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# Chapter 1
# Predictive Control for National Water Flow Optimization in The Netherlands

P.J. van Overloop, R.R. Negenborn, B. De Schutter, and N.C. van de Giesen

**Abstract** The river delta in The Netherlands consists of interconnected rivers and large water bodies. Structures, such as large sluices and pumps, are available to control the local water levels and flows. The national water board is responsible for the management of the system. Its main management objectives are: protection against overtopping of dikes due to high river flows and high sea tides, supply of water during dry periods, and navigation. The system is, due to its size, divided into several subsystems that are managed by separate regional divisions of the national water board. Due to changes in local land-use, local climate, and the need for energy savings, the currently existing control systems have to be upgraded from local manual control schemes to regional model predictive control (MPC) schemes. In principle, the national objectives for the total delta require a centralized control approach integrating all regional MPC schemes. However, such centralized control is on the one hand not feasible, due to computational limitations, and on the other hand unwanted, due to the existing regional structure of the organization of the national water board. In this chapter the application of MPC is discussed for both individual regional control and coordinated national control. Results of a local MPC scheme applied to the actual water system of the North Sea Canal/Amsterdam-Rhine Canal are presented and a framework for coordination between several distributed MPC schemes is proposed.

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## 1.1 Introduction

### *1.1.1 Water infrastructures*

Water is the most vital element in human life. It is used for drinking, agriculture, navigation, recreation, energy production, etc. For these reasons, people tend to live close to water systems and therefore run an increased risk of getting flooded. In The Netherlands, improper management of water systems has led to higher damage than necessary as is clearly illustrated by the flooding of polders in February 1998, the shutting down of power plants in June 2003 (due to the limited availability of cooling water), and the yearly high mortality rate of fish due to algae bloom and low oxygen levels in the water.

To manage the human interaction with water systems, societies have formed organizations that were made responsible for managing certain tasks on particular water systems. This has resulted in a complex system of responsibilities that is not governed by the behavior of the water infrastructures themselves, but by the existing societal and organizational structures. These structures are hereby divided at a *spatial* level and at a *working field* level:

- At a spatial level the management of large rivers is divided into several parts. These large rivers almost always run through various countries. The management of the river in each country is an important national issue in which the inflows from and the outflows to the other counties are considered as given boundary conditions.
- A division by working field is apparent from the separated departments that manage a water system with their own isolated objectives. Water boards usually have one department that is responsible for the management of the water quantity variables and processes, such as water availability and flood protection, and another department that is responsible for water quality variables and processes, such as salinity control and water treatment. In reality, these variables and processes are all part of the same water network and therefore interact.

The spatial and working field division of water management is generally considered undesirable, but difficult to change. Many studies have been carried out on trans-boundary water management of rivers and the potential of integrated water management of water quantity and quality for canal systems [14, 15]. These studies have resulted in the formation of international agreements on river inflows and outflows at a national level and agreements on target values for water quantity and water quality variables, which are used by the different departments. The agreements are updated once every couple of years, but it is evident that the dynamic behavior of water systems requires coordination at a much higher frequency, e.g., daily or even hourly. The effects of climate change only add to this need: It is expected that precipitation will intensify on the one hand, while on the other hand periods of drought will last longer [5, 9]. In order to still guarantee safety and availability of water,

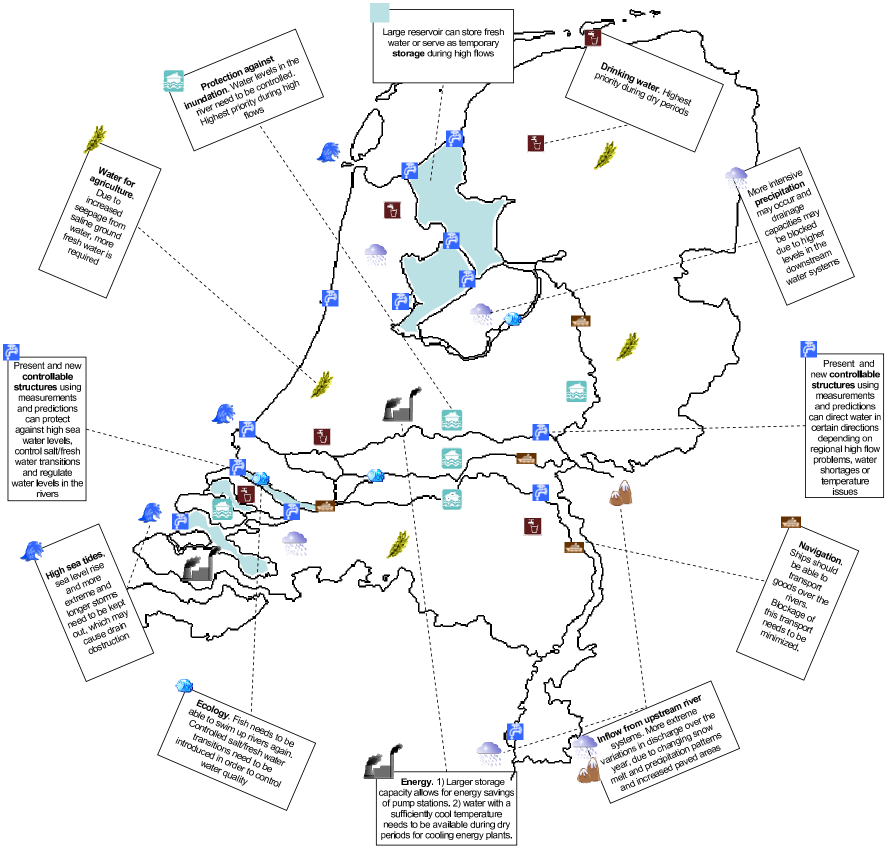



**Fig. 1.1:** The water system of The Netherlands with disturbances (High sea tides, Precipitation, Inflow from upstream rivers), objectives (Ecology, Water for agriculture, Drinking water, Navigation, Energy) and control structures (Controllable structures at the sea-side and in the river, storage).

more flexible agreements, that are updated continuously and take into account the limitations and possibilities of the infrastructure, have to be implemented.

### *1.1.2 Water system of The Netherlands*

Figure 1.1 presents the main rivers and lakes of the Dutch water system and a summary of the objectives, major control structures, and disturbances. In the East, the River Rhine enters The Netherlands at Lobith and in the South the Meuse River enters at Borgharen. Their combined flow varies over the year from 1000 to 10000 m$^3$/s. These rivers run from the South and the East to the sea in the North and the

(a) The Neder-Rhine at Driel.

(b) The Lower-Delta in Haringvliet.

**Fig. 1.2:** Controllable structures.

West. To protect the country from water excesses, in the last century, the main part of the Dutch estuary has been closed off from the sea by large dams and controllable gates and pumps. This has resulted in large reservoirs in the West and the North of the country. Downstream of these reservoirs, the fluctuating sea tide is present. Under normal conditions this sea tide fluctuates between -1 and +1 m. However, during storms, the water level can reach up to +4 m. During such extreme events, there is an excess of water that has to be prevented from flowing into the western and northern parts of the country, which lie below the mean sea level. However, in the summer time there is frequently a water deficit and water from the reservoirs has to be used as efficiently as possible.

Below a more detailed outline is given on the desired behavior of the Dutch water system, the actuators that can be manipulated in order to get as close as possible to this desired behavior, and the disturbances that complicate this.

#### 1.1.2.1 Objectives

There are a large number of different objectives with respect to water quantity and quality [5]. Water levels in the river have to be controlled in order to prevent inundation and flooding. This has the highest priority during periods with high river flows. It should also be ensured that sufficient drinking water is available for consumption. This has the highest priority during dry periods. Moreover, ships should be able to transport goods over the rivers. Blockage of this transport function has to be kept at a minimum. Also, energy savings of pump stations should be maximized, and water with a sufficiently cool temperature should be provided to energy plants. Fish should be able to swim up rivers again, and salt/fresh water transitions should be controlled to ensure a good water quality. Furthermore, increased seepage from saline ground water should be counteracted.

#### 1.1.2.2 Actuators

Several control options are available to manipulate the flows and water levels in The Netherlands. Water can be pumped into and taken out from the large reservoirs that can store fresh water or serve as temporary storage during high flows. Controllable structures can direct water in certain directions depending on regional high flow problems, water shortages, or temperature issues. Furthermore, controllable structures can also protect against high sea water levels, control salt/fresh water transitions, and regulate water levels in the downstream parts of rivers. Figure 2(a) presents a typical river structure, while Figure 2(b) is a controllable structure that blocks the sea from entering the inland [20, 21].

#### 1.1.2.3 Disturbances

The main disturbances that influence the large open water reaches and lakes of The Netherlands are the inflow of the rivers from Belgium and Germany, high sea tides that block the drainage capacity precipitation, and the water demand for agriculture, drinking water, and ecology. Predictions of these disturbances are currently available for up to 4 days ahead with sufficient accuracy based on measurements of river flows in the two upstream lying counties and forecasts of precipitation and snow melt [6]. The quality of these predictions is expected to improve over the coming decade. It is expected that future prediction systems can provide forecasts with a prediction horizon of up to 10 days.

#### 1.1.2.4 Complexity of the control problem

Whereas originally the main objective with respect to water systems was safety, as illustrated above, nowadays many different additional objectives need to be taken into account as well. Consequently, when considering the complete river delta as a single system, control of this system involves solving a large-scale, multi-objective, constrained control problem. This overall control problem cannot be solved by optimizing local control actions alone. Novel control approaches have to be developed in order to be able to coordinate locally optimizing control actions.

### *1.1.3 Automatic control of water systems*

Over the last decades, an evolution of automatic control applications in water systems has taken place. Concerning the day-to-day operation, the first attempts to implement automatic control were made by civil engineers and were based purely on feedforward control. The reason for this was that although accurate models useful for inverse modeling were available, no knowledge on feedback control was available among these civil engineers. The first successful implementations were based on feedback control [12, 19], either without or with feedforward control added. These controllers were able to keep water levels close to set-points and in this way ensured the availability of water in canals and reservoirs. Later on, projects in which controllers were successfully implemented were characterized by the collaboration between civil engineers and control engineers [2, 4, 10, 13]. A next generation of controllers that was then realized was based on the use of model predictive control (MPC) [22, 25]. This generation was able to take constraints into account. Currently, the integration of various objectives on different variables (water quantity and water quality) by means of MPC is being investigated to further improve performance.

Based on the present knowledge, the control problems for most water systems (rivers, canal networks, sewer systems, irrigation canals, and reservoirs) can be formalized into simplified models, objective functions, and constraints. These small-scale control problems can be solved with MPC. However, solving larger-scale control problems is not feasible for two reasons:

1. For larger-scale systems different parts of the system are owned by different organizational structures, that are not willing to give up their autonomy.
2. Even if all organizational structures would be willing to give up their autonomy, solving the optimization problem involved in MPC for the resulting large-scale system would require too much computational time.

Therefore, in the future, distributed MPC will have to be used in order to solve multiple local MPC problems in a coordinated way, such that overall optimal performance is obtained. Solving the control problems of the separate water systems, while having them negotiate on the interactions with other systems (managed by other organizations), is also in line with the present manner in which large-scale water systems are being managed.

### *1.1.4 Outline*

In this chapter, we present the state of the art in physical water infrastructure control using MPC and propose a framework for coordination between individual local MPC controllers. The chapter is organized as follows. In Section 1.2, the modeling and MPC control of open-water systems is introduced. Section 1.3 focuses on the application of MPC for control of a single physical discharge station in The Netherlands. A framework for achieving coordination among MPC controllers of multiple subsystems is proposed in Section 1.4. Section 1.5 concludes the chapter and provides directions for future research.

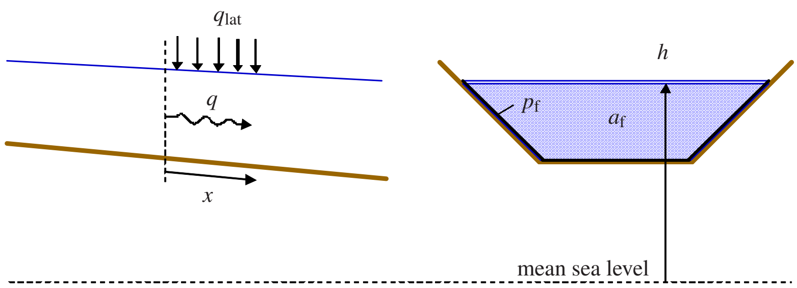



**Fig. 1.3:** Open-water canal variables and parameters.

## 1.2 Modeling and MPC control of open water systems

The Dutch river delta can be considered as a system of interconnected open-water canals, channels, reservoirs, and control structures. Below we describe how these components of the infrastructure are modeled, both for use in detailed simulation studies and for use in controller design. Hereby, the models used for controller design are simplified models derived from the detailed models used for simulations. In this way, solving the optimization problems involved in MPC has lower computational requirements.

### *1.2.1 Open canals and reservoirs*

#### 1.2.1.1 Open canals

The flows and water levels (water quantity variables) in an open canal can be described by the Saint-Venant equations [3]. These nonlinear hyperbolic partial differential equations consist of a mass balance and a momentum balance. The mass balance ensures the conservation of water volume, while the momentum balance is a summation of the descriptions for the inertia, advection, gravitational force, and friction force:

$$\frac{\partial q}{\partial x} + \frac{\partial a_f}{\partial t} = q_{lat} \tag{1.1}$$

$$\frac{\partial q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{q^2}{a_f}\right) + g a_f \frac{\partial h}{\partial x} + \frac{g q |q|}{c^2 r_f a_f} = 0, \tag{1.2}$$

where $q$ represents the flow (m$^3$/s), $t$ is the time (s), $x$ is the distance (m), $a_f$ is the wetted area (i.e., the cross sectional area that is wet) of the flow (m$^2$), $q_{lat}$ is the lateral inflow per unit length (m$^2$/s), $g = 9.81$ m/s$^2$ is the gravitational acceleration, $h$ is the water level (mMSL, i.e., meters above the Mean Sea Level (MSL)), $c$ is

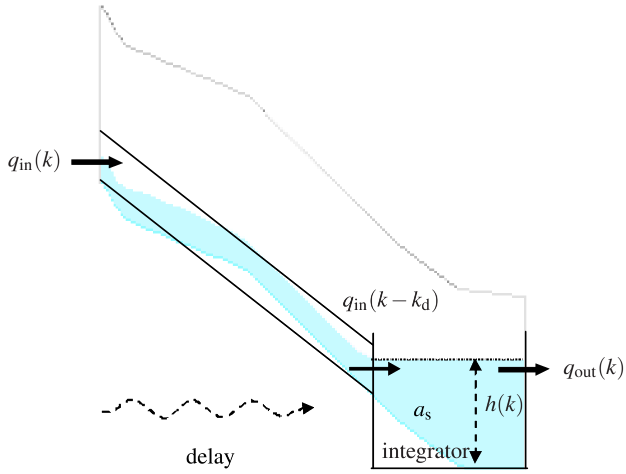



**Fig. 1.4:** Integrator delay model of a canal reach.

the Chézy friction coefficient ($m^{1/2}$/s), and $r_{\mathrm{f}}$ is the hydraulic radius (m), calculated as $r_{\mathrm{f}} = a_{\mathrm{f}}/p_{\mathrm{f}}$, where $p_{\mathrm{f}}$ is the wetted perimeter (m) (i.e., the perimeter of the cross sectional area that is wet). Figure 1.3 gives a schematic representation of a typical open canal with its parameters.

To use the formulas in a numerical model of a canal reach, the partial differential equations are discretized in time ($\Delta t$) and space ($\Delta x$). In case these discretized formulas are simulated, the model results in time series solutions of water levels and flows at discrete locations along the reach. Also, the time series are discrete solutions in time.

#### 1.2.1.2 Reservoirs

Reservoirs, such as lakes are modeled in a different way. Here, only the mass balance, as given in (1.1), is applied as an ordinary differential equation. The water level $h$ (m) is calculated as a function of the inflows and the outflows:

$$\frac{\mathrm{d}h(t)}{\mathrm{d}t} = \frac{q_{\mathrm{in}}(t) - q_{\mathrm{out}}(t)}{a_{\mathrm{s}}}, \tag{1.3}$$

where $q_{\mathrm{in}}$ represents the sum of the inflows in the lake ($m^3$/s), $q_{\mathrm{out}}$ the sum of the outflows ($m^3$/s), and $a_{\mathrm{s}}$ the surface area ($m^2$).

#### 1.2.1.3 Simplified models for controller design

For model-based controller design, the nonlinear partial differential equations are usually transformed into simplified, linearized, and discrete-time models. A popular model for the water quantity variables is the integrator delay model [19]. In this model, a discrete time step index $k$ is considered, and a discretized pure delay is placed in series with a discretized integrator. Figure 1.4 presents this model. The integrator delay model is given by the following equation:

$$h(k) = \frac{q_{\mathrm{in}}(k-k_{\mathrm{d}})\Delta t}{a_{\mathrm{s}}} - \frac{q_{\mathrm{out}}(k)\Delta t}{a_{\mathrm{s}}}, \tag{1.4}$$

where $k$ is the time step index and $k_{\mathrm{d}}$ is the number of delay steps.

This model functions properly for long canal reaches. For shorter reaches, resonance waves can occur that reflect on the boundaries of the reach. These need to be filtered out by means of low-pass filtering, before an integrator delay model can be fitted on the reach. The parameters can be derived from simple step tests or by application of system identification techniques [11]. Alternatively, a higher-order model can be used that does represent these waves [23].

### *1.2.2 Control structures*

Between canal reaches, river reaches, lakes, and the sea, controllable structures are present. By adjusting the setting of these structures, the flows between the water elements can be manipulated. The most common structures in a river delta are pumps and undershot gates.

#### 1.2.2.1 Pumps

Pumps can be modeled in a straightforward way by imposing a flow at a certain value. In case of frequency-driven pumps this value can be between zero and the maximum capacity of the pump. Many pumps can only be switched off or set to run at maximum pump capacity. In general, pump flows are usually only slightly influenced by the surrounding water levels. The maximum pump capacity reduces though, when the difference between the upstream and the downstream water level increases. In that case, with the same amount of energy being brought into the pump, the water needs to be lifted higher and consequently the flow decreases.

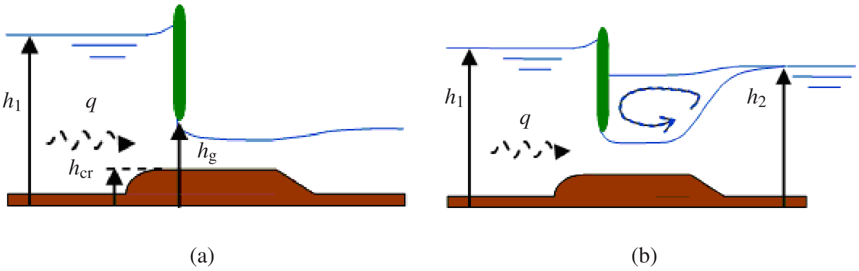



**Fig. 1.5:** Free-flowing (a) and submerged undershot gate (b).

#### 1.2.2.2 Undershot gates

Undershot gates have a gate that is put into the water from the top down. The water flows under the gate. The stream lines of the upper part of the flow, just before the gate, bend down to pass the gate opening, causing the actual flow opening to be contracted. Usually, the factor $\mu_g$ by which the flow opening is contracted when compared to the gate opening is 0.63 [1].

The flow through the undershot gate can be free or submerged. In general, an undershot gate is free flowing when the downstream water level is lower than the gate height, i.e., the bottom of the gate. It is submerged, when the downstream water level is higher than the gate height. Figure 1.5 presents a free-flowing and a submerged undershot gate.

The flow through a free-flowing and a submerged undershot gate are given by:

$$q(k) = c_g w_g \mu_g (h_g(k) - h_{cr}) \sqrt{2g \left( h_1(k) - h_{cr} + \mu_g (h_g(k) - h_{cr}) \right)} \tag{1.5}$$

and

$$q(k) = c_g w_g \mu_g (h_g(k) - h_{cr}) \sqrt{2g \left( h_1(k) - h_2(k) \right)}, \tag{1.6}$$

respectively, where $q$ represents the flow through the structure (m$^3$/s), $c_g$ is a calibration coefficient, $w_g$ is the width of the gate (m), $\mu_g$ is the contraction coefficient, $h_1$ is the upstream water level (mMSL), $h_2$ is the downstream water level (mMSL), $h_g$ is the gate height (mMSL), $h_{cr}$ is the crest level (mMSL), $g$ is the gravitational acceleration (m/s$^2$), and $k$ is the discrete time index.

As the natural feedback mechanism between the increase in the upstream water level and the increase in the flow is described by a square root, the flow is not very sensitive to upstream water level fluctuations. Consequently, without adjusting the gate height, an undershot gate is not well suited to control this water level. However, by changing the gate height, the flow can be set precisely, allowing a well-controlled water level.

#### 1.2.2.3 Simplified model for undershot gates

As a simplified model for the flow through a free-flowing undershot gate, the following model can be employed by linearizing (1.5) around a particular $h_1(k)$ and $h_{\mathrm{g}}(k)$:

$$
\begin{aligned}
q(k+1) = q(k) + & \frac{g c_{\mathrm{g}} w_{\mathrm{g}} \mu_{\mathrm{g}} \left(h_{\mathrm{g}}(k) - h_{\mathrm{cr}}\right)}{\sqrt{2g\left(h_1(k) - \left(h_{\mathrm{cr}} + \mu_{\mathrm{g}}\left(h_{\mathrm{g}}(k) - h_{\mathrm{cr}}\right)\right)\right)}} \Delta h_1(k) \\
& + \Bigg( c_{\mathrm{g}} w_{\mathrm{g}} \mu_{\mathrm{g}} \sqrt{2g\left(h_1(k) - \left(h_{\mathrm{cr}} + \mu_{\mathrm{g}}\left(h_{\mathrm{g}}(k) - h_{\mathrm{cr}}\right)\right)\right)} \\
& \quad - \frac{g c_{\mathrm{g}} w_{\mathrm{g}} \mu_{\mathrm{g}}^2 \left(h_{\mathrm{g}}(k) - h_{\mathrm{cr}}\right)}{\sqrt{2g\left(h_1(k) - \left(h_{\mathrm{cr}} + \mu_{\mathrm{g}}\left(h_{\mathrm{g}}(k) - h_{\mathrm{cr}}\right)\right)\right)}} \Bigg) \Delta h_{\mathrm{g}}(k), \qquad (1.7)
\end{aligned}
$$

where $\Delta h_1$ is the change in upstream water level (m) and $\Delta h_{\mathrm{g}}$ the change in gate height (m).

Considering the flow through a submerged undershot gate, the following simplified model can be obtained by linearizing (1.6) around a particular $h_1(k)$, $h_1(k)$, and $h_{\mathrm{g}}(k)$:

$$
\begin{aligned}
q(k+1) = q(k) + & \frac{g c_{\mathrm{g}} w_{\mathrm{g}} \mu_g \left(h_{\mathrm{g}}(k) - h_{\mathrm{cr}}\right)}{\sqrt{2g\left(h_1(k) - h_2(k)\right)}} \Delta h_1(k) - \frac{g c_{\mathrm{g}} w_{\mathrm{g}} \mu_{\mathrm{g}} \left(h_{\mathrm{g}}(k) - h_{\mathrm{cr}}\right)}{\sqrt{2g\left(h_1(k) - h_2(k)\right)}} \Delta h_2(k) \\
& + c_{\mathrm{g}} w_{\mathrm{g}} \mu_{\mathrm{g}} \sqrt{2g\left(h_1(k) - h_2(k)\right)} \Delta h_{\mathrm{g}}(k), \qquad (1.8)
\end{aligned}
$$

where $\Delta h_2$ is the change in downstream water level (m).

#### 1.2.2.4 Principle of MPC for water systems

MPC is a model-based control methodology meant for operational on-line control. At each decision step control actions are decided upon by solving an optimization problem. In this optimization problem an objective function that represents the control goals is minimized over a certain prediction horizon. Dynamics of the system to be controlled, operational constraints, and forecasts on, e.g., expected precipitation are hereby taken into account. The actions obtained are implemented until the next decision step, at which a new optimization is instantiated.

As the optimization has to run in real-time, it has to be fast and it has to result in a feasible solution. Therefore, typically the prediction models used are linearized and simplified models of reality, and the objective function that is optimized is usually formulated as a quadratic function. The constraints are formulated in such a way that, together with the quadratic objective function and the linear prediction model, the optimization problem is a convex optimization problem. Fast and reliable solvers are available for such quadratic programming problems.

For control of open-water systems, a typical objective function that is minimized over a finite prediction horizon is the following function, which represents the objectives for a water system with one water element controlled by a gate (via control variable $\Delta h_{\mathrm{g}}$) and a pump (via control variable $q_{\mathrm{p}}$):

$$J = \sum_{l=0}^{N-1} \left( w_e \left( e\left(k+1+l\right)\right)^2 + w_{\Delta h_{\mathrm{g}}} \left(\Delta h_{\mathrm{g}}\left(k+l\right)\right)^2 + w_{q_{\mathrm{p}}} \left(q_{\mathrm{p}}\left(k+l\right)\right)^2 \right), \tag{1.9}$$

where $J$ is the objective function or performance criterion, $\Delta h_{\mathrm{g}}$ is the change in the gate position (m), $q_{\mathrm{p}}$ is the pump flow ($\mathrm{m}^3$/s), $N$ is the length of the prediction horizon, $e$ is the error between the value of a water level variable and the target value of this variable, $w_e$ is the penalty on the error, $w_{\Delta h_{\mathrm{g}}}$ is the penalty on the change in the gate position, and $w_{q_{\mathrm{p}}}$ is the penalty on the pump flow. This objective function encodes the control objectives of minimizing deviations from target values weighted against minimizing control effort and energy. It is straightforward to extend this objective function to multiple water elements and actuators.

For open-water systems, the physical and operational constraints are usually time-varying limitations on variables. Physical constraints, such as minimum and maximum pump flows, and minimum and maximum gate positions, are implemented as hard constraints. Operational constraints, such as maxima of water levels for safety, minima on water levels for navigation, minima on water flows for agriculture, drinking water, and ecology, are implemented as soft constraints. Slack variables, representing the amount of violation of such constraints are added to the objective function with a penalty term.

The problem of minimizing the quadratic objective function, including the slack variables for the soft constraints, subject to a linearized prediction model and linear constraints is then a quadratic programming problem with linear constraints. Many efficient solvers for such problems are available, e.g., quadprog, CPLEX, and GAMS.

## 1.3 MPC for the North Sea Canal and Amsterdam-Rhine Canal

Since August 2008, an MPC controller has been in operation to support the operators of the discharge complex at IJmuiden, from where the North Sea Canal and the Amsterdam-Rhine Canal is operated. Here, the precipitation of a catchment area of approximately 2300 $\mathrm{km}^2$ drains into the sea. The main water ways that transport the water are the Amsterdam-Rhine Canal that continues into the North Sea Canal, as illustrated in Figure 1.6. At the end of the North Sea Canal the water can be discharged by seven gates during low tide and by six pumps when the sea water level is higher than the water level in the canal. Currently, the installed pump capacity of 260 $\mathrm{m}^3$/s is the largest in Europe. Below, we detail how MPC is used there to optimize the operation of the discharge complex every hour of the day.

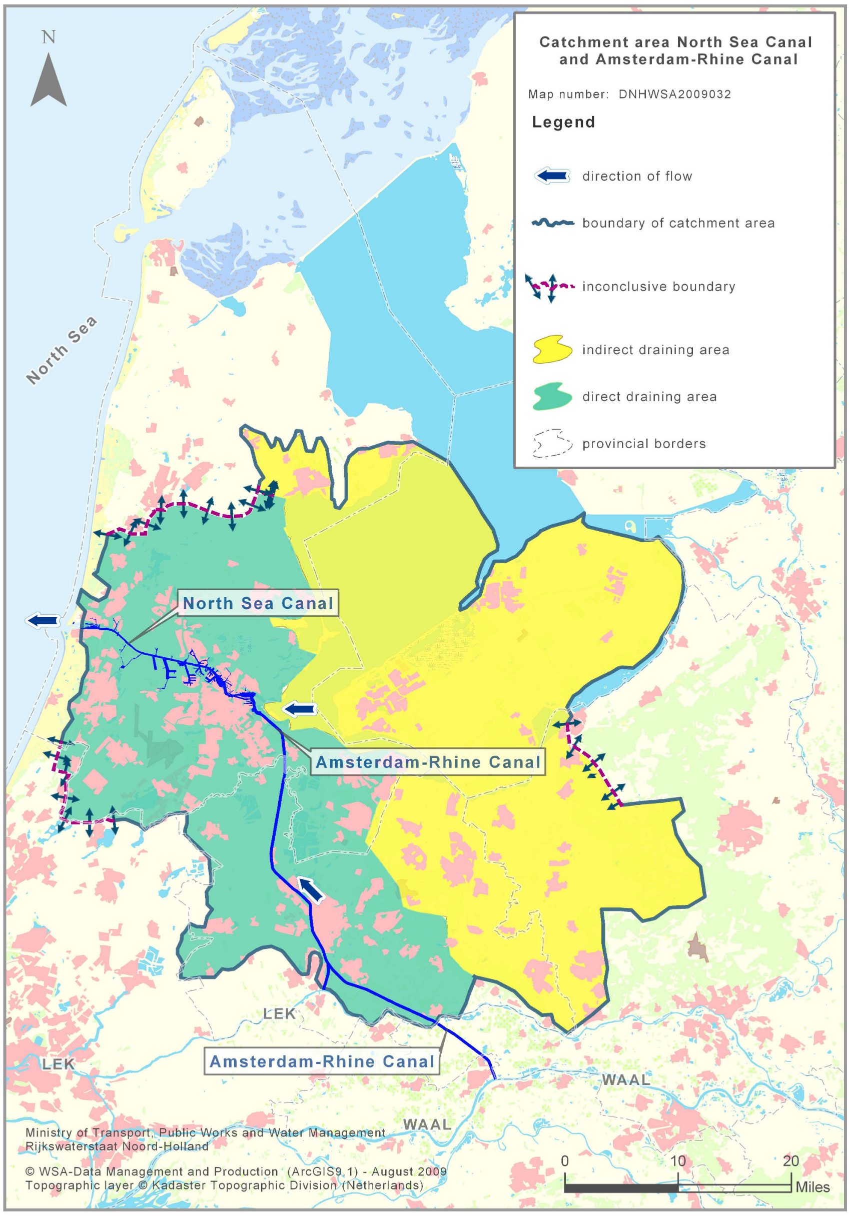



**Fig. 1.6:** Map of the catchment area of the North Sea Canal and the Amsterdam-Rhine Canal (Illustration courtesy of Ministry of Transport, Public Works and Water Management, Rijkswaterstaat Noord-Holland).

## 1.3.1 Setup of the MPC scheme

### 1.3.1.1 Objectives

The main objectives in the area of IJmuiden are navigation, minimum energy consumption, and to a smaller extent safety. The conflict in the operational management of the canal levels in this system is that navigation and safety require the water levels to remain close to the target level of -0.40 m with respect to mean sea level, whereas for achieving minimal energy consumption the water levels should fluctuate: During high sea water levels water should be stored in the canal, such that it can be discharged for free during low sea water levels. The objective function $J$ is therefore defined as:

$$J = \sum_{l=0}^{N-1} \left( w_{\mathrm{e}} \left( e\left(k+1+l\right)\right)^2 + w_{\Delta q_{\mathrm{g}}} \left( \Delta q_{\mathrm{g}} \left(k+l\right)\right)^2 + \sum_{i=1}^{6} w_{q_{\mathrm{p},i}} \left( q_{\mathrm{p},i} \left(k+l\right)\right)^2 \right), \tag{1.10}$$

where $e(k) = h_{\mathrm{ref}}(k) - h(k)$ is the difference (m) between the target water level $h_{\mathrm{ref}}$ (mMSL) and the water level of the canal $h$ (mMSL), $N$ is the number of steps in the prediction horizon, $\Delta q_i$ is the change in the discharge of all gates (m$^3$/s), $q_i$ is the discharge of pump $i$ (m$^3$/s), $w_{\mathrm{e}}$ is the penalty on the water level deviation, $w_{\Delta q_{\mathrm{g}}}$ is the penalty on the change in the discharge of all gates, $w_{q_i}$ is the penalty on the discharge of pump $i$ (for $i = 1, \ldots, 6$), and $k$ is the time step index. As the energy consumption of the pumps is directly linked to the pump flow, minimizing the pump flow results in minimization of the energy consumption.

### 1.3.1.2 Prediction model for the canals

The North Sea Canal and the Amsterdam-Rhine Canal are wide and the prediction model can therefore be modeled as the prediction model for a single reservoir:

$$h\left(k+1+l\right) = h\left(k+l\right) + \frac{q_{\mathrm{d}}\left(k+l\right)\Delta T}{a_{\mathrm{s}}} - \frac{q_{\mathrm{g}}\left(k+l\right)\Delta T}{a_{\mathrm{s}}} - \frac{q_{\mathrm{p}}\left(k+l\right)\Delta T}{a_{\mathrm{s}}}, \tag{1.11}$$

for $l = 0, \ldots, N-1$, where $h$ is the water level (m) of the North Sea Canal, $q_{\mathrm{d}}$ is the disturbance inflow (m$^3$/s), $\Delta T$ is the time step (s), $a_{\mathrm{s}}$ is the storage (surface) area (m$^2$), $q_{\mathrm{g}}$ is the sum of the flows through the seven gates (m$^3$/s), $q_{\mathrm{p}}$ is the sum of the flows through all six pumps (m$^3$/s). The time step $\Delta T$ is 600 s.

### 1.3.1.3 Constraints

Soft constraints are applied over the prediction horizon to impose the following limitation for navigation and safety:

$$-0.55 \leq h(k+1+l) \leq -0.30, \tag{1.12}$$

for $l = 0, \ldots, N-1$. Time-varying hard constraints over the prediction horizon are imposed on the flow through all gates and on the flow through the individual pumps:

$$0 \leq q_{\mathrm{g}}(k+l) \leq q_{\mathrm{g,max}}(k+l) \tag{1.13}$$

$$0 \leq q_{\mathrm{p},i}(k+l) \leq q_{\mathrm{p},i,\mathrm{max}}(k+l), \tag{1.14}$$

for $i = 1, \ldots, 6$, $l = 0, \ldots, N-1$. The maximum gate flow $q_{\mathrm{g,max}}(k+l)$ is derived from (1.8), using predictions of the sea water level and the canal water level, and assuming that all gates are completely open. The maximum pump capacity $q_{\mathrm{p},i,\mathrm{max}}(k+l)$ is set to 0 when the water level of the canal is higher than the sea water level; it is set to the maximum capacity as specified by the pump manufacturer otherwise.

The predicted disturbance flow is calculated by means of an auto-regressive model that uses as input the measured precipitation and evaporation over the past 15 days and the predicted precipitation for the next 12 hours. The parameters of this model are identified for each month of the year.

#### 1.3.1.4 The control scheme

The MPC optimization has been implemented in a decision support system. In this system a prediction horizon is considered with a length of $N = 144$ steps of 10 minutes, corresponding to 24 hours. Each hour the MPC controller performs its optimization, taking only a few minutes to complete. When the optimization has finished, a human operator checks the advice for the upcoming hour and accepts the advice by clicking on an acceptance button. Then, the advice for the next hour is automatically executed by implementing the actions determined for the first 6 steps of the prediction horizon.

### *1.3.2 Results*

#### 1.3.2.1 Without MPC control

Figure 1.7 shows the evolution of water levels, flows, and energy consumption two days before the MPC controller was activated. It can be observed that the operators maintain the water levels of the canal close to the target level (i.e., -0.4 m) by keeping a proper number of pumps running over a long period (from 14 to 23 hours and from 34 to 46 hours). The flow of these pumps balances the disturbance inflow. It can also be observed that the actions of the operators result in low power usage (around 1000 kW), when the sea water level is low, but a power usage that is at least 2.5 times higher (at least 2500 kW), when the sea water level is at its peak.

The worst situation appears when the canal water level is decreased below the target level by discharging water for free through the gates when the sea level is below the target level. After the low tide, the operator stops discharging water through the gates. However, when the water level becomes higher than the target level, the operator is triggered again to start discharging (at 6 hours on the first day), now using the pumps. By starting multiple pumps, the canal water level is kept close to the target level. However, the timing of the pumping action is costly as the starting the pumps coincides with the peak sea water level. In the considered scenario, the total amount of energy consumption is 79.5 kWh and the peak power usage is 4956 kW.

#### 1.3.2.2 With MPC control

A period of two other days is selected after the MPC controller has been taken into operation. This period has approximately the same inflow volume and potential gravity (gates) discharge volume as the previously described days at which the MPC controller was not used. As can be observed in Figure 1.8, the result of the controller is that the canal water levels fluctuate slightly more, although still within the allowed limits. However, the pumps switch approximately twice as often in order to achieve a much more cost-effective usage. The energy consumption is 54.5 kWh and the peak power is 3528 kW. This results in a reduction of 34% in costs compared to the case without MPC. For other situations average cost reductions over a year in the order of 20% were computed. The average energy costs for operating the pumps over the past five years (2003 to 2007) has been almost 1 million euro per year. Hence, using MPC can result in a cost reduction of 200.000 euro per year.

## 1.4 Distributed MPC for control of the Dutch water system

As was shown in the previous section, when using MPC for control of an individual discharge station a significant performance improvement can be achieved. Ideally, such an MPC controller would be implemented for the complete Dutch water system. However, due to the complexity and the size of this large-scale water system, this is not feasible. Controlling such systems in a centralized way in which at a single location measurements are collected from the whole system and actions are determined for the whole system would impose a too large computational burden. To illustrate this the following estimation of the size of the Dutch water system can be given: In The Netherlands, 15000 pumps, and a multiple number of gates can be controlled. Water levels in 1000 km of rivers, 1000 different canals, and a multiple number of ditches have to be controlled. These control structures and water ways are controlled by the national water board and 26 different regional water boards.

Instead of defining an overall control problem, it should be accepted that there are multiple MPC controllers spread across the network, each controlling their own part of the network. Local control actions include activation of pumps, filling or

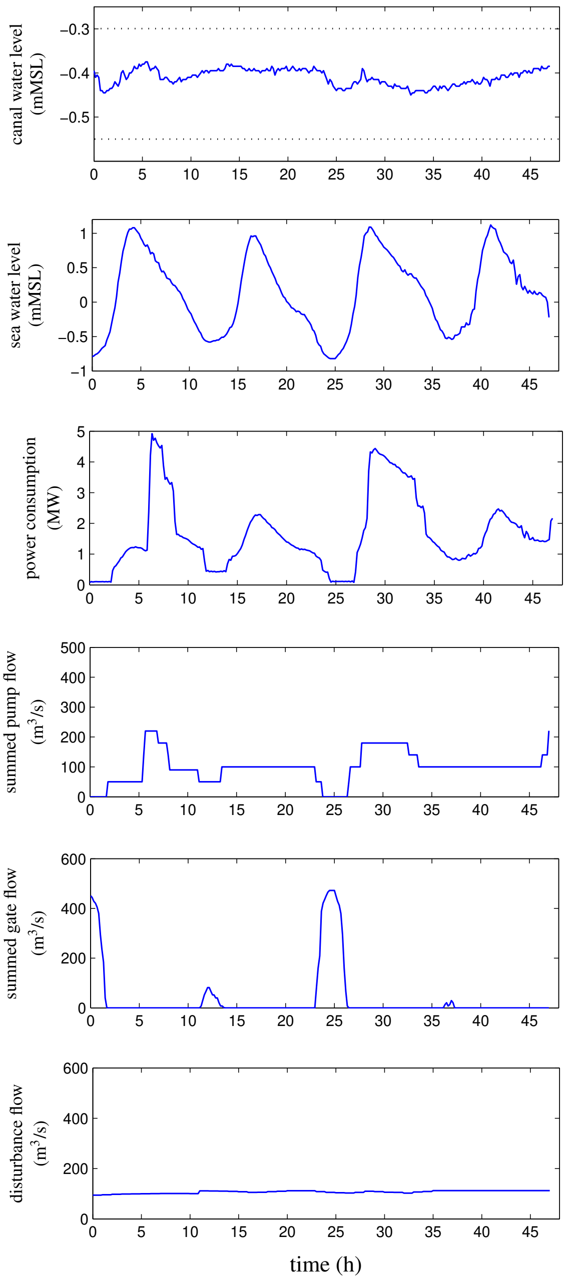



**Fig. 1.7:** Evolution of the water levels, the flows, and the energy consumption when not using MPC.

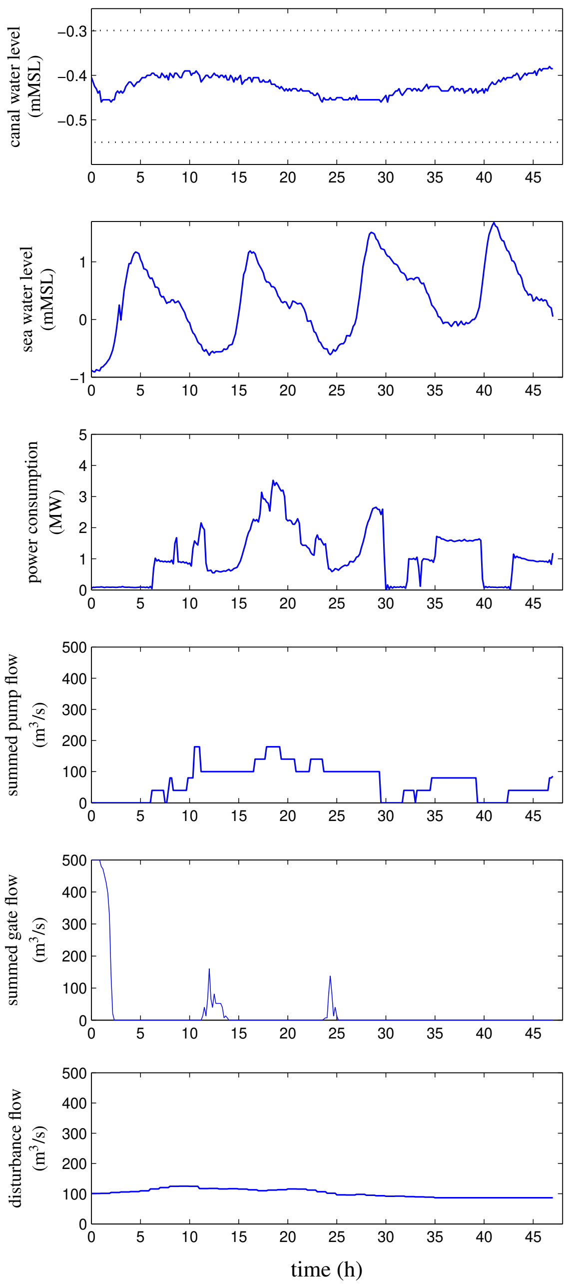



**Fig. 1.8:** Evolution of the water levels, the flows, and the energy consumption when using MPC.

emptying of water reservoirs, manipulating flows in certain directions of the country, closing off parts that are under threat of high sea water levels. Due to the continuing developments in information and communications technology, exchange of information between local controllers becomes practically and economically feasible, such that the local controllers have the possibility to take one another's actions into account. In order to achieve overall optimal performance, the local MPC controllers have to be designed in such a way that they account for the effects of local actions at a system-wide level using information exchange. The local controllers should thus be able to perform cooperation and negotiation with other controllers with the aim of achieving the best system-wide performance. Distributed MPC is aiming to enable this.

Similarly as in centralized MPC, the controllers in distributed MPC choose their actions at discrete control steps. The goal of each controller is to determine those actions that optimize the behavior of the overall system by minimizing costs as specified through a commonly agreed upon performance criterion that has been translated into desired water levels and flows. To make accurate predictions of the evolution of a subsystem over the prediction horizon for a given sequence of actions, each controller requires the current state of its subsystem and predictions of the values of variables that interconnect the model of its subsystem with the model of other subsystems. The predictions of the values of these so-called *interconnecting variables* are based on the information exchanged with the neighboring controllers. Usually, these interconnecting variables for water systems represent inflows and outflows between different parts of the water infrastructure.

Several authors have proposed distributed MPC strategies for control of large-scale water systems, e.g., in [7, 8, 17, 18]. These algorithms achieve cooperation among controllers in an iterative way, in which controllers perform several iterations consisting of local problem solving and communication within each control cycle. In each iteration, controllers then obtain information about the plans of neighboring controllers. This iterative process is designed to converge to local control actions that lead to overall optimal performance.

In order to employ any distributed MPC technique, first the subsystems, objectives, and constraints need to be determined. For The Netherlands this is currently being investigated. Below we first propose a division of the water system of The Netherlands into subsystems. Then, in Section 1.4.2 we illustrate the workings of a previously proposed distributed MPC scheme for control of subsystems in an irrigation canal. This illustration forms an example of how the water subsystems of The Netherlands could be coordinated in the future.

### *1.4.1 The subsystems of the Dutch water system*

Figure 1.9 illustrates our proposal for distributed MPC control of the complete Dutch water system. In the figure, 6 major water network regions are indicated. All regions together cover the major flows in The Netherlands. Each region by it-

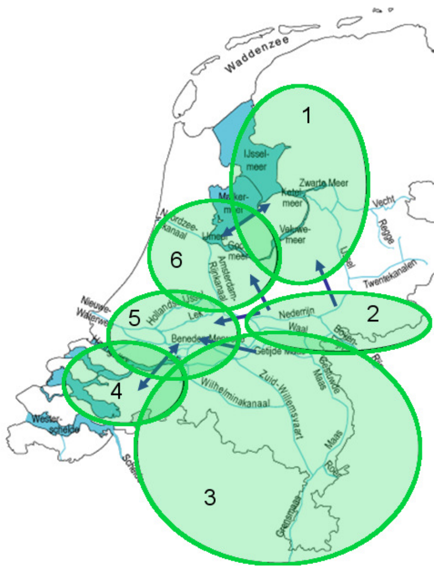



**Fig. 1.9:** The Dutch water system divided into 6 major regions.

self is defined in such a way that on the one hand the flow dependencies with the other regions are minimal, whereas on the other hand the flow dependencies within each region are strong. These regions are therefore already associated with separate divisions of the Dutch national water board. For each of these regions, local control objectives are formulated. To achieve these objectives an MPC controller is associated with each region. In order to be able to take into account the interaction between the different regions, the controllers can communicate in order to coordinate their actions. The 6 major regions and their control objectives are the following:

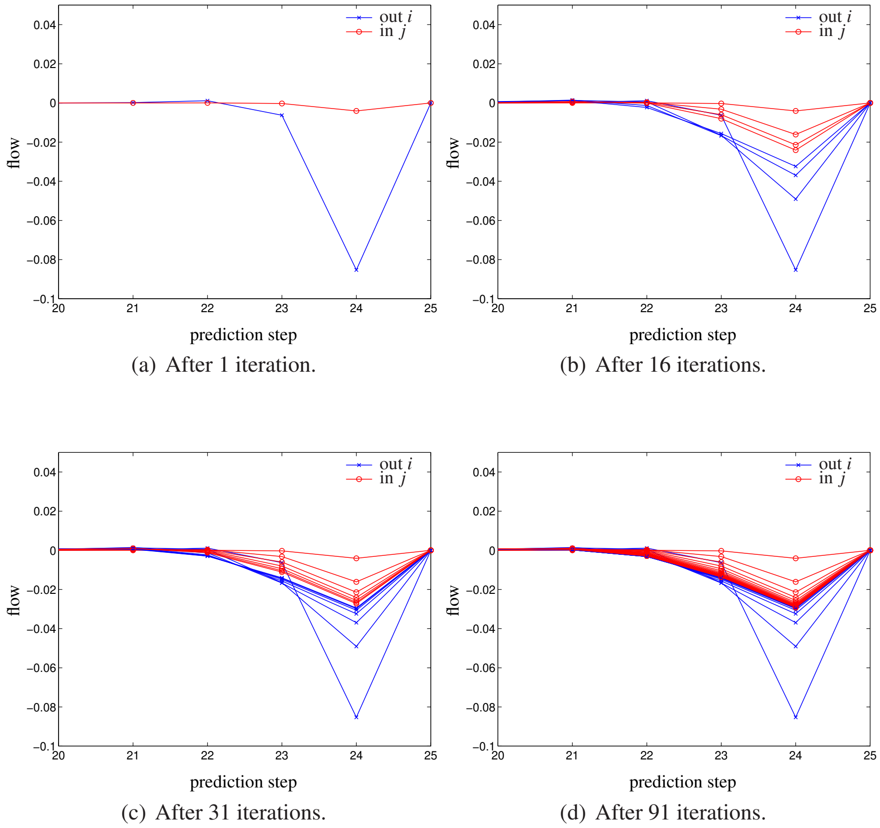
1. **Lake IJssel** is the large water reservoir in the North of The Netherlands. This reservoir should be used for the provision of drinking water and water for agriculture in the North and West. Water should also be flowing in such a way that algae bloom is reduced, encouraging a good ecology. Furthermore, lake IJssel should store water that can be used as cooling water for power plants.
2. The **Rhine River** is the largest river of The Netherlands. In addition to the provision of water for drinking, agriculture, cooling, and ecology in the West, navigation should also be possible. Hereby, safety has to be taken into account, as the Rhine River flows through densely populated areas.
3. The **Meuse River** has to provide water for agriculture and drinking in the South. Navigation and safety are two other important aspects that have to be taken into account when managing the water levels of the Meuse.

4. The **delta of Zeeland** is the second largest water reservoir of The Netherlands. Safety in the estuary has to be ensured, while water for agriculture, drinking, and algae bloom reduction should be sufficient. Moreover, also ships navigate through the Delta of Zeeland, and hence, water levels should be sufficiently high for this.
5. The **delta of Rijnmond** should provide safety in the estuary, while providing drinking water and water for agriculture. By exploiting the open connections to the sea, the ecological state can be improved.
6. The **North Sea Canal and Amsterdam-Rhine Canal** should have a sufficiently high water level to allow navigation. Pumps at the discharge station in IJmuiden should be employed taking into account their energy consumption. Water levels should not be too high, in order to ensure the safety in the area surrounding the canals.

### *1.4.2 Illustration of concept using control of irrigation canals*

Current research efforts are addressing controlling the water flows in The Netherlands based on the division into regions as presented above. Recently, as a first proof of concept for distributed control of water systems, we have implemented a distributed MPC scheme for control of an irrigation canal consisting of 8 individual canal reaches. Cooperating control systems for irrigation districts that have inter-dependent water demand schedules can yield a better spreading of the available water towards areas that are under increased water stress. On a larger scale, less water will be wasted. An illustration of the importance of coordination between sub-systems is the avoidance of disturbance amplification in canals consisting of canal reaches in series. When the water level in separate canal reaches are controlled simultaneously with proportional integral controllers that are tuned to give a high performance, problems could occur during operation [24]. Disturbances that occur at the downstream side are amplified at each control gate further upstream. Coordination between the canal reaches is required or a global tuning procedure for all PI-controllers needs to be used that minimizes the deviations from the set-points in all reaches [24].

In our application of distributed MPC for irrigation canals, each canal reach is controlled by an MPC controller. In order to obtain the best overall performance, the controllers have to reach an agreement concerning the amount of water flowing from one canal reach to the next over the full prediction horizon. At each decision step, the controllers therefore perform a number of iterations, in each of which they inform one another about desired inflows and outflows. This iterative procedure is illustrated in Figure 1.10. This figure shows at a particular decision step how the desires on the outflow from one controller become consistent with the desires on the inflow from a downstream controller over the iterations. Depending on a threshold specified in a stopping criterion the performance of the coordinated MPC scheme




(a) After 1 iteration. (b) After 16 iterations.

(c) After 31 iterations. (d) After 91 iterations.

**Fig. 1.10:** Two MPC controllers obtaining agreement over iterations on how much water should flow between two canal reaches [17].

can be balanced with the required computational time. Using a smaller threshold can result in a performance that is less than 1% worse compared to the performance obtained by a hypothetical centralized MPC controller. However, this is at the price of a significant computational effort. With a larger threshold, the performance becomes closer to the performance of a centralized controller, although computational time requirements improve. Further details on the actual implementation are found in [16, 17].

## 1.5 Conclusions and future research

In this chapter, the present knowledge on control of water systems is described. Control of rivers, canals, and lakes has been discussed. Most of the control problems of individual water systems can be solved. The next step to be taken is to set objectives for multiple water systems in a larger area, in this case the water systems in The Netherlands. This requires coordination between the sub-systems in that area. A promising solution for solving this large-scale optimization challenge is to create a distributed control framework for interacting water systems that can take into account the complex dynamics of the water system on the one hand and the often conflicting objectives on the other. Future research will have to further develop this framework in order to utilize water infrastructures to their fullest potential.

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