Selecting meteorological boundary conditions for use in a Decision Support System for a low land water system

Thesis for Applied Mathematics written by

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Preface

The research presented in this thesis is a result of my graduation project in order to obtain the degree Master of Science in Applied Mathematics from the University of Groningen. The project was carried out at the Water Department of the firm Ordina (formerly known as Vertis) in Groningen. The research question of this project originates from 2006, when a Decision Support System was built to support the management of a low land water system. To make this project into a success, I have taken a dive into the specialism of civil engineering.

Certain individuals and institutions deserve acknowledgement for their help throughout this project. Special thanks goes to my supervisor dr. ir. Peter Jules van Overloop for his guidance and support. I have appreciated the time and effort he made and I could not have wished for a more pleasant collaboration. At Ordina I would like to thank my supervisor drs. Igor Hemmers for his guidance and Maarten Wetterauw for his involvement. Furthermore, I want to thank Pier Schaper and Gerben Talsma for supplying the necessary data and for organizing a fun sailing trip to the Lauwersmeer. I also want to thank my colleagues for a pleasant work climate. Last but not least, I want to express my gratitude to my supervisor prof. dr. Harry Trentelman for his guidance.
Thank you all.

Contents

Chapter 1

Introduction

Water is essential for human life. When talking about drainage systems, water can also become a threat to human society, in particular if it leaves its boundaries. This gives rise to the question of how to manage open water systems in such a way that the chance of flooding is limited. The easy solution would be to make the (storage) canals sufficiently large, so that they can store all the excess water until it can be drained off. When looking at Dutch society, this is of course not a realistic solution. Society is expanding and therefore nature is getting more restrictions. Of course it is possible to change the present situation of the open water system in such a way that there is more storage for the excess water inside the system. However, there are many restrictions, like land use and water quality, which have to be taken into account. Decision Support Systems (DSS-policy) have been built to help answering such long term questions.
Another question is how to manage the open water system in the present situation, when meteorological boundary conditions disturb the water system. Decision Support Systems (DSS-operational) have been built to help answering such short term questions.

Decision Support Systems (DSS-operational) can be used to support the operator in making choices about managing the present state of the open water system. See Figure 1.1 below, where operator A was the case before DSS and operator B with DSS. In high water situations good control actions have to be taken, if not, damage to the land and/or cities and the costs that go along is the consequence.
In order to built a DSS for an open water system it is crucial to have an accurate view of the system. Hydrodynamic modelling packages have been developed and can be used for this purpose. Such models are able to describe and simulate open water systems in a very detailed way. For modelling the initial state of the open water system, measured data are needed from the actual system. There are located a number of measurement points in The Netherlands in such a way that they give a good representation of the amount of rainfall, temperature, wind direction and wind force in that area. By use of radar images an even better representation can be made. Also information about soil type, channel shapes, water levels in the canals and the water levels outside the water system are essential for modelling the open water system. All such data are stored in a DSS-database.
To know how to manage the open water system in the next few hours or days, a simulation of the open water system is made. Therefore it is necessary to have an idea about the upcoming weather or disturbances into the system. This information is, in the Netherlands, delivered

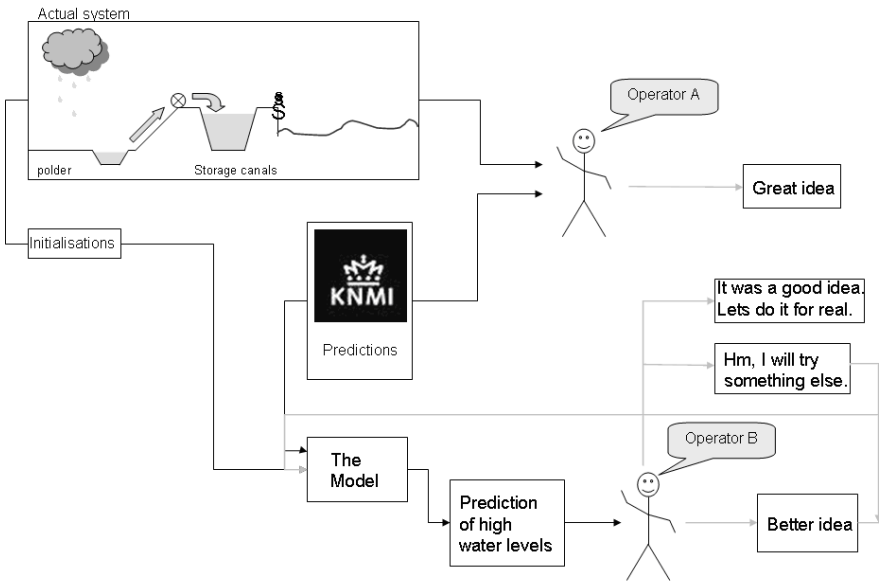


Figure 1.1: *Operator A was the case before DSS and operator B with help of DSS.*

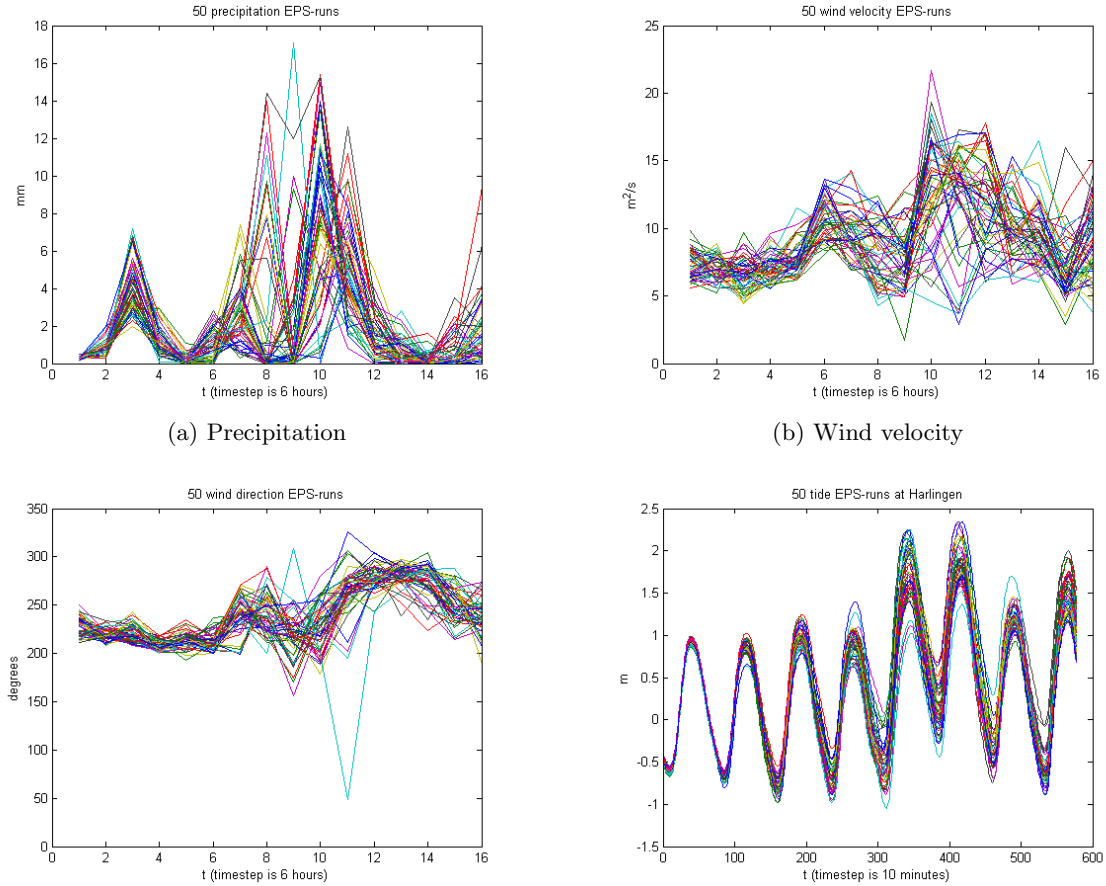
by the Royal Dutch Meteorological Institute (KNMI) in the form of Ensemble Prediction System-runs (more about this in the next section). The disturbance is used in the model of the open water system in order to obtain a prediction of the upcoming water levels. The predicted water levels are derived assuming the use of a standard control procedure for the main structures (more on this subject in Chapter 3). The simulation might show water levels that are above target level. In particular with a standard control procedure, it can happen that a high water situation occurs. If this is the case, the operator has to decide which extra or different control action(s) to take. The user interface, implemented with Geographic Information System tools, gives the operator a good overview of the water system and the polder areas. The operator can run the extra control action(s) in the DSS, such that the effects of the action(s) can be seen. If this is not satisfying, a decision to add another control action or try a different control action can be made. In this way, the use of the DSS will lead to better decisions.
From now on we will look at one typical open water system, namely the network of Frisian storage canals.

1.1 Problem statement

Also for Friesland a Decision Support System has been developed. One of the inputs that are used in the DSS are the Ensemble Prediction System-runs. The European Centre for Medium-Range Weather Forecasts (ECMWF; England) calculates daily, 10 days ahead, the weather forecasts using atmospheric models. In this way 1 deterministic prediction run is ob-

tained, which is the best prediction of the forecasts. The uncertainties of the forecasts grow with time, so the Ensemble Prediction System (EPS) is used to make a prediction of these uncertainties. The EPS generates 51 runs; 1 verification-run and 50 additional runs are made by making a small disturbance in the initial state. This means that the larger the difference between the runs become, the larger the spread of the forecasts within the ensemble and thus the more uncertain the forecast is. The reliability of the EPS-runs is not considered in this project though.
The Royal Dutch Meteorological Institute delivers the EPS-runs. The EPS-runs for precipitation, wind velocity, wind direction and the tide-setup are used in the DSS. The tide-setup is derived from the effects of wind and air pressure on the water level deviation related to the astronomic tide, and has become recently available. The EPS-runs for the different predictions are linked, which means that precipitation run i, 'happens together with' wind velocity run i, wind direction run i and tide-setup run i. Together they represent EPS-scenario i. In Figure 1.2 the ensembles for the weather forecasts used in this project can be seen. The chosen period to work with in this project and which is used in the examples is January 2007 (16-01-2007 up to 20-01-2007). In this period a heavy (wind)storm passed Friesland, so the windeffects are for sure recognizable. Also for this year the tide-setup ensembles for the boundary points of the Frisian open water system are available.
Because of the real time use of the DSS, making a simulation with all EPS-runs will take too much computing time and therefore 3 EPS-runs are selected for use. The 50 EPS-runs are sorted based on the amount of precipitation over the next 4 days and out of this, 2 runs are selected; the 'maximum' and 'minimum' precipitation run. Each EPS-scenario has a 2% chance of appearance. EPS-run number 46, of the sorted EPS-runs (with largest amount first) based on precipitation, is taken as the maximum run, since there is a chance of 90% that the actual amount of precipitation appears below that run. In the same way, the minimum run is taken to be EPS-run number 5, since there is a 10% chance that the actual amount of precipitation appears below that run.
These selected maximum and minimum precipitation runs are simulated in the DSS, together with the deterministic prediction run for the wind velocity, wind direction and tide-setup. It is clear that these together do not form an EPS-scenario. The complete deterministic prediction run is taken as the average run, which does form an EPS-scenario.

The choice of sorting the EPS-runs based on precipitation is made because it is assumed that precipitation is responsible for high water. This assumption is based upon the idea that the polders, which represent a large area, capture most of the precipitation. The main part of this water has to be drained off to the storage canals and therefore the water level in the channals will rise.
Although the assumption holds in many cases, the problem is that it is not always true that more precipitation results in higher water levels. When for example the inflow (Q_{in}), due to heavy precipitation, is large, but the outflow (Q_{out}), due to low low tides, is also large then there is no problem as regards to high water. When the inflow is low, the weighted average water level of the storage canals (WASL, more about this in Chapter 2) will remain close to target level, but due to wind effects, the water can set up and local water levels may become high.
For Friesland the year 2002 gives a clear view of the problem. In the month of February of that year no high water problem was expected. It was due to wind effects that flooding occurred.



(a) Precipitation

(b) Wind velocity

(c) Wind direction

(d) Tide at one boundary point

Figure 1.2: *50 EPS-runs for precipitation, wind velocity, wind direction and tide-setup weather forecasts. The situation of January 2007 (16-01-2007 up to 20-01-2007) will function as an example troughout this thesis.*

1.2 Research goal

In the above an explanation was given of why sorting on precipitation EPS-runs does not always reproduce a selection that gives 'real' (maximum) water levels. It is of course very important for the operators to know the real worst case scenario. But also the average- and minimum water levels are of importance. They give an expression of the range of the predicted water levels. The research goal of this thesis is:
To investigate the applicability of deriving realistic maximum, average and minimum EPS-scenarios based on sorted predicted water levels for use in a real time Decision Support System for an open water system.

1.3 Research approach

In general, we can state that it is better to make a selection based on sorted water levels, in order to get a correct maximum, average and minimum prediction. That is why a sorting method for the EPS-scenarios is developed and this method is tested on a detailed model of the Frisian water system. A real time implementation of this sorting method is needed. Since, with the detailed model, this is not conceivable, the possibility of obtaining a simplified model is investigated.

1.4 Outline of the thesis

First of all, the research is done on the Frisian open water system, so the main dynamics about this water system are described in the next chapter. Secondly, an accurate model is needed to be able to sort on water levels. The model that is used in the DSS is such an accurate model and will be described in Chapter 3. In Chapter 4 it is described which water levels are sorted on, and how to sort the EPS-scenarios. The results of two sorting methods are presented at the end of Chapter 4. The possibility of obtaining a simplified model is investigated in Chapter 5. A comparisation of the water levels of both models is presented at the end of Chapter 5. Conclusions that can be drawn from the results are combined in Chapter 6 and finally the recommendations are given.

Chapter 2

The Frisian open water system

The Frisian open water system is managed by an operator who is supported by a Decision Support System. The DSS contains a hydrodynamic model which simulates the water system. The inputs that directly effect the water levels in- and outside the system are the weather circumstances. Lack of computation time makes it impossible to take all Ensemble Prediction System-scenarios into account, so a set of 3 EPS-scenarios must be selected. The choice of these 3 EPS-scenarios is very crucial, since they strongly effect the management of the water system. In the previous chapter it was explained why selecting EPS-scenarios based on sorted water levels give a better range for the predicted water levels than a sorting based on EPS precipitation runs. In doing so, we need an open water system to test the sorting method on. In this report use of the Frisian open water system is made, so a closer look at this typical system is taken in this chapter.

The Frisian open water system is a connected network of lakes, canals and ditches. It is the largest drainage system in The Netherlands (See Figure 2.1).
The storage canals are a link between the lower polders and the higher sloped area on the one hand and the IJsselmeer, the Lauwersmeer and the Waddenzee on the other. Water also drains off onto the storage canals from the province Groningen (See Figure 2.2). There is an area in the north of Friesland that is draining the excess water directly onto the Waddenzee. So when speaking about the Frisian open water system, this area is not included.
Challenges in many areas arise when dealing with the Frisian open water system, such as safety, drainage, water level management, banks[1], eutrophication[2], emissions, channel bed and maintenance. It is the district water board that deals with these issues. One of these water boards is called Wetterskip Fryslân. Among other things, this board is responsible for managing the water level of the system. This means that it is daily trying to keep the water level in the lakes, canals and ditches at target level, by draining off the excess water or supply the shortage water. The target water level of the storage canals is set at NAP[3] -0.52 m during the whole year and has a maximum permissible level of NAP -0.22 m. There are located 6 points at which the water level is measured (See Figure 2.2) and out of these a weighted average water level of the storage canals (WASL) is calculated: WASL = 0.1 h_1+ 0.1 h_2+ 0.1 h_3+ 0.1 h_4+ 0.2 h_5+ 0.4 h_6. The difference between this WASL and the target

[1]Also called a berm.
[2]Enlarging of the river and surface water nutrition resources.
[3]Normaal Amsterdams Peil (or Amsterdam Ordnance Datum). Currently NAP is close to mean sea level at the Dutch coast.

Figure 2.1: *The Netherlands, where the Frisian storage canals are located in the northern part.*

Figure 2.2: *The Frisian open water system in context with its surroundings. Open water outside the storage canals influences, one way or the other, the present situation of the water system. The pump stations (P) and the gates (G) are placed to control the water system.*

level is used to calculate the amount of excess or shortage water. The local water level though can deviate, due to wind effects and water flows.
When precipitation causes the water level in the lower polder-ditches to get too high, the water is pumped into the storage canals by a pump. The water level in the storage canals will rise. To get rid of this water, it can be pumped onto the IJsselmeer with use of the two pumping stations; J.L. Hoogland and ir. D.F. Wouda (respectively P_1 and P_2 in Figure 2.2). The IJsselmeer has a winter target level at NAP -0.40 m and a summer target level at NAP -0.20 m. The excess water can also flow freely onto the Lauwersmeer with use of 5 gates; Dokkumer Nieuwe Zijlen and Friese sluis Zoutkamp (respectively G_3, G_4, G_5, G_6 and G_7 in Figure 2.2). There are additionally located 5 gates at two locations; two of them are called the Tsjerk Hiddessluizen (respectively G_1, G_2) and the other three gates are called the Lauwersoog spuikokers (respectively G_8, G_9 and G_{10} in Figure 2.2) For these 5 gates it holds that, in case of low tide, the excess water can flow freely onto the Waddenzee. See below for a detailed describtion of each structure.

Pumping station P_1: J.L. Hoogland
The J.L. Hoogland pumping station consists of four screw-pumps, each with a diameter of 3.60 m. Each pump can be driven by a so-called small (285 h.p.) or large (570 h.p.) electric motor. The drainage capacity by use of the small motors is 72 m^3/s = 6.2 $\cdot 10^6 m^3/day$ and by inset of the large motors it is 98 m^3/s =8.5 $\cdot 10^6 m^3/day$. The pump station is activated when the WASL tends to exceed NAP -0.46 m. In dry periods, the pump station can let water in from the IJsselmeer onto the storage canals.

Pumping station P_2: ir. D.F. Wouda
The ir. D.F. Wouda pumping station is the only steam pumping-station in the Netherlands, that still operates professionally. It consists of eight centrifugal pumps, each with a diameter of 1.70 m. The pumps are driven in couples by a tandem steam engine. There are four boilers and formerly these were distilled with coals, but since 1967 oil is used. It takes six hours to fire the boilers. Under normal conditions the pumps are driven by two steam-boilers and the pumping station then has a capacity of 65 m^3/s. When the number of revolutions still needs to be raised, a third boiler is added, increasing the maximum capacity to 72 m^3/s. The fourth boiler then functions as a spare boiler.

Gates G_1 and G_2: Tsjerk Hiddessluizen
The Tsjerk Hiddessluizen connects the storage canals, through the Harinxmakanaal, onto the Waddenzee. This sluice-complex consists of a small and a large sluice, which are respectively 7 meter and 12 meter wide. The large sluice has the function to lift-lock[4] ships and both sluices drain off the excess water. In the middle of the sluice there are gates of steel with openings, so that de drainage can be regulated and/or hampered for lift-locking ships. The water level at the Waddenzee is variable and the excess water can only be drained off at low-tide. The drainage capacity then is changeable and is mostly set at 50 or 80 m^3/s. To resist salt seawater entering the storage canals, the gates open and close when the difference between the inside and outside water level is 5 cm.

[4]A lock is a particular type of device for raising or lowering ships between stretches of water at different levels.

Gates G_3, G_4, G_5 and G_6: Dokkumer Nieuwe Zijlen
The Dokkumer Nieuwe Zijlen is a sluice-complex and connects the storage canals with the Lauwersmeer. Excess water is drained off onto the Lauwersmeer and it is also possible to lift-lock ships from a higher water level to a lower water level. Most of the excess water of Friesland is drained off via Dokkumer Nieuwe Zijlen. The water level in the storage canals is usely higher (target level is NAP -0.52 m) than in the Lauwersmeer (target level is NAP -0.93 m), so the water can flow freely to the lower laying lake. The drainage capacity is variable, since it is possible to regulate the drainage with gates, and is influenced by the water level in the Lauwersmeer, but a discharge of 20 to 50 m^3/s is common. The lift-locking part of the sluice-complex is also used for drainage and has gates, which make it possible to open them against the water pressure and close them even in running water.

Gate G_7: Friese sluis Zoutkamp
The Friese sluis Zoutkamp has a combined operation; it is possible to drain off water and lift-lock ships. The gates are made in such a way that the drainage can be regulated. In practice the gate is completely open or when drainage is limited the gate is opened half a meter. The drainage capacity varies, next to the behavior of the water levels, due to wind effects. If the gate is open, this interval ranges from 0, when there is no difference between the water levels, to 1.3 $\cdot 10^6 m^3/day$, if the difference in water levels is large.

Gates G_8, G_9 and G_{10}: Lauwersoog spuikokers
The Lauwersoog spuikokers are a connection between the Lauwersmeer and the Waddenzee. This sluice-complex consists of 3 sluices, each containing 4 tubes. The tubes are each 10 meter wide, so that the maximum drainage width is 120 meters. The bottom of the tubes are at NAP -5 m and the top edge is at NAP -0,5 m. Halfway through the tubes there are two gates, so that they can be closed. During low tide, both gates are completely open. Each of the in total 24 gates, weighs approximately 35000 kg. The dimensions of the sluices are such that when one of the sluices can not be used, because of maintenance, the other two sluices have enough capacity to drain off all the excess water. The drainage capacity of the sluice-complex is approximately 1900 m^3/s.

Chapter 3

The model

In order to sort meteorological boundary conditions based on water levels, it is required to know what the effects of each EPS-scenario is on the water levels. For this we need an accurate model, that describes all crucial dynamics of the system. In this chapter the ins and outs of the model SOBEK are discussed in the necessary detail that is relevant in the context of this report.

3.1 The modelling package SOBEK

SOBEK is a hydro-dynamic modelling package that describes and simulates open water systems in a detailed way. SOBEK has an integrated framework consisting of a number of program modules, which work together to give a comprehensive overview of the water system. Because it is based on high performance computer technology and has a robust numerical core, SOBEK can handle water systems of any size, no matter how complicated the simulation gets. Due to the different program modules it has been made possible to link river, canal and sewer systems so that a total water solution can be obtained. It also has a graphically oriented interface and it is designed to interface with existing software, so it is user friendly as well.
SOBEK has three basic product lines: SOBEK-Rural, SOBEK-Urban and SOBEK-River. These cover any open water situation that appears in River, Urban and Rural systems. Each product line consists of different modules to simulate particular aspects of the water system. These modules can be operated separately or in combination. The data transfer between the modules is fully automatic and modules can be run in series or parallel to facilitate the physical interaction.
When modelling an open water system, many parameters need to be set and inputs need to be given such that a simulation that approaches reality can be obtained. The case manager gives an overview of the different tasks that need to be carried out (See Figure 3.1). Each block represents a specific task. A task can be a model, a set of linked models, the selection of a scenario or strategy, or a (graphical) presentation tool. The arrows between the blocks represent the relations between the tasks and show which tasks have to be completed before going on to the next task.

In the next section a closer look is taken at what is actually modelled.

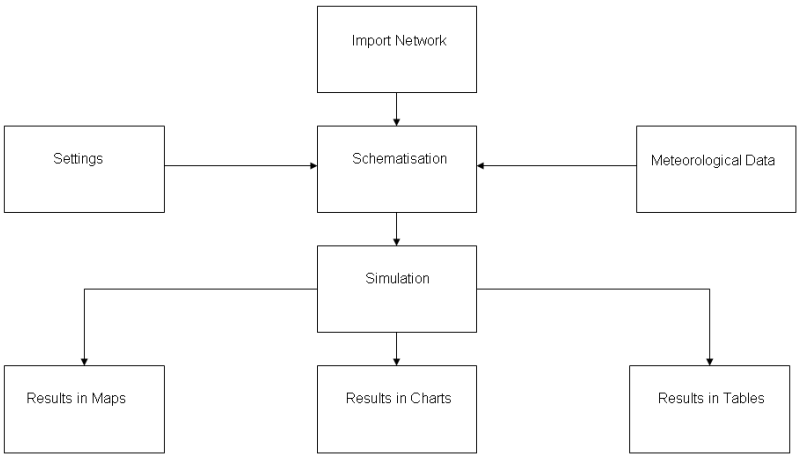


Figure 3.1: *The case manager, which links the different task blocks. The arrows show which tasks have to be completed before going on to the next task.*

3.2 Configuration of the model

The total model is a combination of different dynamical processes. Every dynamical process that is modelled is indicated by a node. Figure 3.2 shows a complete graphical overview of all the nodes that are used for modelling the Frisian water system.

The complete model consist of 2243 flow connection and flow calculation nodes, 2361 reach segments, 89 structures of the storage canals, 881 structures or friction nodes in polder areas, 52 lateral inflow nodes due to direct precipitation, 881 open water nodes of polder areas, 1133 unpaved area nodes, 124 paved area nodes, 85 greenhouse nodes, 681 lateral inflow nodes between polder and storage canals and 3104 connection nodes between polders and between polders and storage canals.
The total model consist, at least in use of this project, of a Rainfall-Runoff module, a Channel Flow module and a Real Time Control module. The Rainfall-Runoff part simulates the hydrology, i.e. the route that the precipitation takes through urban and rural areas. These areas capture the biggest part of the precipitation and drains the excess water off into the storage canals. The flow in these storage canals, i.e. the hydrodynamics, can be simulated by the Channel Flow module. The Real Time Control module makes it possible to combine all sorts of available information to adjust the operation of structures. Each module contains nodes that describe dynamics that are representative for its particular aspect of the water system. These aspects will be discussed in the next subsections.

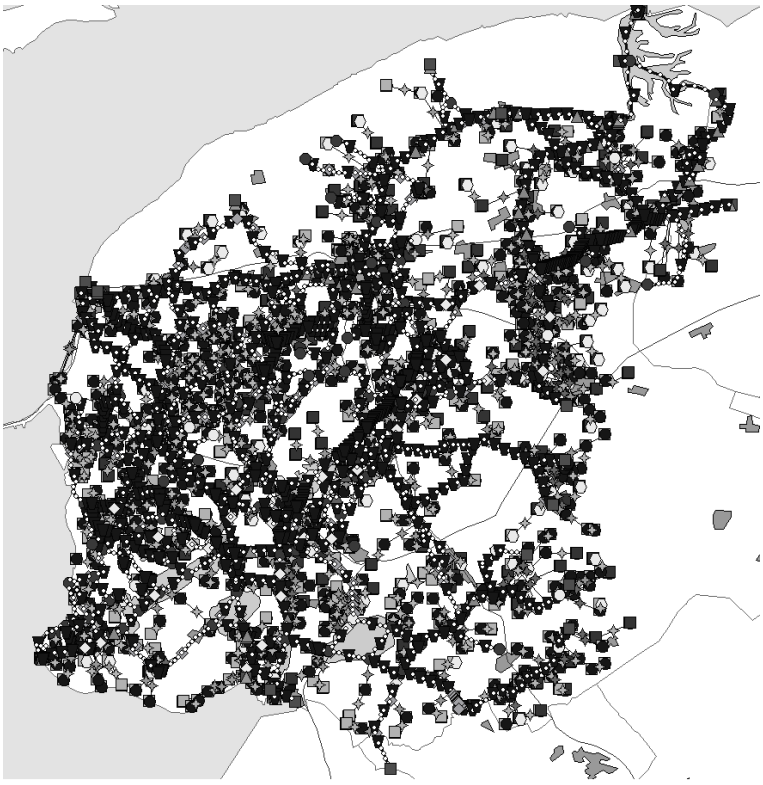


Figure 3.2: *The complete (graphical) model with all nodes that are used in modelling the Frisian water system.*

3.2.1 The Rainfall-Runoff module

The Rainfall-Runoff module models, as mentioned, the urban and rural areas. Typical for these areas is that it contains paved, unpaved and open water areas. Polder areas normally have a lower surface level than the target level in the storage canals. This means that a polder area is in need of a pump station (See Figure 3.3 for a graphical view). Greenhouses can be seen as paved areas, but they have special features, which makes it necessary to model them seperately. The areas that are mentioned here drain their excess water off onto open water nodes, which are connected with a structure to another open water node, or to a RR connection on channel node, representing the boundary of the Rainfall-Runoff part.
The nodes that are involved in the Rainfall-Runoff part of the model must be validated to the actual system. Per node there are parameters that need to be adjusted. Each node represents an area at a certain location and with a certain size. Such information must in any case be given. When the location and size of the area that each node represents are given, it is possible to determine the amount of precipitation that disturbes that particular node. This water then travels a certain route until it reaches the storage canals. This route depends on the area (or node) the precipitation occurs on. In the following the typical features of each node will be discussed.

The *paved area node* is used to simulate the rainfall-runoff process on paved areas. In paved

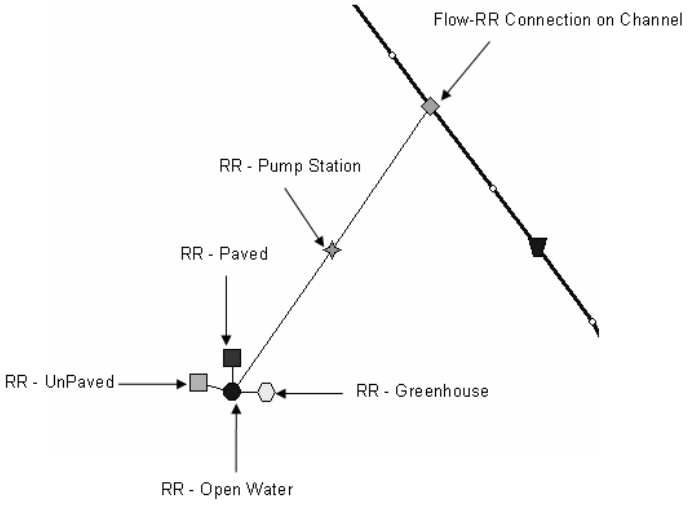


Figure 3.3: *Graphical view of a polder area, which drains the excess water off into the storage canals.*

areas water can be stored on the surface (on-the-street storage) and in a sewer system (See Figure 3.4). The storage on the street and the sewer storage can be considered to be two reservoirs. When precipitation occurs on the paved area, first the on-the-street storage reservoir is filled. If this reservoir is full, it starts spilling into the sewer reservoir. The amount of water that is stored on-the-street is reduced by evaporation and it is explicitly assumed that (in paved areas) there is no infiltration to the groundwater.
There is another way of water entering the sewer system, namely from domestic water use, which is also called dry weather flow (DWF). Depending on the type of sewer system, the inflow from the surface and the dry weather flow are either mixed in one sewer storage reservoir, or put into separate sewer storage reservoirs. When the sewer storage reservoir contains water, the sewer pumps are switched on, and water is pumped from the sewer to the local open water or to a boundary representing a waste-water treatment plant outside the system. If the sewer is full, it can also spill directly into the open water.
So in short we can say that a paved area node is characterized by its area, maximum storage on the street, maximum storage in the sewer, the sewer pump capacity, the sewer type (mixed, separated, or improved separated system) and its DWF (dry weather flow).

The *unpaved area node* is used to simulate the rainfall-runoff process on unpaved areas. In unpaved areas, water can be stored on the surface and in the ground (See Figure 3.5). As soon as water is stored on the surface, it starts infiltrating into the ground. Only when the rainfall intensity exceeds the infiltration capacity, or when the groundwater has risen to ground level, the remaining water is stored on the surface. That is why the surface storage (storage on land) can be seen as an reservoir from which evaporation can take place. When the storage capacity of the surface is exceeded, the excess water will flow to the adjacent open water.

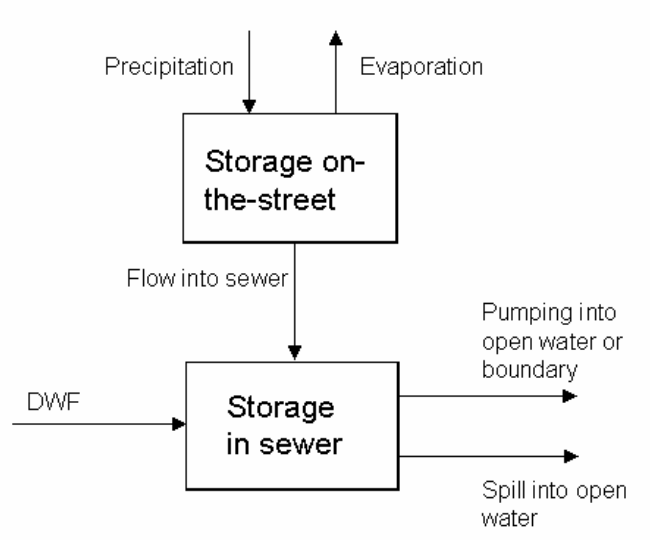


Figure 3.4: *Representation of the rainfall-runoff process in paved areas.*

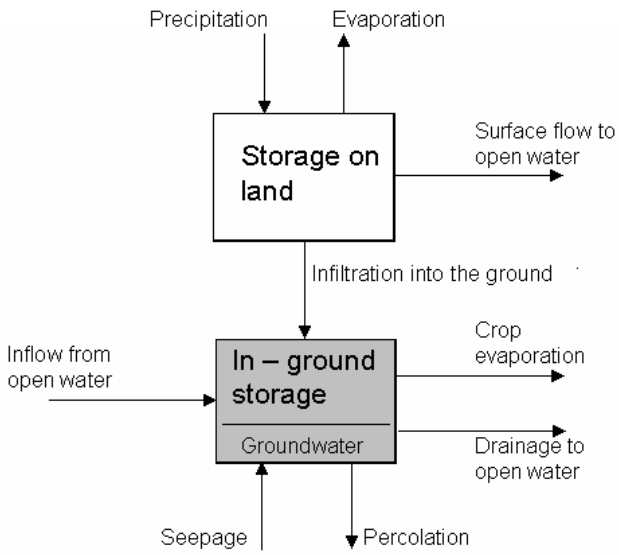


Figure 3.5: *Representation of the rainfall-runoff process in unpaved areas.*

The second reservoir is the groundwater reservoir. The groundwater level can rise as a result of infiltration from the surface, and through seepage from deep groundwater. Groundwater storage can be reduced by crop evapotranspiration and percolation to deep groundwater. How much the ground water level will rise, due to net incoming water, is determined by the storage coefficient. The storage coefficient says something about the available capacity for storing water in the given soil profile. This profile depends on information about the soil type

and the drainage depth, which is the difference between surface level and open water level. If there are different types of soil present in one area, then this is modelled by multiple unpaved nodes.
Now, if the groundwater level is higher than the open water level, drainage from the soil to open water will occur. How fast the water, stored in the ground, flows to the adjacent open water is captured in a drainage resistance value. This value depends mainly on the permeability of the soil. On the other hand, if the open water level is higher than the groundwater level, inflow from the adjacent open water will occur.
So in short, we can say that an unpaved area node is characterized by its area, groundwater area, surface level, soil saturation, soil type, storage coefficient, evaporation, storage on land, infiltration capacity, drainage resistance value, seepage, percolation and surface runoff.

In Figure 3.3 also a *greenhouse node* can be seen. A greenhouse is a paved area, nevertheless it is given as a seperate node because it exhibits some different dynamics. The greenhouse area node is used to simulate the rainfall-runoff process on greenhouse areas. In greenhouse areas precipitation can be stored on the glass surface (roofs) of the greenhouses and in storage basins above- or underground (See Figure 3.6). The aboveground basins take in runoff water from the glass surface as well as from direct precipitation. The amount of water stored is reduced by evaporation, water use in the greenhouses, and possible pumping to the underground (subsoil) storage silos. When the maximum storage capacity is exceeded, the excess water flows into the adjacent open water.

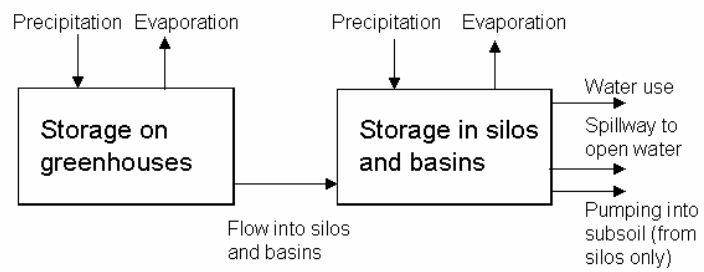


Figure 3.6: *Representation of the rainfall-runoff process in greenhouse areas.*

So in short, we can say that a greenhouse area node is characterized by its area, surface level, maximum and initial storage on roofs, subsoil silo capacity, pumping capacity and greenhouse type. The last characterisation indicates the size of the basins and the amount of water that is used for the crops.

An *open water node* can be considered as a storage basin and is always connected by a structure to another open water node or to a RR connection on channel node representing the boundary of the Rainfall-Runoff part (See Figure 3.3). Storage at open water nodes increases through precipitation, seepage from deep groundwater, flows from adjacent paved area, unpaved area and greenhouse area, and inflow through structures from other open water nodes or from boundaries. Open water storage decreases through evaporation, percolation to deep groundwater, flows to unpaved area (if the open water level is higher than the groundwater level), and flows through structures to other open water nodes or boundaries (See Figure

3.7).

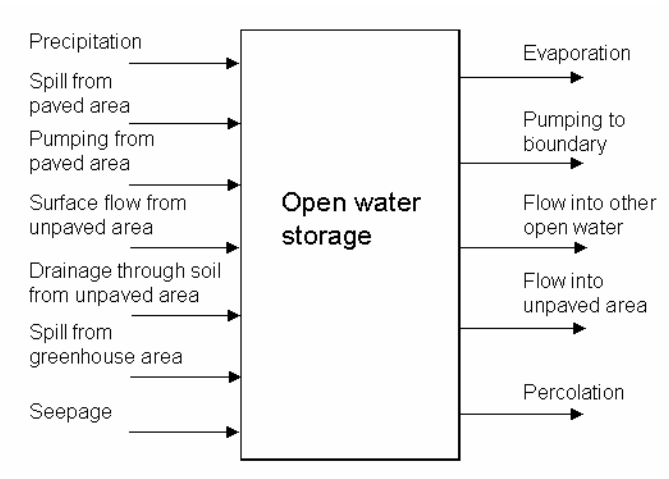


Figure 3.7: *Representation of the rainfall-runoff process in open water areas.*

So in short, we can say that an open water area node is characterized by its surface (which depends on bed level), target level, maximum permissible level and seepage.

Structure nodes can be applied to describe the water exchange between polder areas with different target levels and between polder areas and the storage canals. There are 3 structures used in the Rainfall-Runoff module, namely a weir, a pumping station and a resistance (friction node). Each structure node will be explained in more detail below.
There are different types of weirs, but for each one three different types of flow conditions can occur. These types of flows are: free flow, drowned (submerged) flow and no flow (water levels below crest level). If high tail water conditions do affect the flow, the weir is said to be drowned. Free weir flow appears when $h_2 - z < \frac{2}{3}(h_1 - z)$ and a drowned weir flow appears when $h_2 - z \geq \frac{2}{3}(h_1 - z)$.

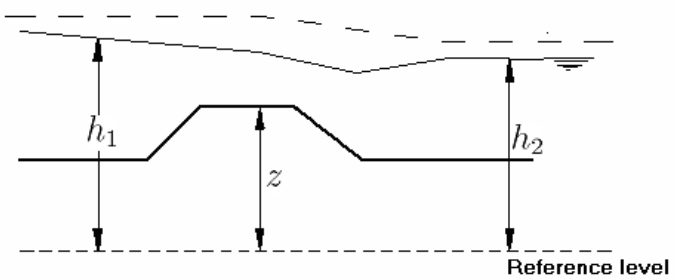


Figure 3.8: *Representation of a weir, where h_1 is the upstream water level, h_2 is the downstream water level and z is the crest level.*

The discharge that passes a weir depends on the weir type, for instance a sharp crest weir or a broad crest weir. Information such as crest level and width must be given to determine

the discharge. When a weir has an adjustable crest, it can be controlled. The crest level may depend on local management strategies, such as a target level one wants to keep in a specific area, which can be due to different seasons. This later is true since in general the target level is higher in the summer, when the crops need to grow, than in the winter period. The crest level may also be controlled based on global management strategies defined in the Real Time Control module. This feature though will not be used for structures defined in the Rainfall-Runoff part.
A pumping station can be used in two ways, namely to remove excess water or to supply water for drought prevention. In this case the first situation is of importance, in which a switch-on and switch-off level must be defined, which may be linked to the capacity at which the pump must work. As soon as the upstream water level exceeds a certain 'switch-on' level, the pump will start working. The pump will stop pumping as soon as the upstream water level drops below the 'switch-off' level. These switch-on and -off levels and capacity may vary during 24 hours, since it can be taken into account that electricity is cheaper during night hours.
The friction node creates a water level difference between two nodes that calculate open water levels. It is designed to simulate the backwater effects of a narrow channel with certain length, bed width, friction and bank slopes.
Now all nodes that are used in the Rainfall-Runoff module are described, and a closer look at the Channel Flow module can be taken.

3.2.2 The Channel Flow module

The flow in the storage canals is simulated by the Channel Flow module. It receives water from the Rainfall-Runoff areas (See Figure 3.9) and by direct precipitation. Excess water is drained off onto the waters outside the water system.
The storage canals can schematically be seen as a network of reaches. These reaches are connected to each other at *connection nodes*. In each reach a number of *calculation points* can be defined. These calculation points and connection nodes each represent a storage area, and together also the spatial numerical grid to be used in the simulation. The momentum equation and continuity equation will be solved numerically on this grid, which results in the hydraulic states, i.e. water level and discharge. The resulting water levels are defined at the calculation points and the connection nodes, while the discharges are defined at the reach segments (staggered grid). Figure 3.10 shows such a staggered grid.
Figure 3.9 shows some other nodes than the ones mentioned until now. These will be discussed in the following, keeping in mind that first of all for each node a location must be assigned.

One of the most important node types when creating a hydrodynamic model, is the *cross section node*. This node defines the dimensions of the channels that are to be modelled. The discharges through a channel are basically determined by the shape of the channel bed. Therefore it is important to add a sufficient number of cross section nodes, in order to obtain a proper schematisation. For these nodes it is required to give the cross section profile or type, and the dimensions that go along, and the bed friction. There are different cross section profiles, such as a symmetrical tabulated cross section, a (a)symmetrical trapezoidal cross section and a egg-shaped cross section (See Figure 3.11). Cross sections can be open

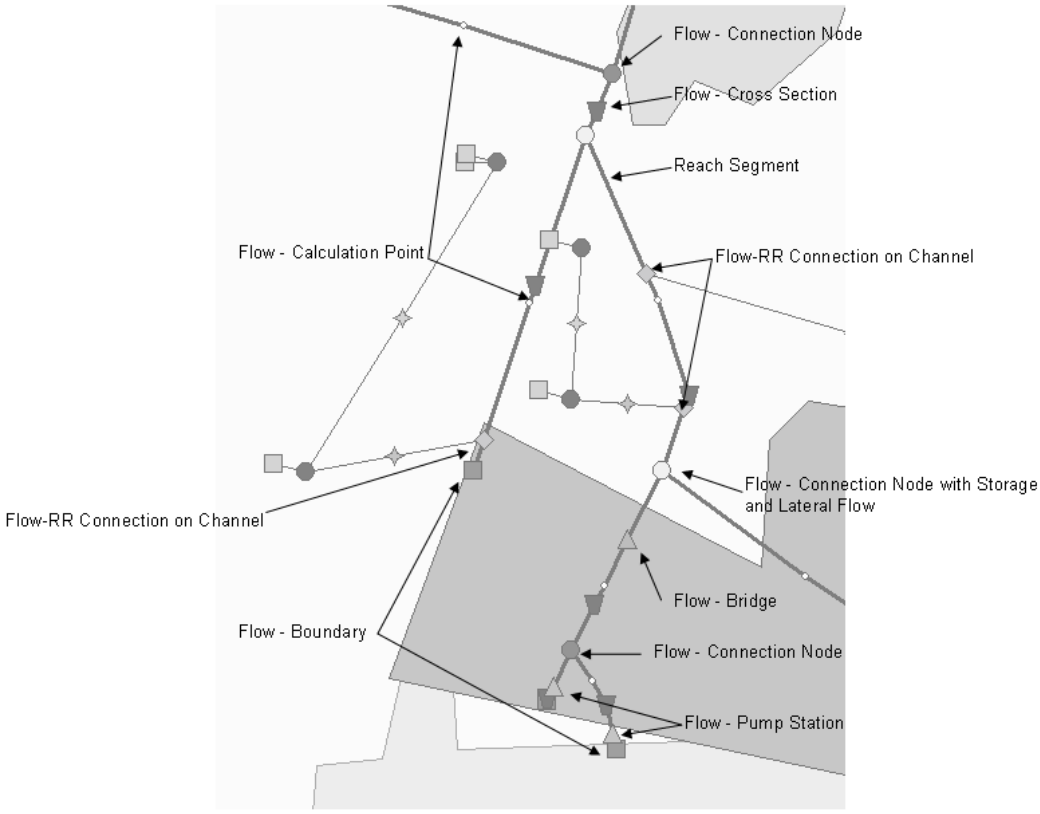


Figure 3.9: *Graphical view of a channel flow section (thick lines), which receives excess water from the Rainfall-Runoff areas and drains the excess water off into the waters outside the (drainage) system.*

or closed. The (a)symmetrical trapezoidal cross section is always open. The symmetrical tabulated cross section can be open or closed. It is considered closed when the width of the highest level is smaller than 10 mm. The egg-shaped cross section is closed. The bed friction is the friction between the flowing water and the channel bed and can be determined using typical coefficients like Chézy or Manning[1]. It exerts a force on the flowing water always in the direction opposite the water flow. This force together with the force caused by earth gravity usually determines the flow conditions.

The Rainfall-Runoff part is connected to the storage canals by a *RR connection on channel node*. This node is a boundary condition for the Rainfall-Runoff part and determines the in- or outflow towards the storage canals. This boundary condition can be set beforehand, which

[1]The most commonly used equations for calculating friction losses are the ones known as Chézy and Manning equations.

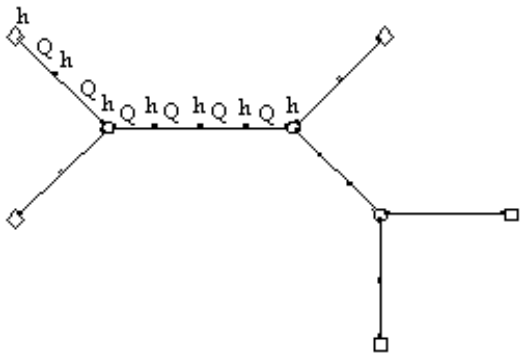
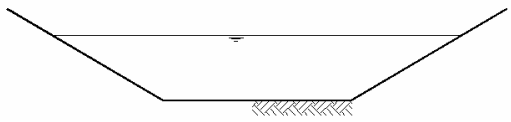
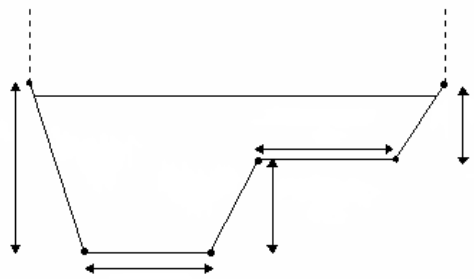


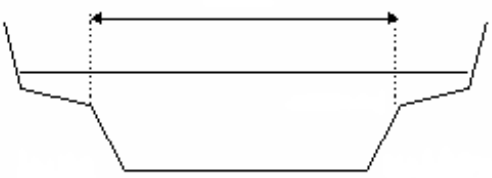
Figure 3.10: *The Channel Flow module uses a staggered grid to define the hydrodynamic states, i.e. water level and discharge.*



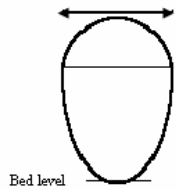
(a) Symmetrical trapezoidal cross section



(b) Asymmetrical trapezoidal cross section



(c) Symmetrical tabulated cross section



(d) Egg-shaped cross section

Figure 3.11: *Four different types of cross sections that can be used in modelling the storage canals.*

means that the changing waterlevels in de storage canals do not influence the in- or outflow of the Rainfall-Runoff module, and it is possible to let the boundary condition vary, based on the change of the water levels in the storage canals.
At the bounds of the storage canals, the model must interact with the surroundings outside the system. Nodes that can be located at the bounds of the model are Connection nodes, *Connection nodes with storage and lateral flow* or *boundary nodes*. Connection nodes with storage and lateral flow are similar to the connection nodes, apart from the fact that it is

possible to assign lateral flow. This node gives the opportunity to add conditions that are not included in the schematisation. Boundary nodes do not have storage, but present a boundary condition such as a discharge that flows into or out of the model and/or a water level, such as tidal movement.

Structure nodes are also part of the Channel Flow module. When modelling the storage canals three structure types are used, namely a bridge, an orifice and a pump station. The later was treated in de last subsection, so here a short description of a bridge and orifice will be given.

A *bridge* can be modelled as a pillar bridge, abutment bridge, fixed bed bridge or a soil bed bridge. See Figure 3.12 for the different shapes. A pillar bridge has one or more pillars that affect the discharge through the bridge. The plate of the bridge is always so high that it does not effect the flow through it. The plate of a abutment bridge can effect the flow through the bridge. A fixed bridge is like a abutment bridge except for its rectangular profile. A soil bridge is like a fixed bridge including the rectangular profile. In addition to this it has a ground layer with a different friction formulation.

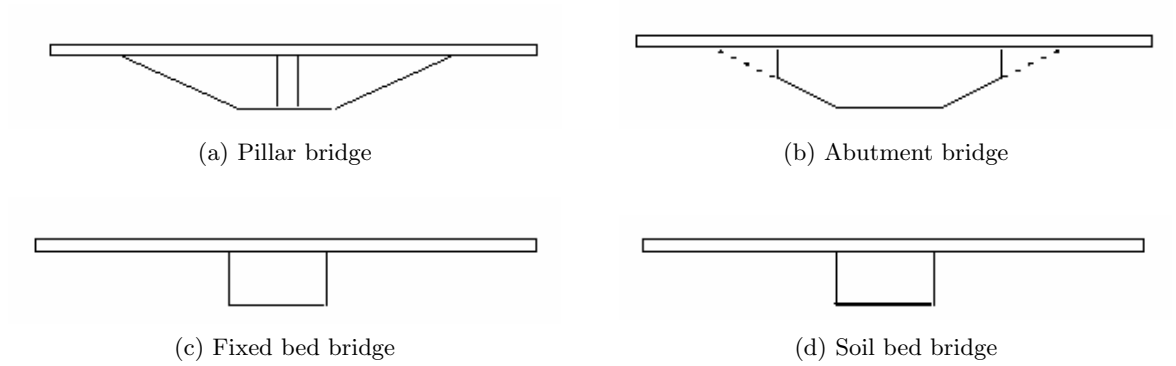


Figure 3.12: *Four different types of bridges that can be used in modelling the storage canals.*

With a *orifice node* it is possible to simulate rectangle-shaped gates through which water within a channel is led. Orifices are used to regulate the water flow through or towards a channel, since it contains a controllable gate height. When the gate height is at a certain level, the water will flow as if it were a weir. In Figure 3.13 it can be seen that the representation of a orifice is similar to a weir, with the exception of a gate. Flow across the orifice can therefore be of the following types: submerged weir flow, free weir flow, submerged orifice flow, free orifice flow or no flow (water levels below crest level or orifice closed) depending on the dimensions of the structure and the flow conditions.
The flow conditions are distinguished as follows: Orifice flow occurs when $h_1 - z \geq \frac{3}{2}d$. It is called free when $h_2 \leq z + d$ and submerged when $h_2 > z + d$. Weir flow occurs when $h_1 - z < \frac{3}{2}d$. It is called free when $h_1 - z > \frac{3}{2}(h_2 - z)$ and submerged when $h_1 - z \leq \frac{3}{2}(h_2 - z)$. The dimensions of the orifice which need to be specified are the width, crest level, initial openings height, contraction coefficient, lateral contraction and possible flow directions. The contraction coefficient and lateral contraction represent the energy loss that is caused by contraction of the flow towards the orifice. This phenomenon generally occurs when the weirs

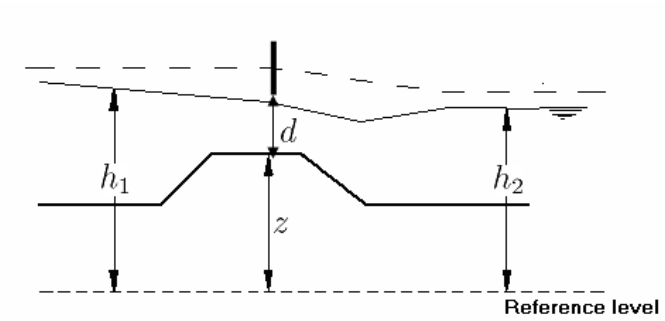


Figure 3.13: *Representation of a orifice, where h_1 is the upstream water level, h_2 is the downstream water level, z is the crest level and d is the opening height of the gate (opening level minus crest level).*

crest is less wide than the channel. In addition to defining the flow directions, it is also possible to define a maximum flow through the gate for each flow direction.
The adjustable part of a pump and orifice can be adjusted by a local controller. The parameters that can be controlled are the opening height of an orifice and the capacity of a pump. As mentioned in the Rainfall-Runoff subsection, it is also possible to control such parameters by global management strategies, which must be defined in the Real Time Control module. Controllable structures defined in the Channel Flow Module are, in this case, exclusively controlled by the Real Time Control module. This module is discussed in the next subsection.

3.2.3 The Real Time Control module

The Real Time Control module makes it possible to simulate complex real time control of all structures (described in Chapter 2) in the channal network. To use the Real Time Control module, it has to be switched on in the Settings taskblock. The module can not be run stand-alone, it always runs simultaneously with the Channel Flow and the Rainfall-Runoff module. In this case the module is linked to Matlab. There are a number of Matlab files, which together form the *standard control procedure*. These rules represent global management strategies or decisions that, under certain conditions, would be taken by the operators of the Frisian water system. It is possible to define multiple decision rules with different priorities. The module can also control the same parameter of more than one hydraulic structure, e.g. it is possible to control the capacity of a series of pumps by only one controller.
The module combines all sorts of global information to adjust the operation of the structures, instead of using only local information. The information that is used in this case are measured water levels and weather forecasts, such as precipitation and tide preditions. The precipitation is used to compute the lateral flow that could disturb the water system and the tide predictions together with the water levels determine how much water at which timesteps the system can drain off. The amount of water that needs to be drained off is computed by the difference between the weighted average water level of the storage canals (WASL), which is calculated using 6 measured water levels (Located as in Figure 2.2) and the target level. When the Real Time Control module has computed the decisions for the structures they are given as feedback to the Channel Flow module.

Chapter 4

Sorting Ensemble Prediction System scenarios

The model simulates the Frisian open water system by making a discretisation of the area into approximately 2300 segments. For all these segments the water level is simulated for the next 4 days. In general, too high and too low water levels lead to greater damage, so we want to decide which water level is likely to lead to greater damage. Hence a criterion on which we should sort is needed.
In the actual system, 6 points are located at which the water level is measured (See Figure 2.2), and out of these a weighted average water level of the storage canals (WASL) is calculated. This means that a water level that is the same everywhere and a water level that is low in one area and high in another will give the same WASL. It is clear that the last possibility is worst; in the area where the water level is high, the water could be passing its allowed limitations. Working with the WASL is therefore only desired in case one is interested in a weighted average of the extreme situations. The idea then is to sort on the highest and lowest weighted average water levels, whenever in the simulated 4 days.

Another criterion, that does not average out the wind influences, is to split up the total area into subareas to get representative local water levels. The aim for the division is, for simplicity reasons, to find as few as possible subareas, while keeping representative local water levels. How many subareas are required is mainly based on the dynamics of the system. In examining the dynamics of the system, it becomes clear which water levels (approximately) exhibit the same patterns and height and which do not. Also structure activity, velocity of the water flow and the physical geography reveal the differences between areas. The water levels with the same dynamics are clustered such that one local subarea is obtained. This is worked out in Appendix A. Then, per subarea 3 water level calculation points are chosen such that they geographically represent that subarea. Out of these 3 points an average water level per subarea is calculated and this water level is used. For further explanation see also Appendix A.
Due to different embankment heights, we must find a way to say something about the height of the water level. The simulation that is done by the model generates water levels with respect to the Mean Sea Level (MSL). So comparing them will give us the highest water level, but not compared to the surface level. In this case, where we are interested in detecting areas of flooding, this is what we need. Therefore the embankment height is taken as the measure

of the water level that locally may occur once in a 100 years. The idea then is to sort on the minimal and maximal differences between embankment height and water level (also called freeboard), whenever in the simulated 4 days and at whatever location.

A third criterion is to sort on the maximal and minimal water levels per subarea, the ones mentioned above and obtain one sorted list by taking the EPS-scenarios that in total occur most often. This means that, for example, the first element is not necessarily causing the absolute maximum, but is the EPS-scenario that causes the highest water levels in most subareas. Also, in some subareas high or low water levels cause more damage then in other subareas. Consideration of the different damage criterions between subareas can also be added in the sorting process.
In this way, many different criterions can be found. In this chapter, the first two criterions are used for sorting and are worked out in the next sections.

4.1 Sorting method 1

Running the model for all 50 EPS-scenarios, while focusing on January 2007 (16-01-2007 up to 20-01-2007), gives 50 different sequences of weighted average water levels for the next 4 days. These water levels are sorted using a sorting method and a sorted list of EPS-scenarios is obtained. The sorting method developed here is a Matlab code, which sorts on the highest and lowest peaks in the next 4 days. The list obtained is ordered in such a way that the first 25 elements (or EPS-scenarios) lead to the 25 highest peaks and the last 25 elements lead to the lowest points in the next 4 days.
For comparison, the precipitation (for all EPS-scenarios) is sorted by taking the sum of the amount of precipitation in the next 4 days, which is also what happens in the DSS. The sorted list obtained is ordered in such a way that the first element leads to the largest amount of precipitation.

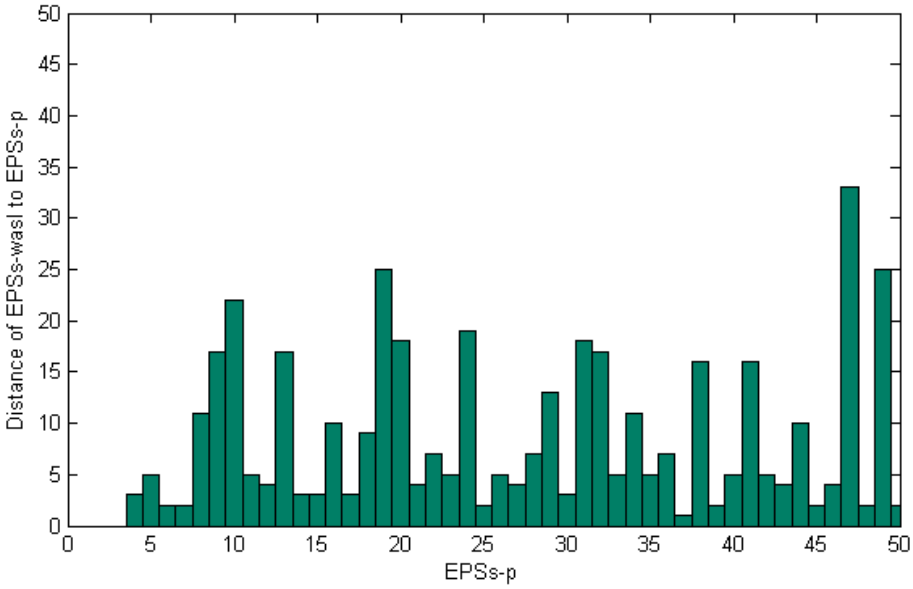
4.1.1 Results of sorting method 1

Sorting the 50 EPS-scenarios based on precipitation and on weighted average water levels, results in the Table 4.1. Here EPSs-p is the index of the sorted EPS-scenarios based on precipitation and EPSs-wasl is the index of the sorted EPS-scenarios based on the weighted average water levels.
In comparing the sorted lists, EPSs-p and EPSs-wasl, we want to say something about how well (or not) they resemble. For this the absolute distance (adist) of EPSs-wasl in relation to EPSs-p is determined and is also given in Table 4.1. The word distance here is used for the number of steps a EPS-scenario has shifted. Thus shifting from position i in EPSs-p to position j in EPSs-wasl gives the absolute distance $|i - j|$. The results are also given in a diagram and can be seen in Figure 4.1.
From these diagrams it can be seen that the number of cases, where the absolute distance of EPSs-wasl to EPSs-p is larger than 10, is 15. This means that the influence of the wind is large for at least 15 cases and is even larger for 4 cases, in which the distance is 20. When the distance for an EPS-scenario i is large, it says that this EPS-scenario has become more of less important for high or low water levels. At the same time, there are also 29 cases in which the absolute distance is less than or equal to 5. This indicates that for 50 % of all EPS-scenarios no major differences take place when sorting on WASL in stead of precipitation. This means

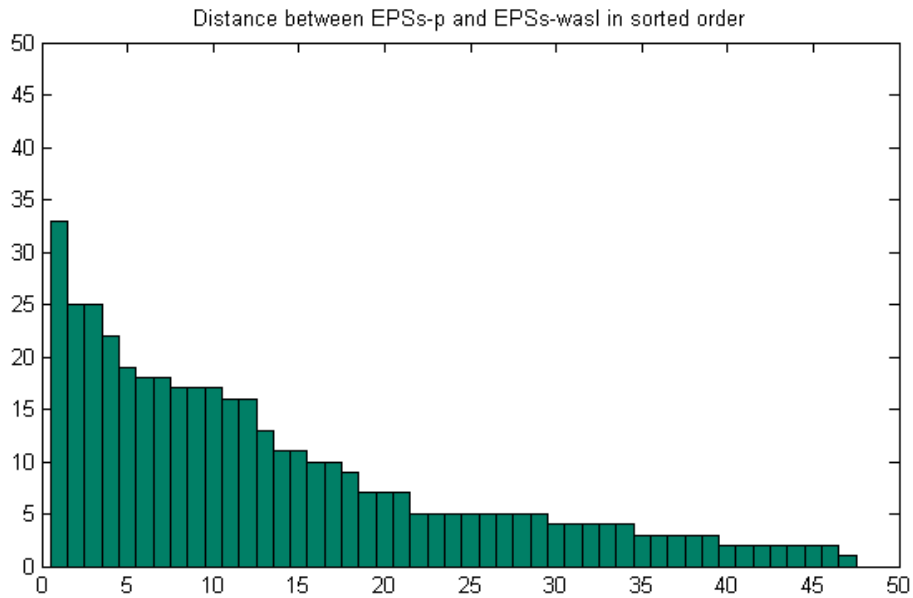
position	EPSs-p	EPSs-wasl	adist	position	EPSs-p	EPSs-wasl	adist
1	49	49	0	26	37	16	5
2	17	17	0	27	35	18	4
3	19	19	0	28	15	33	7
4	38	46	3	29	40	8	13
5	7	12	5	30	47	22	3
6	46	13	2	31	32	35	18
7	12	38	2	32	45	10	17
8	24	30	11	33	33	47	5
9	16	41	17	34	23	20	11
10	10	7	22	35	29	15	5
11	44	34	5	36	8	2	7
12	30	1	4	37	2	26	1
13	22	32	17	38	3	28	16
14	34	9	3	39	25	43	2
15	1	6	3	40	4	29	5
16	13	44	10	41	36	25	16
17	31	39	3	42	26	40	5
18	41	5	9	43	43	42	4
19	14	24	25	44	20	14	10
20	28	31	18	45	48	4	2
21	39	37	4	46	21	27	4
22	6	3	7	47	9	48	33
23	5	23	5	48	27	11	2
24	42	50	19	49	50	45	25
25	18	36	2	50	11	21	2

Table 4.1: *The sorted EPS-scenarios based on precipitation (EPSs-p) and weighted average water levels (EPSs-wasl) and the absolute distance (adist) between them.*

that the sorting based on WASL resembles the sorting based on precipitation. This is related to the fact that the WASL is a weighted average of the storage canals, in which the wind effects are cancelled out.



(a) Distance from EPSs-p to EPSs-wasl



(b) Sorted distance from EPSs-p to EPSs-wasl

Figure 4.1: *Results of sorting method 1.*

4.2 Sorting method 2

In Appendix A 11 subareas have been determined, and for each of these a representative water level exists. In deriving these subareas, it was explained that subarea 11 is an isolated area in which the water level is completely controlled by structures. The water levels of this area will therefore not be taken into account in the sorting method. Running the model for

all 50 EPS-scenarios, while focusing on January 2007 (16-01-2007 up to 20-01-2007), gives 500 different sequences of water levels (10 per EPS-scenario) for the next 4 days. These water levels are sorted using a sorting method and a sorted list of EPS-scenarios is obtained. The sorting method developed here is a Matlab code, which sorts on the highest and lowest peaks in the next 4 days at whatever location in which the highest and lowest points are determined with respect to embankment height. The list obtained is ordered in such a way that the first 25 elements (or EPS-scenarios) result in the highest peaks and the last 25 elements result in the lowest points in the next 4 days and at whatever location.
The precipitation (for all EPS-scenarios) is sorted by summing the amount of precipitation in the next 4 days. This also results in a sorted list of EPS-scenarios. These two lists are then compared.

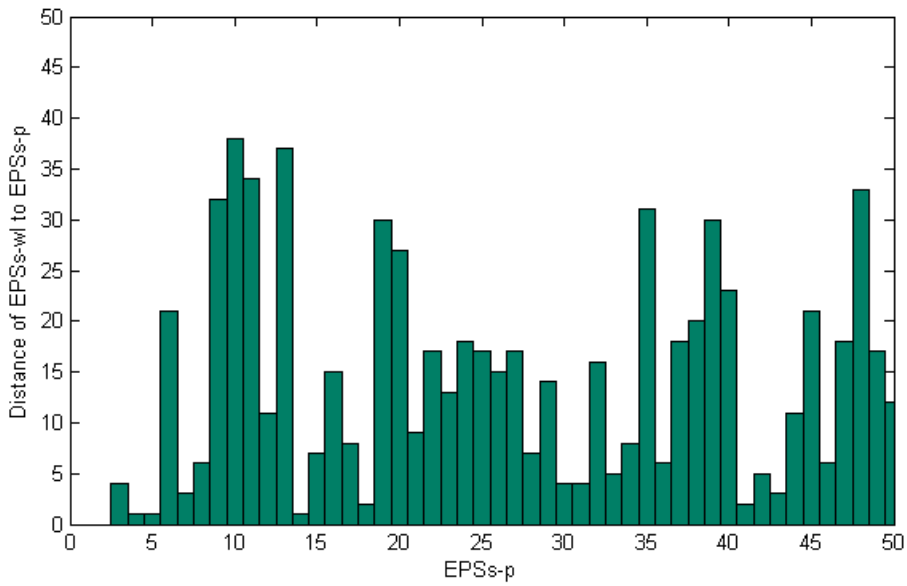
4.2.1 Results of sorting method 2

Sorting the 50 EPS-scenarios based on precipitation and on water levels, results in the following Table 4.2. Here EPSs-p is the index of the sorted EPS-scenarios based on precipitation and EPSs-wl is the index of the sorted EPS-scenarios based on the water levels.

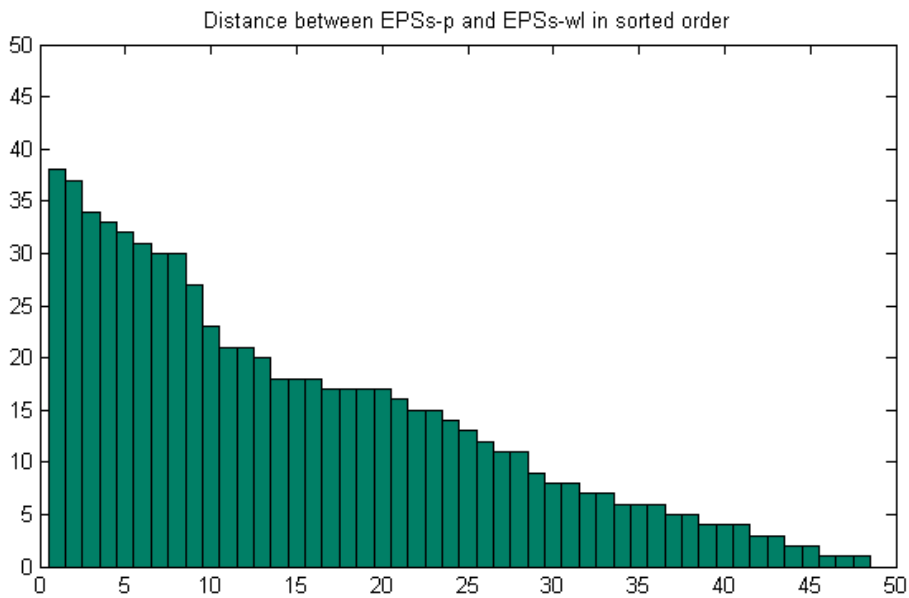
position	EPSs-p	EPSs-wl	adist	position	EPSs-p	EPSs-wl	adist
1	49	49	0	26	37	23	15
2	17	17	0	27	35	46	17
3	19	38	4	28	15	33	7
4	38	29	1	29	40	9	14
5	7	6	1	30	47	8	4
6	46	7	21	31	32	13	4
7	12	19	3	32	45	50	16
8	24	18	6	33	33	20	5
9	16	25	32	34	23	47	8
10	10	12	38	35	29	32	31
11	44	37	34	36	8	5	6
12	30	39	11	37	2	26	18
13	22	34	37	38	3	11	20
14	34	24	1	39	25	36	30
15	1	27	7	40	4	21	23
16	13	45	15	41	36	16	2
17	31	4	8	42	26	42	5
18	41	3	2	43	43	40	3
19	14	2	30	44	20	35	11
20	28	41	27	45	48	44	21
21	39	15	9	46	21	43	6
22	6	1	17	47	9	28	18
23	5	30	13	48	27	10	33
24	42	48	18	49	50	14	17
25	18	31	17	50	11	22	12

Table 4.2: *The sorted EPS-scenarios based on precipitation (EPSs-p) and water levels (EPSs-wl) and the absolute distance (adist) between them.*

In comparing the sorted lists, EPSs-p and EPSs-wl, we want to say something about how well (or not) they resemble. For this the absolute distance (adist) of EPSs-wl in relation to EPSs-p is determined and is also given in Table 4.2. The absolute distance is, like in the previous section, equal to $|i - j|$, where i is the position of an element from EPSs-p and j is the position of this same element from EPSs-wasl. The results are also given in a diagram and can be seen in Figure 4.2.



(a) Distance from EPSs-p to EPSs-wl



(b) Sorted distance from EPSs-p to EPSs-wl

Figure 4.2: *Results of the sorting method.*

In the diagrams it can be seen that the number of cases in which the absolute distance is larger than 10, is 28. This means that the wind effects are large for more than 50 % of all EPS-scenarios. Of these 28 cases, 12 cases have an even larger absolute distance, namely one that is larger than 20. Only 12 cases have an absolute distance that is less than 5. The influence of the wind can thus clearly be seen and it is the reason for the large differences between the sorted lists (EPSs-wl and EPSs-p) of EPS-scenarios.

4.3 Selecting 3 EPS-scenarios

As was explained in Chapter 1, out of the sorted EPS-scenarios, 3 EPS-scenarios must be selected for use in the DSS. There are many ways of selecting 3 EPS-scenarios. It depends on the open water system and the user who is working with the DSS, which selection is best. One way of selecting would be to take EPS-scenario number 46 as the maximum prediction run and EPS-scenario number 5 as minimum prediction run, as in the case of selecting the EPS-runs based on precipitation, which was explained in Chapter 1. This way of selecting though, does not say that there is a 90 % respectively 10 % chance that the water levels will be below EPS-scenario 46 respectively EPS-scenario 5. Because the sorting was based on the highest and lowest water levels whenever in the simulated 4 days, this does not mean that the total water level will apear above or below another water level. Since the sorting is done from maximum to minimum water levels, EPS-run 46 has position 5 and EPS-run 5 has position 46 in the tabels of the previous sections. In this way the selected EPS-scenarios for sorting method 1 and 2 become:

	EPSs-run 46	EPSs-run 5
Sorting method 1	12	27
Sorting method 2	6	43

Table 4.3: *Selecting EPS-scenarios based on sorting method 1 and 2.*

The absolute distances that appear at these EPS-scenarios are not large. The largest absolute distance for sorting method 1 is 5 and for sorting method 2 it is 6. This means that the EPS-scenarios selected here are not very different from the selection based on sorting EPS precipitation runs. This holds for the situation of January 2007, using either one of these sorting methods, but in other situations a larger distance could be found. Since the value of selecting positions 5 and 46 is lost, also other positions may be chosen. For example, if positions 45 and 6 were selected, the largest distance for sorting method 1 would be 2, but for sorting method 2 it would be 21. So, it certainly also depends on which EPS-scenarios are selected. It is also possible to use the EPS-scenarios that are leading to the (overall) maximum and minimum water levels. For the average prediction run the deterministic prediction run can still be used, since it gives the best prediction. Any way of selecting 3 EPS-scenarios can easily be implemented.

Chapter 5

A simplified model

In selecting the input, namely the EPS-scenarios based upon a sorting method on the output, the water levels, the model must run a simulation 50 times. The model models the Frisian open water system by making a discretisation of the area into approximately 2300 segments. For all these segments the water level is simulated for the next 4 days. Obviously the use of this complex model will need quite some computation time. In every day use of the DSS, thus in real time, we will not have that amount of time. Crucial is to make a simplification of the model in which all physical elementary characteristics of the Frisian open water system are included. In this case, since we want to sort on water levels, it is essential that the two models, the model and a simplified model, generate approximately the same output.
After a simplified model has been obtained, the predicted water levels for the model and the simplified model are compared.

5.1 Building a simplified model

The Frisian storage canals were divided into 11 sub areas for sorting purposes (See Chapter 4 and Appendix A). A simplification of the model will be made based on this division. Every time the DSS runs a new simulation, an updated selection of the EPS-scenarios is needed within a few minutes. This means that the simplified model is used for sorting water levels only. In this project use of the Matlab System Identification Toolbox is made, where grey-box theory is applied. Grey-box theory indicates that beforehand a general linear state space structure of a model must be given and that in identifying the outputs, only a number of parameters will be adjusted until a (local) best fit is found. The structure of the simplified model is based on relatively simple physical characteristics and is derived in the next subsections. Although the parameters are the unknowns of the system, a very good initial estimate of the values must be given, because it determines to a high extent the best fit that is found. The latter makes that it is easier, or not to say necessary, to split the identification process up into two parts (See Figure 5.1).
First a system identification is made on the Rainfall-Runoff part of the model. This means that a linear state space structure is derived for this part, and best fitted parameters are to be found, such that the inflow onto the storage canals is similar to that of the model. Due to the model complexity, it is possible to compute and get hold of the inflow and thus use it for system identification. The second part involves the total model, but the best fitted parameters of the first identification part are used as known parameters here.

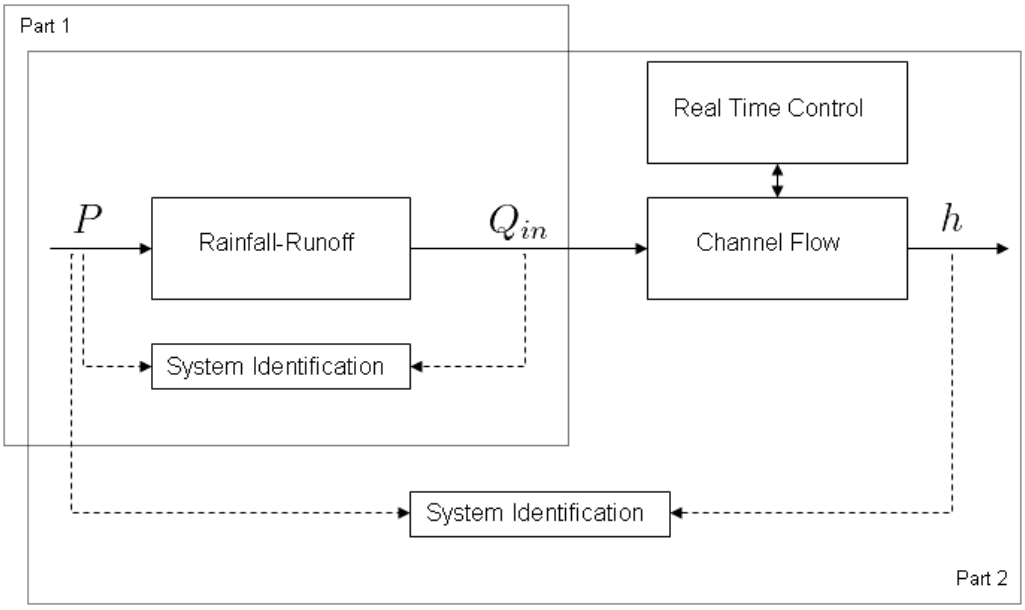


Figure 5.1: *System Identification on the Rainfall-Runoff part of the model (part 1) and on the total model (part 2).*

The situation of February 2002 (02-02-2002 up to 01-03-2002) is used in the identification process to obtain a simplified model, since this period was characterised by heavy storms. The high water levels outside the Frisian water system made it impossible to drain the excess water off and this resulted in local flooding. The situation of January 2007 (16-01-2007 up to 20-01-2007) is used here for validation of the simplified model that we have obtained.

5.1.1 System Identification on the Rainfall-Runoff part

The Rainfall-Runoff process can, for one area and highly simplified, be seen as a reservoir with a certain water level, an inflow due to precipitation and an outflow, which is an inflow onto the storage canals (See Figure 5.2).
This process can be captured into a first order mass balance equation:

$$\frac{dV(t)}{dt} = Q_p(t) - Q_{in}(t) \Longleftrightarrow \frac{dh(t)}{dt} = \frac{Q_p(t) - Q_{in}(t)}{A_s},$$

where V is the volume and A_s is the surface area of the reservoir, which is chosen to be constant. h is the water level in the reservoir. To arrive at the inflow due to precipitation, the latter is multiplied with an unknown constant c_1. The inflow onto the storage canals is mainly influenced by the water level in the reservoir. Therefore the water level is muliplied with an unknown constant c_2. The equations, $Q_p(t) = c_1 P(t)$ and $Q_{in}(t) = c_2 h(t)$, are used in the mass balance equation. The equation obtained is:

$$\frac{dQ_{in}(t)}{dt} = \frac{c_1 c_2}{A_s} P(t) - \frac{c_2}{A_s} Q_{in}(t).$$

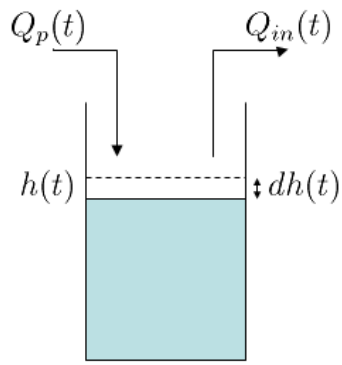


Figure 5.2: *The Rainfall-Runoff part (part 1) can be seen as a reservoir with a certain water level, an inflow due to precipitation and an outflow, which is an inflow onto the storage canals.*

Since the model computes the data per timestep ΔT, a discretization is made:

$$\begin{aligned} \Delta Q_{in}(k) &= \frac{c_1 c_2}{A_s}\Delta T P(k) - \frac{c_2}{A_s}\Delta T Q_{in}(k) \\ \Longleftrightarrow & \\ Q_{in}(k+1) &= bP(k) + aQ_{in}(k), \end{aligned}$$

where $b = \frac{c_1 c_2}{A_s}\Delta T$ and $a = (1 - \frac{c_2}{A_s}\Delta T)$.
Per sub area the parameters a and b are to be found. The simplified model will consist of 11 sub areas, but since sub area 11 is an isolated area, containing only the Lauwersmeer (See Chapter 2) in which other dynamics give the resulting water levels, this part is not considered here, and will be taken into account at a later stage. The Rainfall-Runoff of the 10 sub areas is given as a linear state space representation of the form:

$$\begin{aligned} x(k+1) &= Ax(k) + Bu(k), \\ y(k+1) &= Cx(k) + Du(k). \end{aligned}$$

With the discrete water balance equation for each sub area, the linear state space representation becomes

$$\begin{aligned} x(k+1) &= \begin{pmatrix} a_1 & & & \\ & a_2 & & \\ & & \ddots & \\ & & & a_{10} \end{pmatrix} x(k) + \begin{pmatrix} b_1 & & & \\ & b_2 & & \\ & & \ddots & \\ & & & b_{10} \end{pmatrix} u(k), \\ y(k+1) &= \begin{pmatrix} 1 & & \\ & \ddots & \\ & & 1 \end{pmatrix} x(k), \quad x_0 = x(0), \end{aligned}$$

where the input $u(k) = \begin{pmatrix} P_1(k) \\ \vdots \\ P_{10}(k) \end{pmatrix} \in \mathbb{R}^{10}$ is the vector of precipitation and the state and output $x(k) = y(k) = \begin{pmatrix} Q_{in1}(k) \\ \vdots \\ Q_{in10}(k) \end{pmatrix} \in \mathbb{R}^{10}$ is the vector of inflow onto the storage canals.
A Matlab code has been written to set-up and start the identification process for the Rainfall-Runoff part. The best fitted parameters a_i and b_i, for $i \in \{1, \ldots, 10\}$ that were found here are given in Appendix B. In order to find out how good the simplified Rainfall-Runoff model with the best fitted parameters is, not only for the situation used to derive these parameters, but as a general model to be used in all cases, the simplified model is tested for the situation of January 2007. The total inflow is a result of adding the inflow from the 10 subareas. This total inflow onto the storage canals has been simulated by the model and the simplified model and can be seen in Figure 5.3 and Figure 5.4.

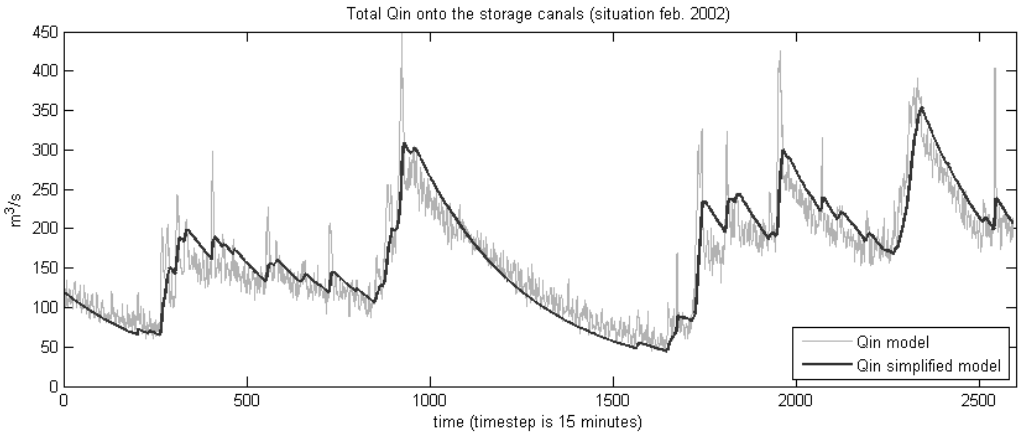


Figure 5.3: *The total inflow (summing the inflow of the 10 subareas) onto the storage canals in case of the situation of February 2002 used for identifying the two models.*

In the first figure it can be seen that the inflow of the obtained simplified model makes a good fit to the actual inflow. The peaks are not reached, but since these appear shortly, the volume of water that enters the storage canals is insignificant and will not influence the water levels much. The second figure shows an inflow that resembles the actual inflow a little less, but it is not bad at all. The two models both show three peaks, thus the reaction to the input is quite the same. Also the total volume that enters the storage canals is nearly the same. With this result, a total simplified model can be obtained.

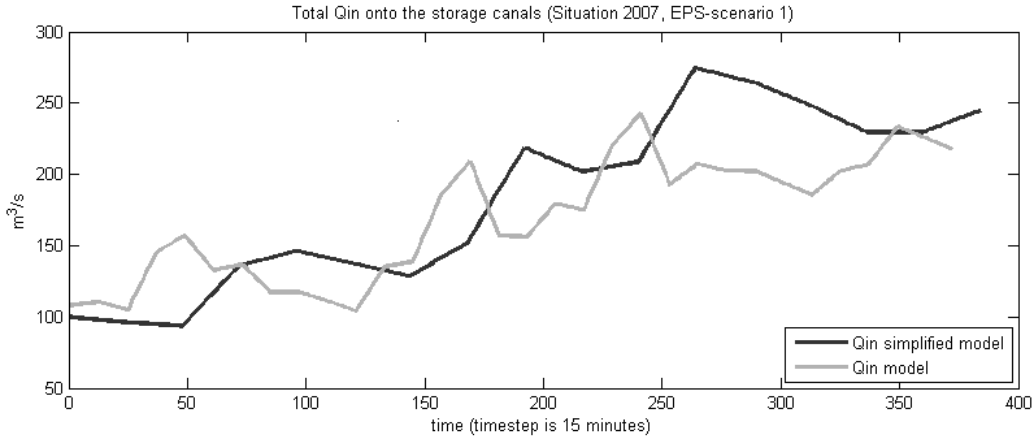


Figure 5.4: *The total inflow onto the storage canals in case of the situation of January 2007 used for validation of the simplified model. The total volume entering the storage canals is approximately the same.*

5.1.2 System Identification on the total model

Part 2 of the system identification process is about identifying the water levels in the 10 sub areas. The model uses the momentum equation and the mass balance equation to describe the flow in one dimension. A simplification of the mass balance equation is made, such that every Channel Flow process per sub area can be seen as a reservoir in which the water level is mainly a result of an inflow (Q_{in}), an outflow (Q_{out}) and an exchange flow (Q_e) between the reservoirs. The inflow is due to precipitation on the Rainfall-Runoff area and a (simplified) first order equation for this process was derived in the above subsection. The outflow is due to local water levels in- and outside the water system, and these are in turn influenced by the wind. The exchange flow is determined by the difference between water levels and the wind effects. Other influences on the water levels, such as evaporation or direct precipitation on the reservoir are not to be taken into account, since these factors are negligible compared to the other influences in high water situations. A first order mass balance equation for this process can thus be written as follows:

$$\frac{dV(t)}{dt} = Q_{in}(t) - Q_{out}(t) \pm Q_e(t) \Longleftrightarrow \frac{dh(t)}{dt} = \frac{Q_{in}(t) - Q_{out}(t) \pm Q_e(t)}{A_s},$$

where V is a volume and A_s is the surface area of the reservoir, which is chosen to be constant. h is the water level in the reservior. A discetization is made, since the data of the model has a certain chosen timestep ΔT:

$$\Delta h(k) = \frac{(Q_{in}(k) - Q_{out}(k) \pm Q_e(k))}{A_s}\Delta T \Longleftrightarrow h(k{+}1) = h(k){+}\frac{(Q_{in}(k) - Q_{out}(k) \pm Q_e(k))}{A_s}\Delta T.$$

The inflow is, as mentioned, generated due to precipitation, described in the first part of the system identification process. The outflow is the data series generated by the Real time Control part of the model for the situation of February 2002. The exchange flow is a dynamical process. A simplification and discretization of the momentum equation is made, so that the

description of the exchange flow between two resevoirs i and j becomes:

$$Q_e(k) = c(h_i(k) - h_j(k)) - w_f \frac{\tau_{wi}(k)}{\rho_w},$$

where c is a constant that acts like an inverse resistance of the flow between the two reservoirs. The symbol h stands for the water level in a reservoir, w_f is the (contant) flow width and ρ_w is the (contant) water density. Here, τ_{wi} is the wind shear stress and is expressed as:

$$\tau_{wi}(k) = -\rho_{air} \cdot C_{wi} \cdot v_{wi}^2(k) \cdot cos(\phi_{wi}(k) - \phi_r).$$

In this equation, ρ_{air} is the (contant) air density, and $C_{wi} = \alpha_{wi,1} + \alpha_{wi,2} v_{wi}(k)$ is called the wind friction coefficient and the values, used in the model and therefore also here, for $\alpha_{wi,1}$ and $\alpha_{wi,2}$ are respectively $0.5 \cdot 10^{-3}$ and $0.06 \cdot 10^{-3}$. v_{wi} is the velocity of the wind and ϕ_{wi} is the direction of the wind, and ϕ_r is the reach direction. The wind velocity and direction are, in the identification process, data series from measurements. The direction of the reaches are calculated with respect to northern wind and are derived in Appendix A. The only unknown parameters for which a best fit is found, are the constants c and w_f.
To get a clearer view of which sub areas are connected, thus where an exchange between reservoirs is possible and where an outflow is assigned, a schematic model has been made (see Figure 5.5). Here, the outgoing discharges represent the structures described in Chapter 2, where $Q_{out1} = P_1$: J.L. Hoogland, $Q_{out2} = P_2$: ir. D.F. Wouda, $Q_{out3} = G_1, Q_{out4} = G_2$: Tsjerk Hiddessluizen, $Q_{out5} = G_3, Q_{out6} = G_4, Q_{out7} = G_5, Q_{out8} = G_6$: Dokkumer Nieuwe Zijlen, $Q_{out9} = G_7$: Friese sluis Zoutkamp, $Q_{out10} = G_8, Q_{out11} = G_9, Q_{out12} = G_{10}$: Lauwersoog spuikokers.
Based on the mass balance equations, for both simplified Rainfall-Runoff reservoirs and Channel Flow reservoirs and the above schematic model, a linear state space model can be build:

$$\begin{aligned} x(k+1) &= \widetilde{A}x(k) + \widetilde{B}u(k), \\ y(k+1) &= \widetilde{C}x(k), \qquad\qquad x_0 = x(0), \end{aligned}$$

where the state $x(k) = \begin{pmatrix} h_1(k) \\ \vdots \\ h_{10}(k) \\ Q_{in1}(k) \\ \vdots \\ Q_{in10}(k) \end{pmatrix} \in \mathbb{R}^{20}$ and the output $y(k) = \begin{pmatrix} h_1(k) \\ \vdots \\ h_{10}(k) \end{pmatrix} \in \mathbb{R}^{10}$. We have $\widetilde{C} = \begin{pmatrix} 1 & & & 0 & & \\ & \ddots & & & \ddots & \\ & & 1 & & & 0 \end{pmatrix} \in \mathbb{R}^{10\times 20}$.

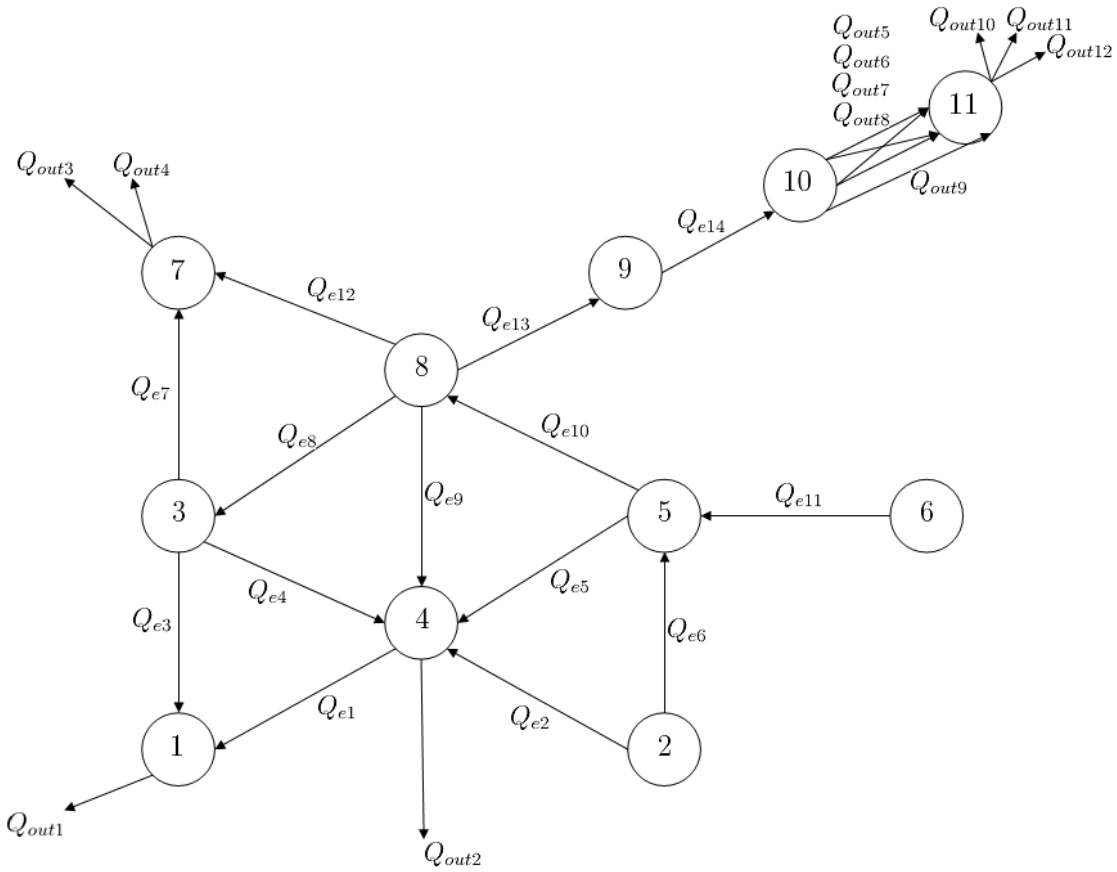


Figure 5.5: *Schematic model of the division made in Appendix A. This division into 11 sub areas is now also used for building a simplified model.*

The input $u(k) = \begin{pmatrix} Q_{out1}(k) \\ \vdots \\ Q_{out10}(k) \\ P_1(k) \\ \vdots \\ P_{10}(k) \\ \frac{\tau_{wi1}(k)}{\rho_w} \\ \vdots \\ \frac{\tau_{wi14}(k)}{\rho_w} \end{pmatrix} \in \mathbb{R}^{34}$. The matrix $\widetilde{A} = \begin{pmatrix} A_{11} & A_{12} \\ 0 & A_{22} \end{pmatrix} \in \mathbb{R}^{20\times 20}$ and $\widetilde{B} = \begin{pmatrix} B_{11} & 0 & B_{13} \\ 0 & B_{22} & 0 \end{pmatrix} \in \mathbb{R}^{20\times 34}$, where $A_{22} = A$ and $B_{22} = B$, which were derived in the previous subsection and the other matrices are presented on the next page.

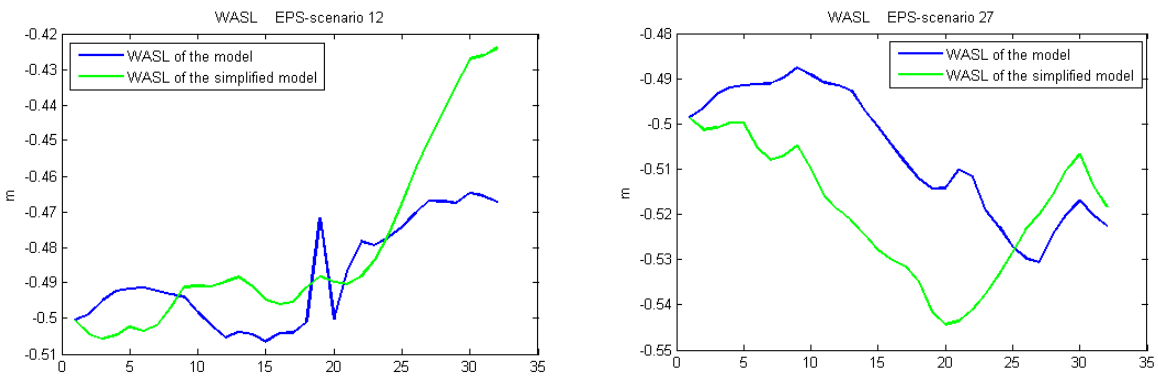
$$A_{11} = \Delta T \begin{pmatrix}
\frac{1+(-c_1-c_3)}{As_1} & 0 & \frac{c_3}{As_1} & \frac{c_1}{As_1} & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & \frac{1+(-c_2-c_6)}{As_2} & 0 & \frac{c_2}{As_2} & \frac{c_6}{As_2} & 0 & 0 & 0 & 0 & 0 \\
\frac{c_3}{As_3} & 0 & \frac{1+(-c_3-c_4-c_8-c_7)}{As_3} & \frac{c_4}{As_3} & 0 & 0 & \frac{c_7}{As_3} & \frac{c_8}{As_3} & 0 & 0 \\
\frac{c_1}{As_4} & \frac{c_2}{As_4} & \frac{c_4}{As_4} & \frac{1+(-c_1-c_4-c_9-c_5-c_2)}{As_4} & \frac{c_5}{As_4} & 0 & 0 & \frac{c_9}{As_4} & 0 & 0 \\
0 & \frac{c_6}{As_5} & 0 & \frac{c_5}{As_5} & \frac{1+(-c_{11}-c_{10}-c_5-c_6)}{As_5} & \frac{c_{11}}{As_5} & 0 & \frac{c_{10}}{As_5} & 0 & 0 \\
0 & 0 & 0 & 0 & \frac{c_{11}}{As_6} & \frac{1-c_{11}}{As_6} & 0 & 0 & 0 & 0 \\
0 & 0 & \frac{c_7}{As_7} & 0 & 0 & 0 & \frac{1+(-c_7-c_{12})}{As_7} & \frac{c_{12}}{As_7} & 0 & 0 \\
0 & 0 & \frac{c_8}{As_8} & \frac{c_9}{As_8} & \frac{c_{10}}{As_8} & 0 & \frac{c_{12}}{As_8} & \frac{1+(-c_{12}-c_8-c_9-c_{10}-c_{13})}{As_8} & \frac{c_{13}}{As_8} & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{c_{13}}{As_9} & \frac{1+(-c_{13}-c_{14})}{As_9} & \frac{c_{14}}{As_9} \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{c_{14}}{As_{10}} & \frac{1-c_{14}}{As_{10}}
\end{pmatrix}$$

$$B_{11} = \Delta T \begin{pmatrix}
\frac{w_{f1}}{As_1} & 0 & \frac{w_{f3}}{As_1} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & \frac{-w_{f2}}{As_2} & 0 & 0 & 0 & \frac{-w_{f6}}{As_2} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & \frac{-w_{f3}}{As_3} & \frac{-w_{f4}}{As_3} & 0 & 0 & \frac{-w_{f7}}{As_3} & \frac{w_{f8}}{As_3} & 0 & 0 & 0 & 0 & 0 & 0 \\
\frac{-w_{f1}}{As_4} & \frac{w_{f2}}{As_4} & 0 & \frac{w_{f4}}{As_4} & \frac{w_{f5}}{As_4} & 0 & 0 & 0 & \frac{w_{f9}}{As_4} & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & \frac{-w_{f5}}{As_5} & \frac{w_{f6}}{As_5} & 0 & 0 & 0 & \frac{-w_{f10}}{As_5} & \frac{w_{f11}}{As_5} & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{-w_{f11}}{As_6} & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & \frac{w_{f7}}{As_7} & 0 & 0 & 0 & 0 & \frac{w_{f12}}{As_7} & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{-w_{f8}}{As_8} & \frac{-w_{f9}}{As_8} & \frac{w_{f10}}{As_8} & 0 & \frac{-w_{f12}}{As_8} & \frac{-w_{f13}}{As_8} & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{w_{f13}}{As_9} & \frac{-w_{f14}}{As_9} \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{w_{f14}}{As_{10}}
\end{pmatrix}$$

$$A_{12} = -B_{11} = \begin{pmatrix}
\frac{\Delta T}{As_1} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & \frac{\Delta T}{As_2} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & \frac{\Delta T}{As_3} & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & \frac{\Delta T}{As_4} & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & \frac{\Delta T}{As_5} & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & \frac{\Delta T}{As_6} & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & \frac{\Delta T}{As_7} & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{\Delta T}{As_8} & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{\Delta T}{As_9} & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{\Delta T}{As_{10}}
\end{pmatrix}$$

5.2 A simplified model as result

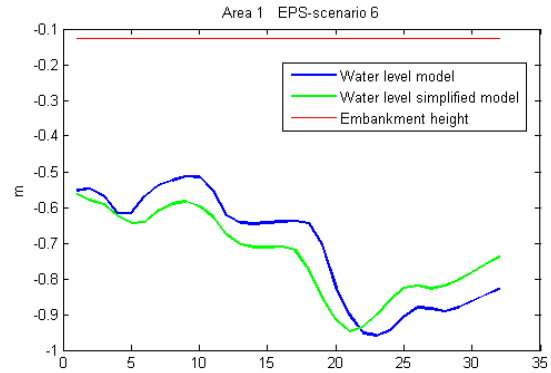
In the system identification process of the Channel Flow, the constants c_i and w_{fi}, for $i = \{1, \ldots, 14\}$, are adjusted in such a way that the output of a simplified model is identified with the output of the model. This is done using the same disturbances, which are precipitation, an outflow and a velocity and direction of the wind, onto both systems. In this way a simplified model is obtained that is found to be sufficient. In Appendix B the best fitted parameters are given. With the fitted parameters, the absolute eigenvalues (See Appendix B) of the matrix $\widetilde{A}$ are less than 1, so the obtained simplified model is stable. Also in Appendix B the water levels of both models, in case of the situation of February 2002 for identifying the two systems, can be seen. More important is though how the simplified model reacts to an other case, namely that of January 2007. To show how well the simplified model simulates the water levels, a comparisation between the water levels of the model (wl) and the water levels of the simplified model (h) is given. Since it is not worth wile to compare all water levels for all EPS-scenarios, only the water levels for the selected EPS-scenarios are shown. In Chapter 4, two selections of the EPS-scenarios were found; one based on sorting weighted average water levels (EPSs-wasl) and an other based on sorting water levels per sub area (EPSs-wl). The maximum prediction run based on EPSs-wasl is EPS-scenario 12 and the minimum prediction run here is EPS-scenario 27. The maximum prediction run based on EPSs-wl is EPS-scenario 6 and the minimum prediction run here is EPS-scenario 43. In Figure 5.6 the WASL for the maximum and minimum prediction run are simulated with both the model and the simplified model. In the Figures 5.7, 5.8, 5.9 and 5.10, the water levels per sub area, for the maximum and minimum prediction run are simulated with both the model and the simplified model.
The figures show a good similairity between the water levels. Local perturbations are not identified perfectly, but the overall behavior is alike. To clarify that the simplified model is good, the maximal difference for the selected EPS-runs is derived. The maximal difference is the (absolute) difference between the water levels of the model and the water levels of the simplified model.



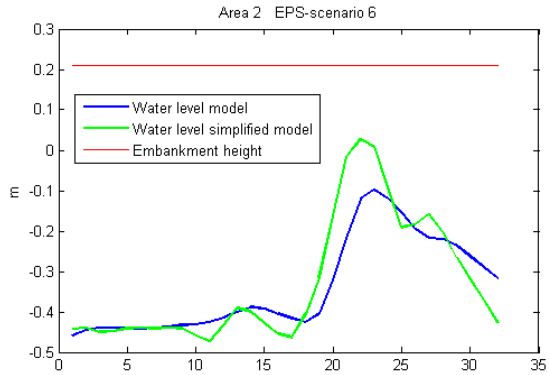
(a) WASL for the maximum prediction run. The difference = 0.0433 m.

(b) WASL for the minimum prediction run. The difference = 0.0334 m.

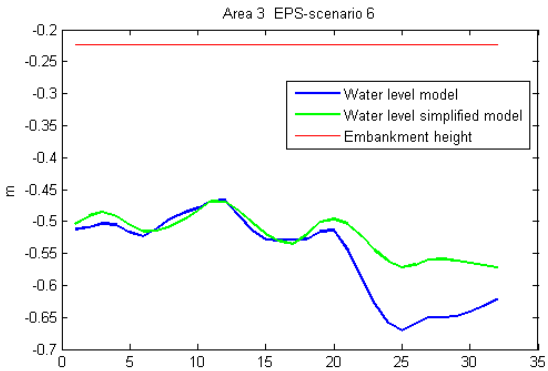
Figure 5.6: *Weighted average water levels of the storage canals for the maximum and minimum prediction runs, simulated with the model and the simplfied model (situation January 2007). The timestep is 3 hours.*



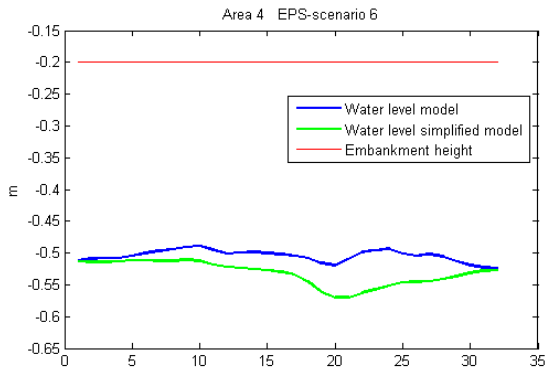
(a) Water levels for area 1. The difference = 0.1494 m.



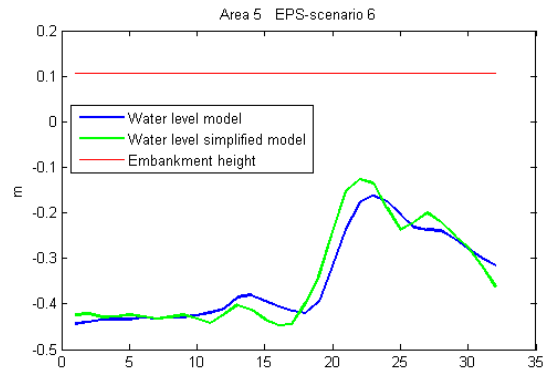
(b) Water levels for area 2. The difference = 0.2017 m.



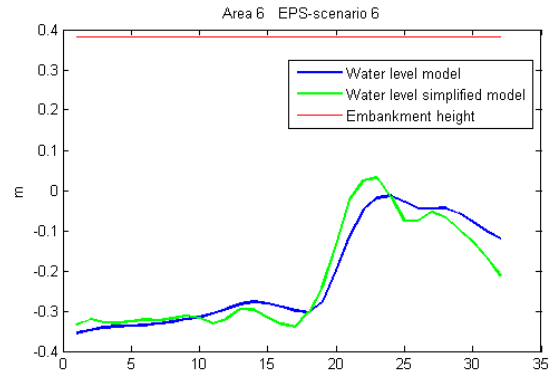
(c) Water levels for area 3. The difference = 0.0981 m.



(d) Water levels for area 4. The difference = 0.0646 m.

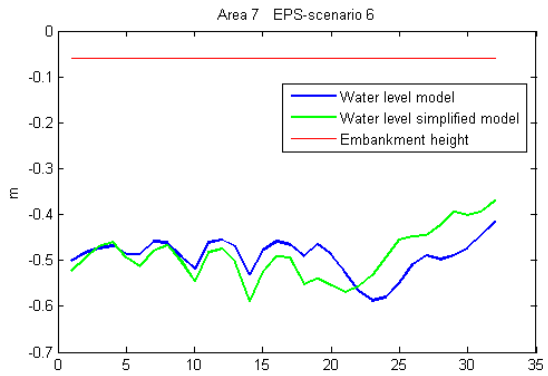


(e) Water levels for area 5. The difference = 0.0836 m.

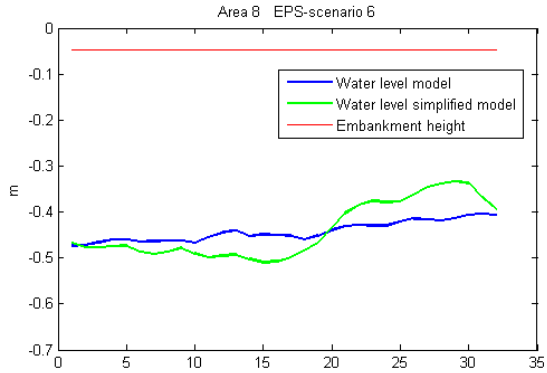


(f) Water levels for area 6. The difference = 0.0916 m.

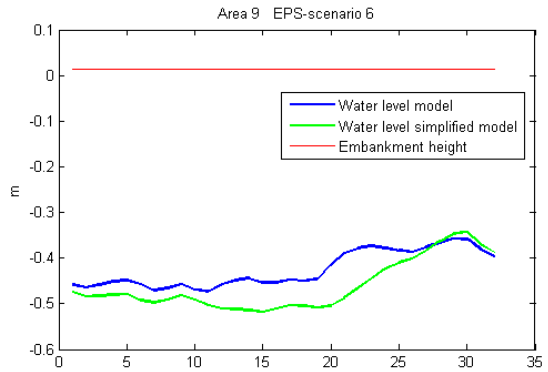
Figure 5.7: *Water levels of the 10 sub areas for the maximum prediction run, simulated with the model and the simplified model (situation January 2007). The timestep is 3 hours.*



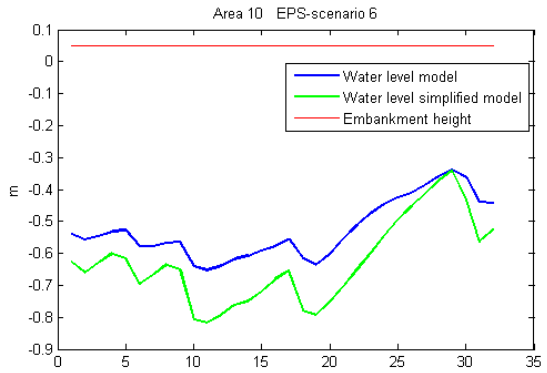
(a) Water levels for area 7. The difference = 0.0950 m.



(b) Water levels for area 8. The difference = 0.0807 m.

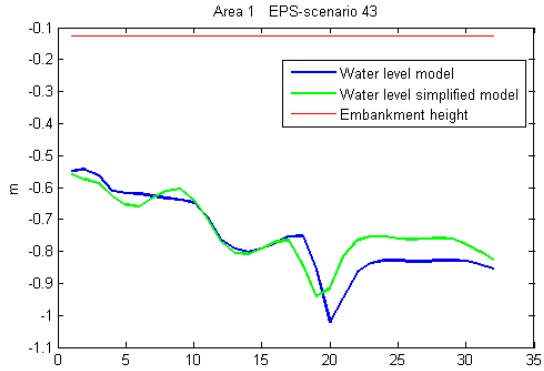


(c) Water levels for area 9. The difference = 0.0978 m.

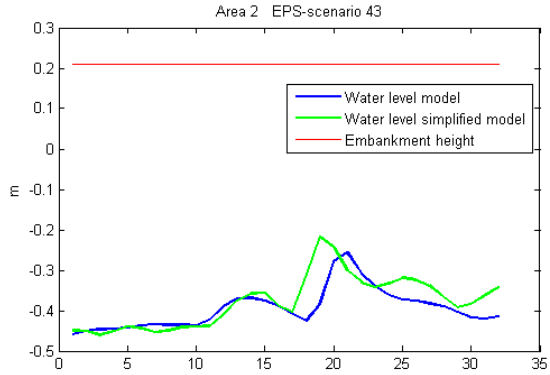


(d) Water levels for area 10. The difference = 0.1670 m.

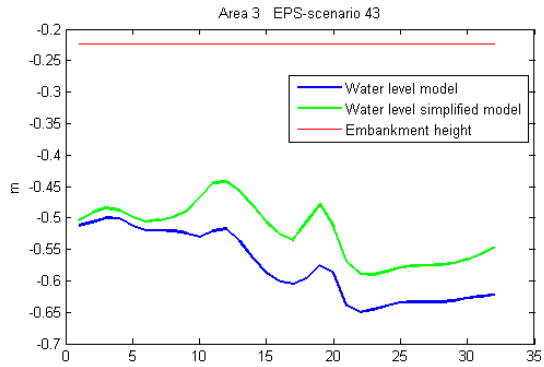
Figure 5.8: *Water levels of the 10 sub areas for the maximum prediction run, simulated with the model and the simplified model (situation January 2007) (continued). The timestep is 3 hours.*



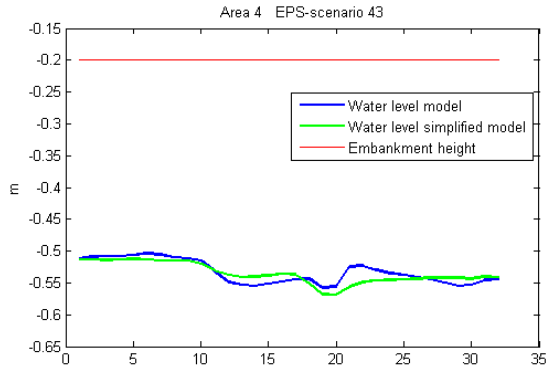
(a) Water levels for area 1. The difference = 0.1305 m.



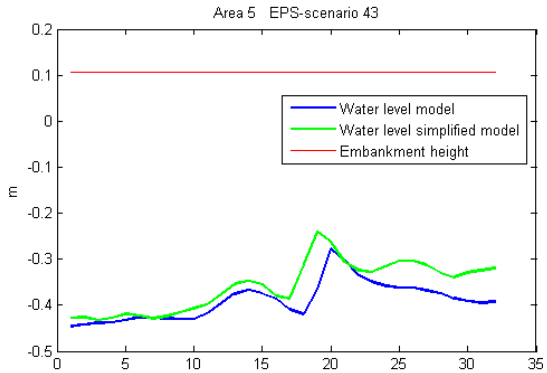
(b) Water levels for area 2. The difference = 0.1680 m.



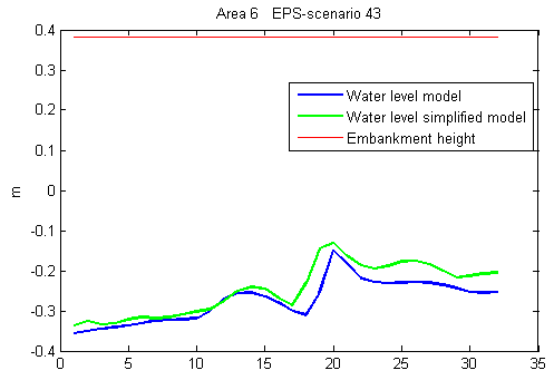
(c) Water levels for area 3. The difference = 0.0975 m.



(d) Water levels for area 4. The difference = 0.0314 m.

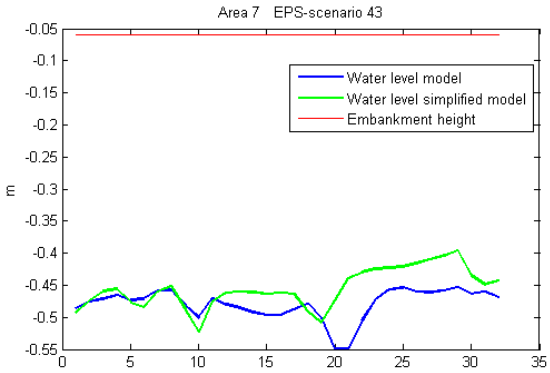


(e) Water levels for area 5. The difference = 0.1250 m.

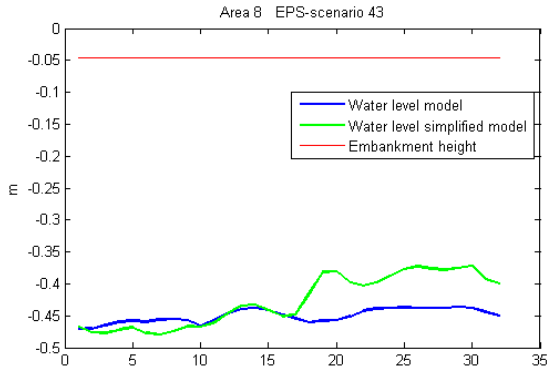


(f) Water levels for area 6. The difference = 0.1103 m.

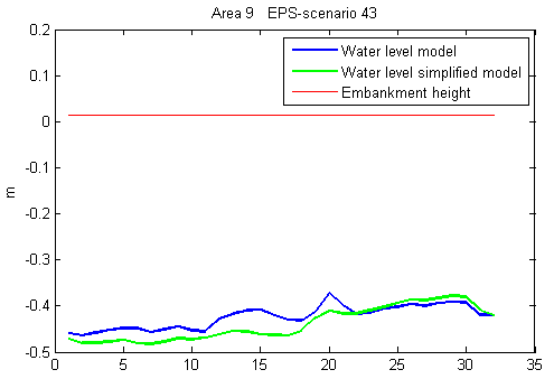
Figure 5.9: *Water levels of the 10 sub areas for the minimum prediction run, simulated with the model and the simplified model (situation January 2007). The timestep is 3 hours.*



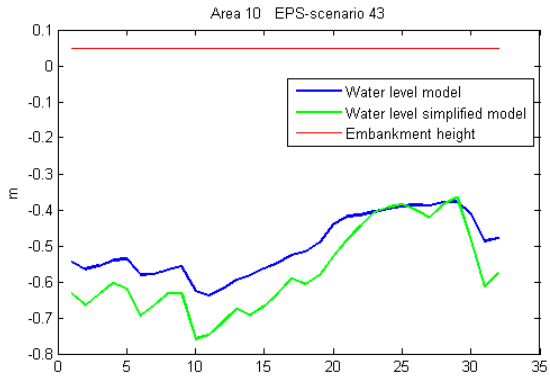
(a) Water levels for area 7. The difference = 0.1098 m.



(b) Water levels for area 8. The difference = 0.0771 m.



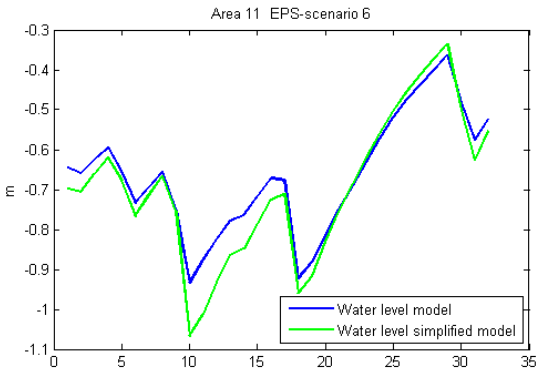
(c) Water levels for area 9. The difference = 0.0528 m.



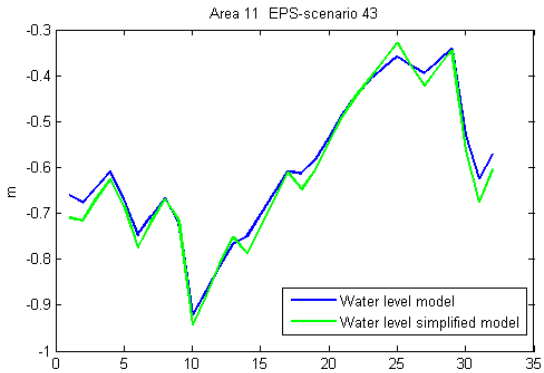
(d) Water levels for area 10. The difference = 0.1324 m.

Figure 5.10: *Water levels of the 10 sub areas for the minimum prediction run, simulated with the model and the simplified model (situation January 2007) (continued). The timestep is 3 hours.*

So far, 10 sub areas have been identified. Sub area 11 is an isolated area, controlled by structures and can therefore be added without the need to fit parameters. The water level in this sub area is, for simplicity reasons, a result of an inflow from sub area 10 and an outflow onto the Waddenzee, which are both controlled by structures, and an inflow from the province Groningen. The model uses the standard control procedure, which was explained in Chapter 3, to simulate the structure activity. These rules are also used in the simplification of the model. This is possible, since the water level in sub area 10 has already been modelled, a prediction for the tide-setup is available and the inflow from Groningen is taken to be the same as the flow simulated in the model. The water level in sub area 11 is simulated this way and is given in Figure 5.11. Also here local perturbations are not exactly the same, but the overall behavior is similar. The simplified model of sub area 11 is good enough. Thus, a total simplified model, which is stable, has now been obtained.



(a) Water levels for area 11 with the maximum prediction run. The difference = 0.1356 m.



(b) Water levels for area 11 with the minimum prediction run. The difference = 0.0516 m.

Figure 5.11: *Sub area 11 is added to the simplified model. The simulated water level for both the model and simplified model (situation January 2007). The timestep is 3 hours.*

Chapter 6

Conclusions and Recommendations

6.1 Conclusions

The aim of this thesis was to investigate the applicability of deriving realistic maximum, average and minimum EPS-scenarios based on sorted water levels for use in a real time Decision Support System for an open water system.

Making a simulation with all Ensemble Prediction System-scenarios has shown to take too much computing time. Therefore, 3 EPS-scenarios were selected for real time use of the DSS. This selection was based on sorting predicted water levels. In order to do this, it is required to know what the effects of each EPS-scenario is on the water levels. In Chapter 3, it has been made clear that the model (SOBEK) contains 3 modules that work together and cover all crucial dynamics, in such a way that a accurate simulation of the Frisian water system is obtained. This means that the model is appropriate for use in sorting predicted water levels. Three different options of water levels that could be used for sorting, were given in Chapter 4. Results of two of the three options were shown. The conclusion of Chapter 4 was that a selection of EPS-scenarios based on sorted water levels gives more realistic maximum and minimum prediction runs. Because the deterministic prediction run is the best prediction run, this run can operate as the average prediction run.
The conclusion of Chapter 5 is that it is possible to obtain a good simplified model, such that a real time implementation of the sorting method is possible.

6.2 Recommendations

- During sorting method 2, the water levels where determined with respect to the local embankment height. The measure of the embankment height was taken as the water level that locally may occur once in a 100 years. Although this is a good measure, the embankment heights in the actual water system are not everywhere as they, according to this measure, should be. Using the actual embankment heights per sub area could make an improvement.

- The conclusions of sorting method 2 can be amplified by investigation of another period in time, where for instance the wind direction is from the North-East. The wind direction that appears most often, in Friesland, is from the South-West and that is why the

embankment heights in the Frisian water system are made for such situations. Wind from the North-East many times results in less precipitation and will cause different problems.

Appendix A

Division of the storage canals into subareas

A.1 Reason for the division

A criterion on which water level we should sort is needed. One of the criterions that is given in Chapter 4, makes use of representative local water levels, such that the wind influences on smaller areas can be taken into account. That is why the total area will be split up into subareas.

A.2 How to make the division

One question is how many subareas we should take such that we get representative local water levels. The aim for the division is, for simplicity reasons, to find as few as possible subareas, while keeping representative local water levels. We will find the number of subareas we need by looking at the dynamics of the system. We will discover dynamics that are similar in one area and not in the next, by comparing different water levels, i.e. by comparing their shape and height. The influence of activating a structure can be clearly recognised in the shape and height of water levels. This is due to the fact that pumping stations may be switched on or off and the gates can only drain the excess water off during (low) low tide. The further away from the structure the less influence it has on the water levels. This is a reason for different dynamics in the water levels.
The velocity of the water flow could also be an indication of change in the dynamics of the water levels. This is because water only flows (fast) when a (large) difference between water levels is found. The large difference between water levels may be due to a constriction in the canal.
The physical geography is also an important aspect on which the division will be based. It is possible that the water levels of a small part of the storage canal coincide better with area n but that it can be more natural to add it to area m. The use of the division will be such that a greater accuracy is not needed. This will be further explained in the section choosing a representative water level per subarea.

A.3 How to make the division in practice

To be able to apply what has been explained in the previous subsection, we assume that the model is the real situation. The model simulates the water levels at calculation and connection nodes, which from now on we will call nodes. The idea is to cluster these nodes. Simulations of high water periods of approximately 18 days over the year 1971 until 2002 are used to make this possible. To make clear how a subarea is created, we discuss one area as an example.
In this example area there is a structure (denoted by s) leading the water into the Waddenzee (see Figure A.1).

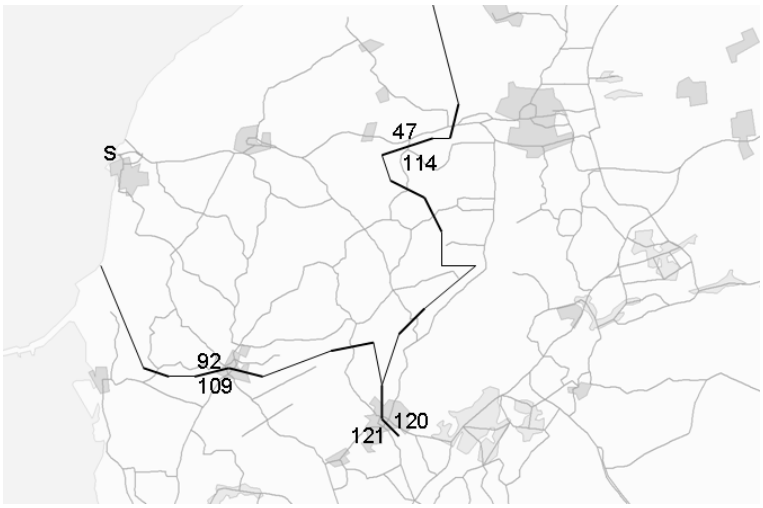


Figure A.1: *Example area, with a structure s. The numbers are located at locations where a cut is made to make a distinction between two areas. These cuts are derived in the following.*

The first thing we investigate are the water levels. Selecting an entire reach and plotting the sequence of nodes and zooming in to the year 2002 shows us the behaviour of the water levels. In this same period the structure discharge changes over time (Figure A.2, Figure A.3). When the structure is switched on and the water is drained off, the water levels close to it show rapid decay in height. At some point, further away from the structure, the water levels exhibit a more smooth behaviour. This is where a cut is made, in this case between the points 109 and 92 (thick black lines).

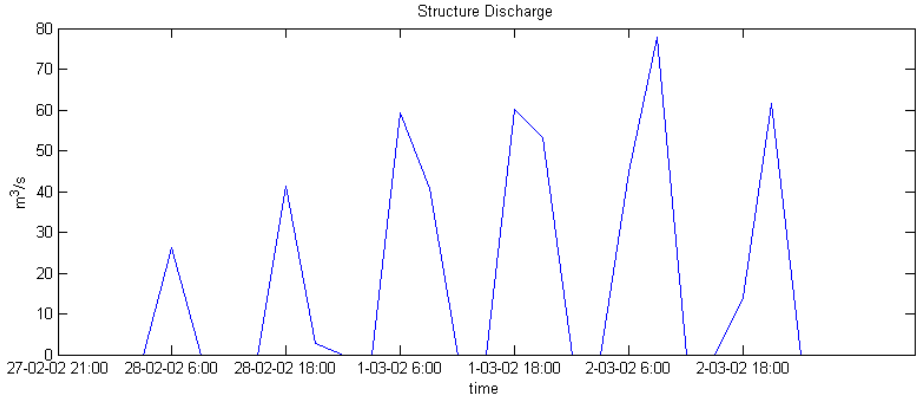


Figure A.2: *Structure discharge (2002)*

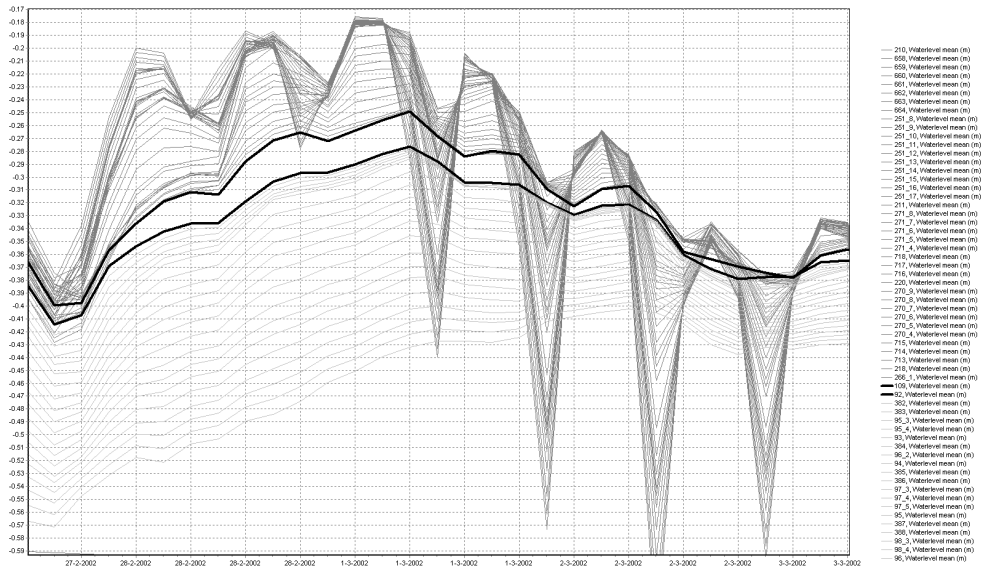


Figure A.3: *Cut between the water level of node 92 and node 109 (the thick lines). The water levels are of the same period as the structure discharge of Figure A.2. This structure activity influences the water levels. The water levels that are below the first thick line are further apart from the structure.*

To find any other important cuts, we can look at the velocities between 2 nodes. At one moment in time Figure A.4 is given.
In this figure, the darker reaches within the enlined region show (for the area we deal with here) a larger velocity then elsewhere. This means that the water levels in these regions may show differences in dynamics, since the velocity of the water flow could be an indication of change in the dynamics. This is because water only flows (fast) when a (large) difference in water levels is found. In Figure A.5 it can be clearly seen that at least between the water level nodes 47 and 114 (thick black lines) there should be made a cut. Since the water levels

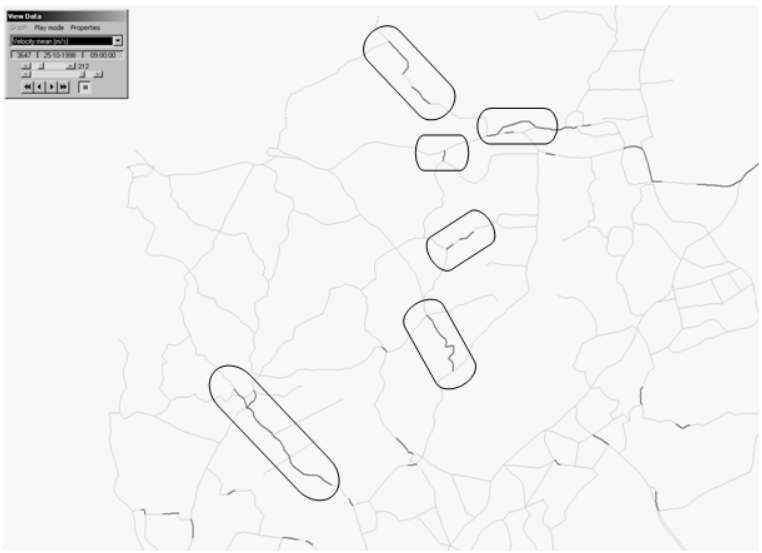


Figure A.4: *Velocities between nodes. The darker reaches show a high velocity.*

after water level node 47 (second thick line) exhibit a different behaviour then the ones before that node.

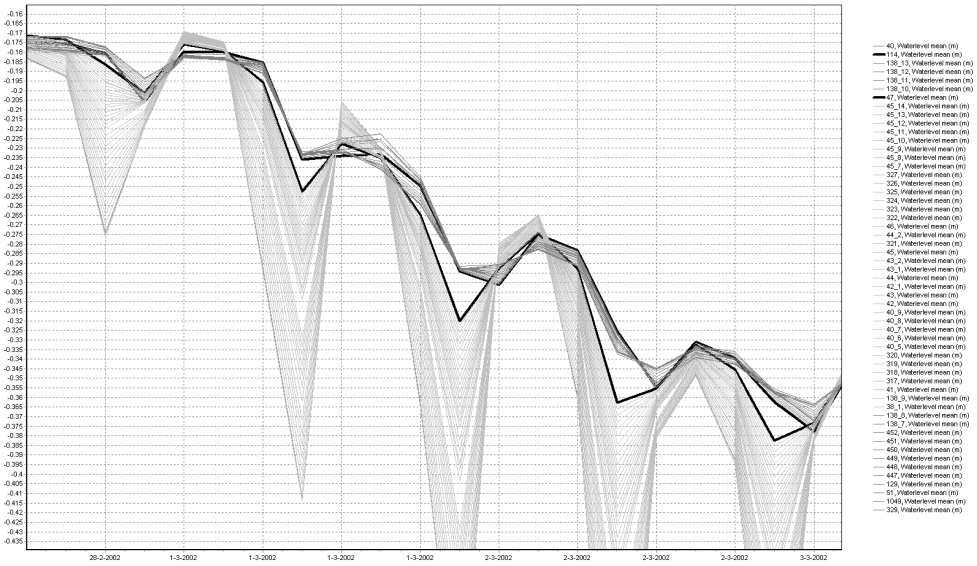


Figure A.5: *Cut between the water level of node 47 and node 114 (the thick lines).*

The other regions are also investigated, but no main cuts can be found. Though, later these regions will be used for an other purpose.
The cut that is made between the nodes 120 and 121 was not found by looking at the same period the structure was activated. Just by selecting an entire row of water level nodes, such that the water levels of a part of the channel can be shown, the change in dynamics can

become clear. In Figure A.6 the water levels that were selected are shown. The transition from water level node 120 to node 121 shows a change in dynamics.

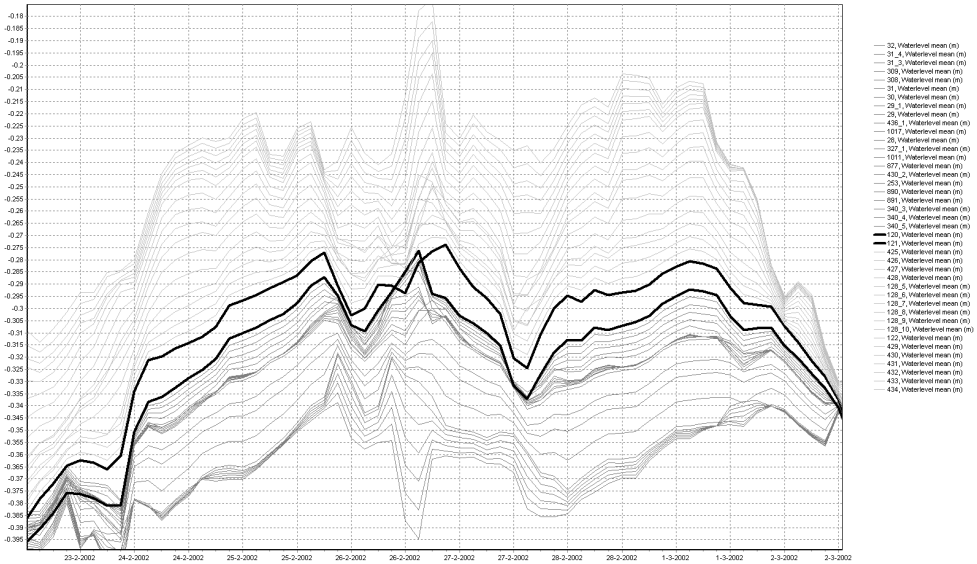


Figure A.6: *Cut between the water level of node 120 and node 121 (the thick lines).*

We now have found three main cuts and other locations where a cut could be made are not found. These three main cuts are used to mark out the subarea. The subarea has to be closed, so yet other cuts have to be made. Here we again use Figure A.4 with the dark reaches within the enlined regions. Searching in these areas and the areas that based on the geography would make a convenient cut, lead to the cuts seen in Figure A.1. Here the main cuts are denoted by a number, these are the numbers of the water level nodes that are located at these cuts.

A.4 A final cluster as result

Applying the actions, described in the previous section, to the total system we obtain a cluster of the storage canals. This can be seen in Figure A.7.

Figure A.7: *Cluster of nodes of the storage canals, where the areas are numbered.*

In this cluster, subarea 11 is created, because it contains a large lake that is completely controlled by structures and is therefore not influenced by the dynamics of other areas.
To get a clearer overview of which subareas are connected, a schematic model is made (see Figure A.8).

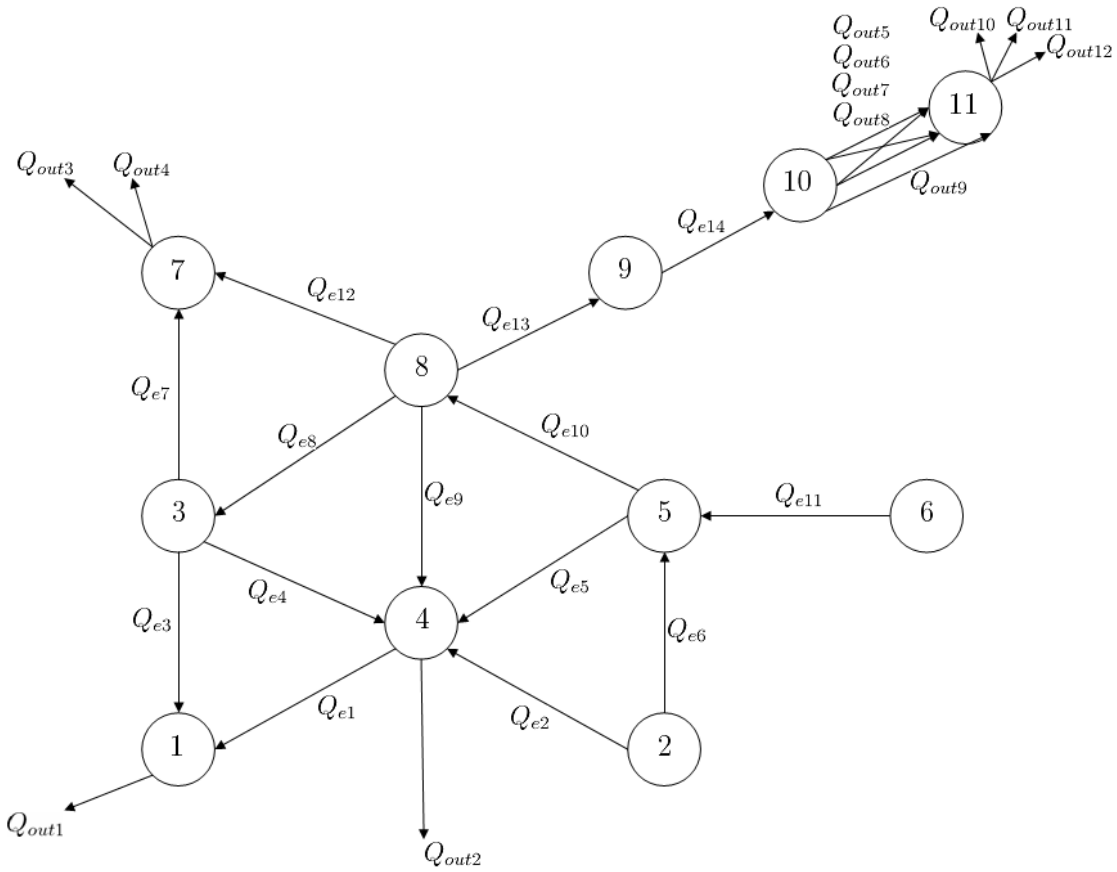


Figure A.8: *Schematic model of the obtained cluster.*

A.5 Obtaining a representative water level per subarea

The Frisian storage canals have now been divided into 11 subareas. The idea is that each subarea has one representative water level. Obtaining such a water level is done by selecting 3 water level points, such that they geographically represent the subarea they are divided into. The location of these points can be seen in Figure A.9. As mentioned in the previous section, area 11 is an isolated area, which really can be seen as having one water level. Due to this, only one water level point is chosen for area 11.
Out of these 3 water level locations, one average water level per subarea is calculated. Sorting EPS-scenarios will thus be based on these local representative water levels.

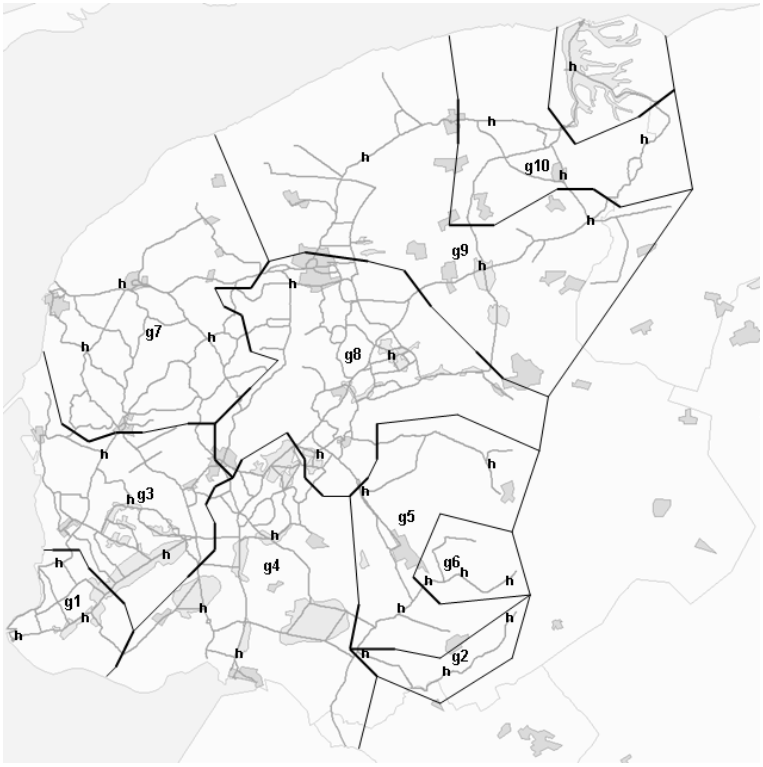


Figure A.9: *Per subarea 3 water level points (h) are chosen such that they geographically represent the subarea they are divided into. Each subarea has assigned a geographical point location, which is denoted by g. Why these geographical locations are needed is described in Appendix B.*

Appendix B

Results of the System Identification process

In Chapter 5 we have used System Identification techniques to obtain a simplified model of the model discussed in Chapter 3. The System Identification process has been divided into two parts; a Rainfall-Runoff part and a part involving the total model (including Channel Flow and Real Time control parts). In the second part the results of the first part are used. Grey-box theory indicates that beforehand a general linear state space structure of a model must be given and that in identifying the outputs, only a number of parameters will be adjusted until a (local) best fit is found.
The values of the (local) best fitted parameters a and b that were found in identifying the Rainfall-Runoff part are:

```
a=[0.9965;0.9967;0.9956;0.9948;0.9970;0.9986;0.9979;0.9968;0.9968;0.9965];
b=[0.2516;0.4549;1.4465;2.0631;1.1468;0.5339;0.8669;1.4606;1.7107;0.6029];
```

The values for the parameters a_i and b_i, for $i = \{1, \ldots, 10\}$, that were found are maybe not the global best fit. Nevertheless, these are used based on the sufficient similarities between the output of the obtained simplified model and the model. This is also true when we validate the obtained simplified model by using a different period (January 2007) than the one (February 2002) used in the identification process.

B.1 Obtaining the directions of the reaches between the subareas

The subareas are connected through reaches between them, as can be seen in Figure A.8. In deriving mass balance equations for the 10 subareas, such that a simplified version of the Channel Flow part is obtained, the exchange flow between two reservoirs must be calculated. This means that the effects of the wind on a reach must be computed. That is why for each reach (as in Figure A.8) the direction in degrees (ϕ_r) with respect to northern wind is derived. In the (graphical) representation of the model, coordinates of points in the middle of a subarea can be found. The geographical location of these points can be seen in Figure A.9 and are denoted by g.

The reaches can be seen as the vectors between the point locations and can therefore be calculated. A reach, where the length is of no importance, then has degree 0 if it has the same direction as wind from the north, which (since length is of no importance) can be expressed as $N = \begin{pmatrix} 0 \\ -1 \end{pmatrix}$. The angle ϕ_r between two vectors can be calculated with $r_i \cdot N = |r_i||N| \cdot cos\phi_r$, for $i = \{1, \ldots, 14\}$. Reaches that lay in the left quadrant, make angles larger than 180 degrees and are therefore calculated with respect to $-N$ and are added by 180 degrees.

The obtained reach directions in degrees

```
phi_r =[81.2889 114.7969 34.7755 61.2517 67.2046 155.2827 175.1685
        55.6696 20.0235 163.7856 128.9636 97.9288 132.2411 136.8381];
```

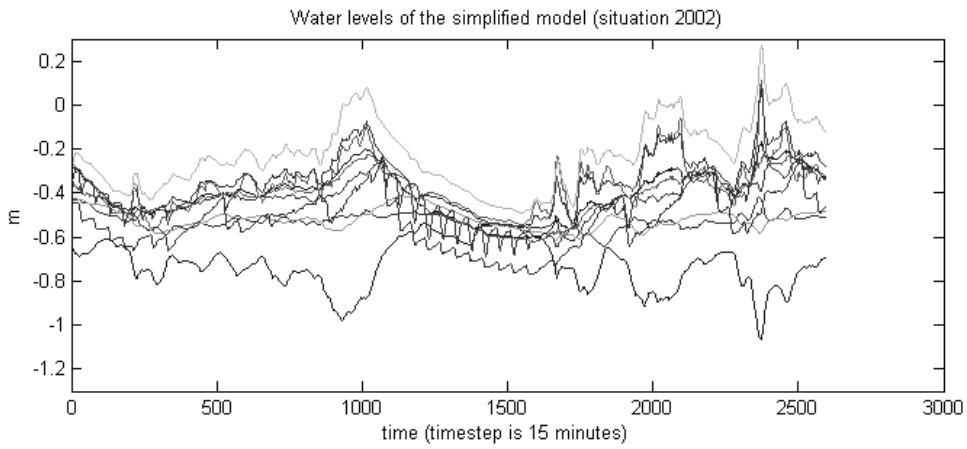
The reach directions have become known and a linear state space structure is derived (see Chapter 5), that is why the system identification process could be started.
The values of the (local) best fitted parameters c_i and w_{fi}, where $i = \{1, \ldots, 14\}$, that were found in identifying the total model are:

```
c1=102; c2=91.5; c3=586; c4=821; c5=183.3; c6=30.1; c7=97.3;
c8=19.7; c9=453; c10=385; c11=160; c12=376.8; c13=8829.6; c14=467;
wf1=0.9014; wf2=0.3; wf3=1.4422; wf4=1.9; wf5=0.7; wf6=0.066; wf7=0.17;
wf8=0.6; wf9=0.069; wf10=0.7; wf11=0.005; wf12=0.6; wf13=5.3; wf14=0.17;
```

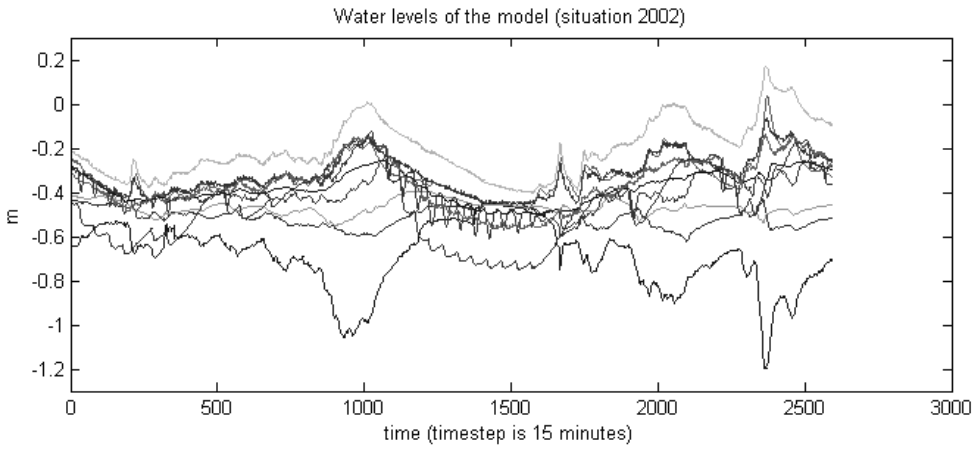
Using the above fitted parameters in the state space matrix $\widetilde{A}$, we find that the absolute eigenvalues of the matrix are less than 1, which means that the obtained simplified model is stable.
The eigenvalues of $\widetilde{A}$ are:

```
-0.214818612369637   0.994800000000000
 0.540139799859277   0.995600000000000
 0.999999999999999   0.996500000000000
 0.978416009789574   0.996500000000000
 0.960355310302331   0.996700000000000
 0.915394311577170   0.996800000000000
 0.894279753387748   0.996800000000000
 0.781636648510716   0.997000000000000
 0.849268384137439   0.997900000000000
 0.819134736917235   0.998600000000000
```

The water levels that have been simulated with the obtained simplified model are given in Figure B.1(a). These water levels can be compared to the simulated water levels of the model B.1(b).



(a) A simplified model



(b) The model

Figure B.1: *Simulated water levels for the 10 subareas with the simplified model and the model for the situation of February 2002.*

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