1.20	2.60
1 000	1.38	12.99	0.19	1.25	2.63
2 000	1.37	12.84	0.19	1.30	2.67
3 000	1.36	12.71	0.20	1.35	2.71
4 000	1.35	12.59	0.20	1.40	2.75
5 000	1.34	12.49	0.20	1.45	2.79
6 000	1.33	12.39	0.20	1.50	2.83
7 000	1.32	12.31	0.20	1.55	2.87
8 000	1.31	12.24	0.20	1.60	2.91
9 000	1.31	12.17	0.21	1.65	2.96
10 000	1.30	12.12	0.21	1.70	3.00

8.5 Layout of roads and canals

The choice of routes for the roads and canals in polders is mainly influenced by the costs of construction of the roads, canals and structures in the polder. In essence three systems can be distinguished (Figure 8-3).

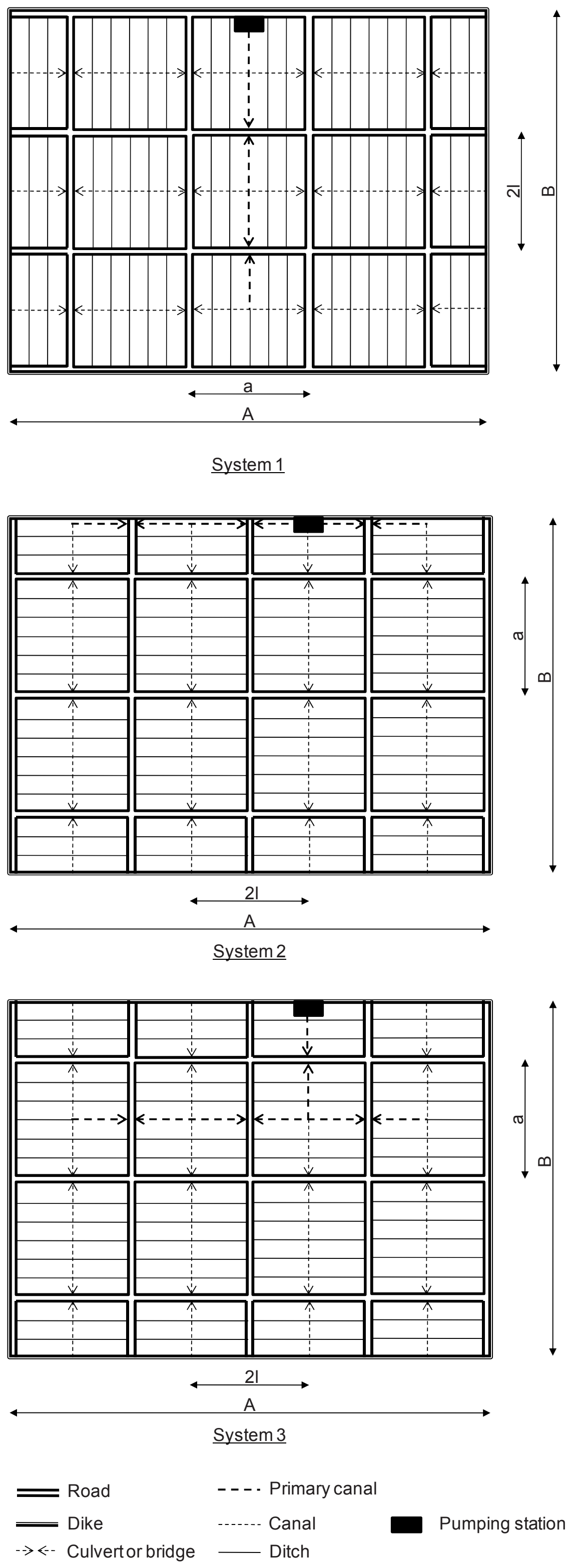


Figure 8-3 Systems of allotment

For each possible alternative system the length of the roads, canals and the number of structures are calculated. *A* is the length of the polder, *B* is the width, the plot length is defined by *l* and the distance to the main road by *a*. The presence of just one primary canal is assumed in all systems.

Length of the roads:

System 1: $\left(\frac{B}{2l}+1\right)\cdot A+\frac{A}{a}\cdot B=\frac{A\cdot B}{2l}+A+\frac{A\cdot B}{a}$

System 2: $\left(\frac{A}{2l}+1\right)\cdot B+\frac{B}{a}\cdot A=\frac{A\cdot B}{2l}+B+\frac{A\cdot B}{a}$

System 3: $\left(\frac{A}{2l}+1\right)\cdot B+\frac{B}{a}\cdot A=\frac{A\cdot B}{2l}+B+\frac{A\cdot B}{a}$

For a rectangular polder (not square shaped) the second and third systems produce the smallest road length. The second system has the primary canal placed next to the dike, which is not desirable. This leaves the third system as the best option.

Length of the waterways:

System 1: $\frac{B}{2l}\cdot A+B-l=\frac{A\cdot B}{2l}+B-l$

System 2: $\frac{A}{2l}\cdot B+A-2l=\frac{A\cdot B}{2l}+A-2l$

System 3: $\frac{A}{2l}\cdot B+A-2l=\frac{A\cdot B}{2l}+A-2l$

The second and third systems have the same length of waterways. For the same reason as before, system 3 is preferable. However, when the difference between length A and width B is larger than plot length 1, the first system is the best option.

Number of structures:

System 1: $\frac{A}{a}\cdot\frac{B}{2l}+\frac{B}{2l}=\frac{A\cdot B}{2al}+\frac{B}{2l}$

System 2: $\frac{B}{a}\cdot\frac{A}{2l}=\frac{A\cdot B}{2al}$

System 3: $\frac{B}{a}\cdot\frac{A}{2l}+\frac{A}{2l}-1=\frac{A\cdot B}{2al}+\frac{A}{2l}-1$

System 2 results in the smallest number of structures, but this system has its drawback in the location of the primary canal. The choice for system 1 or 3 will depend on the size (difference) of A and B.

From this prior analysis, the conclusion is that the three solutions are not very different when looking at the length of roads and waterways and the number of structures. Other factors will be of major influence on the allotment of the polder, such as the connecting roads to towns outside the polder, motorways, location of waterways with respect to the elevation, the presence of natural waterways, the landscape and the placement of farms.

9 Precipitation and evaporation

9.1 Precipitation

The main form of precipitation in the Netherlands is rainfall. This is measured in rain gauges, some of which are recorded. The gauges consist of a funnel-shaped container with measuring storage below (Figure 9-1). Daily precipitation is measured at 325 different Royal Dutch Meteorology Institute stations (Dutch acronym: KNMI). Measurements are carried out at 08.00h GMT/UT (local time: 09.00h in winter, 10.00h in summer).

Figure 9-1 Manual rain gauge with a storage area of 2 dm² and an elevation of 40cm (top of bucket)

In the Netherlands the average precipitation is 793mm per year. There is considerable spatial variation in the yearly precipitation, as shown in Figure 9-2. This figure shows the average yearly precipitation over the period 1971-2000 based on information from 283 locations. The most precipitation is measured near Vaals, behind the dunes in the province of South Holland and east of the 'Utrechtse Heuvelrug' (a lateral moraine in the east of the province of Utrecht). The lowest precipitation is measured in the middle of Limburg, the south-east part of Brabant, east of Drenthe and west of the province of Zeeland.

Besides these manual daily measurements, hourly measurements are carried out at the KNMI primary stations in De Bilt (near Utrecht), Vlissingen, Rotterdam Airport, Schiphol, Den Helder, Maastricht, Twenthe, Eelde, Eindhoven and Leeuwarden. These hourly measurements have been carried out using a pluviograph, tipping bucket (Figure 9-3) or electrical rain gauges. Until 1990 pluviographs were used by the KNMI. The system has a rotating drum around which a paper is fixed. As the drum rotates a pen registers the precipitation. Electrical rain gauges consist of a bucket of 2dm² located 40cm above ground level and a float which is connected to a potentiometer. The bucket is emptied every 12 seconds and the float registers the water level in the bucket.

Because of inconsistent measuring instruments and methods, the 24 hour total of the hourly measurements by pluviographs or electrical rain gauges are usually not identical to the daily measurements by conventional rain gauges. The hourly measurements up to the 1980s were corrected based on the daily measurements.

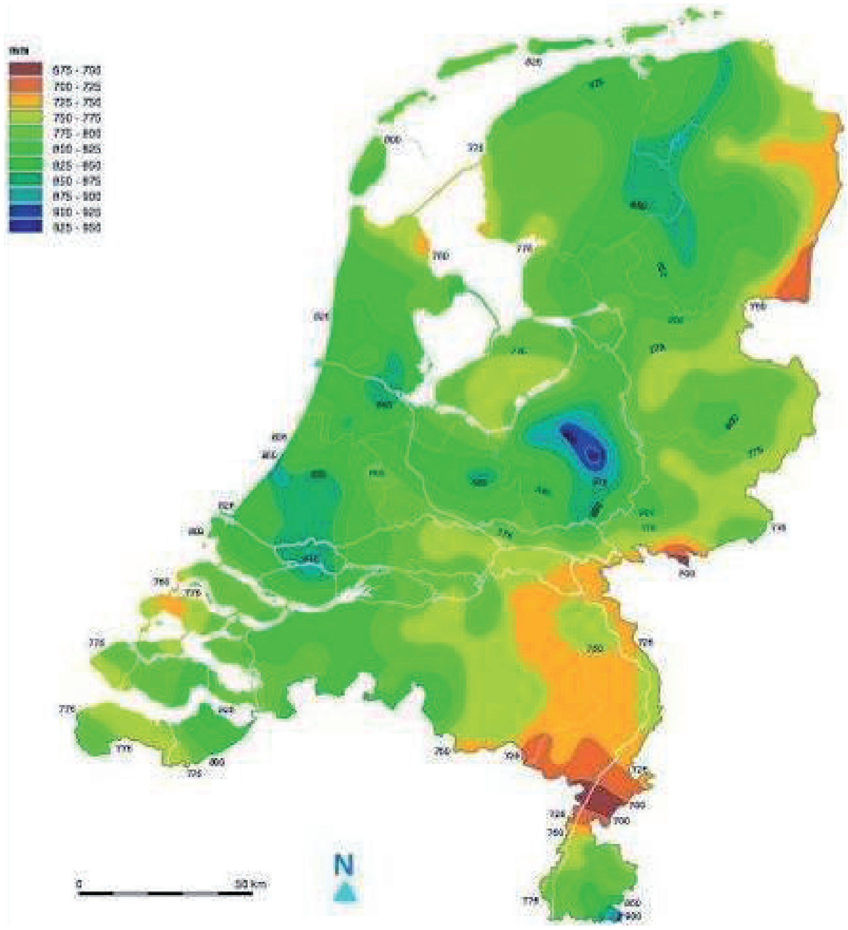


Figure 9-2 Annual average precipitation in the Netherlands from 1971 to 2000 (source: KNMI)

Figure 9-3 Tipping bucket rain gauge; every time the bucket tips over the actual time is registered. The surface area of the funnel and the volume of the bucket determine the precipitation unit of the tipping bucket.

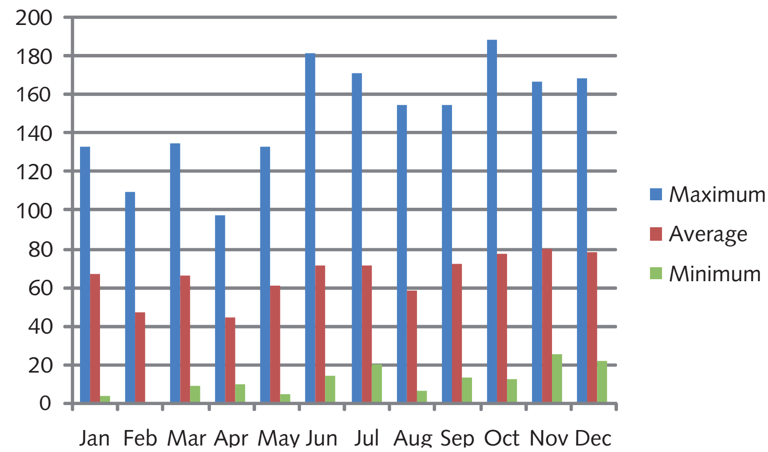


Figure 9-4 Maximum, average and minimum monthly precipitation in mm in De Bilt

An overview of the maximum, average and minimum monthly precipitation over the period 1971-2000 in de Bilt is given in Figure 9-4 (data source www.knmi.nl). The monthly precipitation varies considerably; the highest average amounts are in October, November and December, which are 40% higher than the lowest amounts measured in February and April.

The figure also shows the lowest amounts of monthly precipitation measured over the period. Except for November and December these are all less than 10mm. The highest amounts of monthly precipitation are found in June and October. In August large amounts of precipitation are mainly caused by thundery showers; large temperature differences and high humidity cause large amounts of precipitation in a short period of time.

Table 9-1 Maximum, average and minimum monthly precipitation in mm in De Bilt

	Maximum [mm]	Average [mm]	Minimum [mm]	Average [%]
Jan	133	67	4	8%
Feb	110	47	0	6%
Mar	134	66	9	8%
Apr	98	45	10	6%
May	133	61	4	8%
Jun	181	71	14	9%
Jul	171	71	20	9%
Aug	155	58	6	7%
Sep	155	72	13	9%
Oct	189	77	13	10%
Nov	167	80	25	10%
Dec	169	78	22	10%
Total per year	1240	793	536	100%

The yearly and monthly precipitation totals are much less significant for polders. Although these numbers are important to the soil saturation and the amount of water in fresh water basins, flood problems are normally caused by events where it rains constantly over a period of days. Table 9-2 gives an overview of the top 10 of highest precipitation rates over varying periods of time. Notice that the events in the first column with a duration of 4 hours (convective rainfall) are different from the events in the last column with a duration of 8 days (frontal depressions). Assignment: therefore which of these columns do you need to test a small urban polder or a large rural polder?

Table 9-2 Top10 of the highest maximum of precipitation in de Bilt for 4 hours, 24 hours, 4 days and 8 days over the period 1906-2003

Rank	Date	Rainfall	Date	Rainfall	Date	Rainfall	Date	Rainfall
	(dd-mm-yyyy)	4h (mm)	(dd-mm-yyyy)	24h (mm)	(dd-mm-yyyy)	4 days (mm)	(dd-mm-yyyy)	8 days (mm)
1	6-6-1961	51	3-7-1952	66	12-10-1960	111	8-6-1998	130
2	3-7-1952	49	1-8-1917	65	1-8-1917	95	25-8-1912	128
3	19-7-1966	48	12-10-1960	63	5-3-1998	90	19-7-1987	122
4	13-6-1953	47	8-2-1946	63	25-8-1912	85	1-12-1961	117
5	2-8-1948	43	19-7-1966	63	20-10-1986	85	10-10-1960	115
6	6-6-1998	43	2-8-1948	60	19-7-1966	85	20-8-1969	112
7	6-6-1943	38	1-8-1994	60	26-11-1983	81	5-2-1946	109
8	5-8-1947	37	11-7-1942	59	8-2-1946	79	14-9-1957	106
9	7-5-1931	37	6-6-1998	56	22-8-1969	79	11-3-1981	105
10	27-6-1930	36	7-5-1931	55	10-3-1981	79	22-10-1986	105

Figure 9-5 shows the rain duration curves for several recurring periods of time. The length of the rainfall event is shown horizontally (in hours) and the totalled precipitation amount is shown vertically (in mm). The data from a location closest to a polder is used for the calculation of the pumping capacity for the polder. When an area is larger than 10,000ha it is preferable for datasets from two stations to be used. If this is not possible, it is acceptable to use just one station; the measurements can then be reduced, as the area is so large that the high intensity precipitation will not occur simultaneously over the whole polder.

In some circumstances the discharge of an area can be much larger than the precipitation measurements would imply, i.e. when the soil is frozen and only overland flow occurs as the whole area behaves as a paved impervious layer or when precipitation causes the snow that has already fallen to melt.

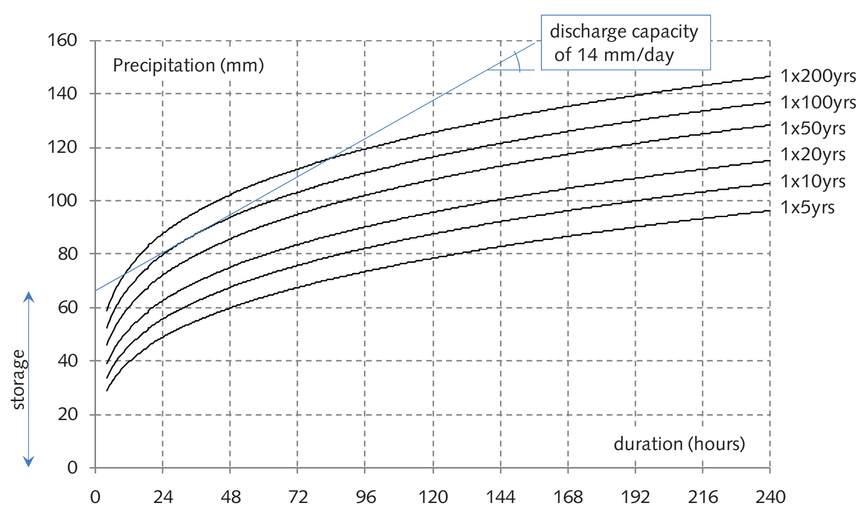


Figure 9-5 Rain duration curves for De Bilt with return periods of once every 5, 10, 20, 50, 100 or 200 years. To handle the 1:100 year rain duration curve at a discharge capacity of 14 mm/day, the storage capacity needs to be 66 mm.

9.2 Evapotranspiration (ET)

Crops mainly consist of water. Nevertheless, the amount of water needed for the formation of the water content in crops (in mm) can be ignored. Most of the water is used for the transportation of nutrients, for which water needs to evaporate through the leaves. The amount of water needed for the formation of 1kg dry matter is called the transpiration coefficient. This coefficient varies from 200 to 600 (average: 400mm/kg dry matter). Also the precipitation that was intercepted by the plants leaves will evaporate; this intercepted water evaporation can be 25% of the total precipitation in a forest. Water also evaporates from water surfaces and the soil; when describing plants one calls it transpiration. However, soils are usually covered with crops, which makes the evaporation irrelevant.

The total of transpiration and evaporation from leaves, soil and water is called evapotranspiration. Grass has the highest evapotranspiration (600mm), followed by clover (500mm), and Lucerne and wheat (both 400-500mm). Over short periods of time the evapotranspiration can exceed the average value drastically; this is strongest for grass, followed by clover, Lucerne, oat, beets, rye and potatoes. During hot, dry days the evapotranspiration can exceed 6mm/24h.

The average evapotranspiration is 450 to 600mm per year, but this is very dependent on the type of crop, growth stage, soil type, available water, temperature, wind and humidity. The density, rooting depth and soil particle composition are also an influence on the evapotranspiration. The exact value for evapotranspiration is difficult to measure. However, evapotranspiration fluctuates less than precipitation and it can be estimated by using several methods.

The estimation of evapotranspiration by means of the water balance of an area is most often applied. The equilibrium is made up of the different components which fill up and relieve a catchment area, using a balance of input and output; evapotranspiration is often used as the remaining balancing component.

A measuring instrument for evapotranspiration is the lysimeter. A lysimeter is a large bin containing a soil sample. The bin with the soil sample is imbedded in its natural surroundings. The precipitation is measured, as well as the discharge leaving the bottom of the bin. The water content is measured by determining the weight of the bin and its content. The disadvantage of lysimeters is that it is not possible to get an undisturbed soil sample, which makes the soil different from its surrounding. For this reason lysimeters are sometimes only separated on the sides with vertical boards from their surroundings. The water content is then measured by soil moisture measurements. In order to cancel out the influence of seepage the pressure head is kept constant both inside and outside the lysimeter.

Evaporation of surface water is measured in evaporation pans by determining the difference in weight, taking precipitation into account. The evaporation measured is however much higher than the actual open water evaporation, because the temperature in such a pan becomes much higher than the temperature in a body of water. Sometimes, the pan is placed in a body of water in an attempt to prevent this, but nevertheless the pan does not show the circulation that occurs in a large body of water.

Sometimes soil evaporation is measured by taking a sod of grass out of the soil, weighing it and putting it in a container back in the ground. The weight difference after one hour is defined as the standard evaporation.

The evaporation can be determined analytically by means of the air temperature and the degree of latitude. Thornwaite developed this method and it was further elaborated on by Penman, who included sunshine duration, wind speed and relative humidity. The method is based on an energy balance and is known as the Penman-Monteith method.

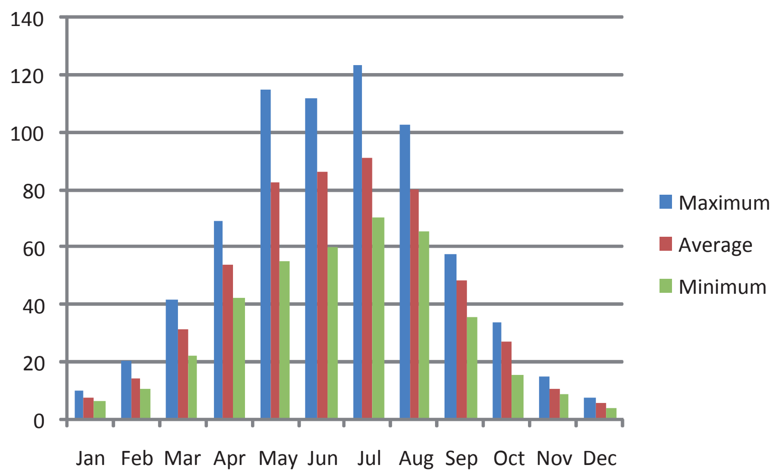


Figure 9-6 Highest, average and lowest reference evapotranspiration

Up until 1987 the Penman-Monteith method was used in The Netherlands, but since then the Makkink reference evapotranspiration method has been used; this takes the 24-hour total of the global evapotranspiration and the 24h temperature to determine evapotranspiration. Figure 9-6 and Table 9-3 show the highest average and lowest Makkink reference evapotranspiration for the period 1971-2000. The values show a strong similarity with the growing season, with low values in winter and high values in June and July.

Table 9-3 Highest, average and lowest reference evapotranspiration

	Highest [mm]	Average [mm]	Lowest [mm]	Average [%]
Jan	10	8	6	1%
Feb	21	15	11	3%
Mar	42	31	23	6%
Apr	69	54	42	10%
May	115	83	56	15%
Jun	112	87	60	16%
Jul	124	92	70	17%
Aug	103	80	66	15%
Sep	58	48	36	9%
Oct	34	27	16	5%
Nov	15	11	9	2%
Dec	8	6	4	1%
Total for year	616	543	492	100%

The Makkink reference evapotranspiration method is used for short grass under favourable conditions, with the soil moisture content at field capacity and sufficient nutrients. The actual evapotranspiration can vary from the reference evapotranspiration due to the cultivation of a different crop or deficit of soil moisture in the root zone. In order to estimate the actual evapotranspiration the reference evapotranspiration is multiplied by a crop factor (Table 9-4). The effects of the soil and weather conditions on the crop development can be calculated using a model simulation which takes into account a time series of evaporation and precipitation measurements (e.g. the Soil Water Atmosphere Plant model -www.swap.alterra.nl)

Table 9-4 Decade values for the crop factors required for the determination of the actual evapotranspiration (from Penman to Makkink, CHO-TNO, reports and memorandum no 19)

Month	*April*			*May*			*June*			*July*			*August*			*September*		
Decade	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III
Grass	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Cereals	0.7	0.8	0.9	1.0	1.0	1.0	1.2	1.2	1.2	1.0	0.9	0.8	0.6	-	-	-	-	-
Maize	-	-	-	0.5	0.7	0.8	0.9	1.0	1.2	1.3	1.3	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Potatoes	-	-	-	-	0.7	0.9	1.0	1.2	1.2	1.2	1.1	1.1	1.1	1.1	1.1	0.7	-	-
Sugar beet	-	-	-	0.5	0.5	0.5	0.8	1.0	1.0	1.2	1.1	1.1	1.1	1.2	1.2	1.2	1.1	1.1
Pulse crops	-	0.5	0.7	0.8	0.9	1.0	1.2	1.2	1.2	1.0	0.8	-	-	-	-	-	-	-
Planting onions	0.5	0.7	0.7	0.8	0.8	0.9	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-	-	-	-	-
Sowing onions	-	0.4	0.5	0.5	0.7	0.7	0.8	0.8	0.9	1.	1.0	1.0	1.0	1.0	0.9	0.7	-	-
Chicory	-	-	-	-	-	-	0.5	0.5	0.5	0.8	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Winter carrots	-	-	-	-	-	-	0.5	0.5	0.5	0.8	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Leeks	-	-	-	-	0.5	0.5	0.5	0.5	0.7	0.7	0.8	0.8	0.8	1.0	0.9	0.9	0.9	0.9

Figure 9-7 A pyrano meter to measure global radiation

The Makkink equation reads:

$$ET = 0.65 \frac{\Delta}{\Delta + \gamma} \frac{R_s}{\lambda}$$

in which:

ET	= Potential Evapotranspiration	(mm/dag)
R_s	= Solar Radiation	(joule/cm^2/day)
Δ	= Slope of saturation vapour pressure	(kPa/°C)
λ	= Latent heat	(joule/g)
γ	= Psychromatic constant	(kPa/°C)

Solar Radiation is measured by a pyranometer. Typical daily radiation values in the Netherlands are 1,800 joule/cm^2/day in the summer, and 400 joule/cm^2/day in the winter. The slope or derivation of the saturation vapour pressure (‘*helling van de verzadigde dampdrukcurve'*)'can be calculated with the following formula:

$$\Delta = \frac{4098\, e_s}{(T + 237.3)^2} \qquad \text{(kPa/°C)}$$

e_s	= Saturation vapour pressure	(kPa)
T	= Average air temperature	(°C)

The saturation vapour pressure can be estimated by:

$$e_s = 0.6108 \cdot \exp\left(\frac{17.27T}{T + 237.3}\right) \qquad \text{(kPa)}$$

This means that at 20 °C there is a pressure of 2.34 kPa and a slope Δ of 0.144 kPa/°C.

The latent heat λ is the necessary energy to convert water to vapour. The latent energy for water amounts to 2501-2.361*T (joule/g). Therefore at 20 °C the latent heat λ = 2454 joule/g.

The psychromatic constant γ can be estimated by:

$$\gamma = \frac{c_p \cdot P}{0.622 \cdot \lambda} \qquad \text{(kPa/°C)}$$

c_p	=Specific heat of moist air	(joule/g/°C)
P	= Atmospheric pressure (at sea level this is 101.3 kPa)	(kPa)
λ	= Latent heat	(joule/g)

The specific heat of moist air c_p is the heat energy required to increase the temperature of 1 g of water by a temperature interval of 1 °C and amounts to 1.012 joule/g/°C. Therefore, at 20 °C the psychromatic constant is (1.012*101.3)/(0.622*2453) = 0.067 kPa/°C. Thus the evapotranspiration (ET) at 20° on a day with 2,500 joule/cm^2/day solar radiation amounts to:

$$ET = 0.65 \frac{\Delta}{\Delta + \gamma} \frac{R_s}{\lambda} = 0.65 \frac{0.144}{0.144 + 0.067} \frac{2500}{2453} = 0.45\ g/cm^2/day = 4.5\ mm/day$$

9.3 Precipitation and evapotranspiration

The monthly course of the precipitation and the evapotranspiration is given in Table 9-5 and Figure 9-8. The monthly averages of precipitation are taken from the time series for De Bilt over the period 1971-2000.

Table 9-5 Averaged monthly precipitation and evapotranspiration from De Bilt (1971-2000)

	Precipitation [mm]	Evapotranspiration [mm]	P-ET (mm)
Jan	67	8	59
Feb	47	15	32
Mar	66	31	35
Apr	45	54	-9
May	61	83	-22
Jun	71	87	-16
Jul	71	92	-21
Aug	58	80	-22
Sep	72	48	24
Oct	77	27	50
Nov	80	11	69
Dec	78	6	72
Total for year	793	543	250

The yearly average precipitation is 793mm and evapotranspiration is 543mm per year. The yearly precipitation surplus is 250mm. However, during the growing season evapotranspiration exceeds precipitation. In April and September the precipitation and evaporation are balanced.

At the start of April the winter surplus is drained and discharged or used for the shift from winter level to summer level. From halfway through April onwards the evapotranspiration is higher than the precipitation and the groundwater levels gradually drop.

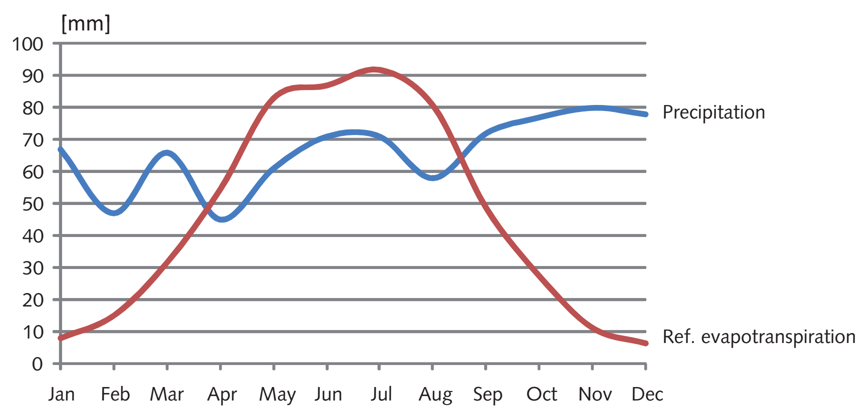


Figure 9-8 The monthly precipitation and reference evapotranspiration (in blue and red respectively)

In Figure 9-9 the totals of precipitation and evapotranspiration are given from 1 April. From this moment on the groundwater levels drop gradually, because the evapotranspiration is greater than the precipitation. This fall continues until halfway through August, when precipitation and evapotranspiration are equal. From this moment on, the precipitation exceeds the evapotranspiration causing the groundwater to be replenished. Halfway through November groundwater levels are levelled.

This is the cycle for a normal year. For the calculation of the total number of operating hours of a pumping station the precipitation surplus of the period 31 October to 31 March is taken into account. Therefore a small safety margin is built in, as the replenishment of the groundwater in November is not taken into account. Next, the precipitation surplus of the first half of April is also not taken into account, since this is used to raise the polder water level to summer level.

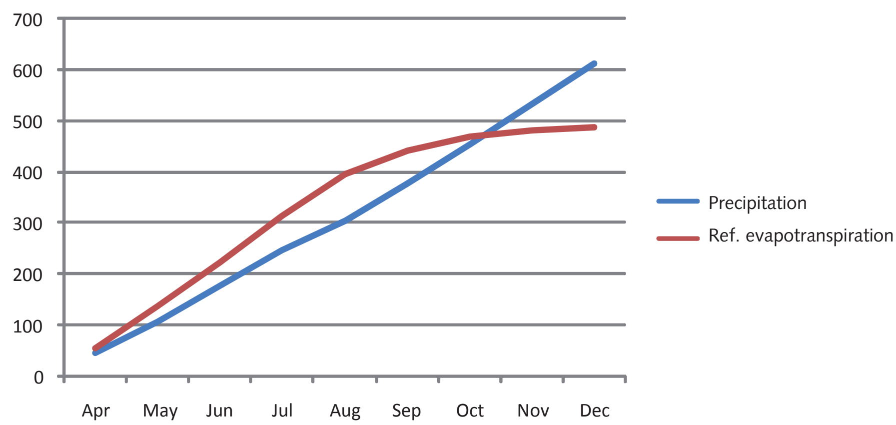


Figure 9-9 Precipitation and evapotranspiration totals in mm

The precipitation surplus from halfway through August until the end of October is not fully taken into account, although this period is known to have heavy (thunder) storms. In an average year the precipitation is used to replenish the soil moisture and the groundwater. This storage is especially important in clay. Here the storage between the wilting point (pF = 4.2) and field capacity (pF = 2) can be as much as 250mm per meter of soil. Sandy soils are far less able to store (130mm/m).

Raising the polder water level in April is done in order to minimize the groundwater level drop in the summer months. Generally, this measure has little effect in soils with low permeability. Better results are found when water is supplied to the soil through the drains, by raising the polder water level above the drain depth. This method is not ideal; in soils with low permeability the water deficit is only neutralized directly around the drain. Midway between the drains the water level will not rise; drains have to be placed with short distances in between in order to make this method work.

A better way of dealing with the moisture deficit is artificial watering by sprinklers. This keeps the groundwater level high, and the water content of the pendular zone at field capacity.

Figure 9-10 Sprinkler

10 Water balance and water surplus

10.1 Water balance

In a polder one can distinguish between factors that increase the amount of water, the so-called loading factors, and the factors that reduce the amount of water, the so-called relieving factors. The balance between all these factors is called the water balance. One way of writing this balance is:

$N+ES+S+L+I+WWTP-ET-IS-P-Q=\Delta bs + \Delta bc$

With:

Loading:		Relieving:		Closing balance:
N	= precipitation	ET	= evapotranspiration	Δ bs = ground storage
ES	= upward seepage	IS	= downward seepage	Δ bc = storage in canals
S	= lock water	P	= production process water	
L	= leakage	Q	= discharge	
I	= intake water			
WWTP	= effluent waste water treatment			

Too much water stays in the polder when loading factors exceed the relieving factors, which is called the *water surplus*. The surplus is neutralized by the discharge, with temporary storage in the ground and canals.

The water balance is the starting point used to determine the discharge in the case of a water surplus, or supply of water in the case of a water deficit. Water balances are also used for the determination of evapotranspiration and seepage. In order to determine these variables, the water balance has to be determined for an area that forms a hydrographic and hydrologic unit. Polders and river catchments are such units. The various factors are expressed in mm water over the whole surface area.

The water balance can be determined for a longer period of time; preferably a period is chosen in which the initial and final conditions are similar so that the closing entries (storage) can be ignored. The above factors will be discussed separately below.

Precipitation occurs through the cooling down of air that has been saturated by vapour. It can fall in liquid form as rain or in solid form as snow or hail. Condensation such as mist, dew and hoar frost is ignored; when it occurs just before a rain event, it will be measured as rain; otherwise it will evaporate and be measured as such. Part of the precipitation will fall on plant leaves or the ground.

Seepage is the discharge of groundwater to the ground surface. This can be divided into dike seepage and deep seepage. Dike seepage occurs when water from a higher water body outside a polder flows through and underneath the dike towards the polder. The amount of seepage depends on the head difference, permeability of the dike and surrounding soil and width of the dike (Figure 10-1).

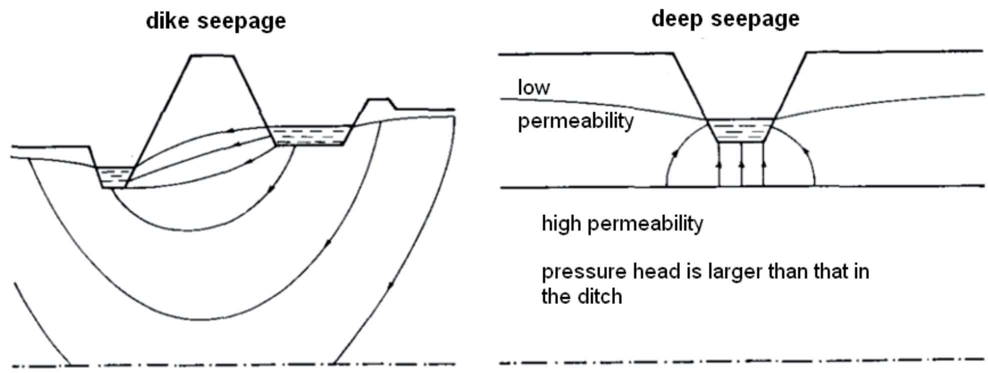


Figure 10-1 Schematic sketch of upward seepage

When a polder ditch is located behind a dike, most of the seepage will discharge into that ditch. Dike seepage can be prevented by:

- Building artificial walls of clay in the dike body;
- Reinforcing the dike with an impermeable layer;
- Drilling sheet pile walls;
- Constructing shallow ditches parallel to the dike to prevent the impermeable layer from being perforated.

When the seepage water is brackish or salty, it will be intercepted in a ditch in the polder and will be discharged separately.

Deep seepage comes from deep, often highly permeable, layers, as the water in the aquifer has a larger piezometric head than the head of the phreatic groundwater table. Because of the shorter pathway through the clay layer, this water ends up in the deeper ditches and canals of a polder. This seepage water can be brackish when the deeper layers have a marine origin, especially in the former lakes in the western part of the Netherlands. In periods with little precipitation the chloride concentration in the Dutch polders might therefore rise to as high as 500 mg/l or more.

The volume of the deep seepage is dependent on:
- The pressure head difference from the aquifer to the polder water level;
- The horizontal permeability of the subsoil;
- The vertical permeability of the top layer; and
- The path length.

The pressure head in the deep layer is usually dependent on the mean sea level. Since most polders have a ground level of several meters below sea level, the overpressure is often large. In these polders the top layer of the soil mainly consists of marine clay, which has a low vertical permeability. The layers in the subsoil are often sediments of mud flats, which have high horizontal permeability. The top layer is usually 2 to 3m thick, with an optimal drainage depth of about 150cm for wheat. This means that the bottom of the ditches will be 2m below ground level, and for canals even deeper. Therefore, the remaining thickness of the clay layer below those ditches and canals is practically non-existent. In these situations, the drainage depth will preferably be limited, e.g. to 120cm. This will cause the yields to be smaller; however brackish water would also have reduced the yield.

The amount of deep seepage in most polders is small. In Lake Haarlem (*Haarlemmermeer*) it is 0.2mm/24 hours, in Wieringermeerpolder 1.2mm/24 hours and in Eastern Flevoland it is 1.0mm/24 hours (Figure 10-2). However some polders have to deal with large amounts of deep seepage; for example, the Broekvelden en Vettenbroek polder near Gouda receives 13mm/24 hours and the Bethune polder gets 16mm/24h seepage (fresh water).

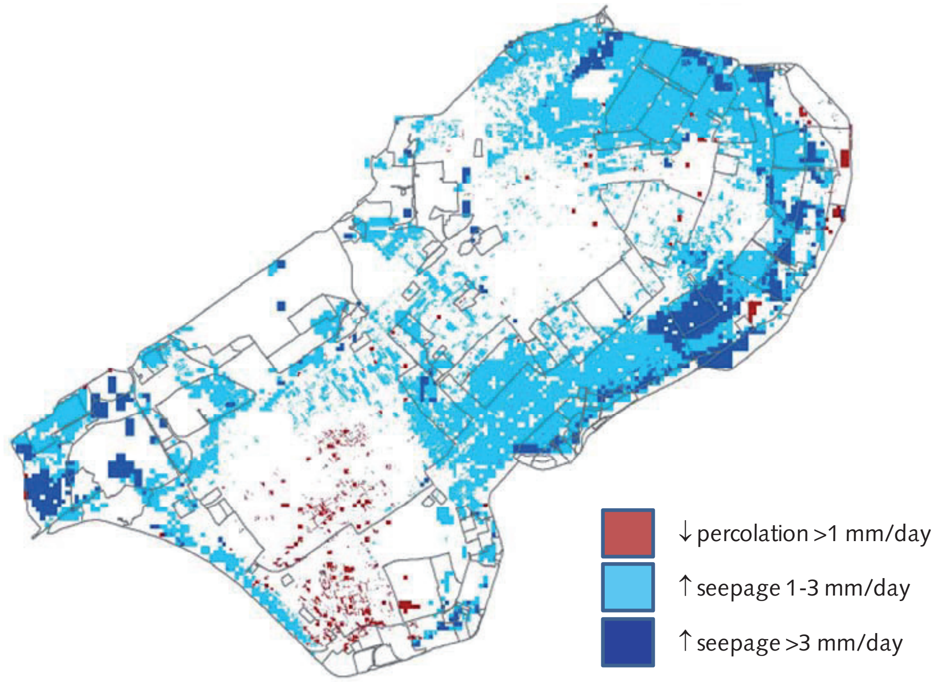


Figure 10-2 Seepage map of the Flevopolder (source: Zuiderzeeland Water Board)

Seepage to polders can cause downward seepage in adjacent land, especially when the adjacent land has a higher elevation, i.e. around Flevoland. The North East polder (*Noordoostpolder*) is connected to the adjacent land, creating a significant downward seepage flow from this land. This resulted in ground water tables that are too low. Southern and Eastern Flevoland were therefore reclaimed with large lakes in an attempt to prevent, or at least reduce, this seepage flow. Both seepage and downward seepage are difficult to quantify; seepage is often used as a closing entry in the water balance or estimated from the pumped volume over a long period in winter without rain (as there is little evaporation in winter).

Lockage water is water that enters the polder at the locks. This lockage water can be both a loading factor and a relieving factor to the water balance of a polder. The quantity can be estimated by multiplying the number of lockages with the surface of the lock chamber and the difference in water level. When the outer water is salt water, the salt load entering the polder is much larger than might be expected from the volume of the lockage water. Since salt water is heavier than fresh water, the exchange during lockage is not equal. Generally the total lock volume is replaced by the salt water. Through the Noordersluis close to IJmuiden 3.7 million metric tons of salt enters the North Sea (*Noordzee*) canal on an annual basis and the two locks between Lake IJssel and the Wadden Sea bring 1.7 million tons of salt to Lake IJssel.

Leakage water enters the system at locks, due to leakage around the doors of the locks. This factor can be negative as well as positive in the water balance and does not depend on the lockage itself. It is true to say that a better construction leads to less leakage. Generally the leakage water can be ignored. Sometimes leakage water can also enter a polder through other constructions in dikes.

Production process water is tap water that is discharged into the polder or belt canal system by industry. For example, the beer brewer Heineken discharges 3 million m^3 a year into the Rijnland belt canal system.

Inlet water is used for flushing waterways in polders. Reasons for flushing can be algae or contamination with sewage water from a mixed sewer system, but also for the reduction of the chloride concentration in a polder. In dry periods inlet water is used in order to decrease the water deficit by refilling the polder water level, by infiltration through the drains or irrigation using sprinklers. The effluent of waste water treatment plants can also be inlet water, when this plant discharges into the polder system. The volume of this water can be measured at the plant.

The *actual evapotranspiration* is dependent on the type of crop, growth period, soil type, available water, temperature, wind, humidity and balance between soil and water surface. The soil particle fraction, root depth and soil compaction are also important.

Discharge from a polder is done by means of a sluice, pumping station or wind mill. The volume of the discharge can be determined by the capacity of the pumping station and the number of pumping hours. When a sluice is used the sluice operation time and the water level difference determine the discharge volume.

Storage in ditches and canals is also called surface water storage. Precipitation that falls on the side slopes or directly into the ditches, water from lockage and inlet and leakage water are all stored in the ditches and canals in the system. However water from seepage and drainage is also discharged into these water bodies. When considered over a long period of time the storage in ditches and canals can be ignored, but it can be significant when shorter periods are being considered.

Old polders had a surface water area of up to 10% of the total polder area, which made the surface water storage possibilities very large. The structures for discharging to the outer waters (sluices and mills) made it necessary to have such large surface water storage possibilities; when outer waters had excessive levels or in situations with too much or too little wind, water had to be stored within the polders. The waterways were also frequently used for transport and therefore wide canals were needed. Nowadays these functions are no longer valid; transport is carried out mainly over land and discharge is not dependent on wind. The use of waterways to restrict the movement of cattle is also no longer needed, since electric fences are now common.

Modern polders have a smaller surface water storage; Lake Wieringer (*Wieringermeer*) has 2% surface water and the North East polder (*Noordoostpolder*) just 1%. This is because drains and reliable pumping stations make it possible to discharge water quickly. In many areas lakes are present in the polder system, which makes extra surface water storage possible. Examples of this are in Friesland (*Fryslan*) and Rijnland.

It is difficult to quantify the amount of *storage in the soil*. When defining the water balance, it is preferable for the final situation to be similar to the initial situation, allowing the storage in the soil to be disregarded. For short periods of time, this storage cannot be ignored. In summer the groundwater table drops drastically because from the middle of April onwards the evapotranspiration exceeds the precipitation. Therefore in the middle of August the groundwater table is at its lowest point. From this point onwards it rises again until it reaches the starting point in November. During this period (April-November) the water content in the pendular zone is reduced to almost wilting point. The storage capacity is the largest at this time; even extreme convective rainfall is hardly ever drained.

When the water content of the soil reaches field capacity, the rise of the groundwater level is determined by the storage coefficient. This is the porosity minus the water content at field capacity. The storage coefficient varies from 4% in clay to 16% in sand; the groundwater levels will therefore show a rise of 25 or 6 times respectively for every mm of precipitation. Preceding rainfall events substantially reduce the available storage capacity.

The water balance can be used for the determination of seepage and evapotranspiration. Over longer period of time the storage in canals and the soil can be ignored, which makes the seepage and evapotranspiration the closing entries in the water balance. In order to distinguish both, it is assumed that the evapotranspiration in winter can be ignored, making it possible to determine the seepage over this period. This seepage is considered constant throughout the year, making it possible to determine the evapotranspiration in summer. Lysimeter tests and test polders are also used for the determination of unknown factors in the water balance.

Table 10-1 The water balance in Rijnland during 2000-2003 (determined by a model!)

IN	[10^6 m^3]	OUT	[10^6 m^3]
Precipitation	3 356	Evapotranspiration	-1511
Waterboard HDSR (Woerden)	347	Evaporation of surface water	-292
Waste water treatment Plants	447	Downward seepage	-145
Seepage	275	Pumping station at Gouda	-219
Inlet at Gouda	175	Pumping station at Halfweg	-1 226
Sluice at Tolhuis	20	Pumping station at Spaarndam	-435
Sluice at Spaarndam	55	Pumping station at Katwijk	-879
Heineken factory	12	Pumping station at Leidschendam	-9
Schiphol airport	4	Cultivation under glass	-20
Total in	**1335**	**Total out**	**-1343**
Δ Storage	-47		

10.2 Water surplus in polders

The majority of precipitation infiltrates vertically into the soil. During summer this water will lead to the replenishment of the water content back to field capacity, before it leads to a groundwater level rise. Usually the groundwater rise does not occur before the middle of August, when precipitation exceeds evapotranspiration. In November the average groundwater level is reached once again. From that point onwards every rainfall event will cause an increase in the groundwater table above the surface water level.

Generally the period that deals with a water surplus is taken from 1 October to 31 March. Although the evapotranspiration will not exceed the precipitation until the middle of April, the surplus of those last two weeks will be used to raise the polder water level to the summer level.

The velocity at which the precipitation infiltrates (the so-called infiltration capacity) depends on the vertical permeability of the soil. In the root zone this velocity is quite high due to preferential flow paths (roots, cracks and earth worm paths). Negative factors can be horizontal clay layers, compressed layers and hardpans. Trenching can increase the infiltration velocity.

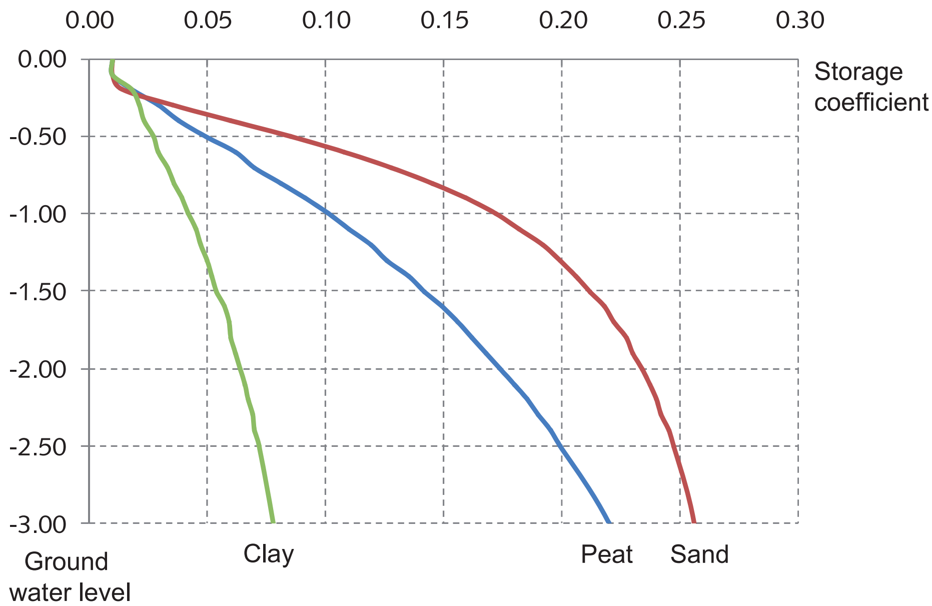


Figure 10-3 Storage coefficient (-) of clay, peat and sand as a function of depth

The rise of the groundwater level is determined by the storage coefficient. Clay has a coefficient of 4% (at a drainage depth of 1m); sand has a coefficient of 17% (Figure 10-3). The groundwater level will rise 25 times the precipitation (in mm) in clay soils and 6 times in sandy soils. The drainage to ditches or drains can be calculated using the Hooghoudt formula:

$$q = \frac{8kDh + 4kh^2}{L^2}$$

Where:

q	= constant precipitation		[m/day]
k	= horizontal hydraulic conductivity		[m/day]
D	= height of the surface water above the aquitard		[m]
L	= plot width between two parallel ditches		[m]
h	= height of the groundwater table above the surface water table at ½L		[m]

The drain distance L has to be calculated in such a way that the discharge q is capable of draining the precipitation, without the groundwater level midway between the drains rising above 40 to 50 cm below surface level.

The watercourses in a polder receive - as well as drainage water - water from shipping locks, leakage, inlet water, seepage and direct precipitation on the watercourses. These factors all form the direct load on waterways.

Seepage is calculated from the water balance. Precipitation directly on the surface water can be calculated from the surface area of those water bodies. In general the water inlet for flushing is closed until the water surplus is taken care of. However, the inlet from waste water treatment plants, factories and sewerage and also water discharged from other polders can still be discharged into the polder water system. The evapotranspiration is negligible in a situation of water surplus. Surface water storage can be calculated from the rise in surface water level. The storage in the soil is difficult to calculate or estimate, which makes the determination or estimation of the total discharge from the polder slightly more difficult.

A pumping station has to have sufficient capacity to discharge the drain discharge and the direct load on the waterways, without letting the surface water level rise too much. Generally the water level is not allowed to rise above the drain depth, because this would cause the drain discharge to drop drastically and might lead to a blockage of the drains near the drain ends.

'Rule of thumb'

An old 'rule of thumb' is that the precipitation from one day is drained evenly over three days (i.e. 1/3-1/3-1/3, including the day the rain fell). Depending on the size of the area, hydraulic conductivity, drainage depth, drain system and storage capacity, this drain period can be considered to be between two and five days. A calculation is given in Figure 10-2 with a three-day drain period and 63mm precipitation. The seepage, leakage and inlet water is assumed to be constant at 0.5mm/24h. The surface water storage is 1/50 of the total polder surface area; variation due area loss from rising water levels on slopes is ignored. The storage capacity is assumed to be 4% (clay). In the table the average surface water level rise (in m) is determined by multiplying the total surface water storage by 50/1000. The groundwater level rise in m is calculated from the multiplication of the total storage in the soil in mm by 25/1000. Initial conditions are assumed to be a water level of 0.80m below ground level for the groundwater and 1.20m for surface water.

During the first two days of this rain event, the groundwater level is higher than the surface water level, which also happens in real life. However, after the first few days, on the 10th day, the surface water level rises to 28cm above the groundwater level. This is not realistic; therefore this calculation method seems to be inaccurate. By increasing the pumping capacity this can be corrected, but this leads to very high pumping capacities. The precipitation can also be drained over a longer period of time, which would lead to lower surface water levels.

Table 10-2 Simple calculation of discharge and surface and groundwater levels

Date	7	8	9	10	11	12
Precipitation (mm)	18	33	12			
Discharge of precipitation to the waterways:						
first day	6	6	6			
second day		11	11	11		
third day			4	4	4	
total:	6	17	21	15	4	
Seepage (mm)	0.5	0.5	0.5	0.5	0.5	0.5
Waterway load (mm)	6.5	17.5	21.5	15.5	4.5	0.5
Pumping (13mm/dag)	6.5	13	13	13	13	7.5
Storage in waterways per day (mm)	0	4.5	8.5	2.5	-8.5	-7
Storage in waterways cumulative (mm)		4.5	13	15.5	7	0
Rise in surface water level (m)	0.00	0.23	0.65	0.78	0.35	0.00
Water level below surface level (m)	1.20	0.98	0.55	0.43	0.85	1.20
Storage in soil per day (mm)	12	16	-9	-15	-4	0
Storage in soil cumulative (mm)	12	28	19	4	0	0
Rise in groundwater level (m)	0.30	0.70	0.48	0.10	0.00	0.00
Groundwater level below surface level (m)	0.50	0.10	0.33	0.70	0.80	0.80

A better adjustment would be to distribute the precipitation in such a way that the majority of the precipitation is discharged on the second day, and then gradually decreases to zero over the next three days, instead of evenly between the day of precipitation and the two succeeding days. Nevertheless, the assumption that the precipitation will be discharged over three days is arbitrary. Furthermore, this method needs a lot of understanding of a catchment and its rainfall-runoff relation. This method is therefore actually outdated.

The rainfall duration curve

Instead of using an arbitrary period, the runoff factor can also be determined using, for example, the precipitation of a five-day period with a return period of five years.

These types of values are given in a rainfall duration curve, assembled from measurements over a period of at least 50 years. An example of such a duration curve is given in Figure 10-4. The pump capacity for a station can be roughly determined from this curve, assuming a rainfall event of five days totalled with the lockage, leakage, inlet and seepage over this period. In this example the extra water to be discharged is assumed to be 1mm/day. The pumping capacity should be: (67+5*1)/5=14.4 mm/day.

For this calculation it is assumed that the total precipitation amount will reach the groundwater table without delay and that pumping starts at the start of the rainfall event. According to these assumptions, the precipitation volume is discharged after five days. In reality pumping does not start until the water level in canals has risen to a threshold value (the switch-on level). Although this assumption is questionable, the value found for the runoff factor is in fact reasonable (according to experience in the Netherlands). A major disadvantage is that this method gives no understanding of the groundwater levels or surface water levels.

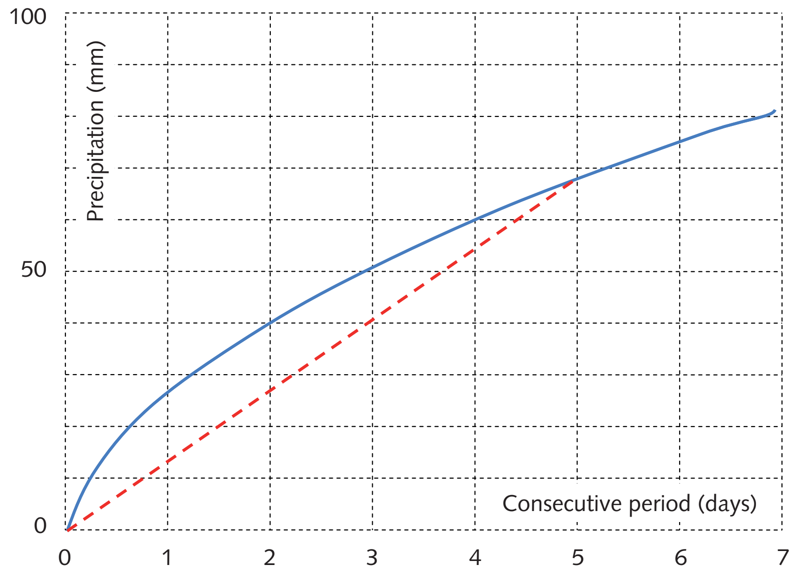


Figure 10-4 Calculating discharge from the rainfall duration curve; assuming five days' worth of precipitation can be discharged in five days

The improved rainfall duration curve

The calculation method using the rainfall duration curve can be improved by taking the delay of groundwater replenishment, surface water and groundwater levels into account (Figure 10-5). The starting point remains the total rainfall in a continuous rainfall period of five days with a return period of five years. For the calculation an initial condition is assumed for the groundwater level, usually set at the drain depth. In the figure below a balanced situation is given with average precipitation of 1.6mm for the winter season. Hooghoudt's formula is used to calculate the height of the groundwater table above the drain depth in between two drains, calculating the average groundwater level at the start of the rainfall event.

The calculation is now carried out over such small time intervals that the flow can be considered to be a steady flow. For the first period the time interval is taken as the time needed for the water to infiltrate the ground and percolate to the groundwater. This time interval is equal to the quotient of the height of the groundwater level below ground level and the infiltration capacity.

The precipitation volume reaching the groundwater table in the following periods is determined by reading the increase in precipitation from the rainfall duration curve at the end of the time interval at the point in time that is equal to the infiltration capacity. These values are totalled and in the figure below shown in the shifted rainfall duration curve.

The increase in the average groundwater level over the time intervals is determined using the portion of precipitation reaching the groundwater level, the discharge and the storage coefficient of the soil. The direct water surplus consists of the lockage, leakage, inlet and seepage water and the precipitation directly on the waterways and side slopes. The figure gives the totalled discharge from the soil to the waterways and the direct water surplus.

In actual practice, pumping starts when the surface water level exceeds the switch-on level at the pumping station. In this example this takes at least one day. Usually the demand is for the surface water level to return to the polder water level after five days. Using the figure, a pumping capacity of 14mm/d can be determined. The five-day period is not that important; it is more important that the groundwater and surface water level do not rise too much.

The average surface water level rise can be calculated from the direct precipitation, drainage water, pumping capacity and surface water storage. In the figure the surface water level drops in the first few days of the pumping period; later on it rises. The surface water level must remain below the drain depth. If this occurs, a decrease in drainage has to be taken into account. In this case it is not the difference in head between the drains and the ground water level midway (= convexity) that is determining the drainage discharge, but the difference in head with the surface water level. A short period of flooded drains is acceptable, but not desirable. More important is a timely discharge through pumping.

In day to day practice, pumping is started after the precipitation has made the surface water level rise. The initial groundwater level determines this level rise. When the initial groundwater level is below the drain depth, drainage does not start until the groundwater level has risen to a level above the drains.

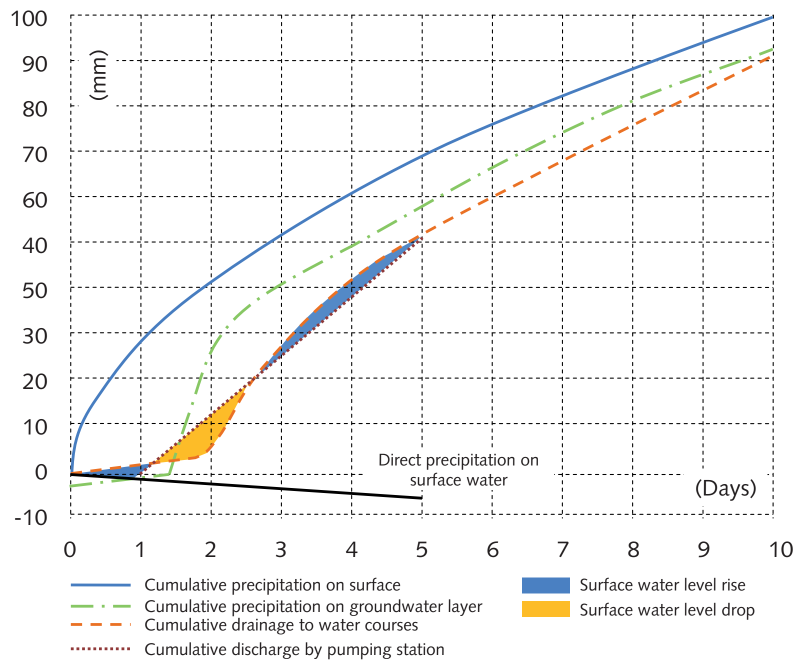


Figure 10-5 Calculation of the discharge from the rainfall duration curve; the groundwater and surface water levels can be determined for each point in time

10.3 Design discharge

The design discharge of a polder is dependent on several factors. For example: the slope of the terrain, seepage, crops under cultivation, plant cover, storage capacity of the soil, drainage methods, and flood damage. The drainage volume is mainly determined using measurements, experience or a hydrological model simulation.

The size of the area is an important factor; an increase in surface area results in a decrease in discharge factor. In general this is given by the formula:

$q = a \log A + b.$

With:

q	= discharge factor	[mm/day]
A	= surface area	[ha]
a	= constant	[mm/day/ha]
b	= constant	[mm/day]

Figure 10-6 gives an example of this relation for the polders in the *Hoogheemraadschap Hollands Noorderkwartier* water board area. The constant a is -6 and b is 32.

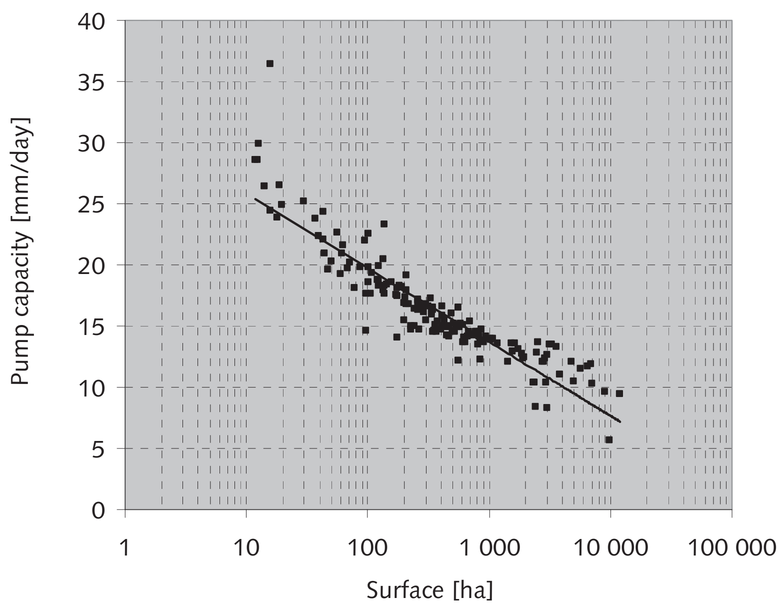


Figure 10-6 Discharge factor for 225 polders in the Hollands Noorderkwartier area

The rational method

A lot of research has been carried out in order to determine the maximum discharge of catchment areas. In initial attempts the relation between maximum discharge and catchment area were given in formula such as:

$$Q = C \cdot i \cdot A$$

This is the rational method formula, where:

Q	= maximum discharge	[m^3/d]
C	= discharge coefficient	[-]
i	= precipitation intensity	[m/d]
A	= catchment surface area	[m^2]

The precipitation intensity i is the intensity that corresponds with the concentration time of the catchment; this is the period of time required for water to flow from the most remote point of a catchment or drainage area to the outlet or point under consideration. This method is only applicable for catchment areas for which this relation is derived.

Unit hydrograph

Another method is the analytical determination of the discharge using a unit hydrograph. This method was developed in 1932 by Sherman (USA) and it uses the assumption that rain is built up by equally distributed precipitation events of the same length but with different intensities.

It also assumes that the discharge caused is proportional to the size of these precipitation events, and that the time span within which discharge takes places is equal for all events.

The influence of several rainfall events can be determined using the superposition of unit discharges, caused by stationary rainfall units of 1 hour or 1 day.

Fitting a statistical distribution

When a time series of discharge measurements is available for a catchment, a statistical distribution can be fitted in order to determine the design discharge. The database should consist of measurements that are independent and representative. To deal with the first requirement, only the highest discharges in a year are taken into account. The second requirement is more difficult to handle, as it is not certain whether the time series also contains a sufficient number of high discharges. It is also possible that the few high discharges have a different statistical distribution than the low discharges.

The rainfall-runoff formula

Krayenhof van de Leur and Wesseling carried out a lot of research on rainfall-runoff in the 1960s and 70s, which led to a better understanding of infiltrated water added to ground water, groundwater levels as a function of time and also runoff. The storage coefficient of the soil was assumed to be constant during their research. Later on it became possible to calculate a varying storage coefficient or to work with varying drainage levels (Hellinga and De Zeeuw, Ernst and Visser).

De Zeeuw created a method to relate water supply to the saturated zone with the discharge; an analysis is made of the measured discharge at the outlet as a function of the effective precipitation. Characteristics of the catchment, such as hydraulic conductivity, average depth of the aquifer, distance of open waterways and storage coefficient, are combined in one or more so-called reservoir coefficient. These coefficients are estimated using topographical and soil properties.

Discharge can be determined using precipitation and evapotranspiration data. With these calculated discharges and the actual measured discharges, the estimates for future discharges can be refined. When a model simulates the discharges correctly, the model can be used to predict discharges using any precipitation data available.

Design rainfall events - made with rainfall duration curves - determine the discharge for which the water levels do not exceed a maximum with a return period of *x* years. The assumption is that the return period for the rainfall event is equal to the return period of the water level calculated from that rainfall event. This is of course not true! To cope with this, engineers nowadays prefer to use continuous simulations with annual precipitation records; e.g. the record from the Bilt for the period 1906-2007 in hourly values.

Hellinga-de Zeeuw started with a simple equation to describe the discharge:

$q = \alpha \mu h$ [1]

with:

q	= discharge intensity	[m/day]
α	= drainage factor	[1/day]
μ	= storage coefficient (porosity – soil moisture at field capacity)	[-]
h	= head difference	[m]

and a water balance: $$\frac{(N-V-q)dt}{\mu} = dh$$ [2]

with:

N	= precipitation
V	= evaporation
q	= discharge intensity
dt	= time step
μ	= storage coefficient
dh	= change in groundwater level

Substitute [1] in [2]:

$$\frac{dh_{(t)}}{dt} = \frac{N-V}{\mu} - \alpha h_{(t)}$$ [3]

This is a first order inhomogeneous differential equation, for which the solution can be determined in two steps; first the general solution for the homogeneous part, and then the solution for the inhomogeneous part. The general solution for the homogeneous part reads:

$$\frac{dh_{(t)}}{dt} = -\alpha h_{(t)} \quad \rightarrow \quad h_{(t)} = C \cdot e^{-\alpha t}$$ [4]

Then vary the constant to determine C:

$h_{(t)} = C_{(t)} \cdot e^{-\alpha t}$ and $\frac{dh_{(t)}}{dt} = \frac{N-V}{\mu} - \alpha h_{(t)}$ gives:

$$C'_{(t)}e^{-\alpha t} - \alpha C_{(t)}e^{-\alpha t} = \frac{N-V}{\mu} - \alpha C_{(t)}e^{-\alpha t}$$ →

$$C'_{(t)}e^{-\alpha t} = \frac{N-V}{\mu}$$ →

$$C'_{(t)} = \frac{N-V}{\mu}e^{\alpha t}$$ →

$$C_{(t)} = \frac{N-V}{\mu}\alpha^{-1}e^{\alpha t} + A$$ [5]

Now [5] in [4]:

$$h_{(t)} = \frac{N-V}{\mu}\alpha^{-1}e^{\alpha t}e^{-\alpha t} + A \cdot e^{-\alpha t}$$ →

$$h_{(t)} = \frac{N-V}{\alpha\mu} + A \cdot e^{-\alpha t}$$ [6]

We need an initial value to determine A. So at t=0 we know that h(t)=h0, thus:

$$h_{(0)} = \frac{N-V}{\alpha\mu} + A = h_0$$ →

$$A = h_0 - \frac{N-V}{\alpha\mu}$$ [7]

Substitute [6] in [7]:

$$h_{(t)} = h_0 \cdot e^{-\alpha t} + \frac{N-V}{\alpha\mu}(1-e^{-\alpha t})$$

And with q= α μ h:

$$q_{(t)} = q_0 \cdot e^{-\alpha t} + (N-V)(1-e^{-\alpha t}) \quad [8]$$

10.4 Water surplus in urban areas

Discharge of precipitation surplus in urban areas is partly carried out by sewage systems and partly by drainage. Sewage systems not only discharge precipitation, but also domestic and industrial waste water and leakage water. Precipitation in urban areas also infiltrates into the soils in gardens, parks, etc. and causes the groundwater level to rise.

Precipitation on rooftops and roads is mainly discharged through the sewage system. Generally it is assumed that in the city centres 60% and in larger towns about 15% of the precipitation is discharged through the sewage system. On average one third of precipitation in urban areas ends up in the sewer.

For water surplus in sewage systems design rainfall events with different return periods are a design standard for Dutch systems (available in ‘Leidraad Rioleringen’). The sewage can either be a combined system, one system discharging both precipitation and waste water, or a separated system, with separate systems for discharging excess precipitation and waste water. Sewage water is not supposed to be discharged into surface water, although sometimes it is necessary to relieve the sewerage system, as they are not designed to cope with every event.

For the sewage system the pumping capacity is determined similar to the pumping capacity of a polder pumping station (Figure 10-7). The figure shows the rainfall duration curves with a return period of once every two and once every ten years. It is assumed that the storage capacity within the sewerage system is 7mm. The discharge curve of the sewerage has a capacity of 60 l/s/ha for paved surfaces, which is the generally recommended value in the Netherlands. The surface between this discharge curve and the rainfall duration curve is not discharged to the sewage system and has to be stored on the streets.

The second straight line is the capacity of the waste water treatment plant, which has a capacity of approximately 3 l/s/ha in addition to its standard dry weather discharge. The surface in the figure between the two discharge curves is the amount of water that cannot be discharged by the sewerage pumps and will end up in the surface water (13 and 23 mm).

Because of excessive rainfall this sewerage water is diluted three or more times, apart from any possible re-suspended sludge. The overflow water from the sewerage system to the surface water contains a maximum of 30% of actual domestic waste water and during extreme rainfall events this can drop to 5%. Nonetheless an overflow can cause local problems such as massive fish mortality due to a lack of oxygen.

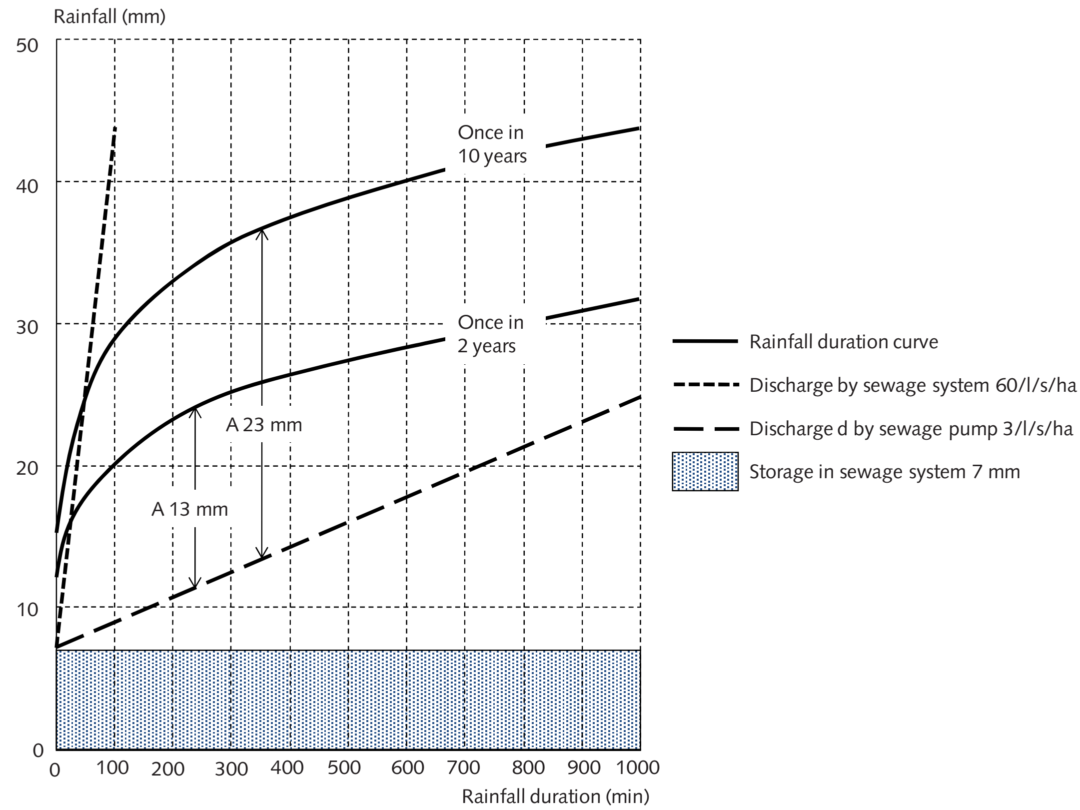


Figure 10-7 Example of discharge in urban areas. The volume 'A' needs to be stored outside the sewage system.

When the relation between water surface and paved surface area is known, the rise in water level can be calculated. This rise must be limited in order to prevent the overflow from the sewage to the surface water being blocked. If not, streets can overflow and basements and low-lying areas will be inundated. So for extreme rainfall events, storage of surface water needs to be much larger. The general assumption is that inundations in urban areas should be limited to a return period of 100 years.

The water surplus can be minimized by discharging water from the urban surface water to watercourses outside the urban area. In urbanized polders, pumping stations need to have larger capacities and the receiving water bodies will have an extra water surplus. The influence of that surplus on the pumping capacity of the polder pumping station, is related to the discharge coefficient and the relation of paved area/total area in the receiving polder.

Urban areas with sewerage systems and surface water can be considered polders. The amount of water that flows into the surface water through the overflow is direct surface water surplus. The surface water also needs to deal with drainage water from unpaved areas.

Drainage needs to be sufficient; therefore soils with low hydraulic conductivity are in need of extra measures. For short rainfall events, sewerage systems will be able to deal with the discharge. In practice, drainage in urban areas is poor. During preparation for building, ditches are filled with sand. This sand has a high conductivity, allowing it to drain the area, but clay dams are often built at the end of the former ditch. New water bodies are created in built-up areas, but often the drain distance is too long to properly function as a drainage system.

Often drainage of peat, loam and clay soils is too poor to maintain sufficiently deep groundwater tables in urban areas. This results in water problems in basements and low lying areas. If land is filled with sand during preparation for building, this can be prevented. However, the groundwater tables cannot always be sufficiently lowered.

When the old waterways in a built-up area are filled with sand and they are properly drained, the water problems can be minimized. Other solutions are minimizing the distance between water bodies, thickening of the sand layer and adding sideways drainage of these layers into water bodies or sewerage system or the implementation of a drain system. Deeper layers of sand under roads can be very effective, especially when sewerage pipes are also surrounded with sand, and the discharge of the excess water from the layer of sand is taken care of. Stability of the sand layer and lower chances of freezing of the system also lead to lower maintenance costs.

Figure 10-1 Mijdrecht on 26 August 2002 (photo Sjaak Ursem) and 31 January 2007 (photo Olivier Hoes)

11 Discharge by gravity flow

11.1 General introduction

The oldest form of discharging excess water from a polder is by gravity flow. This was practically the only available method until the invention of windmills. When discharge by gravity flow is applied, a weir or orifice is used to discharge into another polder, a belt canal system, river, brook or the sea. For this gravity flow a lower water level in the outer water is necessary. When discharging into the sea, this can happen at low tide. When discharging into rivers both low and average water levels are used.

In the past if water levels were high in the outer water, discharge was not possible and water needed to be stored in the surface water system of the polder. Thus, the outer water determined the period needed for storage: the sea has a tide twice a day, which indicated short storage periods; on the other hand, rivers could have long periods of high water levels. In order to minimize this disadvantage, the drainage sluices were implemented in the lowest, furthest downstream part of the polder. If even then the difference in water level was too small for discharge by gravity flow then parallel waterways - with little friction - were constructed to discharge water further downstream of the polder.

Settlement of the soil, sinking subsoil (Pleistocene) and rising of the sea level all cause the discharge to become more and more complicated. Most gravity flow discharge outlets have been replaced by pumping stations; these are sometimes combined with a tunnel that can be closed off at high tide and opened during ebb tide. Other reasons for installing pumping stations are the increased demands for agricultural land use; incidents with high surface water levels are no longer acceptable.

Nowadays discharge with gravity flow is only implemented for discharging from one polder to another or into a belt canal system when the up and downstream water levels make this possible. Discharging from one polder to another has a number of drawbacks; therefore it is recommended that the design discharge is not larger than the discharge capacity of the pumping station or sluice of the receiving polder.

11.2 Discharge sluices

Figure 11-3 shows a cross section of a drainage sluice, which is usually used for gravity flow discharge. When the surface water levels upstream and downstream of the sluice are more or less similar, the discharge formula of a submerged broad crested weir is:

$$Q = m \cdot b \cdot h_2 \sqrt{2g\,(H_1 - h_2)} \qquad \text{(providing } H_1 - H_3 < {}^1/_3\, H_1\text{)}$$

when:

Q	= discharge	[m^3/s]
b	= width of the sluice	[m]
m	= (entry) contraction coefficient	[m]
h_2	= water level above sluice bottom in the sluice	[m]
g	= gravity acceleration	[m^2/s]
H_1	= energy head upstream of the sluice above sluice bed	[m]

The entry and outlet opening are very important in this discharge calculation. The entry contraction coefficient is generally less than 1, but under favourable circumstances and for small discharges it can be greater than 1.

In practice the water level downstream of the sluice h_3 is known and h_2 is not. Furthermore, $H_1 \approx h_1$ when the velocity is limited. Therefore, the formula can be rewritten as:

$$Q = c_{sl} \cdot b \cdot h_3 \sqrt{2g\left(h_1 - h_3\right)}$$

when:

h_1	= water level upstream of the sluice, regarding the sluice bed	[m]
h_3	= water level downstream of the sluice, w.r.t. sluice bed	[m]
c_{sl}	= discharge coefficient for submerged long weir	[-]

The factor c_{sl} is a correction factor for the error made in the replacement of H_1 by h_1 and H_2 by h_3. This factor also includes the (minor) influence of friction and the sluice shape. For sluices that function as a submerged broad crested weir the factor is $0.7 < c_{sl} < 1.4$:

- A smooth weir with a round crest and small head difference gives $c_{sl} \approx 1.3$;
- A weir with a streamlined downstream end of the structure and small head difference gives $c_{sl} \approx 1.1$;
- A rough weir with sharp crest and large head difference gives $c_{sl} \approx 0.9$.

Figure 11-1 gives a time series of water levels of the outer and inner waters of a sluice discharging into the sea. During low tide, water is discharged, and so the water level in the polder drops. During high tide the water level in the polder rises, as no water is being discharged.

The discharge starts when the polder water level is just a few centimetres higher than the sea water level. This buffer of a few centimetres is needed to overcome the larger pressure of the salty water and the friction of the shutters. From the moment of discharge onwards, the discharge increases according to the formula given above for submerged broad crested weirs. For the situations of $h_3 < 2/3\ H_1$ the maximum discharge is described by:

$$Q = m \cdot \tfrac{2}{3} b H_1 \sqrt{\tfrac{2}{3} g H_1}$$

At the end of the discharge period a small head difference of the outer water level is enough to close the shutters, as the water downstream of the sluice will be freshwater. In practice the calculation is done by dividing the discharge period into short intervals, in which the flow can be assumed to be steady and h and H can be assumed to be constant.

In tidal areas automatic discharge sluices are usually used (in Dutch: *suatiesluizen*). The shutters open when the water level of the outer water is a few centimetres lower than that on the polder side and close when the outer water level becomes higher than the polder water level. When the shutters are opened, they are not completely pushed into their casings, in order to prevent them from getting stuck; wooden panels or steel springs are placed in the casing of the shutter. When flow velocity drops, the panels or springs push the shutters out of the casing and it closes. In order to prevent the shutters from closing every time a wave hits the sluice, the sluices are usually placed inwards and the dikes curve in towards the land.

Often a second sluice is placed somewhere behind the discharge sluice. This so-called *guardian sluice* is placed for safety but also functions when the discharge sluice is being inspected or repaired. The canal between the two sluices is well protected by dikes and can also function as a storage basin. When the channel from the discharge sluice towards the sea needs to be flushed, this basin is used to store water during high tide, so it can flush the channel during low tide. In spite of this flushing possibility, the channel will need to be dredged every now and then.

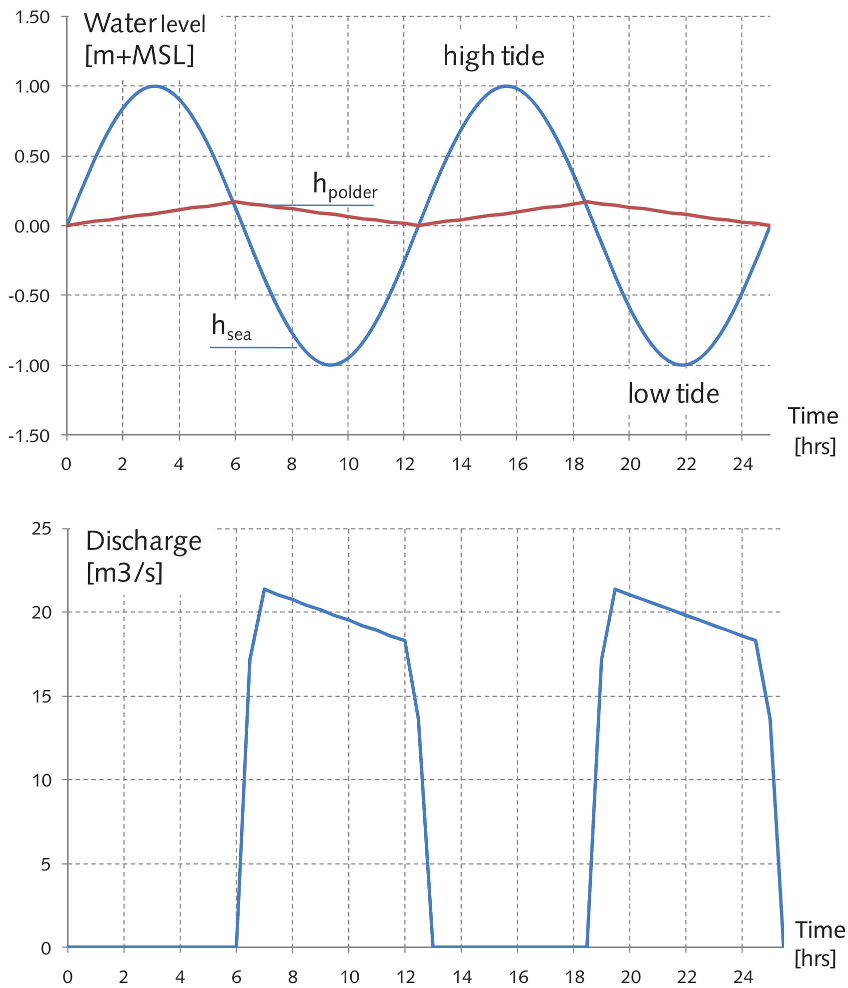


Figure 11-1 Time series of water levels and discharge of a sluice discharging into the sea

Sluice design in tidal areas

The sluice discharges water from a polder of 6,000ha. The surface water area of the polder is 125ha and the waterways are designed at 14.4 mm/day. The elevation of the sluice is -1.20m +MSL. The tide is an M2 tide with a period of approximately 12h 30min. After the discharge, the water level should be at the target level.

When the sluice is closed the water level rises for approximately 6 hours, resulting in a rise in water level of: (6/24)*0.0144*(6000/125) =0.17m. The total volume that has to be discharged is: (12.5/24)*0.0144*6000*10,000 = 450,000m³.

During the discharging period, clear fall occurs although at the beginning of the discharge period submerged flow occurs (as long as $z < {}^{1}/_{3}$ H):

- At the beginning of the discharging period the water level is +0.17m +MSL and $^{1}/_{3}$ H is 0.46m. That means that up to -0.29m+MSL the flow will be submerged.

- At the end of the discharging period the water level is +0.00m+MSL and $^{1}/_{3}$ H is 0.40m. This means that up to -0.40m+MSL the flow will be submerged.

The calculation for this sluice can be done in Excel with a time interval of 30min. The total volume is 55,063 x width m³. Because of the total volume determined above this would mean the sluice would have a width of 8.0m.

Table 11-1 Design of a discharge sluice

Time	h_{sea}	h_{polder}	y_{sea}	y_{polder}	loss			
[hrs]	[m+MSL]	[m+MSL]	[m]	[m]	[m]			
0	0.00	0.00	1.20	1.20	0.00			
0.5	0.25	0.01	1.45	1.21	-0.23			
1.0	0.48	0.03	1.68	1.23	-0.45			
1.5	0.68	0.04	1.88	1.24	-0.64			
2.0	0.84	0.06	2.04	1.26	-0.79			
2.5	0.95	0.07	2.15	1.27	-0.88			
3.0	1.00	0.09	2.20	1.29	-0.91			
3.5	0.98	0.10	2.18	1.30	-0.88			
4.0	0.90	0.12	2.10	1.32	-0.79			
4.5	0.77	0.13	1.97	1.33	-0.64			
5.0	0.59	0.14	1.79	1.34	-0.44			
5.5	0.37	0.16	1.57	1.36	-0.21			
6.0	0.13	0.17	1.33	1.37	0.05			
6.5	-0.13	0.16	1.07	1.36	0.28	submerged	= 4.4*((1.07+1.33)/2)* sqrt((0.05+0.28)/2)*b*1800 =	3 861*b
7.0	-0.37	0.15	0.83	1.35	0.51	free flow	= 1.7*(1.35^1.5)*b*1800 =	4 815*b
7.5	-0.59	0.13	0.61	1.33	0.72	free flow	= 1.7*(1.34^1.5)*b*1800 =	4 744*b
8.0	-0.77	0.12	0.43	1.32	0.89	free flow	= 1.7*(1.32^1.5)*b*1800 =	4 674*b
8.5	-0.90	0.11	0.30	1.31	1.01	free flow	= 1.7*(1.31^1.5)*b*1800 =	4 604*b
9.0	-0.98	0.09	0.22	1.29	1.08	free flow	= 1.7*(1.30^1.5)*b*1800 =	4 534*b
9.5	-1.00	0.08	0.20	1.28	1.08	free flow	= 1.7*(1.28^1.5)*b*1800 =	4 465*b
10.0	-0.95	0.07	0.25	1.27	1.02	free flow	= 1.7*(1.27^1.5)*b*1800 =	4 396*b
10.5	-0.84	0.05	0.36	1.25	0.90	free flow	= 1.7*(1.26^1.5)*b*1800 =	4 327*b
11.0	-0.68	0.04	0.52	1.24	0.72	free flow	= 1.7*(1.24^1.5)*b*1800 =	4 259*b
11.5	-0.48	0.03	0.72	1.23	0.51	free flow	= 1.7*(1.23^1.5)*b*1800 =	4 191*b
12.0	-0.25	0.01	0.95	1.21	0.26	free flow	= 1.7*(1.22^1.5)*b*1800 =	4 123*b
12.5	0.00	0.00	1.20	1.20	0.00	submerged	= 4.4*((0.95+1.20)/2)* sqrt((0.26+0.00)/2)*b*1800 =	3 072*b
								56 063*b
							b = 450 000 / 56 063=	8.03 m

The basin in between the two sluices has another important function in the discharging of excess water. During the discharging period as much water as possible needs to be discharged. Therefore the basin is often bowl-shaped; steep sides would prevent the water from flowing in as quickly as needed. Sometimes a small pumping station pumps water from the polder to the basin, so that it can be discharged as soon as low tide sets in.

The necessary capacity of a discharge sluice can be calculated using the design discharge of the belt canal system, the storage surface, the number of hours that discharge is possible and the difference in water levels. It is important is to take periods of high outer water levels into account, which can be found using statistical analysis. Astronomical influences and storm surges are important factors to take into account.

When discharging into a river, periods of high outer water levels are common, due to melt water and precipitation. These periods can last for quite some time and the water surplus in polders will then be significant.

Figure 11-2 Shutter in the Noordersluis at Lutje-Schardam

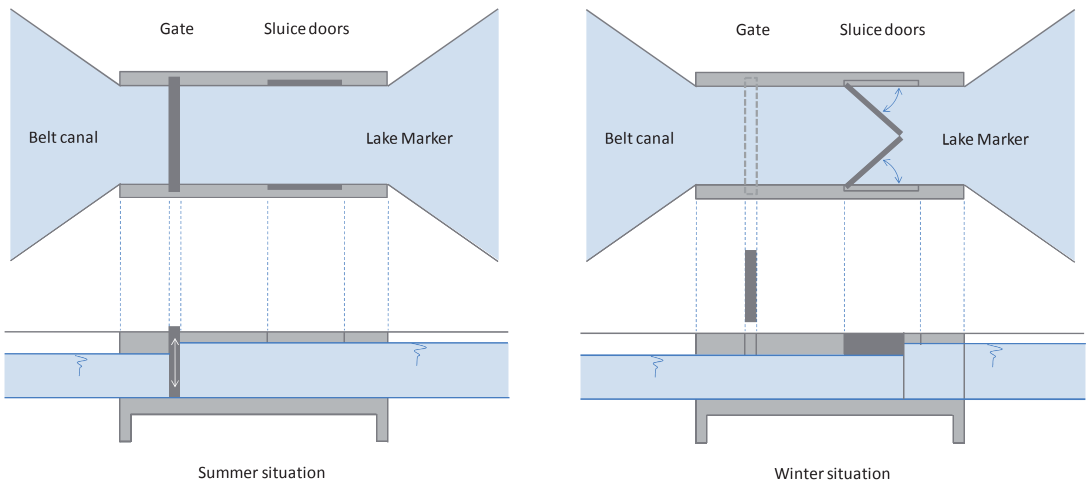


Figure 11-3 Discharge sluice at Lutje Schardam; on the left is the sluice in the summer situation: the doors behind the shutter are open and opening the shutter will let water in. On the right is the sluice in the winter situation: the shutter is up and the doors are closed. When the water level in the belt canal system is high, the doors will be pushed open and water will be let out. Low outer water levels only occur here during persistent westerly winds.

11.3 Shutter in a sluice

When water flows under a shutter, the formula for an opening under water is:

$$Q = \frac{\mu}{\sqrt{1-\left(\frac{\mu \cdot h_2}{h_1}\right)}} b \cdot h_2 \sqrt{2g(h_1 - h_3)}$$

When:

Q	= discharge	[m^3/s]
μ	= contraction coefficient	[-]
b	= width of the shutter	[m]
h_1	= water level in front of the sluice	[m]
h_2	= shutter opening	[m]
h_3	= water level behind of the sluice	[m]
g	= gravitation constant	[m/s^2]

The contraction coefficient μ is not a constant; but depends on the relation between the shutter opening and the water level in front of the sluice ($h2/h1\uparrow : \mu\uparrow$); and the head difference over the shutter ($z\uparrow : \mu\uparrow$). On average the contraction coefficient $\mu = 0.63$ when $h2/h1 \in (0,¼]$. When $h2/h1 \in (¼,1]$ the contraction coefficient is on average 0.75 .

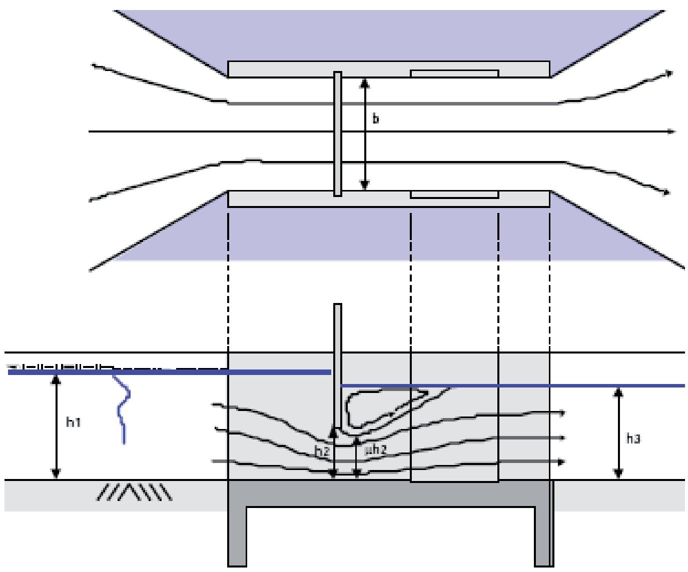


Figure 11-4 Shutter in a sluice

12 Pumping stations

12.1 Introduction

When gravity flow is not possible, excess water is raised from a polder using a form of water-lifting device driven by an engine. There can be a direct link between the two, but usually a speed-reducing transmission is needed to convert the engine's revolutions. In the early days the transmission was carried out by a belt pulley, but nowadays gear boxes and winding transmissions are used. The volume of water pumped out of the polder or belt canal system is called the capacity and is expressed in m^3/min for polder pumping stations.

The net capacity is given by multiplying the capacity (m^3/s) with the discharge head (m) and the factor ρg. The gross capacity is calculated by dividing the net capacity by the efficiency**.** Turbulence, friction and water flow cause losses in the system, which results in an efficiency of less than 100%. Acceptable values of the efficiency of the power supply are 80-95%, the engine 60-85% and the transmission 50-75%, resulting in a total efficiency of 50-75%.

The pressure produced by any pump can be divided into:

- Static head: the difference in water levels between the inner and outer water; and
- Dynamic head: the head needed to compensate for the local losses and friction.

The static head of a pumping station in a polder is the difference in water level of the polder (upstream) and the outer water (downstream). The water level in the polder near the pumping station is slightly lower than the polder water level, due to the discharge. The outer water level is not defined by its highest water level, because this would lead to unnecessary large capacities. Usually a frequently measured water level is taken as a standard. When this water level is exceeded, the pumping station can still discharge water, albeit with restricted capacity.

> Pumping station Y in polder X has four pumps, each with a capacity of 700 m^3/min and a head of 6.2 meter. The net power consumption is:
>
> $P_{netto} = Q \cdot h \cdot \rho \cdot g = (700/60) \cdot 6.2 \cdot 1000 \cdot 10 = 7.2 \cdot 10^5$ ($kg \cdot m^2 \cdot s^{-2}$) which is also ($J \cdot s^{-1}$) = 720 kWatt/pump
>
> When the efficiency is 75%, the gross consumption is 960 kWatt per pump.
>
> Therefore, the total power consumption of the four pumps is 3840 kWatt. With an energy price of €0.10/ kWh, the energy cost is € 384.00/hour. How many houses does this roughly correspond to?

The optimal site for the pumping station will be the lowest point of the polder. In this way the bed slope is used for gravity flow and deep watercourses are avoided as far as possible. When no slopes are present, the preferred siting for the outlet will be in the middle of the long side of a rectangle polder. However, other factors can also be taken into account when choosing the location of the outlet, such as the main wind direction, and the location of the outer water or belt canal. The compromise is often an economic balance between the location of the polder outlet and minimizing the excavation costs by using existing watercourses and natural flow paths. Examples of several possibilities of discharge from a polder that has three sections are given in Figure 12-1.

Solution A has weirs which will lead the water to the lowest section, where it is discharged with the capacity: $Q1h_1 + Q2h_1 + Q_3h_1$. Only one pump and one engine are needed. This alternative is simplest for operational management. However the capacity needed is large, because all the water from the two higher sections is first allowed to flow to the lowest section, from where it has to be pumped back up.

Solution B has separate pumping stations for each of the three sections. The total capacity is minimal: $Q_1h_1 + Q_2h_2 + Q_3h_3$. However, three pumping stations are expensive and a high belt canal needs to be available for the middle section.

Solution C needs two pumping stations and a high belt canal for the middle section. The capacity $Q_1h_1 + Q_2h_2 + Q_3h_2$ is larger than for solution B.

Solution D pumps the water from the lowest section to the middle section and discharges the water by gravity from the highest section to the middle section. The water is pumped from the middle section to the belt canal. The capacity needed is the same as for solution C. Two pumping stations are needed, but these can sometimes be combined in one building.

Solution E pumps the water step-by-step to the highest section and is discharged there. The capacity is similar to that of solution B and there are also three pumping stations needed, but the belt canal can be much lower.

High belt canals can often be avoided when this borders onto the outer water and pumping stations can be combined. An example of this is the Colijn pumping station in Flevoland, which separately pumps two sections with different water levels.

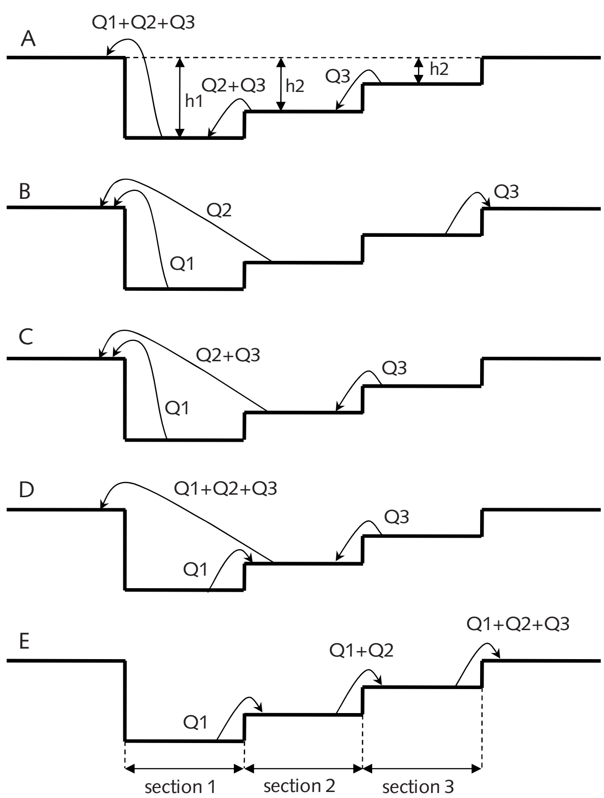


Figure 12-1 Examples of polders with three sections

12.2 Pumping stations

Of the pumping stations presently in the Netherlands, steam-driven pumping stations are the oldest. The first steam-driven pumping station in the Netherlands was built in 1844 for the Nootdorp test polder. Until then windmills were the only devices used, and they could only pump when the wind speed was between 6 and 11 meter per second. This restriction made it necessary to have 10% surface water storage in the polder. The steam-driven pumping station reduced this storage to 4%. However these stations needed large investments for the boiler, the heavy foundations and the chimney. In addition, the fuel was bulky and difficult to transport.

Although the steam-driven pumping stations are extremely efficient, they are no longer suitable for modern water management. Heating up the boiler is a very time-consuming job which requires a lot of extra fuel. Steam-driven pumping stations are only suitable for continuous pumping over longer periods of time. The biggest drawback of the steam-driven pumping stations is the long period of being out of use whilst repairs and inspections are taking place. A well-known working steam-driven pumping station is the. D.F. Wouda pumping station in Tacozijl (Friesland); this pumping station was coal-powered until 1967, since when oil has been used to heat the boiler.

Nowadays most pumping stations are driven by electricity. Some stations are driven by diesel engines, but more and more of these are being converted to electricity. An electric pumping station is simple in operation, smaller in size, and cheaper to build than a diesel station. Moreover, an electric pumping station is also easier to operate with a laptop or PDA from any location 24 hours a day.

Electricity charges can be minimized by using cheaper night rates. However, if pumping is limited to the night time then the capacity has to be increased to compensate for the reduction in the amount of time available. The surface water storage has to be increased in order to prevent water levels from exceeding the maximum.

A diesel engine-driven pumping station is more expensive in terms of construction and its operation is also less straight forward. Nevertheless, fuel is relatively cheap and it can operate continuously. Overload can easily be handled, although this reduces the efficiency. When all costs are added up (maintenance, operation, fuel, electricity, grease, etc.), a diesel engine pumping station can be more expensive than an electrical pumping station, but the opposite can also be true, depending on the price of fuel, electricity and the length of cables needed to connect the pumping station.

In large polders, pumping stations with different energy sources are installed to ensure pumping capacity at all times. For example: Flevoland has the Colijn and Lovink electric pumping stations and the de Blocq and Wortman diesel-driven pumping stations. The diesel engines at the De Blocq pumping station are currently being changed to electric motors.

12.3 Water-lifting devices

Two categories of devices can be distinguished to lift water:

- Devices that can transport water from one open water surface to another open water surface, without any confined flow in a pipe in between; e.g. a waterwheel or an Archimedean screw;
- Devices within which water flows through a pipe from one to another open water surface; e.g. centrifugal, screw pumps etc.

The oldest water-lifting machine is the waterwheel; a large vertical turbine that lifts the water between walls from the rear waterway to the waterway ahead (Figure 12-2 and Figure 12-3). The depth of the turbine blades in the water (in Dutch: *tasting*) determines the efficiency of the waterwheel. This makes the waterwheel unsuitable for water bodies with fluctuating water levels. The waterwheel is a slow spinning device, which makes it difficult to be combined with modern fast pumping stations. When water levels in the outer water are too high, the water will flow back, so a shutter is placed to prevent this from happening. Waterwheels were used in steam-driven pumping stations and are still used today in slow spinning mills. The maximum discharge head is 2m.

Figure 12-2 Aaerial photo of a belt canal pumping station in Spaarndam (www.rijnland.net)

Figure 12-3 Waterwheel insideSpaarndam pumping station (www.rijnland.net)

Archimedean screw

Archimedean screws (*vijzelgemaal*) have been used in polders since 1650. The construction consists of a rotating shaft (5-50 rev/min) with usually 3 flights in a 26º to 40º sloping concrete or steel trough, with a minimal gap of only a few millimeters between the flights and the trough.

The largest screws have a diameter of 4.8 meters and a static head of 9.5 meter and a flow rate of 11 m^3/sec. The efficiency η = 60-75 %. The Q-H curve of an Archimedean screw is almost independent of the static head. The most favorable performance occurs at a lower water level equal to or just above the filling point, and higher water level which is 0.1 á 0.15 D below the intersection point of the shaft and the end of the flights.

The classic trough of an Archimedean screw is made from concrete. For the proper closing of the mortar in a concrete trough, the screw is first constructed in a somewhat too wide concrete gutter. Then, fine mortar is deposited at the bottom of the screw and the engine started. The screw lifts the mortar and finishes the trough with a smooth surface. The barrel mill (*tonmolen*) is a variant in which the trough rotates as a tube fixed to the flights.

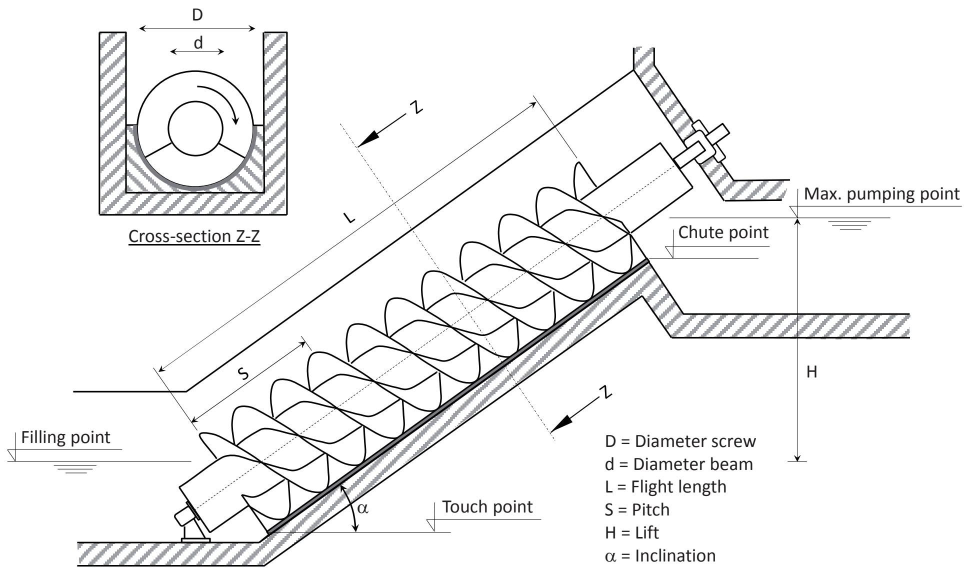


Figure 12-4 Cross section of an Archimedean screw pumping station

The discharge of an Archimedean Screw can be estimated with:

$Q = \beta \cdot n \cdot D^3$

In which:
Q = discharge (m^3/min)
β = factor
n = revolutions per minute ($n_{max} = 50/D^{2/3}$)
D = diameter screw (m)

The factor β depends on the inclination α of the beam. The steeper the inclination, the smaller the volume that can be scooped. For an Archimedean Screw with three flights the following B's give a good first approximation of the discharge:

Inclination α	β-factor
22°	0.36
26°	0.34
30°	0.31
35°	0.25
38°	0.22

Compared to other pumps, Archimedean screws have several advantages and disadvantages:

- \+ An Archimedean screw has a constant discharge independent from a high water level at the lower side or a low water level at the higher side; however
- – Water might flow back with high water levels at the higher side (therefore some screws have an adjustable slope to prevent this), and the screw hardly scoops any water with low water levels at the lower side;
- \+ The construction is simple, directly accessible, not sensitive for any debris, and the pump will not break if there is no water;
- – Maximum head is limited to 10 meter (though, you could put them in series);

Impeller pumps

These pumps add energy by means of an impeller: at first in the form of a speed increase that subsequently is converted into an increase in pressure. Depending on the shape of the impeller and the pump housing, the flow can be radial , axial, or something in between. This means that impeller pumps can be subdivided in centrifugal pumps, axial flow pumps or mixed flow pump respectively.

In order to work properly, a non-return shutter must be built on the discharge side to prevent water from flowing back when the pump is turned off. Furthermore, these pumps are not self-priming, therefore the pump should be either placed under water , or the air has to be sucked out of the pump housing with a vacuum pump prior to starting the pump itself.

Axial flow pumps

These pumps boost the water from the impeller in the axial direction. The position of the impeller vanes of an axial flow pump are comparable with the impeller of an outboard motor. The pump housing is very simple, namely, not much more than a piece of tube in which the impeller rotates. Due to the limited curvature of the impeller blades, the maximum head is quite low, but the possibility exists to handle high flow rates (H_{max} = 8 to 9 m and Q_{max} = 20 m^3/s). This pump has a very steep Q-H and Q-P characteristics. Without water, the pump bearings run hot, and also pump operation against a closed valve is not permissible. The efficiency η = 75-85 %. Technically it is possible to provide the pump impeller with adjustable vanes. However, this is quite expensive, and therefore only used on pumps with large capacities that discharge water to the sea with a fluctuating water level. The maximum angular displacement is usually between -6% and + 6%. By rotating the angle 1°, the flow improves and Q changes by approximately 6%.

Centrifugal pumps

The impeller of a centrifugal pump consists of vanes on a back plate. In centrifugal pumps, water enters the pump housing in the middle, and is then caught by the vanes and thrown out. During the passage of the impeller, the pressure is boosted. The maximum head is 70 to 80 meters and the max flow rate is 6 m^3/sec. For large pumps a maximum efficiency of η = 85 % is possible. When preparing the placement of centrifugal pumps in a pumping station, one has the choice between a vertical and horizontal axis. Local circumstances will determine which configuration is preferable. The application of additional bends should be avoided. With a vertical axis, the pump housing may be placed under water. This has the advantage that the pump housing doesn't need to be primed, which would require quite some time with large pumps. This is at the expense of one or two extra turns and means that the pump is less accessible for maintenance .

Mixed flow pumps

Mixed flow pumps have an impeller between the previous two pumps. The maximum head is around 25 meters and the max flow rate is approximately 8 m^3/sec. For large pumps, a maximum efficiency of η = 85 % is possible. Adjustable vanes are also possible. A special application of mixed flow pumps are deep well pumps, in which a series of impellers are mounted on an axle powered by an electric motor. These pumps can discharge water out of wells up to just 10 cm in diameter.

Figure 12-5 Old impeller of the Zaan pumping station in Zaandam with a maximum capacity of 750 m^3 per minute. The present optimized impeller in the same pump housing delivers 1200 m^3 per minute

12.4 System characteristic

A pump has to be able to lift a discharge Q over a height H, as was already explained in paragraph 12.1. The height $H = H_s + \Delta H_f + \Delta H_v = H_s + f\frac{L}{D}\frac{u^2}{2g} + \sum \xi \frac{u^2}{2g}$

When:

H	= total head	(m)
H_s	= static head	(m)
ΔH_w	= friction losses	(m)
ΔH_v	= local losses	(m)
f	= Darcy-Weisbach friction coefficient	(-)
L	= pipe length	(m)
D	= pipe diameter (or 4R for non-circular)	(m)
u	= average velocity	(m/s)
g	= gravitation constant	(m/s^2)
$\Sigma\xi$	= total of local losses (entrance, exit, bends etc)	(-)

In which f is a non-dimensional friction coefficient, between 0.01 (smooth) and 0.1 (rough). This coefficient can be determined using Colebrook and White's equation, which reads for circular pipes as follows:

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{k}{3.7D} + \frac{2.51}{\mathrm{Re}\sqrt{f}}\right] \qquad \text{(Equation 2)}$$

When:

f	= Darcy-Weisbach friction coefficient	(-)
k	= roughness height	(m)
D	= pipe diameter	(m)
Re	= Reynolds number	(-)

Equation1 can be rewritten with the relation $u = Q/A = Q/(\frac{1}{4}\pi D^2)$ as:

$$H = H_s + \left(\frac{f \cdot L}{2g \cdot \left(\frac{\pi}{4}\right) \cdot D^5} + \frac{\sum \xi}{2g \cdot \left(\frac{\pi}{4}\right) \cdot D^4} \right) Q^2 \qquad \text{(Equation 3)}$$

This equation is called the system characteristic. The length of the pipe is limited to about 50 meters in a usual configuration of a polder or belt canal. The local losses are limited to the entrance loss and the exit loss. Some right angles (90°), combined angles (2*45°, 3*30°, 4*22.5°) or bends can also be used to change the direction. To prevent large local losses the direction should be changed gradually.

The entrance loss occurs when the pipe opening is insufficiently round, which causes contraction. After the contraction the stream widens; this reduces the velocity and energy loss occurs. If the contraction coefficient μ is known, the loss coefficient ξ can be calculated. For practical situations the values of *Figure 12-6* can be applied.

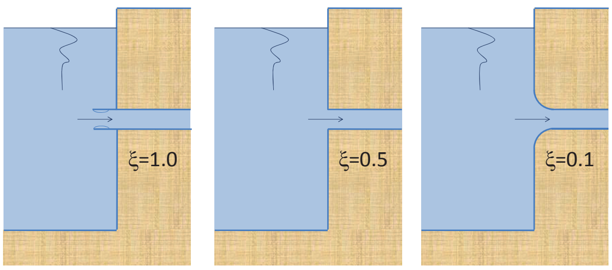


Figure 12-6 Entrance losses

If the discharge flows into a reservoir then the complete velocity head is lost, and the exit loss coefficient ξ = 1. This loss can be reduced by gradually widening the cross section before the exit. In order to limit the velocity and avoid a large dynamic head, a sufficiently large cross section should be selected.

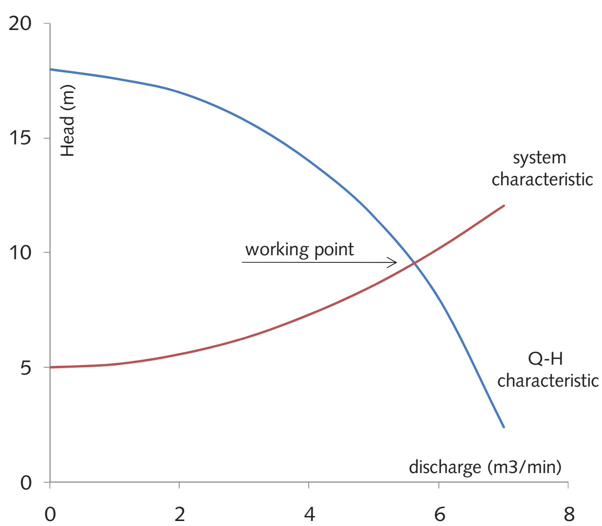


Figure 12-7 The intersection between the system characteristic and the pumps Q-H characteristic is the pump's working point

12.5 Pump characteristics

The hydraulic properties of a pump are contained in pump characteristics:

1. The Q-H characteristic gives the relation between the discharge Q and the head H (see Figure 12-7*).* The intersection gives the working point of the pump;
2. The Q-η characteristic describes the relation between the discharge Q and the hydraulic efficiency η. The hydraulic efficiency is the ratio between the energy captured by the water, and the energy delivered to the axes by the engine;
3. The Q-P characteristic gives the relation between the discharge Q, and the power P supplied to the pump axes;
4. The NPSH characteristic describes the relation between the discharge Q and the necessary margin between the energy head at the suction side of the pump and the vapour level. This margin is necessary to avoid cavitation.

Q-H characteristic
The Q-H characteristic can be determined by installing a flow meter, an adjustable shut-off valve and pressure gauges just before and after the pump. Now, the flow can be squeezed by closing the valves, so the Q-H curve be composed by reading off the pressures at different flow rates.

Hydraulic efficiency
A pump transforms mechanical energy into hydraulic energy at the expense of energy loss. Because of this loss the hydraulic efficiency is:

$\eta = P_{eff}/P_{as}$

When:

η	= hydraulic efficiency	[-]
P_{eff}	= hydraulic power ρQgH	[Watt]
P_{as}	= gross power	[Watt]

For each pump there is only one combination of Q and H for which the total loss is minimal per pump speed. In other words, there is only one working point at which the pump will work at maximal hydraulic efficiency.

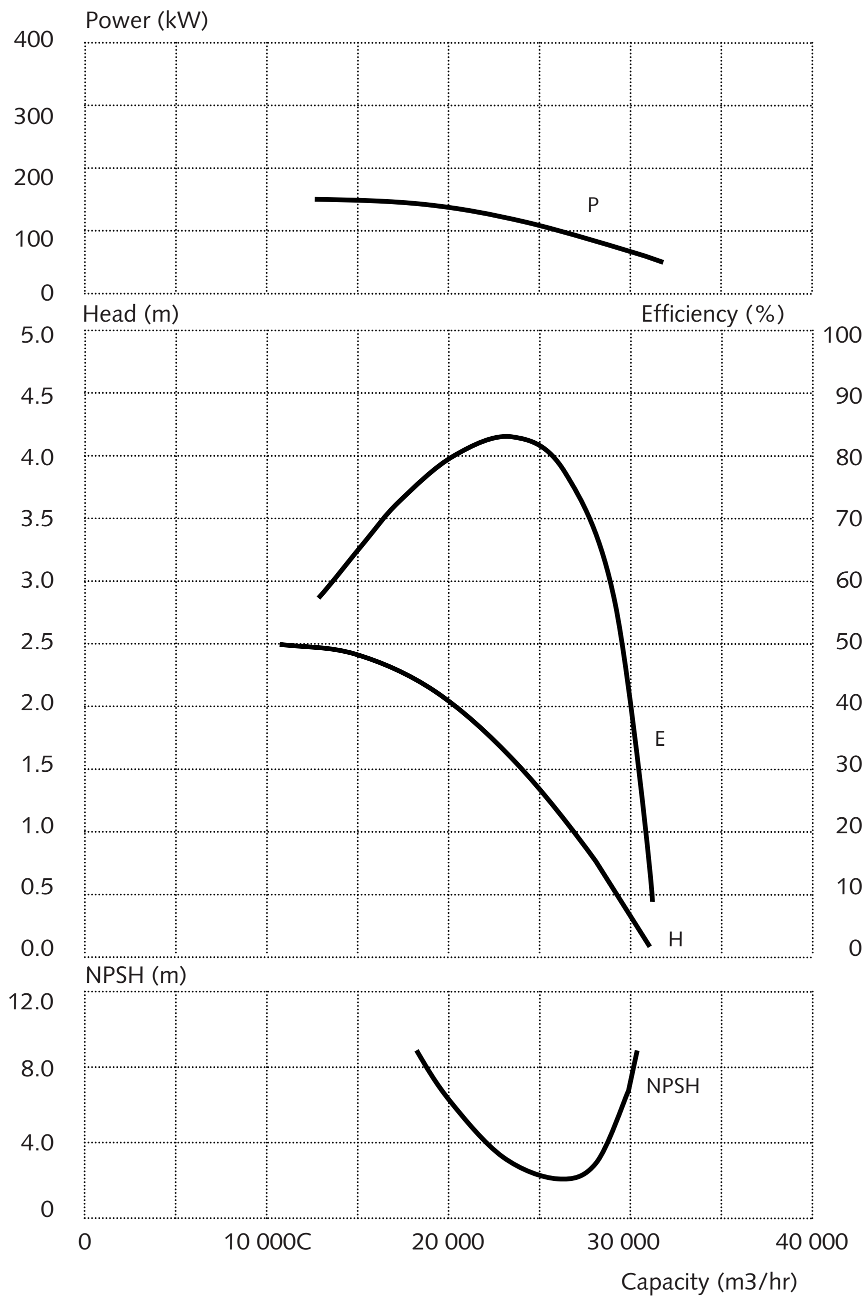
The NPSH characteristic
Underpressure occurs at the suction side of a pumping station. However this underpressure cannot become lower than the vapour pressure of water, otherwise vapour-filled bubbles form at the suction side. Next, these bubbles are pulled into the pump, and implode at the delivery side. This phenomenon is called cavitation and causes wear and dimples at the impeller's surface, as high pressure occurs when the bubbles implode.

The NPSH curve (Net Positive Suction Head) gives the relation between the flow rate Q and the necessary margin between the energy head at the suction side of the pump and vapour pressure of water.

$$NPSH = h_0 + H - \Delta H - h_v - h_s$$

NPSH	= Net Positive Suction Head	[m]
h_0	= atmospheric pressure (10.3 m at MSL and 20°C)	[m]
H	= distance between pump axis to the water level	[m]
ΔH	= friction and local losses in the suction pipe	[m]
h_v	= vapour pressure (0. 4 m at sea level at 20°C)	[m]
h_s	= safety margin	[m]

H can either be positive if the pump axis is below the water level, or negative if above the water level.



Stork vertical pump OVOP 185-V, 127 rev/min

Discharge:	292 / 417 m3/min	Fan diameter:	1840 mm
Static head:	0.60 / 1.16 m	Blade diameter:	1840 mm
Bowl head:	1.30 / 2.04 m	Entrance surface :	6590 cm^2
Number of revolutions	110 / 145 min^{-1}	Blade angle:	17.1 degrees
Capacity:	88 / 164 kW	Weight:	8900 kg

Figure 12-8 Pump characteristics and technical specification of the pumping station at Morsestraat, Den Haag

Specific speed of impeller pumps

The actual head and flow rate that can be generated with a pump depends on the diameter of the impeller, the rotation speed, the number, position and shape of the vanes. The interdependence between these can be expressed in the so called 'specific speed' with which the maximum efficiency can be calculated for each number of revolutions:

$$n_s = \frac{n\sqrt{Q_0}}{H_0^{3/4}}$$

Where:

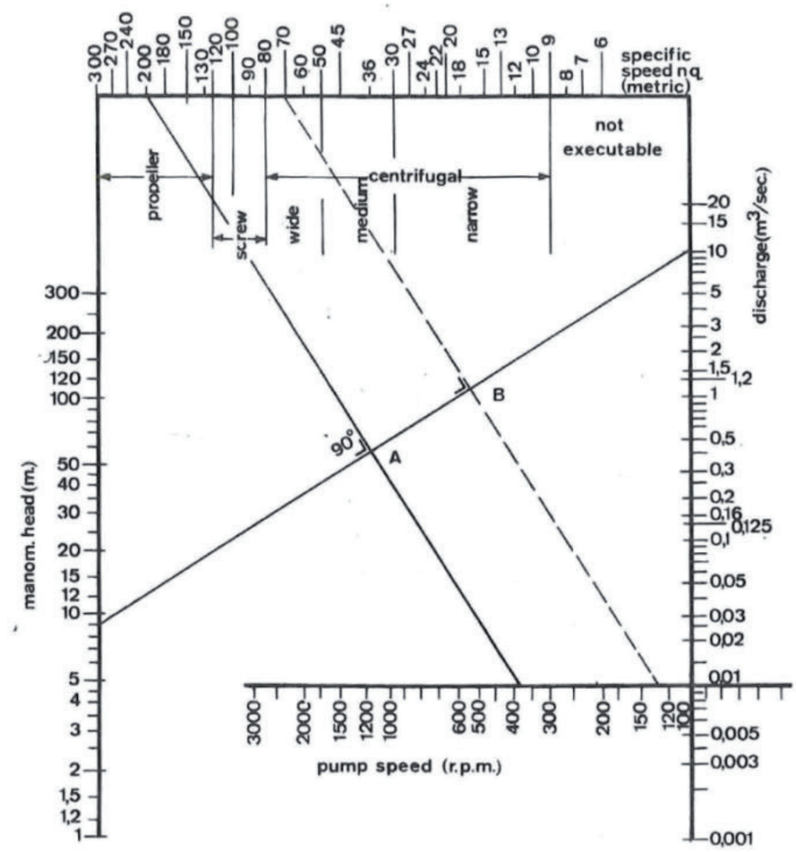
n_s	= specific speed	[$m^{3/4}/min \cdot s^{1/2}$]
n	= pump speed in revolutions	[min^{-1}]
Q_0	= rate of flow at optimal efficiency	[m3/s]
H_0	= bowl head H at optimal efficiency	[m]

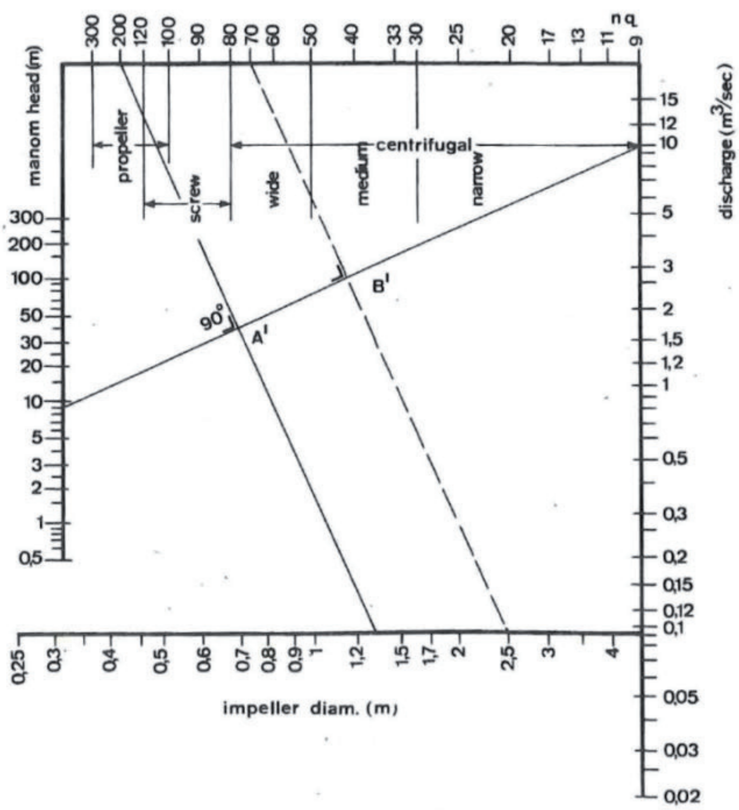
Practically, this means that an arbitrary pump, when set to supply a head of 1 m and rotating on a speed equal to the specific speed, will deliver a flow rate 1 m^3/s. This specific speed for centrifugal pumps is between 10 and 80 , for axial pumps between 120 and 300. Knowing the specific speed might feel like redundant information, however with special graphs, it becomes pretty simple to determine the necessary revolutions and impeller diameter of a pump given the desired H and Q of a pump.

Example: Assume I need a pump that delivers 10 m^3/s at a head of 9 m. Step one is to connect these points with a straight line in both graphs. Every line drawn perpendicular to the line that connect H and Q delivers a pump type with a specific speed, required pump speed and impeller diameter:

A) is an axial/propeller pump with a specific speed of 200. This pump needs a pump speed of 375 revolutions per minute (r.p.m.) and an impeller diameter of 1.35 m;

B) is a centrifugal pump with a specific speed of 70. The pump needs a pump speed of 130 r.p.m. and an impeller diameter of 2.5 m. This pump is running much slower than A). This means less wear, less risk of cavitation , a higher efficiency, but a more expensive pump due to the larger impeller diameter (which needs a larger pump housing, heavier pump, larger foundation etc).


specific speed nq (metric)
not executable
propeller
screw
centrifugal
wide
medium
narrow
discharge (m3/sec.)
manom. head (m.)
pump speed (r.p.m.)
90°
A
B


nq
manom head (m)
propeller
screw
centrifugal
wide
medium
narrow
discharge (m3/sec.)
impeller diam. (m)
90°
A'
B'

12.6 Additional remarks

One big pump or several smaller pumps?

When a new pumping station has to be built, the cheapest configuration will probably be one big pump and engine with variable speed. The disadvantage of this pump is that it is difficult to pump efficiently at different combinations of Q and H. Moreover, at times of maintenance or failure the whole pumping station is out of operation. For moderate polders two identical pump units are often applied each with half the necessary capacity, for the sake of easy maintenance and the storage of spare parts. Three or four pump units are only applied in large pumping stations, and in really large polders, several pumping stations per polder might even be necessary. In that case it might be interesting to divide the pumping stations among electrically and diesel driven motors. This allows you to pump even in case of a power failure, albeit with a limited capacity.

Vertical or horizontal axis?

A pump can be placed horizontally or vertically. In a vertical position the engine can be placed in a machine room, and the impeller under water in the basement. The advantage is that the pump is continuously submerged, and the bearings are evenly loaded. With a horizontal position above the water level a vacuum should be drawn on the pump casing.

Pump building

The primary requirement of a pump building is to provide a solid foundation for the motor, pump , the suction and delivery pipes, appendages and trash rack. A reinforced concrete base is practically always selected, as the pump basement and suction and delivery pipes are easier to pour with mortar than to make with stone masonry. In addition, the building needs to provide shelter to instruments, tools, spare parts, a small workshop, and auxiliary equipment such as an overhead crane.

Cavitation

Cavitation is the formation of vapor-filled bubbles as a result of a steep drop in pressure down to vapor pressure. The bubbles implode as they pass the impeller, and these implosions cause high pressures and ruin your impeller. Cavitation can be prevented by installing the pump close to the water surface such that the suction height is low or the NPSH is not exceeded <u>for all working points</u>. Further, it is recommended that the pump cellars and suction pipes are designed in such a way that a gradual acceleration of the water (without any sharp bends or kinks) to the pumps is assured.

Although there are a wide variety of pumping stations possible, some ground rules are:

- Choose a pump where the working points are as close to the maximal efficiency as possible;
- Ensure that the water velocity is either constant or accelerating at the pump's entrance to avoid any undesirable whirls;
- Try to minimize energy losses at the suction side by using short lengths and wide bends;
- Make the pumps entrance deep enough to prevent air from being sucked in;
- Construct shutters and doors to prevent water from flowing back (the suction-delivery pipe might work as a siphon!);
- The capacity of the pump should fit the capacity of the water-providing canal to prevent the pump from being switched on and off too frequently.
- The engines need to be protected against flooding by installing them (if possible) above the highest water level;
- The location should be chosen such that a connection to electrical grid is close by or where the supply of fuel is as favorable as possible;
- The pumping station must be accessible for the supply of heavy machinery;
- Also within the pumping station spacious aisles are necessary in order to bring large components in or out;
- The pumping station should be provided with facilities to isolate and empty the basement and suction and delivery lines for inspection, repair, or replacement of parts (practically: double grooves in the side walls in which beams can be lowered and clay deposited to make a water-tight seal, or in Dutch: schotbalksponning);
- When floating debris can be expected, the pumping station must be provided with a trash rack and an accessible location for the storage of floating debris;
- Piping under the foundation or along the side walls of the structure must be prevented by seepage screens (in Dutch achter- en onderloopsheid).

13 Water supply

13.1 Water deficit

The relation between drainage depth and yield can give the optimal yield for a certain drainage depth, which must be adhered to as closely as possible and which is mainly dependent on soil type and the crops cultivated. Excess precipitation, that increases the groundwater level (and decreases drainage depth), can reduce the yield. This situation usually occurs during the winter period and not during the growing season, because of high evapotranspiration.

Precipitation deficit can also lead to a reduction in yield and a deficit often does occur during the growing season. The actual yield loss depends on the drop in the groundwater table, soil type and crop. Soils such as clay, loam, peat and fine sand with humus can contain a lot of water and can usually supply sufficient water to crops. With deep clay and loam soils this can be as much as 250mm water per meter in the groundwater table (between the field capacity and wilting point). Deep-rooting crops such as wheat, Lucerne and beets can give a good yield on these types of soils despite a precipitation deficit. Shallow-rooting crops such as vegetables and grass do not get enough water from these soils during long dry periods and yields can be drastically reduced. However, during the growing season some precipitation occurs, preventing these crops from completely wilting and dying.

Sandy soils have considerably less water available between the field capacity and wilting point (approximately 130mm per meter drop in the groundwater table). These soils usually contain sufficient water for deep-rooting crops with short growth periods, such as summer grains. Deep-rooting crops with long growth periods and shallow-rooting crops are almost always hampered by a water deficit in these soils. Although grass has a long growth period and shallow roots, this does not have a direct effect on the yield, as the first yield is harvested in June/July and the water deficit is not very large then. Vegetables are considered to be crops with a long growth period, because of the sequence of crops cultivated with short growth periods.

The result of a water deficit can be seen in dry summers when crops are damaged. Good examples are the summers of 1976 and 2003. If we define the yield of wet summers, such as that of 2002, as 100%, then a normal year, such as 1997, has a yield of 80%; an extremely dry summer such as 2003 only 55%. The yield of these extremely dry summers could be increased if enough water was supplied. This extra water supply could also increase the yield in normal years.

Since a water deficit leads to more damage than a water surplus, the water supply is calculated from a water deficit with a smaller return period than that of the water surplus. In the Netherlands the frequency used for calculations is once every 25 years.

A water balance can be used to determine the need for water supply for several crops and soil types. Although there are many uncertainties in this type of research, it gives sufficient insight. Table 13-1 gives an overview of the maximum water supply for the middle and west of the Netherlands.

Table 13-1 Need for water supply

Nr.	Description	Maximum need [mm/day]
1	Large waterways	8
2	Small waterways	6
3	Horticulture:	
	- open soil	6
	- under glass	7
	- bulbs	4
	- orchards	3-4
4	Sandy soils:	
	- grass	4-6
	- arable land	3
	- forests, parks	2
5	Peat soils:	
	- grass	4
	- arable farming	3
6	Clay soils:	
	- grass	3
	- arable farming	0-1
7	Dunes, built-up areas	0

From Table 13-1 and other publications, the total water requirement for the Netherlands can be calculated for the unfavourable period of ten dry days during the growing season. The results of these calculations are given in Table 13-2. The standards will probably not be exactly the same for each of the separate areas. However, in spite of this flaw, the table gives a good overview of the volumes of water required for water supply during a ten-day period of drought in the growing season.

Table 13-2 Total water requirement in the Netherlands for agriculture over an unfavourable period of ten days during the growing season

	Water requirement [m^3/s]
North Holland, Friesland, Groningen and surroundings	300
Evaporation from Lake IJssel	50
Lake IJssel Polders and surroundings	50
Urban area in North and South Holland	130
East Brabant and Limburg	85
West Brabant and Zeeland	70
Remaining areas	70
Total	750

The water volume required has to be taken from the large rivers, the Rhine and the Meuse. The Rhine gives the largest discharge of the two. The lowest discharge of the Rhine at the 'inlet' of the Netherlands, Lobith, is 600m^3/s; the lowest inlet discharge of the Meuse measured at Lith is 25m^3/s. The discharges of the small rivers and brooks entering the Netherlands can be disregarded during the growing season.

The river Rhine discharges water from both snow melt and precipitation, which leads to generally constant but also rather high discharges during the growing season: 1000m³/s, which is enough to cover shortages. However, a problem can be encountered, as fresh water is required to flush belt canal systems because of pollution and salination. This means that in dry years there is often not enough water left over for the water supply. On the other hand, if the belt canal systems were not flushed, the water in the canals would be too polluted and salty to be used for the water supply.

Freshwater reservoirs are a solution for this problem. The largest freshwater reservoir in the Netherlands is Lake IJssel (180,000ha); if the water level of Lake IJssel were to be raised by 0.40m, a total volume of 720 million m³ could be stored. However, if during dry periods this were to be used at a rate of 750m³/s, this would only be enough for a period of less than two weeks. This illustrates that water shortage is also a potential problem during long dry spells in a country like the Netherlands.

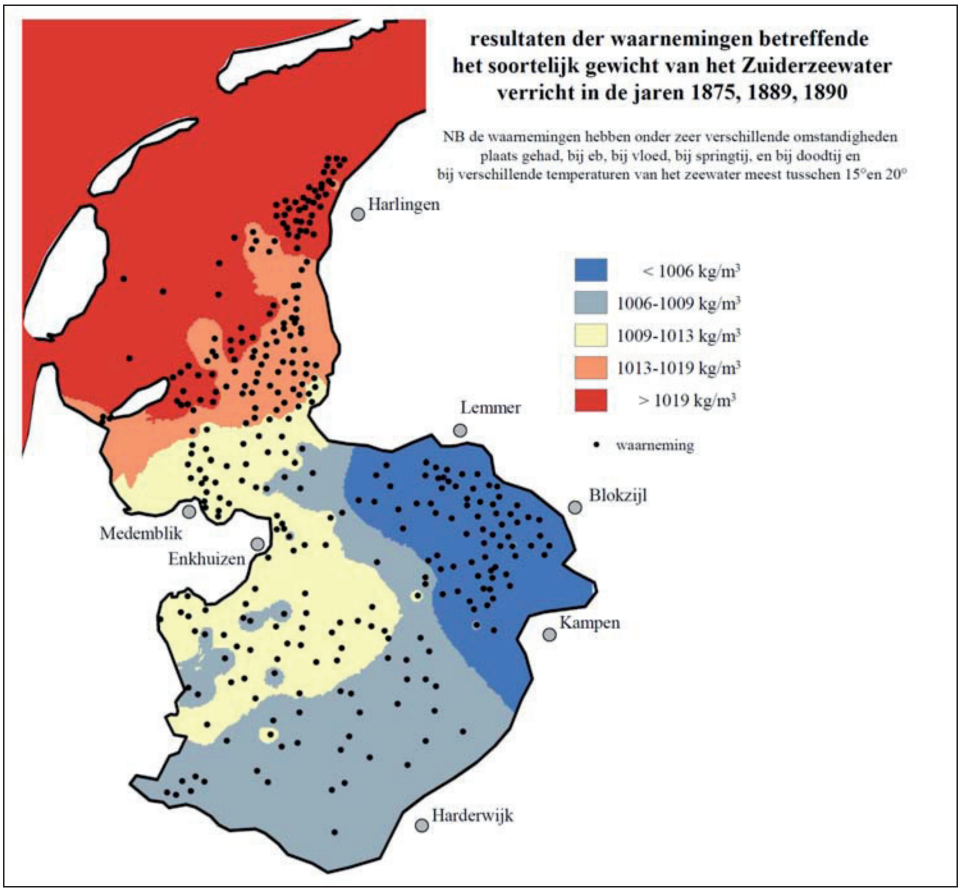


Figure 13-1 Density of water in what is nowadays Lake IJssel measured in 1875, 1889 and 1890 (source: redrawn after a map in the Civil Engineering Library-ITB, Bandung, Indonesia).

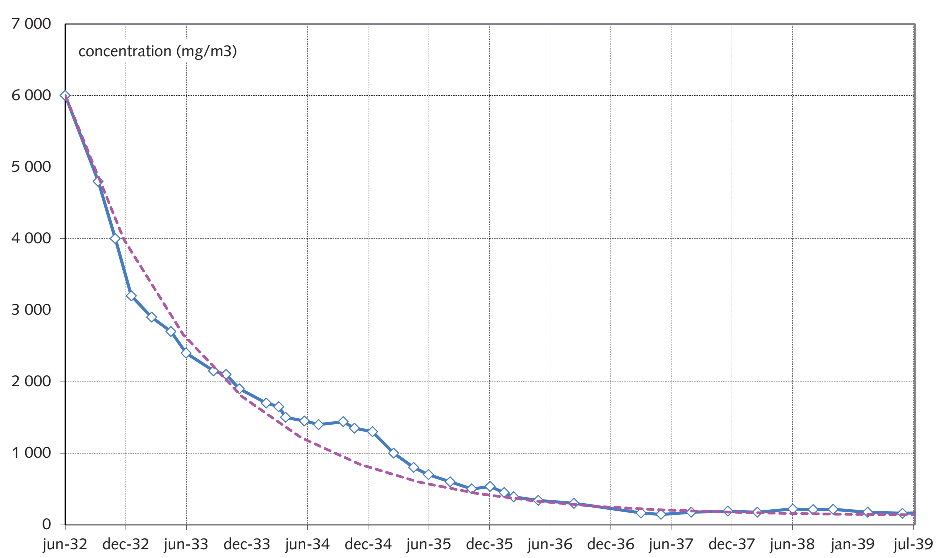
The chloride concentration in lake IJssel just after the construction of the closure dam

The huge fresh water lake (called lake IJssel or IJsselmeer in Dutch) in the center of the Netherlands used to be the South Sea (in Dutch Zuiderzee) prior to the construction of of the closure dam in 1932. In those days this South Sea was in open connection through the Wadden Sea with the North Sea. Nevertheless, this South Sea was never as salty as the North Sea. The North Sea contains approximately 34 grams of salt per liter. In the South Sea, this was approximately 16 grams per liter as the river IJssel was continuously diluting the South Sea with roughly 400 m^3/s of fresh water.

From the start, it was clear that Lake IJssel had to play an important role as a reservoir for the fresh water supply of the Western part of the Netherlands.This means for a) drinking water, b) water level control and flushing of surface water of polders around Lake IJssel. During the construction of the closure dam, the in- and outflow of water became gradually more and more difficult, and thus at the day at which the final part was closed (29 May 1932) the chlorides concentration had dropped to 6000 mg per liter. Currently, the chlorides concentration of the water in lake IJssel is about 100 mg per liter . The drinking water standard is 200 mg chlorides per liter. But you may have noticed, salt and chloride are used interchangeably. They are however related, but not the same!

Chlorides are a salt , but not all salt is composed of chlorides. As mentioned, seawater in the North Sea contains about 34 grams of salt per liter. This salt is composed of 24 grams per liter of sodium chloride (NaCl), 5 grams of magnesium chloride ($MgCl_2$), and 4 grams of sodium sulfate (Na_2SO_4), and 1 gram of some other salts. By dissolving these salts, they break up ions. The same sea water then contains about 19 grams per liter of chloride ions, and 11 grams per liter of sodium ions. Dissolved in water, the density is just 1,025 g per liter. As when the salt dissolves, the mass increases more than the volume. So there is 1.8 times (34/19) more salt in seawater, than that there are chloride ions. It is quite laborious to measure the chloride content. Therefore, the chloride content is practically almost always indirectly measured via the conductivity of the water, with a so-called EC-meter.

And now, how quickly did the chloride concentration in Lake IJssel drop after 1932 to drinking water level? To monitor the decrease the Dienst Zuiderzee Werken (which was a special governmental division in charge of the construction works) took samples along a fixed route in Lake IJssel at a regular interval. These measurements were published in their quarterly journal.



The reduction in chlorides from 6 000 to 200 mg/l could be estimated in advance with:

$$C(t) = C_o \cdot \exp\left(\left[k - \frac{Q}{V}\right]t\right) - C_{in} \cdot \frac{Q}{V} \cdot \frac{1 - \exp\left(\left[k - \frac{Q}{V}\right]t\right)}{k - \frac{Q}{V}}$$

In which:

$C(t)$	= concentration at time t =t	(g/m^3)
C_0	= concentration at t = 0	(g/m^3)
C_{in}	= concentration of incoming water	(g/m^3)
k	= decay or growth factor	(day^{-1})
Q	= discharge	(m^3/day)
V	= volume of the lake	(m^3)
t	= time step	(day)

This formula assumes perfect mixing over the lake! Furthermore this formula only has a solution if V·k < Q. For V·k ≥ Q, the concentration in time C(t) goes to infinity. The dashed line above was constructed with Lake IJssel data. The area is $3.4 \cdot 10^9$ m^2 (or 340 000 hectare), the average depth is 3.5 meter. The volume then becomes $11.9 \cdot 10^9$ m^3. The concentration at t=0 was,as stated,about 6 000 g/m^3. The river IJssel discharges 400 m^3/s with a concentration of 100 g/m^3.

But what does the decay or growth factor do in the formula? Salt surely doesn't reproduce! Well, without the River IJssel flushing Lake IJssel the water level would drop and the concentration would grow due to evaporation of water. Now, if we assume an annual average evaporation E of surface water in the Netherlands of 2 mm per day. This makes the growth factor *k* equal to $(E \cdot A)/V = 0.002 \cdot 3.4 \cdot 10^9 / 11.9 \cdot 10^9 = 0.000571$ day^{-1}. With these numbers the drop in concentration becomes below 200 mg per liter after 5 years and 42 days.

13.2 Water quality

Water quality is mainly determined by the pollution and salinization of a water body. Pollution is caused by non-purified discharges, such as combined sewage system overflows (E-coli, Enterococces etc.) and drainage water (flushing of nitrogen and phosphate). In order to prevent this pollution, locations with sewage overflows can be cleaned, sedimentation basins constructed and legislation controlling fertilizer deposition can be passed.

Water quality problems that occur are blue green algae, botulism and a lack of oxygen. In order to prevent or minimize these situations in the belt canal systems, they need to be flushed. Large volumes of water are required for this flushing. This water is taken from the large rivers and fresh water reservoirs such as Lake IJssel, Lake Marken and the lakes in Zeeland.

Salinization of belt canal water occurs mainly when salt water enters via the shipping locks to the North Sea and from seepage into polders close to the sea. Wells and gas wells can also cause saltwater to seep through the impermeable layers. The Haarlemmermeer polder, Tempel polder, Noordplas polder and Nieuwkoop polder are all well known for salt seepage.

Table 13-3 Suitability of water with various chloride concentrations

Chloride concentration [mg/l]	Suitability
<300	Suitable for all agricultural uses and drinking water for humans.
300-500	Can cause yield loss when used for water supply for cultivation under glass.
500-1,000	Not suitable for water supply for vegetables and questionable for use as sprinkler water in disease prevention for fruits.
1,000-2,000	Questionable suitability for infiltration water and sprinkler water on arable land. Probably still suitable for disease prevention and drinking water for cattle.
2,000-5,000	When clear still suitable as drinking water for cattle.
>5,000	Unsuitable for all crop cultivation.

The large rivers have an increasing salt water intrusion, due to the deepening of the rivers for shipping. Water inlet in the western part of the Netherlands is becoming more and more difficult because of this. However, salinization can be prevented by flushing with fresh water. Nevertheless, at locks it is impossible to prevent and the key here is to minimize salt water intrusion. The volumes of water required to control salinization are large; in general this is 1,000m³/s during short periods, which is equal to the lowest discharges of the Rhine in the growing season.

Illustration

The Helsdeur pumping station in Den Helder discharges water from the Schermer belt canal system. In the past this pumping station discharged between 300, 000 and 350,000 m³ on a daily basis in order to keep the chloride concentration at an acceptable level. When the chloride concentrations rose to over 1,000mg/l on the bed or 600mg/l on the surface of the Noordhollandskanaal near the Kooy, the discharge was set to at least 500,000m³ a day.

Near the Zaangemaal pumping station at the southern end of the 'Noordhollandskanaal' channel the oxygen levels were also measured continuously. In order to maintain a particular oxygen level the following volumes were removed at the southern pumping station:

O_2 concentration	*Discharge at the Zaangemaal*
> 10 mg/l	For two days none, on the third day 250,000 m³
> 7 mg/l	For one day none, on the second day 250,000 m³
> 5 mg/l	Every day 250,000 m³
> 4 mg/l	Every day 500,000 m³
> 3 mg/l	Every day 1,000,000 m³
< 3 mg/l	Continuously with one pump

This discharge had to be simultaneous with the discharges at Helsdeur, in order to prevent a flow to the south in the 'Noordhollandskanaal' channel. The lake near Alkmaar is not allowed to receive water from the wastewater treatment plant (WWTP) in Heiloo.

(In recent years the water board is trying to decrease the above mentioned discharges!)

13.3 Water supply

The large rivers, Lake IJssel and Lake Zeeland are available for the supply of fresh water. The discharges from the brooks are too small to contribute to this supply, and are used locally when available. Polders adjacent to the large rivers can let water in from those rivers; polders adjacent to belt canal systems use these canals for water supply. This belt canal system lets water in from freshwater reservoirs or the rivers. Often water has to travel over a long distance to get to the polder where the water supply is required. In example: the Hollandse IJssel river is the fresh water inlet for the belt canal systems in Rijnland, Delfland and Schieland. In the province of Friesland the belt canal takes its necessary water from Lake IJssel.

For artificial water supply one could use surface irrigation, spray treatment, trench irrigation, infiltration and sprinkler irrigation. Water supply by surface irrigation is very uncommon in the Netherlands. Horticulture is a very intensive type of cultivation and both costs and profits are high; here spray treatments are common. When soils have a high conductivity, the water level can easily be controlled by a dense network of trenches and ditches with a well-maintained target level. The trenches are also used for infiltration of water.

Infiltration is often used in soils with high conductivity where drains are flooded. This will lead to the desired groundwater level close to the drains, but between two drains the water level will drop, depending on the water use and water flow through the soil.

The water level in the ditches and canals can be raised to the desired level using weirs and shutters. In the Noordoost Polder (Northern Flevoland) this is done using two systems: the Vollenhoven system (Figure 13-2) and the Ramspol system (Figure 13-3) where drains discharge into a collector drain on both ends. In the Ramspol system the water flows to the drains from a higher 'wet' ditch, through a shutter. Discharge of the excess water is carried out through a collector drain and a shutter to the so-called 'dry' ditch.

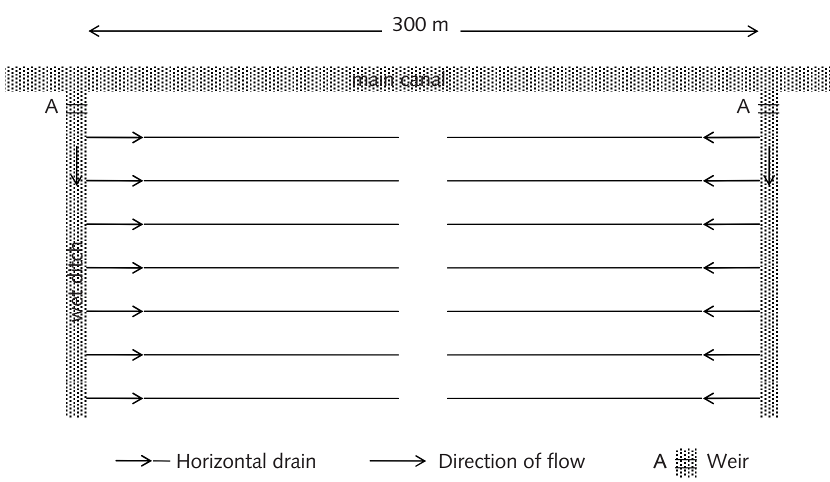


Figure 13-2 Infiltration by means of the Vollenhoven system

When a well permeable soil is sufficiently inflexible, a combined system of drain infiltration and mole tunnels is implemented. The mole tunnels are put in at a short distance from each other perpendicular to the drains just above. Although a mole tunnel usually collapses within a few years, the implementation of the combined system is much cheaper, because a lot less drains are required. The drain distance is now purely based on the user's drainage demands (Figure 13-4).

Sprinkler irrigation is carried out using sprinkler installations. The water is pumped using a system of pipes to a number of sprinklers that can either be fixed or rotating. Some systems are fixed, but others are movable. The volume of water required for sprinkler irrigation is smaller than the volume required for infiltration. Sprinkler irrigation can be successful on sand soils with limited water-retaining capacity, sandy soils with irregular elevation or well-retaining soils where shallow rooting crops are cultivated. The water is usually drained from wells or pumped from ditches.

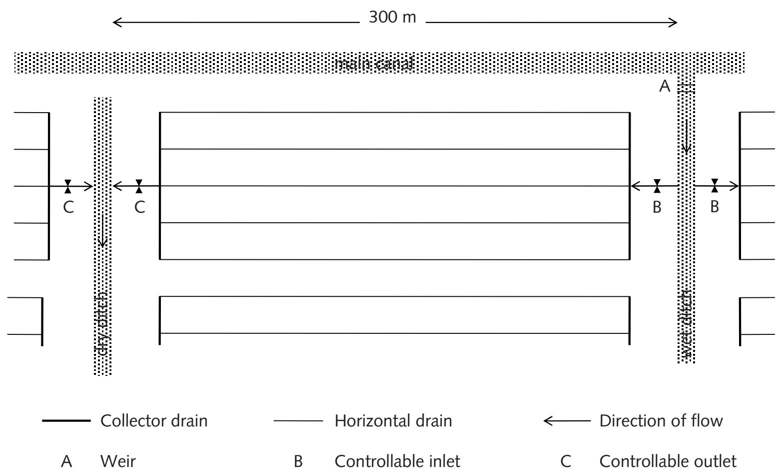


Figure 13-3 Infiltration by means of the Ramspol system

The timing of irrigation depends on the water content of the soil, weather forecasts and the crop being cultivated. When irrigation is applied, the water content of the soil is brought up to field capacity. Research has shown that for some crops there are definite periods where irrigation can lead to a large increase in yield; other crops like grass and potatoes require a constant wet soil.

The efficiency of water supply is never 100%. The water can evaporate before it reaches the ground or before it infiltrates. Crops also tend to use more water when irrigation is applied.

Soils that are infiltrated or irrigated by sprinklers have a highly reduced storage capacity, because groundwater levels are elevated during summer as well as winter. During a rainfall event the groundwater level will rise immediately, because the soil just above the groundwater table is already at field capacity. For these soils it is necessary to calculate the discharge over the precipitation of the whole year, because rain events during the summer will also discharge into the surface water. The pumping station in such an area will require a larger capacity.

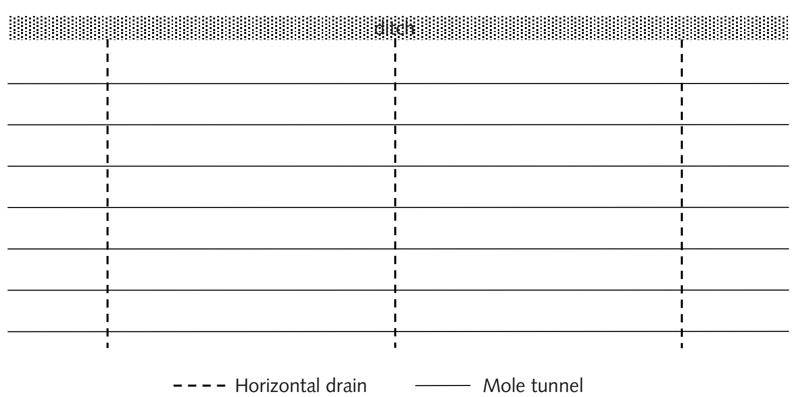


Figure 13-4 Infiltration by means of a combined system of drains and mole tunnels

13.4 The Dutch national close-off sequence

In times of water scarcity the Water Boards adjacent to Lake IJssel and Lake Marken can take in water using their own sluices, locks and siphons. In order to reach a balance on what this water is used for, an agreement is reached which also prescribes the maximum capacity at which water can be taken in.

When these agreements cannot be adhered to because of extreme scarcity, the national close-off sequence stipulates how the available water must be divided (Table 13-4). Categories 1 and 2 have a priority sequence. Categories 3 and 4 are prioritized in order to minimalise the economic and social damage.

The total water demand is 350m³/s from Lake IJssel and Lake Marken, from which a large portion is the evaporation of the surface water itself (Lake IJssel 1100km² and Lake Marken 700km² * evaporation 5mm/day=104m³/s). The regional implementation is carried out by a study group of the Department of Transport and Public Works. These tables will be the key for the division of fresh water. Managers will have their demand for water cut back *pro rata*.

The theoretical net inlet requirement for agriculture is – depending on the crop cultivated - approximately a gross crop water requirement of a few mm/day minus the precipitation and upward seepage. This water is required for the replenishment of the soil moisture in the rooting zone. In dry periods water is also required for water level management and flushing of the surface water.

In practice more water is needed than the net inlet requirement, because irrigation is not carried out efficiently: irrigation water flows into cracks and disappears from the rooting zone, hysteresis needs to be overcome and farmers tend to apply more water than needed. On the other hand water inlet will be less than needed, because of restrictions on the capacity and possibilities of irrigation systems.

Table 13-4 National close-off sequence in the Netherlands

Nr	Category	Discharge [m³/s]
1	***Safety and prevention of irreversible damage***	
1.1	Maintaining water level in main system regarding safety dikes	
1.2	Maintaining water level in peat areas regarding settlement	
1.3	Maintaining water level regarding fragile nature reserves	
2	***Public utilities***	
2.1	Withdrawal and flushing regarding drinking water facilities	
2.2	Flushing regarding cooling water of electricity plants	
3	***Local high-quality uses***	
3.1	Flushing regarding public health (botulism, blue-green algae)	
3.2	Withdrawal process water industry	
3.3	Flushing surface water from which process water is withdrawn	
3.4	Irrigation of the high investment crop cultivation (described in 4.1)	
4	***Other functions***	
4.1	Irrigation of cultivated grounds, sport fields and greens	
4.2	Flushing regarding irrigation of cultivated grounds	
4.3	Maintaining water level in clay and sand soils	
4.4	Maintaining water level and flushing of non-fragile nature reserves	
4.5	Irrigation of grass/corn	
4.6	Raising and maintaining water level in peat districts	
4.7	Discharge for fish migration	
4.8	Flushing preventing botulism and blue-green algae	
4.9	Flushing electricity plant Noord-Bergum and salt intrusion North Sea channel	
4.10	Economising with lockage	
	TOTAL	

14 Economic evaluation of measures

A cost-benefit analysis is often applied to evaluate how worthwhile an improvement measure in a water system would be. This approach was first used for the determination of the height of the Dutch Delta. In a cost-benefit analysis, the benefits are an increase in prosperity and the costs are a decrease in prosperity. The foundation of modern economic theory is that both society and the individual try to maximize their own welfare, within certain boundaries of scarcity and conflicts between needs and desires. According to the Pareto principle (Jackson, 1992) an investment is acceptable when the welfare of at least one individual is increased and no one's welfare is decreased. A maximum is reached when it is no longer possible to improve the welfare of one without harming someone else's welfare. However, the reality is that when a product is redistributed some individuals will gain slightly and others will lose. Each individual's loss is a loss for society of which the individual is part. On the other hand, society has also gained because some of its individuals have gained from the redistribution. The Hicks-Caldor principle (Jackson, 1992) states that society gains when the increases in welfare (benefits) are great enough to compensate for the decreases in welfare (costs).

(See Michael Schnell's book *Cost benefit analysis for engineers and planners* for further reading)

14.1 Points of particular interest for determining costs and benefits

Differences in units

A problem with comparing costs and benefits is the differences in units and time scales. An example of these differences is the choice between building a larger pumping station or alternatively financially compensating the fruit farmers in a polder where there is a recurrent water nuisance.

If a pumping station is built, a larger discharge (Δx m^3/min) can be carried out, which prevents the water nuisance. On the other hand, if the farmers are financially compensated for their loss of yield (y ton apples), the larger pumping station does not have to be built and no investment is needed. The choice is made based on the question of what is worth more: Δx m^3/min pumping capacity or y tons of apples? In order to compare the two they have to be expressed in the same monetary unit.

Differences in time scales

In the example described above, the building of a new pumping station incurs high costs immediately whereas the potential damage to the apple yield may not occur for another few years. The question is therefore whether this year's apples will have the same value as the apples in, for example, 5 years. A water board might prefer to pay compensation for an apple harvest in 5 years' time rather than make the investment in the pumping station now. Reasons for this choice could be the flexibility that it gives and the possibility to invest the money in other changes.

The costs and benefits from different points in time cannot be compared with each other. A hundred Euros could have a higher value now than the same amount of money in the future. For this reason sums or investments need to be converted to the *present value*. The sum is therefore multiplied by a factor that decreases over time. The general description of the present value calculation is:

$$CW = M(1+i)^{-t} \qquad \textit{(Equation 14-1)}$$

When:

CW = Present value when t=0

M = Total at time = t

i = Interest rate

For the calculation of cash values with a fixed price level an real interest rate is used. This means that the interest rate is corrected for inflation. The value of the interest rate *i* has a large influence on the cash value of the costs and benefits. Projects with a small initial investment are more attractive at a high interest rate, while projects with long term benefits are more attractive at a low interest rate.

The interest rate only represents the time preference for money and this is not equal to the interest which would have to be paid when the money was borrowed, because this interest would be determined by the conditions prevailing at the time.

In the Netherlands the interest rate is 2.5% and also sometimes a risk increase of 3% to take project-specific risks into account that are not included in the cost-benefit calculation. This 3% increase in risk is questionable, because the uncertainties in the costs and benefits have nothing to do with the time preference of the money involved. It is mainly benefits that have accrued over the long term that are reduced by the increase in risk.

Time horizon

The time horizon T or analysis period is the period for which the consequences for a project will be taken into account. The technical lifespan, economic lifespan or a shorter or longer analysis period can be chosen as possible considerations. When the analysis period is much shorter than the economic lifespan, the residual value has to be taken into consideration in the cost balance.

The economic lifespan has passed when the marginal benefits are smaller than the marginal costs. In other words: the anticipated profit until the next repair is smaller than the management and maintenance costs. The economic lifespan is always smaller or equal to the technical lifespan.

The analysis period which is chosen is sometimes shorter than the technical or economical lifespan in order to leave uncertainties in the distant future out of the equation. The shipping route projects and the infrastructure projects in The Netherlands have a time horizon of 30 or 50 years. Taking a shorter time horizon in order to disregard uncertainties is just as questionable as raising the interest rate, because uncertainties with respect to costs and benefits have nothing to do with the point in time that the costs or profits are made.

Shadow prices or market prices?

The value of costs and benefits in currency can be taken as market prices or shadow prices. The choice between the two depends on the type of cost-benefit analysis and the extent to which the prices are determined by the market. In a financial cost-benefit analysis the focus is on the position of the individual, and goods and services are valued according to market prices.

In an economic cost-benefit analysis the focus is on the welfare of a group. Although market prices for goods and services are the starting point for the economic cost-benefit analysis, market prices are not always in the best interest of the group. The market prices are adjusted with a shadow price factor to a price that would be paid in an ideal market. This correction is necessary because the market prices vary due to political decisions, monopoly, taxes, subsidies, incomplete or misleading information and unfair competition.

A social cost-benefit analysis has the same focus as the economic cost-benefit analysis, but the prices of the costs and benefits are further corrected according to management preferences. An example is choosing water storage in combination with environmental development in a region where the natural environment is deficient.

Settling old investments

Only future costs or benefits should be taken into account. Expenses from past investments, so-called *sunk costs,* are not allowed to be taken into account in the analysis so that these costs do not influence current decisions.

Nevertheless, past investments are often used in order to force a choice between several alternatives. Two reasons can be given for this:

1. The water board has political reasons or management obligations to wind a project up, so the expenses were not wasted;
2. Unvalued assets in the accounting system have no value and can limit future investments.

In spite of this, past mistakes should never be a reason to continue with a project that is financially unfounded. Continuation of the project is not in the economic interest of the inhabitants or stakeholders.

Inflation

The principle of a cost-benefit analysis is to express all costs and benefits in the same units and compare them. However, inflation means that the Euro of 2006 has a different value than the Euro of 2007. This means that for a cost-benefit analysis all the values have to be consistent. Inflation is taken into account by valuing future costs and benefits with an real interest rate. The real interest rate is the nominal interest rate corrected for inflation.

Depreciation

The depreciating value of goods is not taken into consideration. The decreasing book value of capital goods due to wear and tear is an accounting adjustment made to the end-of-year balance. In a cost-benefit analysis the replacement value is taken for the day on which something has to be replaced.

14.2 Comparing costs and benefits

For comparing costs and benefits four methods have been developed:

1. Nett present value (in Dutch*: netto contante waarde)*;
2. Cost-benefit ratio (in Dutch*: kosten-baten verhouding)*;
3. Internal rate of return *(*in Dutch*: interne rentabiliteit)* and
4. Annual cost (in Dutch*: jaarlijkse kosten)*

Present value

The present value method assesses several alternatives and chooses the alternative with the largest present value of the benefits minus costs.

$$PV = \sum_{t=0}^{T} \frac{(B_t - C_t)}{(1+i)^t} \qquad \textit{(Equation 14-2)}$$

When:

NCW	= present value
t	= year
B_t	= benefits in Year t
C_t	= costs in Year t
i	= interest rate
T	= time horizon

All costs and benefits have to be converted to the same start year with the same interest rate and the same analysis period. Even when the economic lifespan of the several alternatives differs, the equation has to be based on a common period. This can be done by lengthening the service after the termination of the alternative with the shortest lifespan or the calculation of the residual value of the alternative with the longer lifespan.

Cost-benefit ratio

This method does not look at the difference, but instead at the quotient of the costs and benefits: the cash value of the costs divided by the cash value of the benefits:

$$CBR = \frac{\sum_{t=0}^{T} \frac{C_t}{(1+i)^t}}{\sum_{t=0}^{T} \frac{B_t}{(1+i)^t}} \qquad \textit{(Equation 14-3)}$$

When:

CBR = cost-benefit ratio
t = year
B_t = benefits in Year t
C_t = costs in Year t
i = interest rate
T = time horizon

When several alternatives are compared, the alternative with the lowest cost-benefit ratio is not the same as the preferred alternative determined using the present value method. Figure 14-1 illustrates this for different protection levels with their costs and benefits.

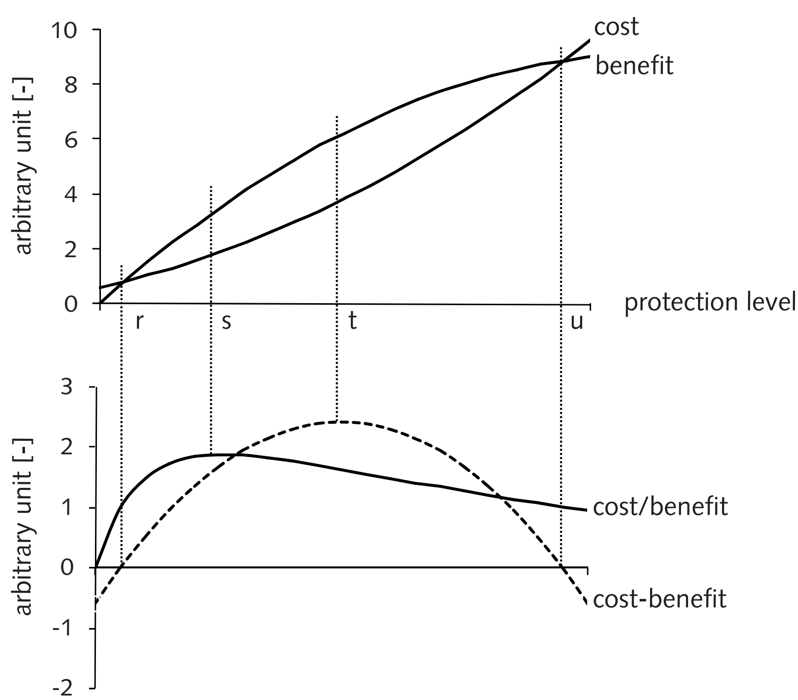


Figure 14-1 Theoretic relation between costs and benefits for several protection levels.

For all protection levels between ‘r’ and ‘u’ there are more benefits than costs. The alternative ‘r’ is preferred when money is scarce but the project has to be realized anyway. The lower boundary is where the benefits are at least equal to the costs and costs are minimal. Alternative ‘u’ is preferred when a lot of money is available and only a few projects have to be realized. The upper boundary is where costs are equal to the benefits.

The alternative with the largest cost-benefit ratio ‘s’ is preferred when money is available, but limited (optimum return). When money is not an issue, alternative ‘t’ is preferred, since this gives the largest benefits. Note: whether an item is a cost or a negative benefit is often not clear, but nevertheless this has a large influence on the cost-benefit ratio.

Internal rate of return

The internal rate of return tells us at which interest rate the present value is equal to zero. When it is not clear what the correct interest rate is, the internal rate can give an indication of the expected rate of return.

$$IRR = i \quad waarbij \quad \sum_{t=0}^{T} \frac{(B_t - C_t)}{(1+i)^t} = 0 \qquad \textit{(Equation 14-4)}$$

Where:
IRR = internal rate of return
t = year
B_t = benefits in Year t
C_t = costs in Year t
i = interest rate
T = time horizon

Annual costs

The annual costs method recalculates all costs and benefits as annual costs and benefits, by multiplying the cash value by a constant factor. The variation in time of costs and benefits is redistributed over the analysis period.

$$\sum_{t=0}^{T} \frac{AC}{(1+i)^t} = PV = \sum_{t=0}^{T} \frac{(B_t - C_t)}{(1+i)^t} \qquad \textit{(Equation 14-5)}$$

Where:
AC = annual costs
t = year
B_t = benefits in Year t
C_t = costs in Year t
i = interest rate
T = time horizon

Each of the methods explained above has its own pros and cons:
- The present value method is easy to apply, but is not suitable to arrange alternatives in a sequence according to size;
- The internal rate of return method has the advantage that a interest rate is not necessary;
- A disadvantage of the IRR method is that more than one alternative can be the preferred solution;
- The cost-benefit ratio is very commonly used;
- The annual cost method is similar to the present value method, but is often used because people are accustomed to using annual costs instead of present values.

The purpose of the analysis determines the method used. You will not be told which method to use in these lecture notes. The goal is to prevent measures that are 'too expensive' from being implemented.

14.3 Determining the costs of improvement measures

The cost of improvement measures in water management is the expenditure for the reduction of risks. In almost all projects it is easier to determine the costs than to determine the benefits. The costs are given by the investment costs and the costs of management and maintenance. The costs can be divided into:

1. *Goods and services*

Identifying possible measures such as the replacement of earth or the implementation of weirs is not difficult. The costs per unit are also easily calculated. The difficulty in estimating the costs of goods and services lies in the determination of when and where;

2. *Labour*

The labour costs are all the employment costs incurred during planning and implementation and for management and maintenance. As with goods and services, the costs per unit are easily determined. However, the determination of how much and when labour is needed, is often more difficult to estimate;

3. *Land*

The economic value of land is difficult to determine, due to land use changes. A good estimation of the value would be the sum of all future yearly net yields that would be gained if the project was not implemented. When a piece of land is valued at the beginning of the project when the land has to be bought, the price would be as if the market was balanced. However this is hardly ever the case: a) the land is already the property of the project developer and purchase costs cannot be taken into account or b) the price is inaccurate, due to non-economic considerations. A possibility is to value the land according to the cash value of its annual lease. Since this price is calculated annually, the price is more likely to be accurate.

14.4 Determining the benefits of improvement measures

Water management is a collective service. The main goals are function support and sufficient protection. The principles of a cost-benefit analysis prescribe that the investment costs counterbalance the benefits. Examples of benefits of a measure are:

1. Larger yields due to:
    - larger production
    - quality improvement
    - change in time and/or place of where a product is sold
    - changes in shape
2. Cost reductions due to:
    - computerization
    - reduced transportation costs
    - prevention of losses

For the improvement of a water system the most appropriate technique for determining the benefits is to calculate the cost reductions due to the prevention of losses as a consequence of a sub-optimal function support.

The benefit of preventing water nuisance is profit in the form of a reduction in future costs due to damages in money or goods, which can vary per project. In other words: the cash value of the difference between the expected value of the annual damage with and the expected value of the annual damages without a measure.