Abstract:
In the last times, infiltration facilities to reduce the load on the drainage systems have been introduced in many urban areas. These facilities allow water to infiltrate or to drain depending of the level it reaches in the ground. It results easy to conclude that this infiltrating and draining process is likely to be controlled in real time so extra storage can be created on the urban subsurface when necessary or, on the other way round, store water when a shortage is expected. To study the possibilities of controlling the groundwater levels to create extra storage, an area of study was selected in the municipality of Delft, which was interested as well in being able to create extra storage. This area includes an infiltration system. After insight on the area was acquired, the only element which is likely to be controlled in the whole process turns to be the head levels at the infiltration facility. Therefore, it becomes necessary to find the influence the infiltration facility has on the groundwater system on its surroundings and obtain a model which allows tuning a proper controller afterwards. A mathematical model of the response of the groundwater levels within the area of study was created by using system identification. The inputs to this identification process are net precipitation, the historical precipitation and the water level at the infiltration facility. The output is, of course, the groundwater levels. All the data was obtained after a measurement campaign using both “in situ” measurements and remote sensing. Once a proper model was obtained, a simple feedback controller for the infiltration facility levels was tuned. A visible improvement on the groundwater levels behavior was observed when the controlled system was simulated. It results clear then that groundwater levels, within an area where an infiltration facility is present, can be controlled in theory. A simple actuator was designed and built to be able to run practical experiments in the area later on. With this, the real relation of the infiltration system with the surrounding grounds can be checked as well as different control methods can be tried in further research.
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Optimization of the rainfall-runoff response in urban areas by using controllable drains

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Delft, August 2009
Summary

In the last times, infiltration facilities to reduce the load on the drainage systems have been introduced in many urban areas. These facilities allow water to infiltrate or to drain depending of the level it reaches in the ground. It results easy to conclude that this infiltrating and draining process is likely to be controlled in real time so extra storage can be created on the urban subsurface when necessary or, on the other way round, store water when a shortage is expected.

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Once a proper model was obtained, a simple feedback controller for the infiltration facility levels was tuned. A visible improvement on the groundwater levels behavior was observed when the controlled system was simulated. It results clear then that groundwater levels, within an area where an infiltration facility is present, can be controlled in theory.

A simple actuator was designed and built to be able to run practical experiments in the area later on. With this, the real relation of the infiltration system with the surrounding grounds can be checked as well as different control methods can be tried in further research.
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Chapter 1

Introduction

That the urbanization process supposes a strong impact on the natural system is a fact that is no longer discussed. Every new urban development at any place in the world implies deep changes in many aspects of the environment, from displacement or removal of animal and vegetal species to the influence in the natural flow paths of water.

Both the surface and the subsurface water systems are substantially influenced and modified by urban developments. The runoff is increased, the transpiration is dramatically reduced and major changes in the frequency and the volume of groundwater recharge are introduced.

The impermeabilization of the land surface by the construction of roofs and paved areas tends to reduce the infiltration rate while the runoff is increased and often accelerated. The use of permeable pavements can lower the intensity of this effect, though it is also possible that, because the evaporation is anyway reduced, the contrary effect is achieved, increasing the amount of water going to the subsurface. Leakage from pipelines (both water supply and sewerage) and irrigation of amenity areas suppose an extra amount in the recharge regime. Lerner (1990) and several more authors state that, even though the common factor to most urbanization is the impermeabilization of a grand part of the land surface, with the consequent reduction of the normal soil infiltration, the net effect of all the previous factors is an increased groundwater recharge.

The later will suppose a deep impact on the urban groundwater levels. Changes in groundwater levels can affect the stability, integrity and operation of subsurface engineering structures and installations (Foster, 2001). Too low groundwater levels may affect the stability of old buildings because of differential settlements or deterioration of the poles on which they are founded. On the other hand, high water levels not only affect the integrity of constructions but also threaten public health in the form of fungi growing on humid basements or the affection to the operation of the sewer and water supply systems. Proper actuations of drainage and infiltration may mitigate these problems by maintaining the groundwater levels in urban areas within certain boundary limits. The optimization in the performance of these techniques can positively improve the behavior of the whole urban water system.
This text will deal with the possibility of achieving the mentioned optimization by the real time control of the groundwater levels within an urban area. To do this, the research will be based on the control of the drain/infiltration levels of an infiltration facility with a model predictive control system. In chapter 2, the groundwater flow basics as well as a theoretical justification to the stated control form will be given. Later on, in chapter 3, the research question as well the approach to the problem and the research objectives will be specified. The area of study where the research was carried out will be presented in chapter 4. Chapters 5 and 6 will deal, respectively, with the identification of the processes affecting the groundwater levels within the area and the design of a feedback control system. Finally, conclusions will be derived from the research process and some recommendations will be given.
Chapter 2

Groundwater levels control. Possibilities and limitations.

2.1. Introduction

Groundwater is commonly considered all the water stored (or flowing) in the saturated area of the soil below the surface. The permeable region or layer in the subsoil is usually called aquifer. There are several ways water can get into the subsurface. When a precipitation event takes place water is brought on the land surface, and from there it can flow over land, evaporate or infiltrate into the soil. All that infiltrated water can either flow to a surface water body as interflow or can move downwards and reach the saturated zone as recharge. Water from the saturated zone can leave it by discharge on a surface water body or another groundwater body, transpiration or well discharge.

All these fluxes in and out the saturated zone will determine the storage in the groundwater body and, therefore, the groundwater levels respect the surface. This can be easily determined by carrying out a hydrologic balance like the following:

\[ \text{Flux in} - \text{flux out} = \text{rate of change in the stored water} \quad (2.1) \]

the previous relation can be generally written like:

\[ R + G_{\text{in}} - G_{\text{out}} - G_s - ET_d - Q_w = \frac{dV}{dt} \quad (2.2) \]

where \( R \) represents the recharge from precipitation, \( G_{\text{in}} \) and \( G_{\text{out}} \) are groundwater inflows or outflows through the lateral boundaries and bottom of the aquifer, \( G_s \) is groundwater discharge into streams or other surface water bodies, \( ET_d \) is deep evapotranspiration from the saturated zone and \( Q_w \) is well discharge. All the previous description can be seen in figure 2.1.
The volume of water stored in the subsurface also depends on the properties of the materials forming the subsoil (aquifers). Porosity of the media determines the volume of water which can be stored in the aquifer. In a similar way, the grain size determines how much pore space is available to hold water, and how easily water is transmitted through the material. Associated to these properties there is hydraulic conductivity $K$ of the medium, it is a measure of the ease with which a medium transmits water; higher $K$ materials transmit water more easily than low $K$ materials. In some cases, it is only practical to measure the hydraulic conductivity as an integrated parameter over the thickness of an aquifer\(^1\). This parameter is called transmisivity of the aquifer and if the hydraulic conductivity of the aquifer can be assumed constant over the thickness $D$ of the aquifer, then the transmisivity is simply

$$T = KD$$ \hspace{1cm} (2.3)$$

Transmisivity is a measure of how easily a layer transmits water.

Controlling any of the terms of the water balance would allow changing the stored volume in the aquifer and set it to the most beneficial situation. Processes governing the water balance as well as their influence in the total of the equation must be known. Of course, knowing the way groundwater moves along the system is also a primary necessity in order to carry out further actions.

### 2.2. Groundwater movement

Controlling groundwater implies taking water from one spot and putting it into another place. This makes necessary understanding how water flows through the soil and how it reacts to changes which may be gradual or sudden.

\(^1\) It is being considered that the aquifer consists of one layer only.
2.2.1. Why the water moves

Water flows from one point to another in response to the different energies existing at each one. Water always flows from points with high mechanical energy towards points with lower mechanical energy. This mechanical energy takes three different forms namely, elastic potential energy, gravitational potential energy and kinetic energy. Elastic energy depends on how compressed the water is, gravitational energy is determined by the elevation of that water respect a reference point and kinetic energy is derived from the velocity of the water. Considering water as a relatively incompressible fluid, all these three forms of mechanical energy come together in the Bernoulli’s equation, which can be written as:

\[ E = PV + mgz + \frac{1}{2}mv^2 \]  

(2.4)

this equation represents the energy \( E \) of a volume of water \( V \) with mass \( m \), pressure \( P \), elevation \( z \) and velocity \( v \). It can also be read as the work required for putting a volume of water \( V \) with mass \( m \) in the previous described state.

For analysis of water flow seems more convenient to use the hydraulic head which is expressed as the energy per weight of water. Dividing the Bernoulli equation by \( mg \) it yields:

\[ h = \frac{E}{mg} = \frac{P}{\rho_v g} + z + \frac{v^2}{2g} \]  

(2.5)

The three terms of the hydraulic head are called pressure head, elevation head and velocity head respectively. Water will flow from regions with high hydraulic heads towards points where they are lower.

Due to the specific characteristic of the soil matrix, groundwater generally flows at very low velocity. This makes the velocity head so low that it can be neglected from equation (2.5) when considering groundwater. In this case the equation will take the form:

\[ h = \frac{P}{\rho_v g} + z \]  

(2.6)

2.2.2. Groundwater flow basics

Even though it has been stated that groundwater velocity is very low and it does not affect the energy of the flow in an important way, water flow still occurs when there is a head difference between two points. The flow between two points with different heads can be calculated with Darcy’s law.
\[ Q = -K_s \frac{dh}{ds} A \] (2.7)

Darcy empirically stated that the flow \( Q \) through a porous medium is directly proportional to the head difference between two of its points and inversely proportional to the distance between those same points. Of course the flow is also proportional to the cross sectional area \( A \). The proportional constant corresponds to the hydraulic conductivity of the medium in the flow direction.

Darcy’s law can also be expressed as the discharge per cross sectional area as:

\[ q_s = \frac{Q_s}{A} = K_s \frac{dh}{ds} \] (2.8)

The term \( q_s \) is also called specific discharge or Darcy’s velocity. It represents the velocity the flow would have when going through an area \( A \). Even though this definition, \( q_s \) can not be considered the velocity of the water in the medium. As long as the subsoil is formed by particles, only part of the cross section can be considered available for water to move through. In other words, the water will flow through the space left between the particles in the cross section. Then the average velocity of the water in a porous media is expressed as:

\[ \bar{v}_s = \frac{q_s}{n_e} \] (2.9)

where \( n_e \) is the effective porosity; it expresses the amount of pores which are interconnected and available for the water flow while porosity measures the total amount of pores in the soil matrix.

It is also useful to consider in this point the one dimensional expression of the Darcy’s law in relation to the aquifer transmissivity. This will allow understanding later on the simplification and particular solutions to the flow equations. As well as the discharge through the porous medium is proportional to its hydraulic conductivity, it will be also proportional to its transmissivity. Combining equations (2.3) and (2.7), it can be shown that the discharge in the x direction through a length of aquifer that extends a distance \( \Delta y \) in the y direction is

\[ Q_x = -T \frac{\partial h}{\partial u} \Delta y = -kD \frac{\partial h}{\partial u} \Delta y \] (2.10)

### 2.2.3. Groundwater storage

As has already been stated, controlling the groundwater volume in an aquifer would require controlling one or several of the processes influencing the hydrological
balance. This fact would make several transient flow phenomena took place in the groundwater system. An important aspect of transient groundwater flow modeling is groundwater storage.

Two main\(^2\) forms of storage may be distinguished:

- Elastic storage, which is due to combined compressibility of the water and the porous matrix.

- Phreatic storage, which is due to filling and emptying of pores above the saturated zone. It takes part mostly in unconfined aquifers.

A basic storage property of saturated materials is the specific storage \(S_s\). It is the amount of water expelled from a unit volume of saturated material when the pore water is subject to a unit decline in head. This can be expressed as:

\[
S_s = -\frac{dV_w}{V_t} \frac{1}{dh}
\]  \hspace{1cm} (2.11)

where \(dV_w\) is the volume of water expelled from the aquifer \(V_t\) when the head changes by \(dh\).

Another useful\(^3\) storage parameter is storativity \(S\), which integrates the storage over the height of the aquifer. In confined aquifers consisting of a single material \(S\) is defined as:

\[
S = S_s b
\]  \hspace{1cm} (2.12)

where \(b\) is the vertical thickness of the aquifer. \(S\) is the decrease in the volume of water stored per unit surface area of aquifer per unit decline in head. Combining equations (2.11) and (2.12), the volume of water removed from an aquifer with area \(A\) when the head changes by \(dh\) is:

\[
dV_w = -SA dh
\]  \hspace{1cm} (2.13)

All the previous can be applied to (semi) confined aquifers where only elastic storage occurs. When head declines, water is expelled from the volume because the water in the pores expands and the soil matrix compresses. This effect also takes place in unconfined aquifers but their elastic storage is usually insignificant compared to storage

\(^2\) The displacement of the interface between fresh and salt water is also considered a form of storage but it will not be considered along this research.

\(^3\) For two dimensional aquifer analysis.
related to drainage of water at the water table. When the head declines, so does the water table. Due to this, the storativity in an unconfined aquifer is called specific yield \( S_y \) and it has the same conceptual signification than storativity \( S \) in a (semi) confined aquifer

\[
S_y = S
\]  

(2.14)

and considering the previous concepts for (semi) confined aquifers, the volume of water removed from storage in an unconfined aquifer can be given by

\[
dV_w = -S_y A dh
\]  

(2.15)

2.2.4. Flow equations

Once the basic concepts of the movement of the groundwater have been mentioned it is time to derive the flow equations which will govern this process. In the case of subsurface flow, the relevant physical principles are Darcy’s law and mass balance. As long as the derivation process of such equations can be found in several documents and text books and to not make this section too long, a simple fast approach will be given.

To derive the flow equations for groundwater flow passes by carrying out a mass balance analysis of a differential element situated in the saturated zone. The net flux of mass (water) through the boundaries of the element is equal to the rate of change of mass within the element. This can be expressed mathematically as in equation(2.1). After some calculus the mass flux within the saturated element may be expressed like:

\[
\frac{\partial}{\partial x} \left( K_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_y \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left( K_z \frac{\partial h}{\partial z} \right) = S \frac{\partial h}{\partial t}
\]  

(2.16)

which is the general equation for three dimensional groundwater flow. For the flow analysis which is going to be performed in the following, it will be enough to consider one dimensional flow. Also the hydraulic conductivity will be considered homogeneous (constant in space). Taking all this into account the flow equation can be written as:

\[
K_x \frac{\partial^2 h}{\partial x^2} = S \frac{\partial h}{\partial t}
\]  

(2.17)

rewriting equation(2.17) in terms of transmisivity and dividing both terms by the storativity it yields in

\[
\frac{kD}{S} \frac{\partial^2 h}{\partial x^2} = \frac{\partial h}{\partial t}
\]  

(2.18)
which has the same form that the diffusion equation with \( \frac{kD}{S} \) the diffusivity of the aquifer.

Moreover, several terms may be added to this equation like recharge from precipitation \( N \) or leakage to an adjacent layer through a resistance \( c \). The addition of these terms gives the equation\(^4\)

\[
\frac{\partial^2 h}{\partial x^2} + \frac{N}{kD} - \frac{h - h_0}{\lambda^2} = \frac{S}{kD} \frac{\partial h}{\partial t}
\]  

(2.19)

with \( \lambda = \sqrt{kc} \) the so called spreading or characteristic length of the system.

To actually be able to solve this equation it will be needed to define initial and boundary conditions. The boundary conditions can be specified as fixed heads or fixed discharges along certain parts of the model boundaries. Only transient solutions require initial conditions, i.e. the head everywhere at time zero.

In the following point a well known analytical solution to equation(2.18) will be shown. It will help to justify/prove that groundwater levels may be controlled under certain conditions.

2.3. Theoretical approach to groundwater levels control

As it has already been mentioned in the previous chapter, the idea of controlling the groundwater levels within a certain area passes by controlling the levels at an infiltration facility present in it. That means setting the level/head at the infiltration facility to bring the groundwater levels towards a desired level. This section deals with the theory behind the idea and the possibilities and the limitations which may be found.

2.3.1. An analytical solution to the one dimensional flow equation for a sudden change of head at \( x = 0 \)

Consider a one dimensional half space, i.e. an aquifer which is in direct contact with a fully penetrating water body at \( x = 0 \). Ignoring leakage and recharge, and assuming a constant transmissivity, the flow equation takes the form of equation(2.18).

The solution for the case given in the title of this point is

\(^4\) \( kD \) is considered a constant. Anyway, in the case of phreatic aquifers, thickness equals the distance from the bottom to the water table and is variable. \( kD \) can still be considered constant as long as the head fluctuation does not vary too much respect the aquifer thickness, otherwise the way to solve the equation would differ.
h = a \text{erfc}(u) \quad (2.20)

where \( u = \sqrt{\frac{x^2 S}{4kDt}} \) and \( a \) is the value of the sudden head change. Erfc(u) is called the complementary error function. By definition

\[
erfc(x) = \frac{2}{\sqrt{\pi}} \int_x^\infty e^{-t^2} dt
\quad (2.21)
\]

so that its derivative is

\[
\frac{d\text{erfc}(x)}{dx} = -\frac{2}{\sqrt{\pi}} e^{-x^2}
\quad (2.22)
\]

A representation of this function can be found in figure 2.2.

![erfc(u)](image)

**Figure 2.2 Function erfc(u)**

It is clear that the solution of head change versus distance to the boundary and time will have the shape of this curve. Of course, its shape will depend on the values of the different parameters in \( u \).

An example case can be done for an aquifer with \( kD = 15 \, \text{m}^2/\text{h} \) and \( S = 0.05 \). Figure 2.3 represents the head change as a function of the distance for the times given in the legend for a sudden head rise of 1m at \( x = 0 \, \text{m} \). Figure 2.4 shows the head change as a
function of time for the distances given in the legend for the same sudden rise than the previous figure.

Figure 2.3 Head change as a function of the distance for the times given in the legend for a sudden head rise of 1m at x = 0 m
The above given solution is only one of an infinite series of solutions where \( h(0,t) = at^{n/2} \), with \( n = 0, 1, 2, 3, \ldots \) and \( h(\infty,t) = 0 \). Several authors have derived solutions to equation (2.18) for different boundary conditions at \( x = 0 \). These solutions may result more efficient than the previous one as long as sudden changes of head are less likely to occur than linear changes in time, for example, which solution can be obtained from the solution obtained by Bruggeman (1999). Anyway, for the scope of this theoretical analysis the first solution can be considered enough.

2.3.2 Superposition of the solutions

Individual solutions of a partial differential equation in a linear system may be superposed (added). To do this, the summed boundary conditions must match the ones of the real system. Both, superposition and convolution are forms of adding two or more solutions of a partial differential equation.

Superposition principle states that the net response at a given place and time caused by two or more stimuli is the sum of the responses which would have been caused by each stimulus individually. Under this assumption and considering the solution for the change of head in the aquifer due to a sudden change of the head \( a \) at \( x = 0 \) at \( t = 0 \) given by equation (2.20), the head due to many changes of the water table, each with its
individual amount $a_i$ and starting at time $t_i$ and continued forever, can be calculated by superposition like

$$h(x,t) = \sum_{i=1}^{n} h_i(x,t) = \sum_{i=1}^{n} \left[ a_i \text{erfc} \left( \sqrt{\frac{x^2 S}{4 k_1 (t-t_i)}} \right) \right]$$

(2.23)

Where the function is considered zero as long as $t < t_i$

An example of this can be found in figure 2.5. On it, a situation where the water level is raised by 1 m at $t = 1h$, lowered by 2 m in $t = 3h$, raised 1.5m again at $t = 3.5h$ and finally lowered again by 0.5 m at $t = 6h$. The aquifer properties are the same than in the example from previous section and the represented point is $x = 15m$. Also the changes in level at $x = 0$ are shown.

Superposition becomes impractical when a large amount of solutions must be computed. Because of this, the use of convolution turns to be more efficient. Convolution is a form of superposition allowing simulation of linear systems with continuous arbitrary time varying inputs. Even though the usefulness of this other method, on the scope of this justification it will be enough with the superposition theorem.

![Example of superposition](image)

**Figure 2.5 Example of superposition**
2.3.3 Possibilities for the active control of groundwater levels

The previous sections help to clarify that groundwater levels can be controlled. It has been said before that a sudden change of groundwater levels is less likely to occur in nature (although they may occur). Anyway, this is less true in the case of human intervention on the groundwater system.

Man modifies the shape of land by digging canals or trenches to bring water to places which need it, or to take it out from where is not desired. At the same time, man also controls, or tries to control, the levels in all the water courses he can. All this makes many groundwater systems are influenced by the new induced fluctuations, which may occur really suddenly.

It has been shown in the previous sections how groundwater reacts to sudden changes of hydraulic head in its boundaries, although it has also been stated that it reacts similarly to gradual changes. It has also been shown that several different changes can bring the groundwater levels to where is desired (see figure 2.5). This allows concluding that controlled changes at the boundaries of the groundwater system would result into controlled groundwater levels. Of course, most of systems are not only driven by their boundary conditions but also from external forces, as has been seen in section 2.1. Therefore, it is very important to identify all the driving forces which may influence the system, as well as their individual influence must be weighted up.

In urban areas, the influence of man results even more obvious. In cities, the whole natural system is strongly modified much more than in rural areas. Because very high groundwater levels may threaten not only the stability or durability of structures, but also public health and infrastructure, man tries to maintain those below a certain security level. In the same way, very low groundwater levels are also negative for the urban environment. Vegetation dries out and buildings foundations can be displaced causing differential settlements. Because of all this, drainage systems are applied in urban areas all around the globe. Drainage facilities allow keeping the groundwater levels below a predetermined level but if a dry period occurs, then these levels can drop till undesired ones. To avoid also low levels, infiltration systems were introduced. A infiltration system allows maintaining the groundwater levels within an area between certain limits by giving water to the soil when the levels are low and draining the water from the soil when the levels are too high. Anyway, an infiltration system does not have capacity to retain water when a dry period is expected or to provide extra storage when a big precipitation episode is going to occur.

Both drainage and infiltration systems can be considered boundaries of a certain groundwater system within an urban area. As long as an infiltration system does the same work as drainage plus infiltrating, focus on these later systems will be kept for the control purposes.

After all the considerations made in the last paragraphs, it becomes clear that controlling the groundwater levels in an urban area by varying the levels at an infiltration
facility should be, at least possible. It results obvious as well that the control method can even include a predictive element which would allow decreasing the groundwater levels before a storm event took place to create extra storage, or to increase the levels in the case a dry period was expected. This would also help to optimize the rainfall-runoff response of the system by shifting and decreasing the peaks.

2.3.4 Limitations

Even though the possibilities for actively controlling groundwater levels within a certain area are interesting, there are several limitations which can make the idea unfeasible or difficult to implement in certain cases.

First of all, the soil characteristics play a fundamental role in the whole process. Very low transmisivities will affect the time it takes the water to infiltrate or drain. This would make the operation process very difficult and inefficient due to the slowness of the system.

Second, the control could only be applied in places where the system can both drain and infiltrate. Or, what would be the same, on places where the infiltration system works as a real boundary condition. If in a certain place the groundwater levels are very deep, the most likely is that they do not affect the urban life at all and then such a system is not really necessary.

Finally, but not less important, is the cost factor. Implementing a control system always implies doing some investments like installing devices for its control, monitoring, etc, apart from the operation costs, which mostly are energy costs. It is always important to weight the costs of introducing this kind of systems against the possible costs of not introducing them.
CHAPTER 3


3.1 Problem statement

The rainfall-runoff response of an urban water system is an important topic inside the urban water management world. Improving/optimizing this response, so the adverse effects associated to it are diminished, has become the main interest of cities, water boards, and urban water managers in general, in the lasts times.

The optimization process is usually achieved by reducing the runoff peaks and shifting them in time so the system is not overloaded. To do this, the most common practice is to store the water before. There are several ways to store the water. Surface storage is the most typical way to delay the entrance of water into the system. It is also possible to build subterranean reservoirs to store the water once it is already in the system. The high prices of soil in densely populated areas can make building surface storage really expensive and, therefore, not advisable. Building subterranean storage facilities is not advisable either if the area is already populated. The only possibility left is to store the water into the ground. The problem this method has is that, in certain areas, the groundwater levels may be too high to even try to increase them more by infiltrating more water. This is the case at certain areas of the municipality of Delft, which wants to study the possibilities of creating extra storage in the ground.

The solution to the last commented problem can be achieved by decreasing those levels before infiltrating more water. The problem which would arise then is that too low groundwater levels are also harmful. Avoiding this problem while still being able to store water in the subsurface could be done if the groundwater levels could be set to the most appropriate level depending on the situation.

In chapter 2 it has been shown by using the basic theory that groundwater levels can be controlled, at least in theory. Problems start when considering translating the theory into practice.
To start, because a real groundwater system is much more complex than the ideal systems used to develop the theory. The homogeneity of the soil matrix in the theoretical approach turns to be highly heterogeneous in the real world, changing even every few meters.

Also the processes governing the system vary from place to place both in nature and in level of influence in the system. While, in some places, recharge from rainfall plays a really important role in the movement of the groundwater levels, in some others it may barely influence them. These differences become even stronger when in an urban area, where they may change from one quarter to the next one. Moreover, all these processes are complex and including them all into a model can suppose months or even years of research and measuring to derive and calibrate all the parameters present on it. Furthermore, some model may be valid for certain areas while completely wrong in some other spots. This would make necessary to look for a new model with the consequent extra time spent.

All the previous problems also affect the control process. As has already been commented, the control process will be more or less effective as long as the characteristics of the system are the appropriate ones. But also the available data suppose an important restrain for the control system. Groundwater levels must be measured and transmitted to the controller so it can take the proper decision depending on them. The measurement frequency must be enough for the degree of control which is desired as well as its precision. But not only are the groundwater levels important. Monitoring of all the processes affecting the system must be done, being the biggest obstacle obtaining these data as well as its accuracy.

### 3.2 Research question

From the previous section it becomes clear that many questions may arise about the implementation of active groundwater levels control systems in urban areas. This MSc Thesis research strives to answer, mainly, the following question:

- Can the groundwater levels from an urban area be actively controlled by using real time control in practice?

Together to this question some sub questions can be answered as well at the end of the research:

- If the answer to the main question is affirmative, how much in advance could these levels be controlled? And how far from the control device?

- How much would this improve the behavior/response of the urban water system?
3.3. Approach

To study the possibility of implementing a predictive control system to actively control the groundwater levels within an urban area, the best is to go into the field. An area of study will be defined, all the processes influencing the groundwater levels will be identified and a model will be obtained, finally a control algorithm will be designed. Also the design of an actuator will be done so it can be checked that the groundwater levels within the system can be affected with this method.

To actually be able to control the groundwater levels, the dynamics of the groundwater system must be known. What causes the levels to vary (inputs), how all these inputs relate to each other or what is the weight of each input in the whole of the process are questions which must be answered before control may be applied. There are actually several ways to achieve this.

From a physical point of view, a physical\textsuperscript{5} model could be derived with formulas which relate different physical processes, such as precipitation, evaporation, etc, to the effect which has to be studied. Afterwards the derived model must be calibrated and validated. This process may result laborious as long as it may require several corrections of the derived model because of unconsidered inputs or factors, or the calibration or validation processes require some long time.

The problem may be also studied from a mathematical point of view. This means obtaining a mathematical\textsuperscript{6} model from the observed data, by more or less easy or difficult techniques, which output acceptably fits the observed response of the system. Applying the necessary techniques to obtain the mentioned model could take a lot of time some years ago, but the advances in computation techniques and the amazing decrease of computation times day after day make the creation of mathematical models a very powerful tool to simulate systems. The obtained mathematical models can be related to physical ones later on. The latter approach to understand the dynamics of the system will be utilized in this research. System Identification techniques will be used to acquire insight on the dynamics of the studied system and to obtain a mathematical model which can be used later on.

Once insight on the processes which govern the groundwater has been gained is the moment to start thinking of a way to actively control the system. To do this a control system is needed. A control system is formed by several components, from an actuator to a control algorithm, which must be designed taking into account the characteristics of the system. It is also necessary to identify what elements of the whole system may be

\textsuperscript{5} In this text it is considered that a physical model is a model where all the components have a physical meaning related to the processes they are describing. For example the hydraulic conductivity value $K$ in Darcy’s formula represents the flow between two points when the head difference between them is equal to one.

\textsuperscript{6} In this text it is considered that a mathematical model is a model which components do not necessarily need to have a physical explanation.
controlled because not all of them will be controllable like the precipitation. As long as this research focuses on an area with an infiltration facility, the most likely is that the control possibilities are reduced to control the level in that infiltration system.

For the design of the control algorithm the philosophy of model predictive control will be followed. Under this scope it will be necessary to know how much in advance the prediction may be available. In relation to the previous it will be also necessary to know how far in space the system can be influenced.

3.4 Research objectives

Along the development of this research the following objectives are aimed to be achieved:

- Create a model by using system identification which simulates the behavior of the groundwater levels at an urban area and, afterwards, use it to simulate the effects that applying a simple real time controller would have on them.

- Design and build a simple actuator to validate the results obtained from the theoretical part.

As well, the following side objectives can be also achieved during the research:

- Acquire insight on the processes governing the groundwater system response.

- Show that system identification is a suitable tool to deal with natural processes, to identify their dynamics and generate valid models which can simulate them.

- Find a general identification method which can be applied on different systems later on.
Chapter 4

The area of study

4.1 Location of the area of study

The Wippolder district consists roughly of three different parts (see figure 4.1). The largest is the western part, between the Schoemakerstraat and Rotterdamseweg, which consists of the Zeeheldenbuurt, the Koningsveldbuurt and surroundings business grounds Rotterdamseweg-North, TU-Campus, TU-North, Delfzicht (Hooikade-Engelsestraat-Leeuwenstein). To the west of the previous areas also the business grounds Schieweg-North and Zuideinde can be found. The eastern part, between the A13 and the Schoemakerstraat, is formed by the neighborhoods Pauwmolen and the Professorenbuurt and the business grounds Delftech, Wippolder North, and Wippolder South. By 2003 the district was expanded to the south by the district Schieweg (consisting of the business grounds Schieweg South and Schieweg Polder) and the district Ruiven (consisting of the neighborhood of Ackersdijk and the business grounds Rotterdamseweg South and Technopolis).

Figure 4.1 Wippolder district.
In 1999 a new urban development was committed in the terrain at the north part of the Wippolder between the streets Kloosterkade, to the south, Professor Krausstraat, to the east, Professor Bosschastraat, to the North, and Sint Aldegondestraat, to the west (see figure 4.2). Due to this new development, a separated sewer system was installed. For the rain water drainage a new infiltration system was installed below the named streets and was connected to the surface water system at Koningin Emmalaan.

![Figure 4.2 Area of study.](image)

In the following, the block resultant of the new urban development on the four already mentioned streets will be considered and, therefore, called the area of study.

### 4.2 Description of the area of study

The area of study is part of a residential area in the Wippolder district. The entire area is paved with bricks with the exception of some small gardens (see figures 4.3 to 4.7). These bricks do not form a completely impervious layer since the joints between them are water-permeable both in the road and the sidewalks. The remaining constructions prior to the new development of the area consist mainly of pre-war single family houses though none of them are present in the area of study.
The new constructed buildings are provided with reservoirs which store the rainwater originating runoff in the roofs. This stored water is used for toilette rinsing. There is also a transfer facility to the infiltration system in the street. Below the new constructions drainage has been placed towards the infiltration system. All the constructions are founded either on steel poles, the older ones, or concrete poles, the newest. The buildings present in the area of study consist uniquely of apartment buildings and single family houses. The center of the block is occupied by a playground for children with a rubber ground cover. The whole area has a total surface of about 11,400m$^2$. 

*Optimization of the rainfall-runoff response in urban areas by using controllable drains*
The soil in the area of study consists, directly below the surface, of a quite pervious sand layer with a thickness of approximately 1 to 2 meters, around NAP -1.5 m. Below the top layer, an about 15 m thickness clay and peat layer is found. Finally, below both this layers, a new sand layer is present.

The top containing water layer is the first sand layer. In-situ mean transmisivity measurements determined a horizontal transmisivity of the top layer of 1 to 2 m/day. The clay and peat layer below forms the top separating layer.

4.3 The infiltration facility in the area of study

4.3.1 Description of the infiltration facility

In the year 2000 an infiltration facility was constructed during the last urban development of the area. This infiltration system was designed to couple the surface water levels with the groundwater levels. Its main purpose is to keep the groundwater of the area between the lowest and highest tolerable levels. These two levels are defined on basis to several criteria from which the two most restrictive levels must be selected at every urban area in the city of Delft. The operational principle of the infiltration system is to drain the groundwater when its level is higher than the level of the system and to infiltrate water when the groundwater level is lower than the one in the facility.

The infiltration system in the area of study is formed by a network of PVC perforated pipes with a 300mm diameter and a construction level of NAP -1.35 m. The pipes were laid in a gravel box (grain size from 10 to 16 mm) with a width of 2 m, the lower side at NAP -1.35 m and the upper part at NAP -1.05 m. Both the pipes and the gravel box were enveloped by a geotextile. The total length of the infiltration system is about 350 m (see figure 4.8 for an overview and appendix () for a complete map).
At every street junction or after a certain length, whatever comes first, a manhole is placed between two pipes in order to provide access to the infiltration system for maintenance or repairing tasks. Three different kinds of manholes, with different purposes, were designed for this infiltration facility namely maintenance, inspection and spillway (doorspuutput, inspectieput and overstortput).

The only difference between the two first manhole types is the size of the manhole itself. A person can easily fit in the inspection manholes but not in the connection ones. The main difference is found in the spillway manhole. This manhole can be considered the end of the infiltration system as long as it is the connection of the infiltration system with the surface water. In this manhole, the infiltration system pipes end has a bended ending which is the specific element that sets the drainage level. The following figure represents a design of this manhole for a better understanding.

Figure 4.8 Overview of the infiltration system in the area of study
The upward bended pipes reach the level at which the drainage is desired to start when the groundwater levels increase. If the groundwater levels are higher than the level of the bend, then the water in the drains will be pushed out from the pipe until the level reaches the one determined by the bended pipe.

In extreme dry periods, the groundwater levels are maintained by increasing the surface water levels above the drainage level. Then the water enters the pipe and infiltrates in the ground making the water levels to increase.

The infiltration facility in the area of study is fed through:

- The precipitation that falls on the roads and the front of the roofs of the new constructions.
- The runoff water from the rain water reservoirs.
- The drainage water from the grounds of the new buildings.

The infiltration system ends in the surface water system at the Emmabrug.

4.3.2 Highest and lowest tolerable levels criteria

As has been stated previously, the main purpose of an infiltration system is to keep the groundwater of the area where it is located between the lowest and highest tolerable levels. These two levels are defined, in the case of this specific area, on basis to the following criteria.

- **Highest tolerable groundwater level under buildings with creeping spaces.**
The criteria for buildings with creeping space defines a most tolerable groundwater position of 0.20 meters below creeping space bottom (on coarse sand) and becomes started from the next requirements:

- Floors of houses lie slightly 0.15 meter above ground level.

- Empty spaces under the lowest floor of a building must have a minimum height of, at least, 0.50 meter, so the pipes and conductions placed below the floor are accessible for maintenance and replacement.

With a floor thickness of 0.20 meters and the above mentioned requirements, it results in a highest tolerable groundwater position of 0.75 meters under the ground level (see figure 4.10).

![Figure 4.10. Highest tolerable groundwater level below buildings with creeping spaces](image)

The criterion of 0.20 meters below the creeping space bottom has been based on coarse sands. For creeping spaces above fine sand or clay material supplementary measures, like a lower groundwater level below the creeping space, may be necessary because of the larger capillary raise on this kind of soils.

- **Highest tolerable groundwater level under buildings without creeping spaces.**

The highest tolerable groundwater position under buildings without creeping space is 0.50 meters below floor level. With a height of the floor construction of 0.15 meters above ground level follows from the above a criterion for the highest tolerable groundwater position of 0.35 meters under the ground level.

- **Lowest tolerable groundwater level below buildings founded on wooden poles.**

This criterion establishes that any part of the building’s wooden deep foundation poles should never run dry. If this happened, the wooden poles could be damaged and, as a consequence of this, also the building. To prevent any damage, the lowest groundwater position must be above the top of the wooden foundation.

- **Lowest tolerable groundwater level below buildings founded on steel poles.**
As a consequence of differential settlements, damage can act on buildings’ steel foundations. Differential settlement is generally a consequence of the increase of the granule tension in the bottom. This increase can be the consequence of an increased load on the ground. A decrease of the groundwater position has an increase of the granule tension as a consequence. When the ground has been properly compacted prior to the construction of the building, the differential settlements as a consequence of an increase in the granule tension will be considerably smaller.

Also, as well as for wooden poles, a decrease of the groundwater levels such that part of the pole is in contact with air and the other one with water, can lead to a fast corrosion of the pole with the consequent damage to the foundation and the construction.

- **Highest tolerable groundwater level below roads.**

  In connection with the location of cables and pipes (above the most acting groundwater position), thaw and stability loss, the highest tolerable groundwater position is 0.70 meters under street level. This criterion may temporarily be exceeded during wet periods.

- **Highest tolerable groundwater level under parking areas.**

  The highest tolerable groundwater position at parking spaces is 0.50 meters under street level. This is in connection with the location of cables and pipes, thaw and stability loss, but a less intensive use than public roads.

- **Highest tolerable groundwater level in gardens and parks.**

  Because of the required root depth of trees and bushes the most tolerable groundwater position in gardens and parks is 0.50 meters below ground level. Furthermore is of interest that the groundwater position can vary a little.

### 4.3.3 Highest and lowest tolerable levels in the area of study

From the previous given criteria, the most restrictive one for the highest tolerable level was the one based on the highest groundwater level below constructions with creeping spaces. In the case of the lowest tolerable level, the most restrictive criterion is the one related to the lowest tolerable groundwater level for steel founded constructions.

Due to small variations in the surface levels of the area of study, the highest tolerable groundwater level is between NAP-0.96m and NAP-1.02m. On the other hand, the lowest tolerable level is between NAP-1.27m and NAP-1.46m.
4.4 Observed behavior by previous studies

In 2001 a monitoring campaign was carried out to check the performance of the infiltration system and its effects on the groundwater levels (and quality). The results of this monitoring did not reveal a univocal relation between the infiltration system and the groundwater. Therefore, a new monitoring campaign was planned and carried out in order to revise the results obtained previously.

The new planned monitoring campaign was carried out between 2002 and 2004. In it, apart from trying to determine the effect of the infiltration facility on the groundwater positions and quality, also the optimization possibilities for the system were studied. The following conclusions were derived from the results of both studies:

- The levels at the infiltration facility follow the levels of the surface water. This is due to the lack of a no-return valve at the end of the system.

- Due to precipitation, the water levels in the infiltration system temporarily rise till above the surrounding groundwater levels because of the accumulation of precipitation towards the street whirlpools. However, these levels go down almost immediately because of the open connection between the infiltration facility and the surface water. This would not occur if a no-return valve at the end of the system had been installed as indicated in the original design of the infiltration system.

- From the measurements of the groundwater levels it is concluded that, during wet periods, the infiltration system works as drainage. In dry periods water from the infiltration system infiltrates towards the direct surroundings until, at most, 10m from the facility. The infiltrated water mainly comes from the surface system.

- After precipitation, the groundwater levels exceed the highest tolerable levels.

From the previous conclusions, some recommendations were done which can be summarized as:

- The temporary exceedance of the highest tolerable groundwater will be accepted.

- The inclusion of a no-return valve at the end of the system so the rain water can be held in the infiltration facility allowing it to infiltrate instead of the surface water.

- Increase the drainage level.

4.5 Considerations to the actual situation of the system
After almost 10 years of its construction and 5 since its performance was checked for the last time, it becomes interesting to revise both the performance and the operational principles of the infiltration facility. New needs and requirements in the water management of urban areas have arisen in the last years. Sustainability has become a primary objective rather than a side effect of urban actuations. Solving problems when they occur has evolved towards avoiding those same problems to happen by prevention of their possible causes.

Under this scope, the performance rules which were applied 10 years ago are no longer valid or, at least, less valid. A new approach becomes then necessary. In the specific area chosen for this research several lacks can be derived from the performance which was studied 5 years ago.

First of all it must be declared that the recommendations given for the system were not followed by the municipality of Delft, which can be checked by a short visit to the area of study. The drainage level setting device (remember the upward bended pipe) is not even installed, as well as the no-return valve has not been placed at the exit of the system (see previous section). Anyway, there are reasons which justify this actuation. Both, the no-return valve and the upward bended pipe represent possible conflicting points which can become against the good performance of the system if they stopped working properly by any cause. As long as avoiding risks is also a premise for water managers, not including these elements may seem a justified action. Anyway, this means that the water levels at the infiltration facility follow the surface water levels and the drainage level is set to the actual surface water level, whatever it is at any moment. When a rain episode takes place, all the surface runoff generated ends up in the infiltration system and, from there, because of the lack of anything which delays this runoff, it goes directly to the surface water system. Taking into account that this is just a small part of a larger system and considering that, most likely, all the other subsystems have the same lacks as the studied one; a strong precipitation occurrence can have as a consequence the overload of the surface water system.

High groundwater levels have a negative effect on buildings both from the health and economic point of view for the householders. Water not only creates problems because of excessive moisture levels in houses but also deteriorates the materials the house is made from. Because of this, high groundwater levels should be avoided at all the possible occasions, thing which is not achieved currently in the area of study.

Excessively low groundwater levels have not been measured in the area. Even though, their actual inexistence does not mean that, in a future, they can not be reached. Because of the rainy environment of the Netherlands this may not seem a problem but, many countries deal with periods of extreme precipitation followed by periods of severe droughts and the actual theory of climate change foresees that this kind of behavior will affect more and more areas in the upcoming years.
Chapter 5

Methods and Measurements

5.1 Analysis of the system with System Identification

It has already been stated previously that, to be able to control the groundwater levels within an area, a model of the whole process must be obtained. In this model the main driving forces of the process must be included.

In previous chapters it has been commented already the difficulties obtaining a physical model\(^7\) suppose. Although creating a physical model has a strong connection with the theoretical concepts and allows gaining insight of the system behavior, the main inconvenient is that it is necessary to include, in the model, many parameters that have to be determined and calibrated from theoretical and/or experimental results. Errors in the parameters determination can result into wrong models which performance is completely different to the expected or needed. Besides that, the amount of measurements and the time required to do them may be too large compared to the time available to obtain the model.

Another form to model dynamic systems is the use of what is called experimental modeling. This method allows obtaining a simple mathematical model based on the results of real experiments which inputs and outputs are measured. The technique to calculate the parameters of such a mathematical model is called System Identification. The substantial difference between physical and experimental modeling is that the second one skips most of the tedious, time consuming part of the modeling process. As a disadvantage, the insight on the individual processes influencing the system achieved with physical modeling is no longer possible or easy to obtain. Anyway, it should be possible trying to relate the mathematical model to the real world later on. Such mentioned differences are also shown in the following figure.

\(^7\) What is understood as a physical model can be found in chapter 3.
5.1.1 What is System Identification?

System identification is the general term used to describe mathematical tools and algorithms that build dynamical models from measured data. The aim of these tools is to find a model structure with adjustable parameters and then adjusting these so that the output of the model matches the measured output from the system.

The system identification procedure allows studying dynamic systems without having any idea about the process physics. In fact, system identification models do not need to have a physical meaning as long as their only purpose is to reproduce the behavior of the studied system. Due to the previous, the design of the experiments from which the system dynamics are going to be identified turns to be a very important step in the process. This is the same as saying that the selection of the proper inputs to the system, as well as the proper response from the many it can have, is a key factor to take into account when using system identification.

5.1.2 General System Identification Process

When thinking of using system identification to obtain a model from a certain process it must be assumed, first, that it can be described as a system $S$, where an input signals $u$ result into an output signal $y$. As in most of physical systems, an unknown disturbance $e$ which influences the output may be also present. A schematization of this is shown in figure 5.2.
The aim of the system identification procedure is to generate a model $M$ of the system $S$. This model is formed by a set of parameters $P$ which have to be determined (see figure 5.3). As long as the model will never match the real system exactly, an error $\varepsilon$ will remain between the model and the system. The way to find the best model passes by determining a set of parameter values which minimizes the error.

To carry out a system identification of a specific process, first, it is necessary to define what is going to be measured. This is usually called experiment design and it is done on the basis of the prior knowledge of the system and the application which is going to be given later on to the model. After this, the measurement campaign has to take place. Once the data which is going to be identified has been obtained, it has to be examined and preprocessed. Once this has been done, the next step is setting the model, this means, choosing all the elements (estimation method, model structure and model complexity) which allow finding the most accurate description of the process. Afterwards the best parameters for the prior setting of the model are calculated. Finally, the obtained model is validated to check whether it fulfills the requirements for its later application.

From the previous paragraph it may be understood that the system identification procedure is an iterative process with a lot of intermediate steps. Only after the conclusion of the last step is that the obtained model is good enough the process can be stopped. Otherwise, changes on the previous steps must be done, although the most common is changing the model settings.
Along the current section separate consideration will be given to each of the described steps of the process together with its application within the scope of this thesis.

5.1.3 Experiment Design

The main objective of using system identification under the scope of this research is to be able to describe the behavior of a groundwater system without knowing almost anything from it. To actually model a groundwater system, several measurements apart from the inputs and the outputs must be carried out. A map of the geology of the area must be done, as well as the hydraulic conductivity or the storativity must be determined in several points and, sometimes, even at different heights. All this makes the modeling process really time and resources consuming, apart from the possible errors which can be done during the measurement campaign, etc. By using system identification, only the inputs and the outputs of the system must be considered.

From the description of the area of study and the infiltration system in it, already some ideas of the inputs, output and measurements which should be done are clear. Of course, if the objective is controlling the groundwater levels within the area, it results obvious that they will be the output of the system and, therefore, they must be measured. From chapter 2, two basic inputs are also clear precipitation and evaporation, present at almost every hydrological process. Also, apart from the rainfall at a certain moment, the history of precipitation must be taken into account due to the dependency of the infiltration capacity of the soil on the amount of moisture content in it.

Finally, considering the infiltration facility as a boundary of the system and that the control implementation is supposed to be from its head/level manipulation, its effect must be clarified. Therefore, the head/level in the infiltration facility must be measured, and as long as it is connected to surface water system and follows its variations, the surface water levels must be also monitored so this dependency can be checked. Anyway, level at the pipe itself does not really represent an input to the process itself. As it has been seen in chapter 2, what makes water moving from one point to another is the head difference between those points. Because of that, measuring the head/level in the infiltration facility will be used afterwards to obtain the head difference between the infiltration pipe and the point where the groundwater levels are being measured.

As can be observed, the process to be described depends of several inputs. It is very important that these processes are independent because they must be added up later on. If the inputs are not independent there is risk that some of them are accounted more than one time. This is the case of the pipe levels which, as well as the groundwater levels, also depend on the precipitation because they are fed from the runoff generated in the surface. If the pipe head/level was fed directly into the system, precipitation would be accounted twice in the process. This is another reason to consider the head difference as an input together with the one given in the previous paragraph.
The duration in time of the experiment has also to be determined. Groundwater processes tend to be quite slow. A long measuring time will be needed to be able to find all the dynamics of the system. Due to the nature of the different inputs it will be more likely they influence the system with a different intensity at different moments. As an example, the potential evaporation will be higher when there are no clouds covering the sky but if there are no clouds then no rain events will take place. What is more, the input processes can not be controlled what means it is possible some of them do not occur for some time. Moreover, as has already been said, the objective is to model a system from which few knowledge about its dynamics exists. For all the previous considerations the measurement time should be as long as possible or, at least, as long as necessary for all the processes influencing the system to be identified. Anyway, because of the limited time for the development of this research, a two months period will be the measuring time.

The last choice which has to be made is the type of experiment. There are actually two kinds of experiments, open loop and closed loop. In the first kind, the output does not have any effect on the input. This is the case of precipitation and evaporation, they freely excitate the system and an independent response is obtained. In closed loop experiments, the output has an effect upon the input, as is the case for the head difference. Although this, the experiments will be carried out on the first form and corrections will be applied afterwards.

5.1.4 Setting the model

In order to obtain an accurate model, several choices must be done about its general shape. This includes the kind of model to be estimated, the estimation method, the model structure and the complexity which is desired for it.

Some authors (Fenicia 2005) have written about the linearity on the behavior of groundwater reservoirs. Under this assumption, linear modeling will be used to approximate the behavior of the system to study.

As it has been commented before, system identification allows estimating models without previous knowledge on the analyzed system, which is what almost occurs with the processes occurring in the area of study. When no knowledge at all exists is useful to go for black-box models, which are very flexible mathematical structures not based in any kind of principles. Of course, after analyzing the data some idea about the system dynamics is already acquired and maybe a grey-box modeling could be used. Anyway, as long as the dynamics may vary from site to site and a general method to identify this kind of groundwater systems is desired, this research will make use of the first modeling type.

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8 Actually the surface water levels and therefore the levels at the infiltration facility can be controlled by the water board but no access to that control has been obtained for the development of this research.
The next step on the model setting is choosing the structure of the model. A general model structure has the form:

\[ y_t = G_p(\theta) u_t + G_D(\theta) \epsilon_t \]  \hspace{1cm} (5.1)

where \( y(t) \) is the output of the system, \( G_p(\theta) \) is the transfer function of the input \( u(t) \) and \( G_D(\theta) \) is the transfer function for the disturbance \( \epsilon(t) \). The most general model structure is the prediction error model on which many other model structures are based. This model structure is described by the equation and represented in figure 5.4:

\[ A(z^{-1}) y_t = B(z^{-1}) u_t + C(z^{-1}) \epsilon_t \]  \hspace{1cm} (5.2)

where:

\[ A(z^{-1}) = 1 + a_1 z^{-1} + \ldots + a_{na} z^{-na} \]
\[ B(z^{-1}) = b_1 z^{-1} + \ldots + b_{nb} z^{-nb} \]
\[ C(z^{-1}) = 1 + c_1 z^{-1} + \ldots + c_{nc} z^{-nc} \]
\[ D(z^{-1}) = 1 + d_1 z^{-1} + \ldots + d_{nd} z^{-nd} \]
\[ F(z^{-1}) = 1 + f_1 z^{-1} + \ldots + f_{nf} z^{-nf} \]

and the parameters to be estimated are:

\[ \theta = [a_1, \ldots, a_{na}, b_1, \ldots, b_{nb}, c_1, \ldots, c_{nc}, d_1, \ldots, d_{nd}, f_1, \ldots, f_{nf}]^T \]

![Figure 5.4 General model and prediction error model structures](image)

By giving different values to the components of the transfer functions of the prediction error model several other model structures can be reached. The use of one structure or another depends on the system to be identified, its dynamics and the noise characteristics. Anyway, it seems interesting to test several of them to determine which one fits the investigation requirements better. After this trial process, the model structure which gave better results was the Box-Jenkins one.

Optimization of the rainfall-runoff response in urban areas by using controllable drains
The Box-Jenkins model structure considers that disturbance can have a completely different model from the process. This results into two different transfer functions with different parameters. Therefore, the Box-Jenkins model structure is a fairly general model which can catch all the relevant dynamics on the process transfer function while disturbances are described separately. All the identifications were done with this structure which representation can be seen on figure 5.5.

\[
A(z^{-1}) y_t = \frac{B_i(z^{-1})}{F_i(z^{-1})} u_{i,t} + \cdots + \frac{B_i(z^{-1})}{F_i(z^{-1})} u_{i,t} + \frac{C(z^{-1})}{D(z^{-1})} e_t
\]

where \( i \) represents the amount of inputs to the system. Trying multi-input modeling resulted to be a pretty difficult process which led to valid wrong results. As can be observed in equation (5.3), each input has its own transfer function of the process, independent of the rest. Because of this it was preferred to identify each input separately and add them later on.

To finalize with the model settings, the order of the model structure must be selected. This means deciding the amount of parameters to be estimated in order to obtain a model. The more the parameters to be calculated, the more complex the model will be. Using more parameters can lead to better results although there is also the risk that inexisten dynamics are modeled as well.

As it has been said before, the reason to be using linear modeling is the similitude of this behavior with the behavior of groundwater reservoirs. In order to attach to this theory and also to keep the obtained model as simpler as possible, the order of the models should be low or even one, what would result in a purely linear behavior as the theory claims.
5.1.5 Model construction and validation

Constructing the model means calculating the parameters of the selected model structure. This was done for the three different inputs by using the System Identification Toolbox for Matlab. Afterwards, the three obtained models were added up in a single model as in equation (5.3). Finally the resultant model was validated.

For the precipitation related inputs, the important dynamics are most likely found during wet periods. Therefore, the initial measurement period was used to calculate the parameters of the models for both of them. On the other side, to find the dynamics related to the head difference it resulted easier waiting for a dry period.

A first order model resulted enough to simulate the general behavior of the groundwater response to the excitation introduced in the system by each of the three inputs.

Afterwards, a single model from the previous three was created in Simulink (see figure 5.6). It can be observed how the head difference input depends on the model output. This has already been commented in section 5.1.3. Due to this, the gain value obtained from identifying the head difference as an open loop experiment was manually corrected to obtain proper results.

![Simulink model](image)

Figure 5.6 Simulink model gathering the three inputs together

The next step after the model is obtained is to use it to tune a controller which permits keeping the groundwater levels within certain limits. This tuning work requires the model to be able to simulate the dynamics of the system properly. It will not be that important if the model simulates larger or lower values of the groundwater levels because the controller can correct them afterwards. But if the system dynamics are not clear in the model, the performance of the controller will be inefficient.
It is important to point that system identification only allows identifying the dynamics of a system at a single point. This point will not represent a whole system the most of the times. That means a different model must be constructed for each point in the system which might be considered important for the general purpose of the project.

5.2 Real-Time control

5.2.1 What is real-time control?

The US Environmental Protection Agency defines real-time control (RTC) as “a system that dynamically adjusts the operation of facilities in response to online measurements in the field to maintain and meet the operational objectives, both during dry and wet weather conditions”. Under this definition are included all the practices and tools for actively managing the operation of a system. Of course, this definition is specifically given for the specific use of RTC in urban drainage networks although the application of RTC is broadly extended throughout the world for a lot of different processes.

RTC is used broadly to improve all kind of systems response and set them within certain operational objectives. The work flow of a RTC system usually is:

- Collect information about the state of the system
- Compare the actual state of the system with the desired state
- Determine the actions to be carried out (setting of the control facilities) to bring the system towards the desired situation
- Implement the previously determined settings on the control elements

The design of a RTC system includes several tasks such as deciding what the desired state of the system will be, design of the information network to keep the system updated, design of the control elements, or the design of the control algorithms of the system.

5.2.2 Generalities of RTC. Methods.

The control loop

An RTC process consists of several parts all of them connected in a loop also called “control loop”. The main components of this control loop are the system/process itself, the controller, the actuator and the sensor/s (see figure 1, annex D).

The system block represents the dynamics of the whole process and its outputs represent the response of that system to the inputs. The sensor measures the system
outputs which are of interest for the control process. These measured outputs are compared with the reference ones obtaining a deviation from the desired state of the system. This deviation enters in the controller which determines the control actions required to bring the system to its desired state. Finally, the actuators are physical devices which carry out the actions determined by the controller. As a result of the actuator decisions, a new input to the system is obtained so the process starts again in a closed loop. There are several variations to the control loop which will be considered in the following sections.

The whole control process must be supported by a communication system. Normally these communication systems are formed by electricity cables connecting the different elements of the process or by modems via the telephone network. New technologies such as mobile telephone communication or wireless networks will allow improve the communication process and, therefore, the implementation of RTC.

**Feedback control method**

Feedback control is actually the most important control method. With this method, the control actions are based on the desired state for the system (the control objective). The following block diagram represents the feedback method applied to the studied groundwater system.

![Figure 5.7 Feedback control applied to a groundwater system](image)

The objective of the controller is to keep the groundwater levels within the system at or around the reference level. To do this, the controller uses the deviation, calculated from the comparison between the target level and the measured level to determine whether water should be infiltrated or drained from the system by setting a certain level in the infiltration facility. This water level has a correcting influence on the system which, at the same time, is disturbed by rain, evaporation or other processes. The new water levels in the system will be measured again, compared to the target level, etc. This control loop is repeated with a fixed time step.

The most used kind of controller used in feedback control processes is the so called proportional integral derivative controller (PID). As can be understood from its name, this controller consists of three parts. The proportional part (gain) makes a change to the output that is proportional to the error value. The integral part (reset) is proportional to the magnitude of the error as well as to its duration. Finally, the derivative part (rate) changes the output proportionally to the rate of change of the error over time. All this can be summarized in the following equation.
\[ h_k(k) = K_p \cdot e(k) + K_i \cdot \sum_{l=0}^{k} e(l) + K_d \cdot (e(k) - e(k-1)) \] (5.4)

The gain values \( K_p, K_i, \) and \( K_d \) on equation (5.4) must be set prior to the implementation of the controller. These parameters depend on several factors unique to each system. Therefore, the most common practice is to set them by conducting an experiment on field or on a hydro-dynamic model. More on the determination of these values can be found in annex D.

**Feedforward control method**

This method bases its control actions on the level at which the disturbances to the system disturb the control objective. The input to the controller, in this case, is a measurement of the disturbance. To determine the effect of the disturbance a model of the system will be required. What the controller will do is to adapt the current state of the system to the upcoming disturbance so the error deviation is minimized. An example of feedforward control can be seen on the following figure.

**Figure 5.8 Feedforward control applied to a groundwater system**

It is obvious that this method requires very accurate measurements as well as a very accurate model to calculate the disturbance on the system. Under these two premises, feedforward control is a powerful control method that directly reacts to disturbances. As long as perfect measurements or models are impossible to achieve, errors will accumulate on time with the consequent erroneous performance of the controller.

The solution to this is to combine feedforward with feedback control. This allows correcting the errors introduced by the model and the measurements, achieving that the overall behavior does not deviate from the set point and, at the same time, obtaining a fast reaction.

**Other control methods**

In the search of optimized behaviors of systems, several control methods are developed continuously. An example of a recently used method is model predictive control (MPC). This is the name given to a family of methods which search to optimize the response of a system by implementing a model, which is updated by measurements and allows predictions to be made. At the same time, MPC methods try to optimize an
objective function under certain structure constrains. This allows obtaining a much more optimized response of the system than by just using feedback or feedback+feedforward control.

Some other control methods implement neural networks, which are trained to transfer a certain input signal to a certain output signal, or fuzzy logic, which translates vague sets of information into concrete control actions. More on this can be found in annex D and in more specialized literature which lies out of the scope of this research.

5.2.3 Applying RTC to the studied system

The main goal of this research, as has already been commented several times along this text, is to be able to control the groundwater levels within an urban area by controlling the head levels in the infiltration system contained in that area.

From the previous sections it can be concluded that applying a RTC system to the area of study will require carrying out several actions. Some of these actions are: the real time monitoring of the processes affecting the response of the groundwater levels within the area, the definition of a desired state for the area of study, to decide what kind of control will be applied, the proper tuning of a controller which is able to take the appropriate decisions to keep or to reach the desired state in the system, and the design of the control elements which will actuate on the system.

In the case of real time control of groundwater levels in urban areas, more specifically in the area of study, the control method to reach can be seen in figure 5.9. The objective is to keep the groundwater levels within certain limits by forcing water to infiltrate or to drain. For this, the controller should compare both the actual and the forecasted groundwater levels and the desired levels, and determine the actions (set the actuator performance) which are needed.
To answer whether the groundwater levels within the study area can be controlled or not, it will be only necessary to check if a simple control method can cope with this task. Any other method would just achieve an optimized behavior of the system which is beyond the reach of the actual research. Therefore, a feedback controller, the most common controller, will be tuned and simulated.

**The desired state for the area of study**

As long as what it is aiming to control are the groundwater levels in the area of study, it follows obviously that the desired state for the system will be defined by the higher and lower admissible groundwater levels. In the description of the area of study (Chapter 4), the upper and lower limits were already defined after considering all the criteria the municipality of Delft applies. As a reminder, those levels are between NAP-0.96m and NAP-1.02m as the highest tolerable ones and between NAP-1.27m and NAP-1.46m the lowest tolerable ones.

**A feedback controller for the area of study**

A model of the dynamics of the area of study was obtained in the previous chapter will be used. In this model, the only input which may be controlled is the head difference between the infiltration facility and the groundwater system, or what is the same, the head levels at the infiltration facility. Based on this model (figure 5.10), a new model was created changing the previous free pipe levels by a PID controller which would control them onwards. The result of this modification can be seen in the following figure.
Figure 5.10 Modified model with a PID controller

As long as one of the questions which are desired to answer is how far in space from the infiltration facility the groundwater levels can be controlled, and the further monitored point is the 28.129, the controller must to be tuned for the results at that point.

To keep the performance of the controller simple, only proportional gain was introduced in the controller settings so it introduces corrections proportional to the error (see section 6.2.2). As well, instead of defining two objective levels (objective function in MPC), an intermediate level was set as objective level (-1.15m NAP). The maximum response of the controller was set between the minimum level possible in the infiltration system (-1.30 m NAP) and the maximum tolerable average level in the area (-1.00 m NAP). The proportional gain value was obtained by trial an error till the behavior of the groundwater levels did not qualitatively improve anymore and the controller response was still logical. The resultant gain value was $K_P = -20$.

5.3 Measurement campaign

5.3.1 Monitoring of groundwater, surface water and infiltration facility levels

To measure all the water levels, a series of divers were placed in the area of study in previously perforated observation wells (see annex A to see all the monitoring wells within the area of study). After, the raw data was retrieved from them every fifteen days approximately by extracting them and reading in the field. Then the raw data was
preprocessed into groundwater levels. Because of the measurements were done every minute, a great amount of noise was present in the new acquired data so a low-pass filter was applied so a more real variation of the groundwater levels could be observed.

**Divers**

A diver is an electronic device which measures pressure and temperature and stores them to a certain capacity dependant of the frequency of the measurements. Communication with the logger is achieved with an intermediate unit which reads the diver data via infrared communication and sends it to the computer through a USB connection. Divers can be programmed in order to change the frequency of measurements or the date and time at when they should start measuring. After programming them and set to start, the divers announce by screen how much time is left till they run out of memory. This allows scheduling fieldworks in a quite efficient way.

Eight of these divers were placed in the area of study at the locations marked as red spots in annex A. The procedure of installation was similar to all of them but for the diver placed at point L15065. As long as the observation wells had been previously perforated for a different research, the only work which was done was to place the divers in their correspondent well. To do this, first, both the ground water in the well and its depth must be measured so a suitable length of the wire from the diver is going to be hanging can be selected. The diver should not rest in the bottom of the well but the risk that the water level goes below the diver level should not be taken either. As a rule of thumb, the total length (wire + diver) is usually taken as the 80% of the depth of the observation well. Finally, the diver is hung from the top of the well pipe. A graphic explanation may be seen in figure 5.11.

The installation of the diver at the point L15065 had to be done in a slightly different way. Point L15065 corresponds to the manhole at the entrance of the infiltration system. This means that no observation well was present nor could be perforated so an alternative solution must to be found. The solution adopted was to hang the diver from the manhole cover itself by tying the wire to a hanger shaped ledge used to lock the manhole if necessary.

Divers located at points with numbering 28.XXX, recorded groundwater levels. The diver placed at point L15065 recorded the water levels in the infiltration system, and the one at Peilbuis Emmabrug recorded the surface water levels. The eighth diver was used as barometer. To actually be able to obtain the water levels measured at the other
locations (remember that the divers measure pressure) the atmospheric pressure also must to be measured. Because of this, a diver programmed as a barometer was placed at the top of the observation well at point 28.128.

By placing the divers in this way, apart from monitoring all the water levels in the area of study, it can also be found out whether there is a relation between the surface water level and the groundwater levels.

The divers were programmed to do a measurement every minute what resulted in a recording time of 16.67 days. Visits to the field were done every 14-16 days to retrieve the data from the loggers and to program them to start measuring again the next half day hour (either 00:00 or 12:00). Although the groundwater processes are slow, it was considered that obtaining the maximum detail was useful towards the control objective. The data could be processed afterwards if less were necessary, thing which is impossible to do on the contrary direction if the measuring time period was longer.

**Raw data**

Once the divers had been read, an excel file could be obtained with the pressure in cm and the temperature in °C data for each programmed time step. These data can be represented in graphics like the ones which are presented next.

![Graphic results from the barometer diver at 28.128 (blue), piezometer diver at 28.128 (green) and piezometer diver at Emmabrug (red) in the period from 5-feb-2009 to 18-feb-2009](image-url)
Calculation of real groundwater levels

The next step is to convert the measured pressures into groundwater levels. In this conversion, the barometer plays a fundamental role.

To obtain the real groundwater levels, just an easy application of the principle of Bernoulli is necessary. As long as the groundwater in the area of study is at an unconfined aquifer, the water table will be equal to the energy at any point in the aquifer. The proof of this can be easily obtained.

The principle of Bernoulli for the analysis of water flow is expressed in equation(2.5). Remembering that the term of velocity can be discarded as long as the flow velocity term in a porous media can be considered very low compared to the other two terms, the equation can be rewritten as equation(2.6).

Taking a look to figure 5.13, the terms in equation(2.6) can be also rewritten so the equation(5.5) is obtained.

\[ P_p = \left( z_p - z_g \right) \cdot \gamma \]  \hspace{1cm} (5.5)

Substituting equation(5.5) into equation(2.6), the following relation is obtained:

\[ H_p = \left( z_p - z_g \right) \cdot \gamma \cdot - z_p \]  \hspace{1cm} (5.6)

so \( H_p = -z_g \)

As long as what is being measured is the pressure at the hypothetic point P, the groundwater level at each point can be obtained as:

\[ z_g = z_p - \frac{P_p}{\gamma} \]  \hspace{1cm} (5.7)

And, as long as the diver measures total pressure, the atmospheric pressure must be subtracted. Including that subtraction into equation(5.7), it yields:

\[ z_g = z_p - \frac{P_{dive} - P_{barometer}}{\gamma} \]  \hspace{1cm} (5.8)
Now that all the values in the equation are known, because they have been measured, this formula may be applied into an excel sheet to obtain the real groundwater levels. Note that all lengths are referred to a certain level. In this case that level is NAP. The following figure shows the results of the application of equation(5.8) to the raw data.

![Groundwater levels calculated in the period from 05-Feb-2009 to 18-Feb-2009](image)

**Figure 5.14** Result of calculating the groundwater levels at points 28.129 (blue) and Emmabrug (green) in the period from 05/02/2009 to 18/02/2009

### 5.3.2 Precipitation

Precipitation is probably the most important input to the system. As long as the groundwater levels were measured with a high degree of detail (due to the frequency of the measurement), detailed precipitation measurements should also be obtained. This will allow feed the system identification process with enough data to obtain good results.

Obtaining precipitation data with the required characteristics expressed on the previous paragraph is a difficult task. Sometimes is just a matter of time to pass through all the bureaucracy official organisms require to give the data. Some others is just a matter of money. Therefore, to skip both obstacles and, at the same time, obtaining useful rainfall data, an application to obtain it from internet based weather services (this means a website providing precipitation data in colored maps) was developed in Matlab. Annex B
describes the application and validates its results. In annex C, the Matlab code for the different m-files required to run the application can be found.

The rainfall is obtained every 5 minutes in millimeters per hour. In figure 5.11 the obtained precipitation for the two months of the measuring campaign is shown. The exact dates of the measuring campaign are from February 5th to April 9th in year 2009. These dates correspond to the end of the winter and the beginning of spring, usually wet periods in the Netherlands, although 2009 has begun as a pretty dry year as can be seen in figure 5.11.

![Figure 5.15 Precipitation data obtained for the area of study from 05-Feb-2009 to 09-Apr-2009](image)

### 5.3.3 Evaporation

The KNMI data base was used to obtain the evaporation values. As long as not all the stations measure evaporation, the nearest one to the area of study had to be used. In this case, that station is the one in the Airport of Rotterdam.

The evaporation data measured by the station at the Airport of Rotterdam are daily potential evaporation values obtained by the Makkink method in mm/d. A representation of these data can be found in figure 5.16.
5.4 Preprocess, data analysis and calculation of inputs

It can be appreciated that all the data obtained have a different measurement period and units. To actually perform a good system identification of the process, as well as to obtain a model with standardized input data, it is better to define a format common to all the data before carrying out the identification procedure.

The time step will be set to 15 minutes, which still allows a detailed analysis while considerably reduces the amount of data to be fed in the identification process. For the units, groundwater levels will still be meters and precipitation and evaporation will be expressed in millimeters as is the common practice in hydrology. To achieve the defined format, different preprocessing techniques will be applied to the data.

5.4.1 Groundwater, surface water and infiltration facility levels

Filtering

As can be observed in figure 5.14, the measurement of the water level every minute results in a lot of noise. This noise, apart from hiding the real behavior of the
signal, would make the control process very difficult. If, at any moment, the groundwater levels got to a level near the set point and that noise made the set point to be exceeded alternatively, the controller would perform in a very inefficient way or would even get unstable. Also the existence of some outliers can be checked.

To achieve a more efficient results and performance of the controller, it is necessary to filter the groundwater levels results obtained in the previous points. To do this, a low pass filter was used.

A low pass filter is a filter that passes low-frequency signals but attenuates (reduces the amplitude of) signals with frequencies higher than the cutoff frequency.

The filter used in this case was:

$$y_f(k) = f_c \cdot y_f(k-1) + (1 - f_c) \cdot y(k)$$  \hspace{1cm} (5.9)

with \( y_f(0) = y(0) \), where \( y \) is the unfiltered signal, \( y_f \) is the filtered signal and \( f_c \) is the filter constant.

![Groundwater levels at point 28.127 before and after applying a low pass filter with fc=0.99](image)

Figure 5.17 Result of filtering the groundwater levels at point 28.127 in the period from 05/02/2009 to 18/02/2009 with fc=0.99
To avoid aliasing, the value of $f_c$ must be chosen higher than 0.667. The smoothness of the data increases with $f_c$ although some of the original behavior is lost during the filtration process. Equilibrium between smoothness and behavior maintenance must be achieved. After some trials, the value which gave the smoother result while preserving the original behavior is $f_c=0.99$. As can be observed in figure 5.17, the filtering process also removes the outliers present in the unfiltered data without affecting the general behavior of the process.

**Resampling**

As long as the measuring period for the water levels is one minute and it has been decided to reduce it to 15 minutes, a resampling of the data must be carried out. To do this the “decimate(x,r)” command in Matlab was used. Decimation reduces the original sampling rate for a sequence to a lower rate, the opposite of interpolation. The result is a data vector 15 times shorter than the original which keeps the general behavior of the water levels fluctuation as can be appreciated when figure 5.18 is compared to figure 5.17.

![Resampled groundwater levels at point 28.127](image)

**Figure 5.18 Result of applying decimation resampling to the filtered groundwater levels of point 28.127**

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9 In signal processing and related disciplines, aliasing refers to an effect that causes different signals to become indistinguishable (or aliases of one another) when sampled. It also refers to the distortion resulting when the signal reconstructed from samples is different than the original continuous signal.

10 `decimate(x,r)` reduces the sample rate of x by a factor r. The decimated vector y is r times shorter in length than the input vector x.
Interpolation

As long as the experiment was decided to last for two months, four different data sets (see section 5.3.1) had to be put together. Due to the scheduling to retrieve the data from the loggers, some periods were missing between data sets. To solve this, the missing data were interpolated from the measured values. There are actually several ways to interpolate data, although to avoid undesired or weird results from the interpolation linear interpolation was used.

5.4.2 Precipitation

As well as for the water levels data, the rainfall data does not match the requirements defined to apply the system identification. The internet based data obtained is expressed in millimeter per hour and measured every 5 minutes while the desired one is millimeters per 15 minutes every 15 minutes.

Resampling

Using decimation to resample the precipitation data is not advisable in this case because the total amount of rain on the 15 minutes should be maintained to achieve a correct identification. What will be done instead is; first, divide the precipitation values by 4 so they are expressed in millimeters per fifteen minutes. Afterwards, the obtained data will be averaged in blocks of 15 minutes (see figure 5.19 for better understanding).

5.4.3 Evaporation

As the measured values are in millimeters per day, they will have to be divided by 96 to have them in millimeters per 15 minutes.
Filtering

Figure 5.20 shows a highly fluctuating behavior of evaporation from day to day. As long as the measuring frequency is the lowest of all the measured processes, it would not be advisable to give it too much importance, or weight, into the identification process. Even though, it is still considered useful to, at least, count with the general behavior of evaporation during the experiment. In order to keep that general behavior a low-pass filter (see section 5.4.1) can be applied to the evaporation data so a more averaged result is obtained. The result of this filtering can be seen in figure 5.15 for $f_c=0.8$.

![Filtered evaporation values vs unfiltered values](image)

**Figure 5.20** Filtered and unfiltered values of evaporation

Interpolation

As long as the evaporation data were obtained daily, an interpolation must be done to have evaporation values every fifteen minutes. To keep it simple, a linear interpolation was done.

5.4.4 Calculation of the data to be used in system identification. Analysis.

The measured processes are just the ones which could be measured although in section 5.1.3 was stated that the identification process would be carried out with slightly different inputs which can be calculated from the obtained data. Also a look to the
obtained groundwater data can help to refine the selection of inputs for the system identification.

**Groundwater levels**

They represent the main focus of the research and the output to be simulated. Already having a quick look to them together with the rest of the measured processes already clarifies how they can relate to each other. In figure 5.21 can be appreciated how a rain episode will result in an increase of the levels (day 5 or day 50) and a period without rain, or very few rain, will make the levels to decrease until they reach the minimum level imposed by the water level at the infiltration facility/surface water system (from days 15 to 25 with few exceptions or after the wet period around day 50).

![Groundwater levels at point 28.129 vs Precipitation](image)

Figure 5.21 Groundwater levels versus precipitation for the whole measurement period

Some other processes may result difficult to observe with a simple look to the data. Like evaporation, which has really low values what make almost impossible to appreciate its influence even though it is there. As in function analysis in mathematics, to find out more about the processes affecting the groundwater it becomes useful to study the first derivative of the process. If this is done to the measured groundwater levels, the result is what can be seen in figure 5.22.
Figure 5.22 First derivative of the groundwater levels at point 28.129 vs precipitation

In this view it can be appreciated how rain episodes make the groundwater levels to quickly increase and then decrease more slowly. Figure 5.23 gives a closer look to this explanation. Although it is an effect which can already be derived without looking at the first derivative, it is easily appreciable how the increase of the groundwater levels (positive values of the first derivative) occurs in less time than the decrease (period with negative values of the first derivative).
If now attention is paid to dryer periods, for example between days 55 and 59, the effect of evaporation during the day can be observed (see figure 5.24). The evaporation effect is very small but existent. In fact, if a step further is taken and the data spectra of the first derivative is studied, a significative peak is found for a frequency of $10^{-4.14}$ Hz, which corresponds to a period of approximate one day. This agrees with the decision of taking evaporation into account for the identification process.
That precipitation and evaporation are essential factors in any hydrological process has been probed by several authors already. Moreover, it has been shown in the previous paragraphs how both processes are present in the response of the system. The question which arises now is whether they should be considered as separate processes affecting the system in a different form each one or, on the contrary, they can be counted as a single process in the form of net precipitation (precipitation – evaporation).

Taking into account that the measured evaporation represents potential evaporation, its measurement is only once a day, while precipitation and groundwater levels are measured more often, and its values are quite low as well, it seems a good solution to choose the second option rose in the previous paragraph. Doing this, the net precipitation will have the form shown in figure 5.25.

**Head difference**

Groundwater movement only takes place when a head difference between two points exists. This head difference was obtained by calculating:

\[
\text{head difference} = \text{gwl} - \text{pipewl}
\]  

(5.10)
Where $gwl$ are the measured groundwater levels and $pipewl$ are the measured levels in the infiltration pipe. The results from this operation allow to know the direction the flow takes (from or towards the pipe) and even to quantify this flow\textsuperscript{11}. An example of this can be found in figure 5.25.

When, in figure 5.25, the head difference value is below zero, the water flows from the ground to the pipe (drainage). If the head difference value is positive instead, the water will go from the infiltration facility to the soil. This head difference graph can be compared with the net precipitation during the same period (same figure 5.25). If this is done, it can be easily seen how, during wet periods, the infiltration facility acts as drainage and, when a dry period occurs, infiltration takes place.

**Historical precipitation**

From the comparison of the net precipitation and head difference graphs (at figure 5.25), also the importance of the rainfall history is reflected. At the beginning of the measurement campaign (from day 0 to day 21 approximately) a fast decrease of the head difference, hence a fast increase of the groundwater levels, due to several rainy days, though without a really high intensity, can be seen. More well, after a dry period (from day 31 to day 47 approximately), during which the levels at the infiltration facility went above the groundwater levels, it can be observed how a more intense but shorter rainy period than the one at the beginning has a weaker effect on the groundwater levels. This evidences the importance of the historical precipitation. Therefore, it will be necessary to define this input value from the obtained rainfall series.

The period of influence of previous precipitation events is related to the residence time of the water in the system before leaving it. This residence time is not known for the area of study, as estimation, a value of one week will be used for the identification. The historical precipitation can be also found in figure 5.25.

\textsuperscript{11} Quantification is possible as long as the flow is one dimensional. Because this is not the case, this quantification would be just an approximation.
Figure 5.25 Net precipitation, head difference and precipitation history from 05-Feb-2009 to 09-Apr-2009.
5.5 Validation of the influence of the infiltration facility on the system

5.5.1 Design of a simple actuator

In order to check whether the proved into theory with system identification and real-time control also works into practice, and to allow further investigation, a taylor-made actuator for the system was designed and built. This actuator will also allow validating the transfer function associated to the infiltration facility influence in the model obtained with system identification.

The requirements for the actuator were:

- Being able to feed water to the system or to extract it depending on the orders of the controller
- To be waterproof so the mechanical components inside it do not get wet
- Not to deteriorate in time so no extra pollutants are introduced in the system

Under these requirements, being the first one the most important, the adopted solution was to design an axial double direction pump.

By using an old fishing boat propeller and a simple 220V AC motor, the construction of the device shown on figure 5.26 was possible. The design details of the device can be found in annex E.
The propeller turns thanks to the action of a 220V motor installed in the upper part of the device which transmits its force to the bar through a transmission belt to a reduction wheel.

5.5.2 Design of the experiment

To check the influence of the infiltration facility on the system, the actuator will be installed inside a manhole of the infiltration facility. The propeller should be placed a few centimeters inside one of the infiltration pipes while, in the other end, a closing valve or a tap should be installed to not affect the whole system as long as the experiment will be carried out only at one of the reaches of the infiltration facility. The section where the experiment will be done is the one between points L15075 and L15071, being the device installed in the second one. In this way, the effects of the experiment should be measured in points 28.127, 28.128 and 28.129.

Water will be pumped in and out at the maximum capacity of the device in cycles of two hours. Therefore, the variation of the water levels in the infiltration system should be reflected in the groundwater levels later on.

Afterwards, the measured levels at the infiltration pipe and in the ground will be used with system identification to obtain a transfer function of the process. The obtained transfer function should be similar to the one obtained from the system identification process carried out previously.
Chapter 6

Results and discussion

6.1 Model obtained with System Identification

Figure 6.1 shows the results of the simulation versus the measured groundwater values measured at point 28.129. It is easy to see how the simulated levels follow the real ones, what indicates that the dynamics of the system have been properly captured by the model, the levels increase when there is a wet period and decrease when there is few rain or not rain at all. Anyway, the errors present in the model have a more quantitative nature. As can be seen in figure 6.1, the peaks are not well explained by the model. This is probably due to the quality of the rainfall data. It can be observed, in annex B, figure B.5, that the radar obtained values for precipitation mostly fail measuring high intensity rainfall episodes. Therefore, it makes sense that the peaks are not well represented as the real value of the precipitation generating them is higher. Nevertheless, it can be concluded that the rainfall part of the model works properly.

It can be discussed now that the peak after day 50 is higher in the model than the measured one. It can be observed that the amount of rain fell around day 50 seems more intense than the rain fell around day 7 (the first peak in the graph). Anyway, as long as the peak after day 50 occurs after a pretty dry period, it is logical that the groundwater levels reach a lower peak level as there is more empty space in the ground to fill. Taking also into account the considerations made for the precipitation data in the previous paragraph, it makes sense thinking maybe the cause of this problem lies on the precipitation history part of the model. The historical precipitation has been considered to be represented by the sum of all the precipitation occurred during the previous week for each time step. If this form to represent it is correct the solution might be increasing or decreasing the period for which the rain is summed until better results are obtained. It might be also possible that the way the historical precipitation has been considered is not correct and then another form should be found.

Even though the previous considerations, the model is considered good enough to allow tuning a controller in the next section.
Once a model which simulates the system properly has been obtained it is time to introduce the element of control and check what would be the results if this was applied in reality. As said in section 5.2.3, only a feedback controller will be used.

As can be observed, comparing figures 6.2 and 6.3, the controller will force full drainage during wet periods while gradually allowing the infiltration in the dry periods. It can also be concluded that the influence of the infiltration facility on the system is of importance. This means that by changing the head levels in the infiltration system,
control over the groundwater levels in the area can be achieved. It also results interesting to observe how a simple feedback controller already improves the behavior of the groundwater system bringing the peak levels below the maximum tolerable level.

Because the state of the system prior to applying the control is not known, the resultant controlled groundwater levels at the beginning of the period may not be representative of the real work of the controller. Anyway, it is clear that the controller pulls the levels down as much as the limitations imposed on its functioning allow it (see section 5.2.3). Therefore, the observed peaks of the controlled levels show the amount of water the controller can not cope with. The drainage level of the system cannot be lowered as long as the facility was built years ago. Anyway, introducing an objective function with a higher and a lower tolerable groundwater levels, instead of just one reference level, would already make the levels to be more under control.

Figure 6.2 Controlled vs no controlled groundwater levels at point 28.129
In figure 6.6, the response of the controller is shown. It must be pointed that this response signal is the one used as control, what means that it is an ideal response. In reality, this signal must be created by the controller; this means that, by switching on and off, the controller should achieve the shown levels at the infiltration facility. Therefore, the shape of the levels in the infiltration facility after a controller is introduced will be very similar to the one shown, but also will show the on/off periods of the controller.
Chapter 7
Conclusions and recommendations

7.1 Conclusions

In the second chapter of this text, after some basic considerations about groundwater movement, it was shown that these same considerations allowed thinking that the groundwater levels might be controllable, at least, in certain cases. Therefore, the arising question was whether the groundwater levels, within an urban area, were susceptible to be controlled by the use of real-time control.

After this research, the answer to the question asked in chapter 3 is yes, the groundwater levels in an urban area can be controlled in real time. Of course, this does not mean a categorical positive response, there are limitations. As it was explained at the end of chapter 3, the soil characteristics play a fundamental role on this process and while, at some places, this control would be beneficial, at some others, the response of the system might be unappreciable. In the same way, the cost is also a factor to take into account.

The distance of influence of the control will depend, again of course, of the kind of soil. The higher the conductivity, the further the water will go and in less time. The further measuring point available in the system was used to check how far the influence of the infiltration facility could reach. Although this point is not even half way between two of the branches of the facility (see map in annex A), it is possible to think the influence of the controlled levels at the infiltration system can reach further distances, at least, in the area of study. Anyway, a form to achieve all the points of the system are under control would be to reduce the distance between the infiltration pipes.

About how much time in advance the levels might be controlled it can only be said that it all depends on how good the prediction of the disturbances (the rain) is. This prediction will depend both on the reliability of the precipitation prediction and the accuracy of the model. Anyway, if this time is too short, probably, there will net be time enough to adapt the system to the coming disturbance. Therefore, the decision of the
amount of time in advance the system is going to be controlled must rely on a compromise between controllability of the system and reliability of the predictions.

The simulation of a feedback controller has shown how the peaks at the groundwater levels within the area of study are decreased. The negative part of this is that the water is taken out of the system at the same time precipitation is occurring. This might overload the surface system with the consequent undesired effects. Optimizing the control method, by including feed forward for example, would allow decrease the peaks by taking water out of the system before the precipitation occurs. This would lead to an optimized utilization of the urban water system by the reduction and shift of the peaks in time.

Finally, it has been also shown that system identification is a useful method to identify the dynamics of a groundwater system. Moreover, the process followed to identify the system in the area of study can be applied to other systems later on as long as all the elements involved can be found on all the systems with similar characteristics.

### 7.2 Recommendations

Independently of the positive results of a research, there are always aspects which can be improved or studied more in deep towards a better understanding of all the processes involved. Under this assumption, a few recommendations are given with the hope they are followed if this research, or related to it, is carried out later on.

The identification process followed should be tried in more sites. This would validate and improve it so an even more general procedure is obtained. As well, longer data series would probably lead to better results of the identification process. Longer data series also allow acquiring more insight on the involved processes. It would be also interesting revisiting the representation of the historical precipitation, and obtaining a better representation form.

Only a feedback controller has been used for this research. The next step lies on the optimization of the groundwater levels control. More advanced methods should be tried so it is possible in the end optimize the response of the system. For example, storage can be created by decreasing the groundwater levels to their minimum before a storm event occurs.

Finally, it would be also interesting to explore the possibility of controlling the groundwater levels by controlling the surface water levels, as long as in the area of study the infiltration system levels were the same than the ones at the surface water system.
References


US Environmental Protection Agency(2006) *Real time control of urban grainage networks, EPA/600/R-06/120*. Washington, DC, USA


*Optimization of the rainfall-runoff response in urban areas by using controllable drains*
Annexes

Annex A
Plane of the area of study

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Annex D
Real-Time Control Applied in Dutch Water Management (by P.J. van Overloop, Delft University of Technology, Delft, The Netherlands)

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Annex B

Obtaining precipitation data from internet based weather services

B.1 Introduction

Calibration and operation of models created to simulate hydrological processes, such as rainfall-runoff response or variation of groundwater levels in certain area, require the use of real and reliable precipitation data.

Many companies and authorities provide precipitation data from their stations which, most likely, will not be at the location we are interested in. Due to the spatial variation of rain this data may be useless or lead us to obtain inaccurate results. Anyway, the installation of a measuring station at each location of interest is not the solution because it is not cheap and sometimes not physically possible either.

Actually there are internet websites which provide the public with real and predicted precipitation maps, apart from some other atmospheric processes, which cover large areas (buienradar.nl covers the whole Netherlands and Belgium, and part of Germany and the North Sea). The question is: can those maps be used to obtain precipitation data at our particular location? And, if they are, how reliable the obtained data is?

The particular website from where the data are going to be obtained is www.buienradar.nl, and the way to obtain it will be with a Matlab script.

An advantage of this website is that it provides with data every 5 minutes which is very useful due to the every minute measurements of water levels. Also from the point of view of real time control, this data provision frequency is very valuable.
B.2 The Matlab script (how it works)

The buienradar.nl website provides a picture, so the location of our area of study is the first step and already the first difficulty. The picture is a .jpg archive of 513x550 pixels representing an area of 1x1 km² each. Finding the particular location of the research will be already an obstacle as big as small the area of interest is. In this case the area of study is very small (less than 1 km²) so only one pixel from the picture is needed. To allocate this pixel a process of trial and error was done using Google Earth. As long as in the picture the location of the cities of The Hague and Rotterdam is represented, a guess about where the Delft pixel should be can be done. After, the color of that pixel is changed to black also with Matlab and the image is exported to Google Earth where it is superimposed and properly allocated over the Netherlands. Also the city of Delft has been marked in Google Earth (see figure B.1) so when the pixel falls under the mark, the proper location can be considered found. If the area of study was larger, a higher amount of pixels should be taken.

Figure B1. Pixel allocation with Google Earth

Once the area has been defined, the next thing to do is to translate the color of those pixels in the map into a value. The amount of precipitation in each pixel of the obtained picture is represented by a certain color from a scale which can be found in the same site. A translation of the color scale into values is necessary so each tone has an assigned value. The color scale goes from 0 mm/h to 200 mm/h. Then the color of each selected pixel is compared with the scale and its correspondent value is stored. It can be observed that there are some colors representing the ground, the sea, the provinces boundaries and some important cities, which do not belong to the color scale. When these colors are present it also means that no precipitation has taken place in that location so a
zero value is given. The precipitation value in that location at a certain time has already been obtained.

If a time series is desired then the previous process must be repeated from the time the first precipitation value is needed till last date required.

**B.3 Results**

A run of the Matlab script was done for each period of groundwater measurements so a relation between the rainfall and the groundwater levels could be found. The following figures show the precipitation values every five minutes obtained after some of the runs.

![Obtained precipitation data for the period from 18/02/2009 to 06/03/2009](image)

*Figure B.2 Obtained precipitation data for the period from 18/02/2009 to 06/03/2009*
B.4 Validation (comparison with real measured data)

To check the reliability of the obtained rainfall data, a comparison between them and in-site measured data was done. The longest time series as possible was selected taking into account the availability of measured data and the first available data in the server of the website. Finally, the period from the 19th of June of 2008 to the 31st of December was chosen. The units of the daily in-site measured data are mm/hour, so that the internet obtained ones were daily averaged for a better comparison.

The source of the in-site measured data is the daily precipitation data provided by KNMI for the municipality of Delft.

The following figure shows both data series together.
From a simple view of both graphs, the differences between them can already be observed. Even though the precipitation episodes seem to take place at the same time, the amount of rain measured in-site is much larger than the radar measured one, mostly in the peaks.

A simple calculation allows knowing that the amount of days with precipitation is the same for both data series, what starts confirming the observation that the rain episodes take place on the same dates. Moreover, if a more accurate comparison is done, the result is that the rainy periods do not exactly match in time for both series. Anyway, in a larger scale, they can be considered similar.

To analyze the reliability of the radar obtained data, the difference between the observed values and the measured ones (residuals) was studied.

First, it is interesting to know how large these residuals are and how large they can get to be (statistically speaking). The mean error in this series is 0.04 mm/h with a standard deviation of 0.19 mm/h. A representation of the errors can be found in figure B.6. Taking into account an approximated precipitation value of 790 mm/year (this makes 2.16 mm/d) and the approximate maximum and minimum residuals, which will be established as the superior and inferior limit of the representation of the 95% of the residuals (0.42 mm/d and -0.34 mm/d), it yields that the radar based internet obtained data may be till about a 20% lower or a 15% higher than the real precipitation. Making a
rough pessimist approximation, it could be said that a 25% difference between real rainfall and measured may be reached, though with a low probability of occurrence.

![Figure B.6 Residuals calculated as the difference of the measured data and the radar observed data](image)

It is also interesting to know whether the residuals are dependent on time. This would mean that the residuals are generated by a recurrent error in time and, therefore, the obtained data would be less valid. To check it a good option is to calculate the autocorrelation of the residuals. To do this, the following hypotheses about the residuals must be done:

- There is a real time dependent function of the rain which can be called $g(t)$.
- Both the in-site measurements (which are being considered as the real values for precipitation) and the radar based ones obtained from internet can be considered a linear combination of the real function of the rain plus an error function dependent of the method and the time. Taking this into account it can be written:

  Gauge values: $g_1(t) = g(t) + \epsilon_1(t)$
  Gauge values: $g_2(t) = g(t) + \epsilon_2(t)$

- For both error functions it can be considered that the mean is zero, the standard deviation has a certain value and their autocorrelation is null. Under this hypothesis it yields:

  $$g_1(t) - g_2(t) = \epsilon_1(t) - \epsilon_2(t) = \epsilon_d(t)$$
And $\varepsilon_d(t)$ is the previous calculated residuals. The less autocorrelated the values of $\varepsilon_d(t)$ are, the more the residuals can be considered white noise and, consequently, the radar internet retrieved data reliable.

![Residuals autocorrelation](image)

**Figure B.7** Residuals autocorrelation together with the confidence interval (Anderson limits)

By observing figure B.7 can be concluded that, even though some initial values are slightly out of the autocorrelation limits, the residuals can be considered random.

### B.5 Discussion

It was easy to foresee that exact results for the precipitation data would not be obtained. Anyway there are some uncertainties which may play for and against the reliability of the results.

First, the allocation of the pixel representing the area of interest may result a difficult task. Better results could be achieved if the coordinates of the center of each pixel were known. The main problem this faces is that the provided map represents more than one country at the same time. As long as each country has its own coordinate system, giving coordinates to each pixel in the map would not be an easy task. The adopted solution of superimposing the map on Google Earth has also its own limitations. The map retrieved from the website is not really exact and the references used to match it with the map in Google Earth do not really match as much as it would be desired. In fact, due to this inaccuracy, only the boundaries of the province of Zuid-Holland, where Delft is located, were matched with Google Earth (as was already shown in figure (1)). The superimposing process also introduces deformations in the superimposed map as well as the process of converting the pixel image into a jpg format file. As long as the
precipitation values vary from pixel to pixel, as well as in reality already exists an important spatial variation in precipitation, the selection of the pixel will already induce an error in the obtained data. Anyway, this effect may be mitigated if a large area is studied. As long as the area of interest grows further than the size of just one pixel the adverse effects of pixel allocation will probably decrease. As long as the area of interest for this research is smaller than the size of one pixel, the results for larger areas are out of boundaries at this investigation and will not be developed any further. However, this question remains open for future investigation.

Second, the limitations associated to weather radars. Radar data interpretation depends on many hypotheses about the atmosphere and the weather targets. Some of them are: International Standard Atmosphere; targets are small enough that they obey the Rayleigh scattering so the return is proportional to the precipitation rate; the volume scanned by the beam of the radar is full of meteorological targets (rain, snow, etc…), all of the same variety and in an uniform concentration; there is no attenuation or amplification; there is no return from multiple reflections. It has to been kept in mind that these hypotheses are not necessarily met in many circumstances so many interpretation errors may be done when translating the raw data to precipitation maps. Of course, these errors are calibrated before the radar starts working continuously but they can not be completely removed.

Of course, also errors may occur due to the performance of the rain gauge measuring the real precipitation. Even the allocation of the measuring device can induce some errors if it is not in a proper place.

Because of all the possible causes on which the differences between the radar data and the in-site measured ones, an analysis of the residuals was done (see previous point) and its results reflected that these residual were independent in time, what already makes a point for the reliability of the obtained data. A major cause of concern about the reliability may lie on the size of the residuals. As has already been calculated, some of the observations may present about a maximum deviation of 25% from the measured values. This value could be considered unacceptable for some applications where accuracy played an important role. In the particular case of this research, these precipitation data will be applied to tune a predictive control system. This means the controller will work on the basis of predicted values but will also perform corrections based on actual real values. This allows relying on the internet obtained data knowing the controller can correct the situation.

B.6 Conclusions

The main question at the beginning of this point was whether it was possible to obtain reliable rainfall data for a specific area from the precipitation maps provided by a weather website.
For the part referent to the data obtaining it is clear that, by using a not too complicated script for a common program for engineers and researchers, it is possible to retrieve precipitation data from internet at any location which is represented in the website. From here, the problem will be to find a website which provides data at the location we are interested in.

About the reliability it can be said that, for the case of study presented, the results were reliable enough for the purpose they were going to be used for. Anyway, it must be analyzed whether, for any different application, the internet retrieved data is reliable or not.
Annex C

Matlab scripts to obtain precipitation data

C.1 Main script

close all

BeginDate=...
    input('Insert the beginning date and time(yyyymm-dd hh:mm): ', 's');

EndDate=...
    input('Insert the ending date and time(yyyymm-dd hh:mm): ', 's');

DateTime=BeginDate;

% First web image request
    RequestString=...
        [['B, map]= IMREAD(''http://www.buienradar.nl/'', ...'
            'images.aspx?soort=1x1h&Jaar=','BeginDate(1:4),&maand=','...
            'BeginDate(6:7),&dag=','BeginDate(9:10),&uur=','...
            'BeginDate(12:13),&minuut=','BeginDate(15:16),&bliksem=0''');

% Evaluation of the string
    eval(RequestString);

    iImage=0;
    DateTimesSelected='';
    DateNumber=datenum(BeginDate);
    counter=1;

    % B is a matrix of the size in pixels of the image
    while DateNumber<datenum(EndDate) && size(B)==[513,550]
        iImage=iImage+1;
        if size(B)==[513,550]
            % If there is no image available a value of zero precipitation
            % is given to that time step
            HOEVEELHEID(:,iImage)=VERDELINGTOT(1);
Nodata(counter)=DateNumber;
counter=counter+1;

else

%The obtained GIF format image is transformed into RGB format
RGB = ind2rgb(B,map);

%Selection of the desired area
GIF=RGB(287:1:287,246:1:246,:);

%Graph with the colors of the selected area
colormap(map);
image(GIF),title(DateTime)

%Color of the selected pixel/s
D=[GIF(1,1,1)];
E=[GIF(1,1,2)];
F=[GIF(1,1,3)];

%Obtention of the color in RGB
RED=D*255;
GREEN=E*255;
BLUE=F*255;
KLEUR=[[RED.' ,GREEN.' ,BLUE.']] ;

%Looking for a match in the legend
LEGENDA0_2
LEGENDA2_5
LEGENDA5_10
LEGENDA10_100
LEGENDA100_250
LEGENDATOTM
LEGENDATOT=[[0 0 0];LEGENDATOT];
[tf,Match] = ismember(KLEUR,LEGENDATOT,'rows');

if
    Match==0
    HOEVEELHEID(:,iImage)=VERDELINGTOT(1);
else
    %Amount of rain measured
    HOEVEELHEID(:,iImage) = VERDELINGTOT(Match);
end

end

%Request of the next image
DateNumber=DateNumber+1/288;
DateTime=datestr(DateNumber,30);
RequestString=...
    ['[B,map]= IMREAD(''http://www.buienradar.nl/','images.aspx?soort=1x1h&jaar=' ,DateTime(1:4), '&maand=',...
        &dag=',DateTime(5:6), '&uur=',DateTime(7:8), '&minuut=',DateTime(10:11),...
        '&bliksem=0');'];
C.2 Legend scripts

The following scripts identify the colors of the legend associated to the map (figure C.1) and assign them a precipitation value. All of them have the same structure, so only the first script has explanations

Figure C.1 Color legend associated to the precipitation maps provided by buienradar

C.2.1 From 0 to 2 mm/h

```matlab
%Reading of the legend image
[L,map]= IMREAD('legendaZC.png');

%Selection of the legend part
LEG0=L(1:1:1,1:1:84,:);

%A0=[254,253,252,251,250,249,248,247,246,245,244,243,242,241, LEG0(:,:,1)];
%B0=[254,253,252,251,250,249,248,247,246,245,244,243,242,241, LEG0(:,:,2)];
%C0=[254,255,255,255,255,255,255,255,255,255,255,255,255,255, LEG0(:,:,3)];

%Color
LEG02U=[[A0.';B0.';C0.']]; 

%Transform to 8-bits units
LEG02 = double(LEG02U) + 1;
q=diff([repmat(NaN,1,size(LEG02,2));LEG02])==0;
LEG02(q)=LEG02(q)-1;

%Assigning the color to the piece of legend
VERDELING0=(0:(2/97):2).';
```

C.2.2 From 2 to 5 mm/h

```matlab
[L,map]= IMREAD('legendaZC.png');
LEG2=L(1:1:1,85:1:108,:);
A2=LEG2(:,:,1);
B2=LEG2(:,:,2);
C2=LEG2(:,:,3);
```
VERDELING2=((2+(3/24)):(3/24):5).';
LEG25U=[[A2.',B2.',C2.']].;
LEG25 = double(LEG25U) + 1;
q=diff([repmat(NaN,1,size(LEG25,2));LEG25])==0;
LEG25(q)=LEG25(q)-1;

C.2.3 From 5 to 10 mm/h

[L,map]= IMREAD('legendaZC.png');
LEG5=L(1:1:1,109:1:128,:);
A5=LEG5(:,1);
B5=LEG5(:,2);
C5=LEG5(:,3);
VERDELING5=((5+(5/20)):(5/20):10).';
LEG510U=[[A5.',B5.',C5.']].;
LEG510 = double(LEG510U) + 1;
q=diff([repmat(NaN,1,size(LEG510,2));LEG510])==0;
LEG510(q)=LEG510(q)-1;

C.2.4 From 10 to 100 mm/h

[L,map]= IMREAD('legendaZC.png');
LEG10=L(1:1:1,129:1:192,:);
A10=LEG10(:,1);
B10=LEG10(:,2);
C10=LEG10(:,3);
VERDELING10=((10+(90/64)):(90/64):100).';
LEG10100U=[[A10.',B10.',C10.']].;
LEG10100 = double(LEG10100U) + 1;
q=diff([repmat(NaN,1,size(LEG10100,2));LEG10100])==0;
LEG10100(q)=LEG10100(q)-1;

C.2.5 From 100 to 250 mm/h

[L,map]= IMREAD('legendaZC.png');
LEG100=L(1:1:1,193:1:214,:);
A100=LEG100(:,1);
B100=LEG100(:,2);
C100=LEG100(:,3);
VERDELING100=((100+(10/22)):(10/22):110).';
LEG100110U=[[A100.',B100.',C100.']].;
LEG100110 = double(LEG100110U) + 1;
q=diff([repmat(NaN,1,size(LEG100110,2));LEG100110])==0;
LEG100110(q)=LEG100110(q)-1;

C.2.6 Merging the legend

% Legend all together
LEGENDATOT=[[LEG02;LEG25;LEG510;LEG10100;LEG100110]].;

xcviii Optimization of the rainfall-runoff response in urban areas by using controllable drains
%Color distribution
VERDELTOT=[[VERDELING0;VERDELING2;VERDELING5;...
             VERDELING10;VERDELING100]];
Annex D

Real-Time Control Applied in Dutch Water Management (by P.J. van Overloop, Delft University of Technology, Delft, The Netherlands)

D.1 Introduction

This survey discusses the theoretical background of measurements and (real-time) control applied to open water systems in The Netherlands. Sequentially, general measurement and control theory, feedback control, feedforward control, model predictive control and more specific control methods are described in separate chapters. Control engineering is an international work field that is applied throughout the world in industry, consumer products and aerospace technology. It is not specifically applied to operational water management. For a control engineer, an open water channel is merely another system with certain characteristics on which existing knowledge and control methods can be applied.

The application of control is essential to Dutch water management. Every low-land pump contains a control loop in which the water level in a ditch is measured. When this water level exceeds a certain level, a signal is sent that switches on the electric motor, forcing the water level to go down again. On a larger scale, canal networks are managed based on water level measurements and weather forecasts. Examples on an even larger scale are gates in series along the Meuse River, the Easter Schelde storm surge barrier and the Maeslant storm surge barrier. These water systems mainly control the water quantity variables. Control of water quality variables is not yet common, especially as in the past systematic measurement of water quantity variables proved to be difficult. Measurement of salinity levels is an exception to this fact, which offers prospects for water quantity control in the ever increasingly saline Western part of The Netherlands. Flushing low-land water systems in dry periods can be seen as (simple) water quality control.

The application of control can be an alternative for (costly) expansion of the water systems infrastructure. Instead of constructing higher embankments or larger pump
installations to avoid inundations, the potential of lowering the water levels before the start of a storm event with the present pumps can be examined. During inundation in downstream areas of a stream, often enough storage is available to cope with the surplus of water at the upstream side of that stream. The extra storage in these two examples can be utilized when, henceforth, water systems are not designed with static dimensions and settings, but are given a certain level of flexibility. The application of real-time control enables the utmost employment of this flexibility.

D.2 General measurement and control theory

A control loop comprises the following standard blocks and connections (blocks shown with solid lines represent parts that are standard in all controlled systems):

Below, the subsystems are described, starting with the most important part, the water system itself.

- System/process. This block contains the dynamic behavior of an open channel. The dynamic behavior of open channels is characterized mainly by the delay time, the storage area (surface area) and extent in which waves reflect on the boundaries of the water course and create standing waves (resonance waves).

- The sensor samples the variables that are of interest to the operational water management. These normally are the water levels, but sometimes also comprise flows or salinity. The most common sensors are pressure transducers and floats for water level measurements, acoustic flow sensors and conductivity sensors for salinity measurements.

- Signal conditioning takes out the information from the measured data that is of relevance to the controller. Fault measurement values can be intercepted by application of minimum, maximum and maximum change checks. When a value overruns a certain maximum, underruns a certain minimum or there is a large change between the previous and new value, the measurement value is rejected.
and needs to be estimated for example by linear interpolation from adjacent values. Another application of signal conditioning is filtering. Here, the frequency content of a signal is considered. Waves with a certain period are let through by a filter, while waves with another period are (partly) blocked. When the water level in a lake needs to be controlled, waves with a long period (for example one hour and more) are relevant, while waves with a shorter period (for example one minute and less) caused by wind or navigation, are considered noise. An example of a common digital first order low-pass filter is presented in the next formula in which the present filtered value, depends on the present measured value and the previous filtered value:

\[ h_f(k) = T_f \cdot h_f(k-1) + (1 - T_f) \cdot h(k) \]

where \( h_f \) represents the filtered water level, \( h \) the measured water level, \( k \) the time step index and \( T_f \) the filter constant. The filter constant \( T_f \) can be calculated from:

\[ T_f = \frac{T_{\text{cutoff}}}{T_{\text{cutoff}} + T_s} \]

Where \( T_s \) stands for the sample time step and \( T_{\text{cutoff}} \) for the period that indicates the partition between either letting through or not letting through the waves (cutoff period). In the example \( T_{\text{cutoff}} \) is selected to be 30 minutes. With a sample time step of one minute, the value of \( T_f \) results in \( 1800/1860 = 0.9677 \). In addition, it is necessary to always use an anti-aliasing filter to eliminate high frequencies from the signal. Often, this filter is already provided in the measurement device.

- The controller is an algorithm in a computer or Programmable Logic Controller (PLC) that based on the measurements and, in certain cases, predictions, determines what control actions are required. In the next chapters, the various control methods relevant to operational water management are described. In the actual practice of the management of large Dutch water systems, humans often act as the controller. These operators are hard to replace by an automatic control loop, because of their extensive experience with the system. What can be demonstrated though, is that these water managers determine their control actions driven by the same control mechanisms.

- The actuators are the control valves in the water system i.e. the adjustable structures. Pumps can be turned on or off or can be infinitely variable from zero to maximum capacity. The crest level of weirs can be adjusted. The opening of an orifice can be changed. The doors of a sluice can be opened. All these actions result in a certain manipulation of the water flow in the water course. This water flow has a time-variant maximum capacity that is determined by the dimensions of the structure.
The arrows between the subsystems in the block diagram are the signals that are communicated in the direction of the arrow. Usually, this is accomplished through electricity cables or by means of modems over the telephone network. Controlling more sites with one controller is referred to as multi-variable control. The arrows then consist of several signals, for example measurement at different locations in a canal network system. Based on these measurements, a weighted average algorithm can determine a representative water level. The signals come together in a central controller that calculates control actions for multiple structures. By definition, a higher performance is achieved compared to configurations consisting of distributed local controllers, simply because the control actions are based on more information of the total system. The disadvantage of central control is the fact that signals have to be sent over large distances and can be disturbed during this stretch. Mobile telephone communication and wireless networks show great promise for data communication in Dutch operational water management.

D.3 Feedback control

This chapter treats the application of feedback control. In all technical fields, Feedback control is the most important control method. This is because the control actions are directly based on the control objective that the controlled system has to achieve. This can be seen in the block diagram of this controller (in the water system block, the dynamic behavior of the structure, the water course and the sensor are combined).

The reference (setpoint) is the input of the block diagram. In this case, it is the target water level that has to be maintained in the water course. The controller uses the deviation, calculated from the comparison between target level and measured water level, to determine the required change in water flow. The water flow has a correcting influence on the water level, which is measured again and compared to the target level, etc. This control loop is repeated with a fixed control time step and, in the end, equalizes the measured water level to the setpoint.

When the water level equalizes to the setpoint, two types of disturbances can occur that again cause a deviation. The setpoint can change or the disturbance flow increases or decreases. In the first case, the control turns the deviation back to zero by step wise correction of the water level until the new setpoint is reached. This is referred to as reference tracking. In Dutch water management this type of feedback control is not very common (yet). A simple example of reference tracking is the difference in summer
and winter target water levels in low-land water systems. On a more dynamic scale day
and night cycles can be implemented to save energy (during the day the water level are
allowed to rise, while during the night pumps run on cheaper energy bring the water
levels back down). Also, by applying reference tracking, extra storage in large water
bodies, such as the IJssel Lake, can function as a buffer for dry periods. The other type of
feedback control, disturbance rejection, is continuously active in each control loop. Every
change in run-off during a storm event or offtakes during droughts, results in a required
change of the control flow to maintain the water at setpoint.

The location of the water level measurement relative to the structure determines
whether water is supplied or demanded. A measurement location upstream from a
structure registers an increase in water level when there is more supply and the controller
will lower a crest, open an orifice or start a pump to equalize the water level to the
setpoint. This type of feedback is referred to as upstream control and is pre-eminently
suitable for drainage systems. The opposite type is downstream control, where the
measurement location is installed downstream from the structure. This manner of control
is practiced in irrigation systems. When a water user withdraws more water from a water
course, the water level drops and the controller will lower a crest, open an orifice or start
a pump to supply more water. In Dutch low-land water systems pumps are used
throughout the year to drain the fields by using upstream feedback control while during
dry periods, gates are used to supply water to the fields by using downstream feedback
control.

By far, the feedback controller that is most often applied worldwide is the
proportional integral derivative controller (PID). More than 80% of all controllers use
this elegant algorithm, which, in the block diagram, is defined by the following function:

\[ Q_c(k) = K_p \cdot e(k) + K_i \cdot \sum_{l=0}^{k} e(l) + K_d \cdot (e(k) - e(k-1)) \]

where \( Q_c \) represents the control flow, \( e \) the deviation (error), \( k \) the time step index and \( K_p \),
\( K_i \) and \( K_d \) the gains of the proportional, integral and differential part, respectively. In
words, the control flow is a function of the deviation, the integral of the deviation and the
derivative of the deviation.

The proportional part is obvious. When a deviation occurs, the control flow is
changed by a factor \( K_p \). In case the deviation increases, the corrective control flow also
increases. The integral part provides an ultimate zero deviation. The proportional part
alone does not suffice, as a deviation is required to generate a control flow with this
algorithm. The summation (integration) of the deviation should not be executed when the
structure is saturated at its maximum capacity to avoid wind-up. The derivative part is not
used for water systems. This is because many waves occur. When a sine wave \( A \cdot \sin(\omega t) \)
is differentiated, it frequently becomes part of the amplitude: \( A \cdot \omega \cdot \cos(\omega t) \). This results in
a controller that strongly reacts to waves with high frequency, while these waves caused
by wind or navigation actually are of no importance to the controlled system.

*Optimization of the rainfall-runoff response in urban areas by using controllable drains*
Standard values for $K_p$ and $K_i$ cannot be given. Various characteristics such as delay time, storage area and the level in which standing waves reflect in the water course influence the proper values. Values that are too low result in a low performance (slow reaction to setpoint changes and in rejecting disturbances). Nevertheless, values that are too high are much more problematic. These can cause instabilities, where resonating water levels can occur and continuously switching structure settings. A practical way of finding proper values is to conduct an experiment on a water course or, with reference to safety, on an (accurate) hydro-dynamic model of the water course. In the experiment, first, the $K_i$ is set to zero and the value of $K_p$ is increased until instability occurs. Next $K_p$ is set to half the value at which instability appeared and the same procedure is repeated with $K_i$. The ultimate values of $K_p$ and $K_i$ have a fair performance and are distant enough from the point of instability.

Finally, another important parameter for tuning the feedback controller is the control time step, or the frequency at which the control loop is repeated. Inevitably, the smaller this value is selected, the better the controlled system functions, also when loop delay times are present in the system. An example of this is the (disturbance rejection) control of the flow through the IJssel River by means of the gate at the location in Driel. It takes about two hours before a change in gate setting effects this flow. Nevertheless, it is important to select a small time step, i.e. 10 minutes. For when a change (disturbance) occurs in the upstream Rhine River, the reaction is 2 hours + 10 minutes delayed at most. In case larger time steps are selected, e.g. 2 hours, the corrective control action can be delayed by $2 + 2 = 4$ hours at most, at which time the flow has drifted off too far from target flow. Naturally, the time step cannot be selected too small due to physical or operational limitations, such as data transmission speed (e.g. when modems are used) or the frequency with which structure settings may be changed to avoid wear and tear.

D.4 Feedforward control

This chapter describes the second most implemented control method (after feedback control) : feedforward control. Feedforward control does not base its control actions on the control objective, but on the level at which measurable disturbance, disturb the control objective. Evidently, the most significant property of feedforward control is that the input of this controller is a measurement of a disturbance. This is in contrast to the feedback controller in which the deviation from setpoint is the input signal. To determine the effect of the disturbance on the control objective, the feedforward controller always implements a model. This can be a table, a formula or a complete dynamic model. The functioning of feedforward control is presented in the following block diagram of a controlled water body that can be represented as a reservoir e.g. a lake.
The dynamic behavior of the reservoir is:

\[
\frac{dh(t)}{dt} = \frac{Q_d(t) - Q_c(t)}{A_s}
\]

Where \( h \) stands for the water level, \( Q_c \) the control flow, \( Q_d \) the disturbance flow and \( A_s \) the storage area. The change in water level is considered small, so the side slope is neglected (which means the side slope can be/may be neglected). To avoid a deviation from setpoint, the derivative of the water level is set to zero. From this follows that the control flow has to equalize the disturbance flow:

\[
\frac{dh(t)}{dt} = \frac{Q_d(t) - Q_c(t)}{A_s} = 0 \Rightarrow Q_c(t) = Q_d(t)
\]

Evidently, the resulting feedforward controller has a very simple transfer function, namely the value of 1. Application of this trivial controller turns out to be problematic, though. An accurate measurement of the inflow into the lake is hard to accomplish. In case of an inflowing river, the measurement is difficult but achievable, though the inaccuracy can be up to 10%, or more. Unmeasurable disturbance flows due to precipitation, evaporation, seepage, etc. have to be added to this. These have to be estimated based on meteorological variables and groundwater levels. A quick calculation shows that, after 24 hours, an error of 15% in a constant inflow of 1 m³/s in a lake that has a storage area of 100000 m² (1000 m times 100 m) and application of the feedback controller leads to a water level deviation of:

\[
\Delta h = \frac{Q_d - Q_c}{A_s} \Delta T = \frac{1.15 - 1.0}{100000} \cdot 86400 = 0.13 \text{ m}
\]

without a corrective mechanism reacting to it. Besides, the inaccurate inflow computation, the implemented control actions can also be off. Manufacturer tables or gauging tables become inaccurate after time. Electric motors lose power, leeway will show in mechanical transmissions, duckweed will block the flow over a weir, etc. Evidently, feedforward control is a powerful control method that directly reacts to a disturbance, but it is definitely not an accurate method. The solution lies in combining feedforward with feedback control. In the given example this combination results in the following algorithm:
\[ Q_c(k) = Q_d(k) + K_p \cdot e(k) + K_i \cdot \sum_{l=0}^{k} e(l) \]

where \( e \) represents the deviation between reference and water level measurement, \( \Sigma e \) the summation of this deviation, \( k \) the time step index, \( K_p \) the proportional gain and \( K_i \) the integral gain. The feedforward part is responsible for a fast reaction, while the feedback part ensures that the overall behavior does not drift from setpoint in the long term.

Flow routing is a feedforward controller that is often applied to irrigation systems. It routes changes of flow through the canal network by adjusting the gates at the right moment. The required flow changes are determined based on a water demand schedule. Every day, this schedule is drawn up on the basis of requests from water users, e.g. farmers, which are ordered beforehand. When the delay times of each reach from upstream to downstream are known, a schedule of all the required structures changes in time can be formulated. An example is given of a long canal reach with an average delay time of 1 hour. When a farmer wants to irrigate for 45 minutes, starting at 9:00 AM, the water has to be release at 8:00 AM at the upstream structure. Under average circumstances, this works well. However, the delay time depends on many factors that vary over the year. The most important parameter, the bed friction, increases during the summer due to the influence of aquatic plants. This leads to a different delay time. Also, the delay time changes by the amount of flow that is running through the canal. Let us assume that, at a certain moment, the delay time is 45 minutes instead of 1 hour. In such a case, the water reaches the location of the farmer’s turnout at 8:45 AM and passes that point unused in the direction of the spillway for 15 minutes. From 9:00 to 9:30, the farmer can use the rest of the ordered water. In this example, the efficiency of the delivery is reduced to:

\[ \frac{V_{\text{supplied}}}{V_{\text{ordered}}} = \frac{Q_{\text{turnout}} \cdot 1800}{Q_{\text{turnout}} \cdot 2700} = 67\% \]

Once again, it becomes clear that feedforward control is only accurate when the model used (in this case the delay times) is perfect. Since, in reality, this is impossible, the implementation of feedback control is imperative. Contrarily, it is possible though to design a water supply system solely based on (downstream) feedback control.

D.5 Model Predictive Control

This chapter discusses the control methodology ‘model based predictive control’. Recently, in literature, the name model predictive control (MPC) is mainly used. In fact, this is a general name for a family of control methods, which have in common that they all implement a model, that is updated by measurements and allows for short term predictions to be made. By actualizing the model based on actual measurements of the water system and subsequently steering this model towards setpoint, implicitly, feedback control is applied. As the model computes into the future and the disturbances over this
prediction horizon are incorporated, model predictive control also functions as feedforward control. Additionally, two important properties make MPC particularly suitable for application in the operational management of certain water systems.

The first important property is the application of an objective function that is optimized. The objective function is a summation of conflicting sub-goals, to which relative importance are assigned by means of weight factors. This is common practice in water systems. Take two adjacent drainage areas into consideration during a heavy storm event. In one drainage area a lot of urbanization is located, while the other is merely grassland. Both drainage areas need to store water, but evidently, the water levels in the water courses of the grassland area are allowed to rise more. By including the water level rises of both areas into the objective function, weighting them with different factors and subsequently minimizing the objective function, the optimal control actions take the different interests into account. The weight factor for the water level rise in the urban area has a higher value in order to achieve a higher penalty on water level rise in this area. Another example is the weighting of the water level in a canal remaining close to setpoint compared to the necessary energy used by the regulating pumps to achieve this. The water levels in the North Sea Canal/Amsterdam-Rhine Canal are kept at target level by the discharge and pump station in IJmuiden. The water levels are allowed to fluctuate within a bandwidth of plus and minus 10 centimeter around setpoint. At low tide, water can be discharge into the sea at negligible cost by means of large undershot gates. In the intermediate periods of high sea water levels, water can be pumped by a large pump installation at very high cost. The two sub-goals, the least possible water level deviation from setpoint and the least possible energy consumption of the pumps, are weighted in an objective function that is minimized. During periods of average supply flow, this results in a cycle in which the water level gradually increases during high tide and the water level strongly drops during low tide induced by gravity discharge possibilities. Only when the supply flow is high, the pumps are turned on.

The second important property that model predictive control possesses is the ability to deal with structure constraints. The internal model applied in the controller is used to look into the near future. Over this prediction horizon, due to extreme disturbances, large deviation from setpoint can occur, especially when the controllability is limited (gates fully open, pumps fully turned on). Before the limitation is reached, model predictive control may decide to implement extra control actions in order to avoid problems further down the prediction horizon. This can be illustrated by the application of MPC on a canal network. The total pump capacity of all pumps is 60 m$^3$/s. Run-off during extreme precipitation is higher than this maximum capacity and the storage capacity in the canals is limited to 10 cm above setpoint. The model predictive controller looks 24 hours into the future and makes use of a rainfall run-off model fed by the forecasted precipitation. In case the combination of initial water levels (derived from measurements), high run-off flow and the pump capacity constraint causes the water levels over the prediction horizon to rise above the 10 cm, MPC advises to start pumping water out of the canals sooner. This results in pumps running at full capacity, even before the actual storm event starts. Obviously, the water levels are not lowered more than is allowed based on the constraint of the water level decrease. The following graphs present...
the results of this example in case feedback control, feedforward control in combination with feedback control and model predictive control is applied.

Feedback control reacts only after a deviation occurs. By making use of an estimate of the actual run-off flow, feedforward control is able to maintain the water level slightly longer at setpoint. By using information of the predicted disturbance flow and structure limitations, model predictive control is able to keep the water levels within the allowed bandwidth around setpoint.

The optimization executed over the prediction horizon is a burdensome computational problem, which for modern computers takes seconds to minutes to solve. Since the most recent measurements from the water system and the most recent predictions of the boundary conditions need to be used, the optimization is repeated at every control step. Each optimization again looks into the future, hence the name receding horizon for this functionality. Consequently, the internal models that MPC uses need not be very accurate. At every control time step, only the calculated control actions for the present time are implemented. These actions are mainly based on the present state of the system and on short term predictions.

The following figure presents the block diagram of the model predictive controller.

D.6 More specific control methods

In this chapter, controllers are discussed that are less generally applicable compared to the previously described control methods. Certain control methods are
suitable for only one specific control problem or invented for just one specific water system.

Heuristic controllers are system specific controllers that are not re-usable for other water systems. They are based on hydraulics and not on control theory. Often, the evolution of such controllers proceeds as follows: At first, the water system is managed by an experienced operator. Due to problems of knowledge transfer caused by absence or departure of the operators, the organization wants to formalize the operator’s task. This results in rules of thumb that are recorded in manuals. Over time, the organization decides to automate the (specific) rules of thumb. In Dutch water management the following heuristic controller is often applied: Based on measurements and precipitation forecasts, a surplus volume is calculated. This volume represents the amount of run-off expected over the next 24 hours. To this value, the volume disc of water above setpoint is added. This disc is calculated by the difference between a weighted average water level (determined from water level measurements in different canals) and the target level, multiplied by the surface area of all canals. Next, the discharge flow is determined by presuming that this total volume needs to be discharged within 24 hours. In case of perfect measurements, predictions and models, after 24 hours this results in an average water level exactly equal to setpoint. (simple forms of) Feedback (bringing back the water level to setpoint) and feedforward (counteracting the effect of the run-off on the water level) can be recognized in this controller. For more complex control problems, simple rules of thumb do not suffice any longer. Decision trees of the type ‘if, then, else’ come into being: ‘If it is dry weather do..., if the precipitation is more than a certain amount of mm do ..., unless, the wind is blowing from the east. But if this Eastern wind is stronger than ... Bft. do it anyway’. Clearly, this soon ends up in an enormous knot of decision rules. System changes, such as expansion of pump stations or construction of an overflow area, require the decision tree to be modified. Another reason why heuristic controllers do not suffice in many cases, is the integral character of operational water management. It is hard to translate the control problem into such concrete decisions and priorities. When a drainage area that mainly consists of grass land is considered less important compared to an urban drainage area, this does not mean that, during extreme precipitation, the controller first completely has to inundate the grassland, while the urban area remains at target level. In reality, conflicting objectives occur and the controller should strive to minimize damage to both areas at the end of the storm event. In this case, an optimal controller that combines conflicting sub-goals is recommended. The different interests of grass land and urban areas need to be defined by different weight factors, not by priorities. Examples of optimal controllers are model predictive control and controllers designed based on linear quadratic regulator theory. The conclusion can be drawn that heuristic controllers are fit for specific water systems, but are not re-usable, difficult to extend and less suitable for complex control problems.

From outside the control community, often, neural networks are suggested for application on complex control problems. Neural networks stem from the science of artificial intelligence and apply a number of neuron layers that are trained to transfer a certain input signal to a certain output signal. This training is executed based on large quantities of historical measurement data. When applied to water systems, a neural
network can be taught what, in certain situations, such as high water levels or heavy precipitation, the control actions have been. After this training, the neural network can generate similar control actions when similar situations arise. There is no guarantee however, that, in case of even higher water levels or more extreme precipitation, the neural network takes the right decisions. Evidently, it has never ‘experienced’ that situation. In literature, no successful applications of neural network controllers for water systems have been reported.

Finally, a controller is described that works comparable to the (average) functioning of various operators. The fuzzy logic controller makes use of fuzzy sets of states and situations, but does translate this vague information into concrete control actions in crisp numbers. The controller can be built based on interviews with various operators. The input signals operators work with, such as measured water levels or forecasted precipitation, can be translated into fuzzy terms such as ‘little precipitation’ or ‘a lot of precipitation’. As operators differ in their opinion on what is exactly ‘a lot of precipitation’, a membership function is formulated consisting of values that they all consider ‘a lot of precipitation’. These membership functions are formulated for all input signals. Next, membership functions for the output signals, the control actions, are drawn up from the information provided by the operators. The functions can be selected as ‘discharge a little’ and ‘discharge a lot’. Finally, a number of logical rules need to be formulated, for example: (1) if ‘little precipitation’ falls ‘discharge a little’, (2) if ‘a lot of precipitation’ falls ‘discharge a lot’. The controller uses the fuzzy sets and the logical rules to determine the discharge flow, given the amount of precipitation. In the figure, the procedure is visualized. Assume that 15 mm of precipitation is forecasted. In the membership functions, this comes down to 55% ‘little precipitation’ and 8% ‘a lot of precipitation’. For each logical rule, these values are projected on the fuzzy membership functions of the control actions and the area below the values is determined. The centre of gravity of both areas results in a discharge flow of 6.64 m$^3$/s. This controller can easily be extended by other input signals, such as measured water levels, the weather condition of the past days, etc. and other output signals, such a pump flows, opening of a undershot gate or the inlet flow.
D.7 Conclusion

The past five chapters present an overview of the application of measurement and control theory of water management in The Netherlands. Control engineering is indissolubly connected to modern water management. It offers perspective as one of the directions in which solutions need to be found for the challenges that confront The Netherlands over the next decennia.
Annex E

Taylor-made pump. Conceptual design and construction planes.