MODEL PREDICTIVE CONTROL APPLIED TO THE DUTCH DELTA, A PROBABILISTIC SAFETY ANALYSIS

Master of Science Thesis

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MSc. Water Resources Management

March 2014

TUDelft Delft University of Technology

Challenge the future

HKV LIJN IN WATER

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Master Thesis Water Management, TU Delft, March 2014

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SUMMARY

As many areas in The Netherlands are located below or slightly above mean sea level, or adjacent to large rivers, a lot of effort is put into ensuring the Dutch keep dry feet. The prevention of flooding is the most important and internationally well-known layer in the Dutch water safety policy. Nowadays this takes place by means of taking physical measures, i.e. making sure flood defences (e.g. dikes, barriers) are of adequate height and strength, or allowing enough space for the river to store water in case of extreme discharges.

Though very robust, taking physical measures for flood prevention is generally also very expensive. Another method to prevent flooding, currently hardly applied in The Netherlands, is anticipatory: the optimization of the control of the large (controllable) flood defence structures in the Dutch water system. This is explored in this thesis in the form of the application of Optimal Control, which utilizes Model Predictive Control. This is the only control method which can deal with large interconnected systems, anticipation on predictions, conflicting goals, and constraints. It is a methodology that originates in the process industries and is throughout the world applied to all sorts of systems and processes. More recently it has found its way into water management. A major benefit of this method is that the costs of realisation, operation and maintenance of such a system are estimated to be orders of magnitude lower than taking (extensive) physical measures.

In previous studies the influence of the application of Model Predictive Control on the water safety in The Netherlands has been determined for specific cases. However, a probabilistic analysis, which can provide a more complete picture of the profit of this technology in general, i.e. the effect on overall system behaviour, and is required by Dutch law for any measure in order to be considered a potential solution for safety against flooding, has not been possible thus far. In this research a computing platform which allows for parallel calculation is used which makes this analysis possible.

In this research, a model framework has been set up allowing for such a probabilistic water safety analysis of The Netherlands using Model Predictive Control. This framework consists of:

- a high resolution Sobek Rural model of the rivers, lakes and estuaries of The Netherlands (LSM) which is used to simulate the real world;
- Optimal Control in Matlab, which includes an internal model, an objective function and constraints;
- Hydra-Zoet probabilistic model to account for the probabilistic analysis.

During this research improvements and adaptations have been made to the models used. Using this framework the probabilistic approach has been followed in order to determine the effect of the application of Model Predictive Control. Additionally five structures, selected considering existing plans for the water system and the effect these structures can have on the water distribution, have been added to these models to further investigate possibilities within the system. The effect of Model Predictive Control in this research is determined largely by a minimization of the objective function which includes many locations, structures and goals, each made explicit by weights set in the controller. Before results could be obtained, an iterative process (trial and error) has been gone through in order to determine a best suiting set of weights to be used for this research.

The required model calculations for the probabilistic analysis used in this research consist of a limited set of 108 calculations determined by previous research of $HKV_{\underline{\text{LIJN IN WATER}}}$, these are considered to be representative for the overall system behaviour. This set consist of nine river discharge levels, combined with six storm levels combined with possible (dependant) failing of the Maeslant barrier and Hartel barrier.

What can be concluded from the results is that, when applying Optimal Control, clear effects can be expected in certain cases, while in other cases differences with current control are minimal. As a result, the effect on the overall system behaviour (normative water levels) is minimal as all scenarios are considered and effects are levelled out. In the upper rivers water system no differences can be observed as in this water system (almost) no structures exist to influence the water distribution. When the new structures are added to the model, more extensive differences can be observed. The effects of these structures are clear when considering individual cases, however the results in terms of differences in normative water levels are not in line with results obtained from individual cases. More detailed inspection of the results obtained from different parts in the model framework revealed some inconsistencies in the outcomes of the Sobek-calculations, which are probably the cause of the deviating results in terms of normative water levels. Due to the complexity of the model framework and enormous amount of data output such inconsistencies can be easily overlooked. Considering this the results displayed in this research should not be considered representative for the differences in overall system behaviour when the new structures are added to the system. Possibly some inconsistencies still exist for the calculations with current control and Optimal Control without new structures as well.

Recommendations have been made for improvements of the model framework and further research, most importantly the addition of the new structures to the objective function.

CONTENTS

1 Introduction

1.1 Background

At Delft University of Technology (TUD), a method has been developed to apply predictive control to optimize national water flow in The Netherlands (van Overloop et al., 2010). With this method, in situations of water shortage or excess, as well as in normal situations, the distribution of the available water over The Netherlands is optimized by applying Model Predictive Control (MPC or Receding Horizon Optimal Control, in this research referred to as Optimal Control, Chapter 4) on the large control structures in The Netherlands. As no tests can be executed on the actual Dutch water network, a SOBEK model of the rivers, lakes and estuaries of The Netherlands is used to test the impact of the application of this optimization.

As of today, several studies have been conducted which prove the use of the application of this optimization in specific cases. In these studies, every hour the application of the large control structures is optimized in a simplified mathematical hydro dynamic model of The Netherlands (the internal model), for a certain prediction horizon. Over this prediction horizon, forecasts for the river discharges and water levels at sea are taken into account, as well as the application of the large control structures and the initial state of the system, in order to calculate flows and water levels throughout the modelled area. The optimized state of the structures is then imposed to the Landelijk SOBEK Model (LSM, nationwide SOBEK model of The Netherlands). The LSM simulates reality as it would react at the execution of the optimized control. With the LSM a situation with high water levels can then be simulated. These studies provide a picture of the influence of the optimized control on water safety in The Netherlands in specific cases. However, a probabilistic analysis (see Chapter 5.4), which can provide a more complete picture of the profit of this technology in general (i.e. the effect on overall system behaviour), has not been possible thus far, since only one calculation (which takes a lot of time) could be performed at once. Though time consuming, such a probabilistic approach is required by Dutch law for any measure in order to be considered a potential solution for safety against flooding (Huizinga-Heringa, 2007; Ministerie van Verkeer en Waterstaat, 2007a,b).

A practical way how this can be realized is using a computing platform, which allows for parallel calculation, where a set of calculations can be performed at once. In this context, a limited set of 108 production calculations, which are considered to be representative for the overall system behaviour, have been set up by $\text{HKV}_{\underline{\text{LIJN IN WATER}}}$ to be used in this research (Geerse et al., 2012). These calculations can be performed in combination with the computing platform of $\text{HKV}_{\underline{\text{LIJN IN WATER}}}$. Before this setup can be used to provide valid results, several steps are required:

- the model framework has to be tested and made compatible with the computing platform;
- targets in the internal model have to be adjusted to accommodate specific goals;
- weights associated with utilization of structures, deviation from setpoints and exeedance of soft constraints in the internal model have to be tuned in order to move towards the best suiting weights for this research, requiring numerous model runs.

1.2 Objective

This thesis aims to find an indication of the benefit of applying Optimal Control on the large control structures in the Netherlands (in terms of lowering normative water levels), in case of high water levels, using a probabilistic analysis in order to determine the effect on overall system behaviour (Chapter 5.4). Furthermore, total calculation time of a complete set of 108 calculations on the computing platform should not exceed 12 hours in order to allow for proper testing and tuning of the model.

1.3 Research Questions

The following research questions are addressed to achieve the objective as described above:

1. *What is the difference in water safety for in The Netherlands when applying Optimal Control?*

   The outcome is a visual overview (map) of the difference in MHW[1] in The Netherlands.

2. *Which means have been utilized when applying Optimal Control and how does this differ from current operational water management?*

   The goal of this question is to quantify the changes which are required when switching from traditional operational water management to Optimal Control.

3. *What is the influence of several proposed new structures on the water safety in different parts of The Netherlands?*

   Several proposed new structures are added to the model to explore opportunities for improvement. These structures have been selected considering existing plans for the water system and the effect these structures can have on the water distribution.

[1] MHW: Maatgevende Hoogwaterstand, the water level corresponding to the norm frequency (water level per return period *T*) of the dike ring (Chapter 2) at the specific point

2 Water Safety in The Netherlands

The risk approach is the foundation for the water safety policy in The Netherlands. This means policy decisions are made based on limiting the probability of flooding as well as limiting the consequences of flooding. The water safety policy is called 'multi-layer safety' (Figure 1) and consists of three layers. It is aimed at limiting social disruption, and mainly focusses at limiting the number of casualties and economic damage.

The first layer is prevention of (major) flooding. This is the most important and internationally well-known layer in the Dutch water safety policy. The prevention of flooding nowadays takes place by means of taking physical measures, i.e. making sure flood defences (e.g. dikes, barriers) are of adequate height and strength, or allowing enough space for the river to store water in case of extreme discharges[2].

The second layer consists of sustainable spatial planning: taking safety into account for spatial developments.

The third layer consists of crisis management: However small, a chance of flooding will always exist. Adequate organizational preparation on flooding is therefore key in limiting casualties.

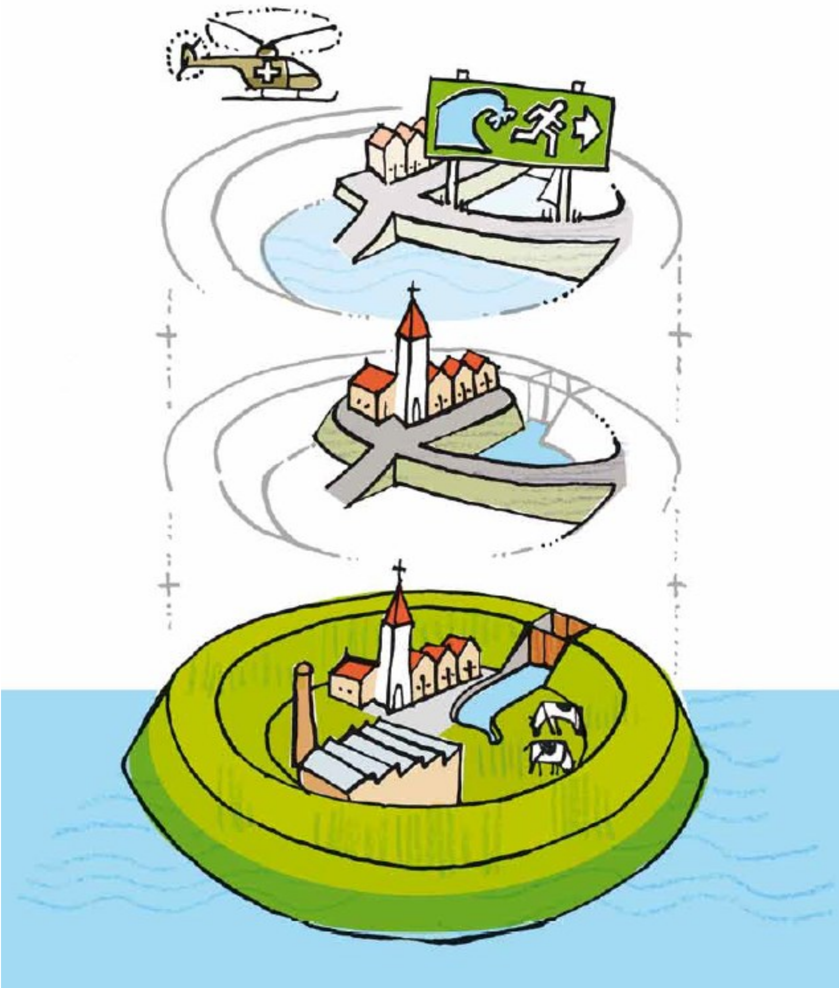

Figure 1: Conceptualization of multi-layer safety (Rijkswaterstaat, 2009)

[2]Specifically the 'Ruimte voor de rivier' and 'Maaswerken' projects in The Netherlands

In The Netherlands water safety differs per location (per dike ring, Figure 2). The Flood Defences Act indicates the safety standards for every dike ring area. Every dike ring is enclosed by a continuous line of flood defence structures (dikes, dunes, high grounds), protecting the area against flooding. This standard is based on the risk approach and therefore determined by the number of economic activities which take place within the ring and the number of inhabitants inside the ring. Other important factors which determine the safety standards of the dike rings are the size of the area liable to flooding, the height to which the water may rise and whether the flood water will be fresh or saline. The standard is expressed in a probability per year that a critical condition (e.g. water level, wave overtopping) will occur, e.g. 1:1,250 per year (or: 'once per 1250 years'). The requirements for flood defence structures in terms of height and strength are derived from that standard. A lower probability will result in a higher required strength.

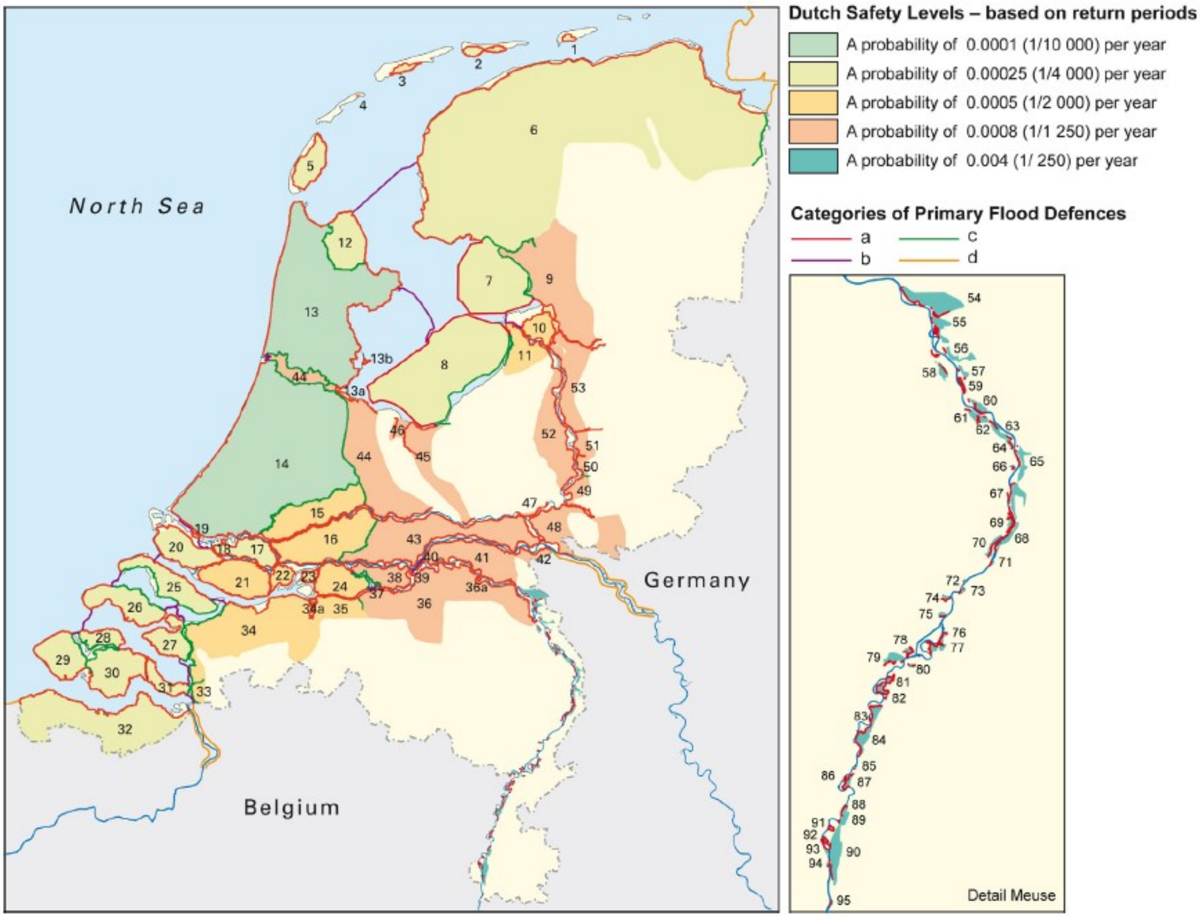

Figure 2: Dutch safety levels (Geerse, 2011)

Recalling the first layer of the multi-layer safety mentioned above, there is another method other than taking physical means to prevent flooding. Physical means, while very robust, are generally also very expensive. Another, currently hardly applied in The Netherlands, method to prevent flooding is anticipatory: the optimization of the control of the large (controllable) flood defence structures in the Dutch water system, which is explored in this thesis. A major benefit of this method is that the costs of realisation, operation and maintenance of such a system are estimated to be orders of magnitude lower than taking (extensive) physical measures. While realising such a system would not necessarily make taking (any) costly physical measures redundant, it could reduce the necessity thereof to a large extent in some areas (van Overloop, 2011).

3 Dutch Water System

This chapter describes the water system of The Netherlands, which is analysed in this thesis.

3.1 Existing water system and structures

In Figure 3 an overview of the main rivers and waters in The Netherlands is presented.

Figure 3: Overview of the main rivers and waters in The Netherlands (Rijkswaterstaat, 2011)

In this thesis the following rivers and water bodies are considered. The selection of these parts of the Dutch water system is based on the extent of influence they can have on the national water flow distribution. Other regional water systems that are managed by the water boards and the smaller water systems that are managed by the national water board are not included. Relevant in- and outflows have been considered (van Overloop, 2011).

- Maas
  Starts in Eijsden with an inflow from Belgium. The main stretch is used, without Julianakanaal. The Noord-Limburgse en Midden-Brabantse Kanalen are considered lateral out- and inflows to the Maas.

- Bovenrijn
  Starts in Lobith where the Rijn enters the country from Germany.

- Pannerdensch Kanaal
  The Rijn bifurcates at the Pannerdensche Kop through this canal to the North.

- IJssel
  This river bifurcates at the IJsselkop towards the North, towards the IJsselmeer. Lateral inflowing rivers, such as Twente Kanalen, are not considered. Instead these are considered lateral inflows on the IJssel.

- IJsselmeer
  This is the largest lake that supplies water for most of Northern water boards. Presently it has a fixed target level in the winter of -0.40 mNAP[3] and a fixed target level in the summer of -0.20 mNAP. Wind plays a significant role in the amounts that can be discharged through the structures.

- Nederrijn
  At the IJsselkop, this river stretch bifurcates in Western direction.

- Lek
  The Nederrijn extends into the Lek that enters the estuary of the Rijnmond area.

- Biesbosch
  This is the location where the rivers Maas and Waal and the tidal influences meet.

- Waal
  This is the largest river in the country. It bifurcates from the Pannerdensche Kop Westwards.

- Nieuwe Waterweg
  This is the connection between the Rijnmond area into the sea. The Hollandsche IJssel is not considered, as it is a dead end reach with limited storage capacity. The inflows and outflows from this stretch is considered as lateral flow.

- Hollandsch Diep
  This estuary part connects the Maas and Waal to the Haringvliet towards the sea. There is a controllable connection to Zeeland.

- Haringvliet
  Large water body with a controllable connection to the sea.

[3] NAP: Normaal Amsterdams Peil, the datum in The Netherlands

- Markermeer
  Lake, approximately half the size of the IJsselmeer. This lake has no direct river inflow and uses the same target levels as the IJsselmeer. Wind plays a significant role in the amounts that can be discharged through the structures.

- Noordzeekanaal (Amsterdam-Rijnkanaal)
  Canal that has controllable connections between Markermeer and Noordzee. The target level = -0.40 mNAP.

- Volkerak-Zoommeer
  Fresh water lake that has controllable connection between the river outflows in the Hollandsch Diep and the Oosterschelde. The target level is 0 mNAP.

- Oosterschelde
  Tidal salt water body that can be closed off from the Noordzee.

Figure 4: Sub-systems of the water system of The Netherlands (Geerse, 2011)

For modelling purposes, based on the characteristics of different parts of the water system, the water system can be categorized into four subsystems (Figure 4, for elaboration on sub-categorization see Chapter 5.4):

- Vecht and IJssel Delta
- Lake area (IJsselmeer and Markermeer)
- Tidal rivers ('Benedenrivieren')
- Upper rivers ('Bovenrivieren')

Additional subsystems can be categorized (coastal waters, regional waters), however these are not considered in this thesis.

The following structures are considered in this thesis:

- Gate Driel (in Nederrijn)
  The width of the gates is 108 meters in total. During low flow, these gates can be controlled to back up the water upstream in order to guarantee the cooling for power generation (Amerongen) and navigation over the IJssel. The head difference can reach as much as 4 to 5 meters. A parallel sluice allows for navigation when the weirs are closed. During high flow however, the function of flood prevention of the gates is limited.

- Haringvlietsluizen (between Haringvliet and Noordzee)
  The Haringvlietsluizen consist of seventeen discharge sluices (each 56.5m wide) and is located at the mouth of the former Haringvliet-estuary. Each discharge sluice has two gates, therefore it can turn water from seaside as well as from riverside. The gate can be partially lifted making different discharges through the sluices possible. It prevents rise of the water levels in the Rhine-Neuse delta due to high water levels at the North Sea by closing off the mouth of the Haringvliet estuary. It keeps the Haringvliet fresh by preventing water flowing into the Haringvliet from the North Sea and it keeps the water level at Moerdijk above 0m NAP.

- Lorentzsluizen (between IJsselmeer and Waddenzee)
  The Lorentzsluizen are located at Kornwerderzand in the North of the IJsselmeer in the Afsluitdijk. There are 10 gates which have a width of 12 meters each. Their function is to discharge water from the IJsselmeer to the Waddenzee. The total flow area when the gates are completely opened is 480 m^2, this means the maximum opening height is 4 meters.

- Stevinsluizen (between IJsselmeer and Waddenzee)
  The Stevinsluizen are located at Den Oever in the north-west of the IJsselmeer in the Afsluitdijk. The complex comprises 15 gates which each have a width of 12 meter. The total flow area when the gates are completely opened is 720 m^2, this means that the maximum opening height is 4 meters.

- Krabbersgatsluizen (between IJsselmeer and Markermeer)
  The Krabbersgatsluizen are located in the west of the IJsselmeer in the Houtribdijk. In order to discharge water from the IJsselmeer to the Markermeer, two gates are available each with a width of 18 meters. The crest level of the gates is -4.50m NAP.

- Houtribsluizen (between IJsselmeer and Markermeer)
  The Houtribsluizen are located in the Houtribdijk in the south of the IJsselmeer and discharges to the Markermeer. There are six gates, each with a width of 18 meters. The crest level of the gates is -4.50m NAP and the maximum capacity of the complex is 1000 m^3/s.

- Schellingwoude (Oranjesluizen, between Markermeer and Noordzeekanaal)
  In Schellingwoude, close to Amsterdam, discharge gates can be used to flush the Noordzeekanaal. This water is taken from the Markermeer. Water can be discharged through a large gate with a width of 9.8 meters and crest level of -4.5m NAP and three small gates, each with a width of 3 meters and crest level of -2.1m NAP.

- IJmuiden sluizen and pumping station (between Noordzeekanaal and Noordzee)
  To discharge water into the sea (and to keep sea water out) a pumping station and discharge sluice are located near IJmuiden. During low tide excess water can be discharged to sea through the discharge sluice (7 gates with a width of 5.25 meters) using gravity flow. The maximum allowed discharge flow is 500 m^3/s. In case the water supply is larger than the discharge capacity of the discharge sluice the pumps are used (6 pumps available). The maximum capacity of the six pumps is 260 m^3/s. Four pumps have a capacity of 40 m^3/s, the other two pumps have a maximum capacity of 50 m^3/s. As these pumps are very large, they need to be on or off for at least 30 minutes to avoid wear and tear.

- Volkeraksluizen (beween Hollandsch Diep and Volkerak)
  Water can be discharged from the Hollandsch Diep to the Volkerak by means of 4 discharge gates each with a width of 30 meters and crest level of -4.25m NAP. The maximum opening height of these gates is up to 1.50m NAP.

- Maeslantkering (between Nieuwe Waterweg and Noordzee)
  The Maeslantkering is a storm surge barrier capable of closing off the Nieuwe Waterweg. The structure consists of two gates that, when it has to close off the Nieuwe Waterweg, are floated out of their dry docks and sunk down to the bottom of the canal. The Maeslantkering therefore prevents the rising of water level in the Rijnmond area, due to high water levels at the Noordzee, by closing off the Nieuwe Waterweg.

- Hartelkering (between Nieuwe Waterweg and Noordzee)
  The Hartelkering is also a storm surge barrier. It has two gates, which can be lowered to close off the Hartelkanaal. Similar to the Maeslantkering, the Hartelkering prevents an increase in the waters levels of the Rijnmond area caused by high water levels at the Noordzee by closing off the Hartelkanaal.

- Krammersluizen (between Volkerak and Oosterschelde)
  Water can be discharged from Volkerak to the Oosterschelde through the locks. When this happens navigation is blocked, but ships can take a detour through the Schelde-Rijn Kanaal.

- Oosterscheldekering (between Oosterschelde and Noordzee)
  The Oosterscheldekering consists of 62 gates each with a width of 42 meters and crest level of -5m NAP. The maximum opening height of the gates is 5.8m NAP. In reality the gates cannot be completely closed and a leakage inflow occurs. In this study, an opening of 0.25m for all gates is used, when these are considered to be closed.

Additionally several other structures exists within the selected parts of the Dutch water system. These structures are included in the Sobek model of the system, but are not included in the Optimal Control and are therefore not controlled. It is possible to add these structures to the Optimal Control in the future. These structures include, but are not limited to:

- Gate Amerongen (gate/sluice structure comparable to Driel, located in the Lek)
- Gate Hagestijn (gate/sluice structure comparable to Driel, located in the Lek)
- Ramspolkering (inflatable rubber dam, located between Ketelmeer and Zwarte Meer)
- 7 Weirs in the Maas
- 6 Weirs in the Vecht

3.2 Proposed new structures

To further explore possibilities within the existing water system, a total of six new structures are proposed. These structures are inspired by research of (Stijnen et al, 2010), (van Overloop, 2011) and (de Jong, 2010), and selected considering existing plans for the water system and the effect these structures can have on the water distribution.

- Spuischuif (between Haringvliet and Nieuwe Waterweg)
  The Spuischijf is a gate with a width of 200m located in the Spui.

- Drechtschuif (between Hollandsch Diep and Nieuwe Waterweg)
  The Drechtschuif is a gate with a width of 200m located in the Dordtse Kil.

- Merwedeschuif (between Hollandsch Diep and Nieuwe Waterweg)
  The Merwedeschuif is a gate with a width of 200m located in the Beneden Merwede. The estimated costs for this gate and the Spuischuif and Merwedeschuif are 500 M€ each.

- Pannerdensche Schuif (between Bovenrijn and Pannerdensch Kanaal)
  The Pannerdensche Schuif is a gate with a width of 150m located in the Pannerdensch Kanaal. This gate can be used to direct water towards the Waal instead of over the Nederrijn and Lek. This way the area downstream of the gate (the Lek and IJssel) is protected against extreme water levels. In combination with the Spuischuif, Drechtschuif and Merwedeschuif it additionally protects the Rijnmond area. The estimated costs for this gate are 800 M€.

- Pumping station Afsluitdijk (between IJsselmeer and Waddenzee)
  Pumping station with a total capacity of 1000m^3/s (de Jong, 2010). With this pumping station the level in the IJsselmeer can be controlled even if the water level at sea is too high for discharge through the Stevinsluizen and Lorentzsluizen, or to otherwise prevent extremely high water levels on the IJsselmeer. In this research the pumping station is set to keep the water level on the IJsselmeer below 0.0 m NAP. The estimated costs of this pumping station are 600 M€.

- Second Maeslantkering
  For the crucial Maeslantkering it could be an option to build a structure in series in the Nieuwe Waterweg or other structures in the Rijnmond area in order to create redundancy. Several designs for such a structure are possible, which may include measures to prevent salt intrusion (Botterhuis et al., 2012). Note that this structure will not be modelled explicitly in Sobek or Matlab, instead the availability of this structure will be simulated by manipulating the probability of failure of the 'original' Maeslantkering in the Hydra-zoet model. The costs for this structure are estimated as 800 M€.

Additionally, measures would have to be taken from the Waal to the Hollandsch Diep and Haringvliet to accommodate for the increased flow along this branch when the Pannerdendsche Schuif is (partially) closed. Dike height along this branch would require to be raised an estimated 1 meter, resulting in an roughly estimated cost of 1000 M€ (assuming these dikes along a section of 160 km to be mostly grass dikes, which cost about 6 M€ per meter dike height raise per km (van der Toorn, 2010)). On the other hand, dikes along the Lek, IJssel, part of the Pannerdensch Kanaal and in the Rijnmond area would require no or less raising. Considering the length of the sections which would require no or less raising, and that these sections are located within more urbanized regions (dikes in urbanized regions cost up to 36 M€ per meter dike height raise per km (van der Toorn, 2010)), these measures, along with the Spuischuif, Drechtschuif, Merwedeschuif and Pannerdensche Schuif could prove to be cost effective.

4 Model Predictive Control

The control strategy used in this thesis is Model Predictive Control (MPC, or Receding Horizon Optimal Control, Figure 5; Maciejowski, 2002). MPC uses a mathematical model of the system under consideration to determine optimal control inputs for a controlled system, considering:

- *The present state of the system*
  Measurements of the present state or output of the system
- *Present and future setpoints*
  Setpoints or ‘goals’ for the system to meet, over a finite prediction horizon
- *Objective function*
  The objective function is used to quantify the interaction between (possibly conflicting) objectives which the MPC strives to achieve. In the MPC the objective function weights are assigned to goals that need to be satisfied as much as possible. In this way, goals are made explicit and an optimum can be searched by minimizing the objective function.
- *Constraints*
  The MPC takes physical and operational constraints into account. Constraints can either be *hard* (e.g. a physical limit on the flow through a structure) or *soft* (e.g. the desired maximum water level at a point).
- *Forecasts of external inputs*
  Forecasts of external inputs (boundary conditions, e.g. discharges and water levels) are considered along the prediction horizon
- *Receding horizon*
  The optimal control inputs for the system along the prediction horizon are calculated every time step, but only the first value is applied, neglecting the rest of the trajectory. At every new time step, new information about currents states and/or measurements are available and a new optimization problem is solved for the new prediction horizon.

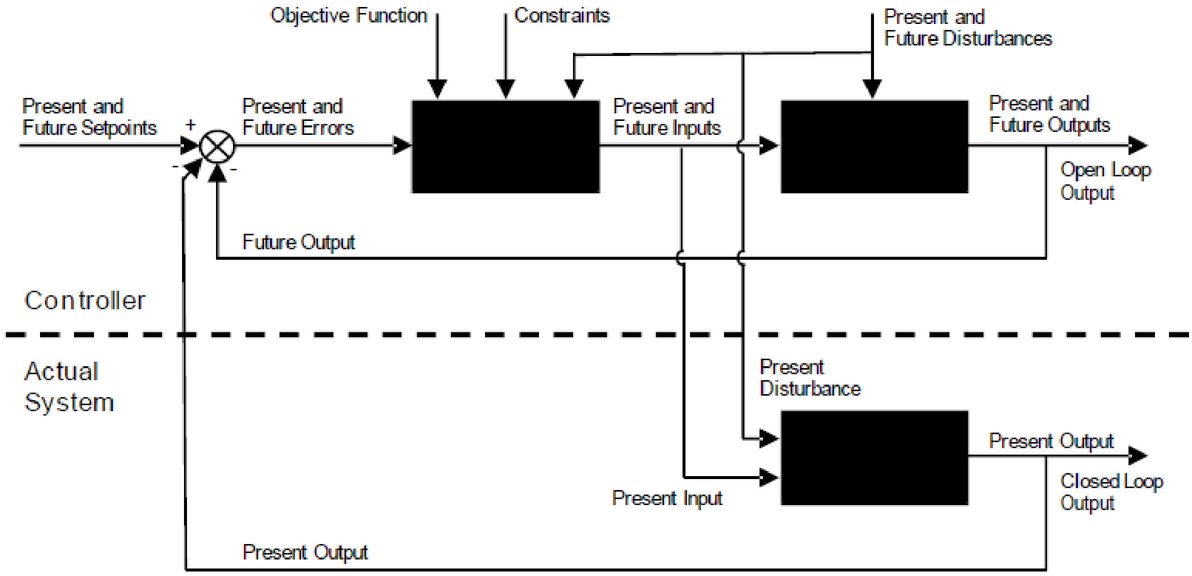

Figure 5: Model Predictive Control applied to an actual system (van Overloop, 2011)

The first development of MPC algorithms started with commercial developments for industrial processes in the 1970's, in order to meet the specific demands of petroleum refineries and power plants. Application of MPC proved to be able to provide large (financial) benefits in these sectors. In the past decades MPC has become more popular and widespread and can be found in a wide variety of application areas including chemicals, food processing, automotive and aerospace applications. (Qin & Badgwell, 2002)

Over the last decade MPC has gained popularity in the field of operational water management as well, with application focussing on different aspects (efficiency, cost reduction) displayed in numerous studies. (Blanco et al., 2008; van Overloop, 2006; van Overloop et al., 2010)

5 MODEL FRAMEWORK

In this thesis it is not possible to conduct the actual water safety calculation used in The Netherlands as included in the DeltaModel[4] for several reasons. Firstly, calculations are performed in the DeltaModel by several different hydraulic models (Sobek RE and Waqua) while the Optimal Control module is only available for the LSM. Secondly in the DeltaModel water systems in The Netherlands are subjected to an individual set of calculations in which the main causes of high water levels are simulated per water system. Because the application of Optimal Control will likely effect the interaction between water systems it is necessary to simulate the entire system as a whole.

As input for the Optimal Control part of the model framework forecasts will be used of river discharges and water levels at sea during the simulated period. Note that in reality these forecasts will always have some uncertainty included, however, with new technologies becoming available and improving performance of hydrological and hydro dynamical models, this uncertainty is estimated to reduce further in the coming decade(s) (van Overloop, 2011). Tools are available to accommodate for forecast uncertainty in modelling in the future (Raso, 2013), though at the upper bound of the performance of the optimization, using perfect predictions is required. This is possible to realise in a model environment where model input is determined beforehand, therefore perfect predictions are available to the internal model.

In this chapter the different parts of the model framework used are described, in which two key assumptions exist:

- Behaviour of water system is limited

Since in this research events are analysed which have not yet occurred in reality, extrapolation is required to analyse the effect of these events. Using the method described above implies that the behaviour of the water system is limited, and the boundary conditions are not; i.e. the behaviour of the given water system is analysed under extrapolated boundary conditions, instead of extrapolating the results of the given water system under historical boundary conditions. A detailed description of this extrapolation can be found in (Ministerie van Verkeer en Waterstaat, 2007a).

- The water is retained

When considering the physical behaviour of the water system it is assumed that all the water stays within the system. This implies that all flood defence structures are strong enough to withstand all imposed loads and are high enough to keep all water within the system, i.e. the flood defence structures do not fail due to any failure mechanism.
The logic behind this assumption is that correctly functioning flood defence structures result in the highest loads for all flood defence structures and thus results in the normative loads. A breach or overtopping would mean that water is disappearing from the system, thus overall less water in the system which results in a lower load on the flood defence structures. A failure upstream in the water system would therefore always to sub-normative loads downstream in the system.

[4] The DeltaModel is a set of models and tools to be used for the hydraulic and water management-wise substantiation of long-term policy decisions.

5.1 Landelijk Sobek Model

In this thesis a high resolution Sobek Rural model of the rivers, lakes and estuaries of The Netherlands (the Landelijk Sobek Model, or LSM) is used to simulate the real world (Figure 6).

The base version of the LSM model is the same as used in previous studies (van Overloop et al., 2010) (van Overloop, 2011). The model, which itself is a combination of models used in various studies of Deltares, incorporates the main rivers and water bodies in The Netherlands, has a calculation grid size of 500m and a calculation time step of 10 minutes. This model is linked with Matlab, through the RTC module of Sobek, in which the predictions and control actions are computed (for which a simplified 'internal' model is applied). The control actions are then used in Sobek for the next time step. In this study, for all cases the simulation period is 10 days and 6 hours. This simulation period is a balance between a period which is long enough to include all relevant processes, though short enough to allow for short calculation times.

In this model only structures are influenced which in reality can be controlled and only measurements can be done similar to what can be done in reality. (van Overloop 2011)

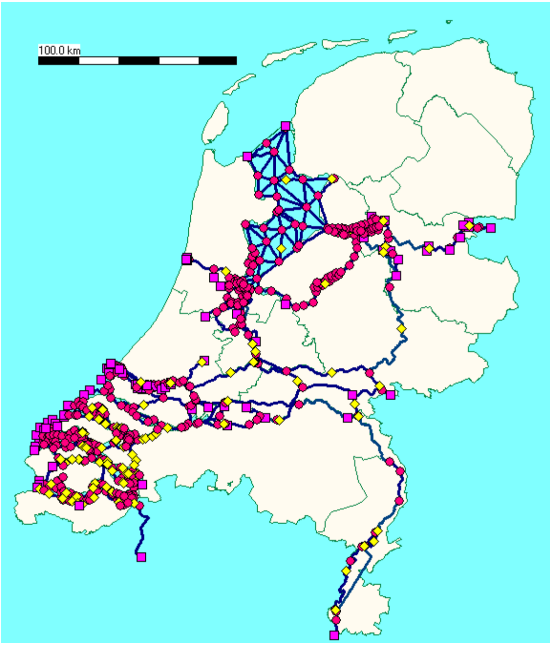

Figure 6: Nodes and reaches of the LSM (van Overloop, 2011)

At several locations in the model boundary conditions are defined. Boundary conditions for the rivers are defined for the Rhine and Meuse. The Scheldt river boundary condition is set at zero since it has no active connection to the water system under investigation, therefore its influence is considered negligible. Influence of the Vecht river is also considered negligible and therefore not varied.

Boundary condition at seaside (North Sea and Waddenzee) are defined for numerous locations along the shore, taking into account the phase-lag of the tide. A uniform one-directional wind field is applied to the entire model, however for different branches of the river hiding factors are applied, reducing the influence of the wind per branch. Evaporation is set at zero since its effect is considered negligible given the simulated period and the focus on cases with high water. For the exact definitions of the boundary conditions see Chapter 5.4.

Initial values in the model are set in such a way that the same initial values can be used for all calculations. IJsselmeer, Markermeer, Hollandsch Diep and Volkerak-Zoommeer are kept at setpoint while water levels in other parts of the model are low enough not to influence results of the low-discharge calculations. This is possible because for the high-discharge calculations the system adapts quickly enough (within the simulated period) to the higher water levels for the initial values to not have significant effect.

5.2 Differences between current control and Optimal Control

The current operational water management of the control structures in The Netherlands can, in general, be characterized as single objective, local and non-anticipatory. Most of the structures serve a single objective, for example safety, for the area in its neighbourhood and bases its actions on local measurements. Also, predictions for the coming days are usually not taken into account (exceptions to this are the structures of the Maeslantkering and Oosterscheldekering). This way of managing the system is very straightforward. On the other hand it can be called conservative as the benefit of new developments and technologies are not utilized. With new technologies coming available (such as systems allowing for real-time measurements throughout The Netherlands with robust communication to centralized locations, improved prediction systems for river inflows and sea tide, improved meteorological, hydrological and hydrodynamic models and faster computers which can run optimal controllers), the performance of the Dutch water system can be improved and problems occurring in the present water system can be counteracted.

Therefore in the models used in this study two settings will be used: the ‘current control’ setting and ‘optimal control’ setting, and some essential differences between the settings of the current control and optimal control exist. The ‘current control’ setting resembles the present operational rules for all structures as much as possible. In the ‘optimal control’ setting several other assets are utilized which currently are not:

- Predictions up to 10 days (240 hours) will be used in an Optimal Control module (which utilizes Model Predictive Control) to optimize water distribution over The Netherlands (more detailed information on the Optimal Control module in Chapter 5.3). Presently the accuracy of the predictions are good enough (within 20 cm) for a horizon of 24 hours. With the ever increasing computer power and knowledge about the hydrological processes, we can expect to extend this accuracy significantly (up to 240 hours) within a time span of several decades. In addition, tools to accommodate for remaining forecast uncertainty can be utilized.

- Anticipation on disturbances
  Using the available predictions it is possible to anticipate on disturbances like increased inflow (high river discharges) and decreased outflow or sluicing possibilities (high sea water levels) in the future. This is done. by lowering water levels below setpoints ahead of the disturbance in order to create storage, resulting in a lower exceedance of respective setpoints at the time of the disturbance.

- Directing water to Markermeer and Noordzeekanaal
  Water is diverted from the IJsselmeer through the Krabbersgatsluizen and the Houtribsluizen to the Markermeer, from the Markermeer through Schellingwoude to the Noordzeekanaal and finally from the Noordzeekanaal through the sluices and pumps at IJmuiden to the Noordzee.

- Storing water in the Zuidwestelijke Delta ('Southwest Delta')
  Water is diverted from the Hollands Diep through the Volkeraksluizen to the Volkerak-Zoommeer and from the Volkerak-Zoommeer through the Krammersluizen to the Oosterschelde. Whenever the Volkeraksluizen need to be used, the Oosterscheldekering is closed in advance, only letting water flow out of the Oosterschelde. At the same time the Krammersluizen start to discharge water from the Volkerak-Zoommeer to create additional storage.

- Creating storage in the Rijnmond area
  Instead of the present 24 hours, the Maeslantkering anticipates 48 hours ahead and is allowed to close multiple times after each other. This way storage is created in the Rijnmond area.

5.3 Optimal Control

The control methodology applied in this thesis is Model Predictive Control (Chapter 4). In this control system, the total optimization problem is solved in three iterations in order to maintain a linearized, time-variant model with time-variant linear constraints. This guarantees a convex problem. The procedure of solving the optimization problem is as follows (van Overloop, 2011):

1. The internal model is updated with measurements using a classical Kalman filter
2. A forward estimation is run with the internal model using a simulation over 240 hours using the present (local, mostly feedback) control
3. The constraints on the flows (minimum flow, maximum flow) are calculated from the water levels of the previous forward estimation
4. Linearization is performed based on previous water levels
5. Model Predictive Control is run, resulting in control flows for the structures
6. The forward estimation is run using the MPC control flows calculated in the previous step
7. Repeat steps 3 to 6 two times (This results in three iterations in total). By applying these iteration steps the linearization approaches the non-linear solution. In this way, all non-linear objects in a water system can be taken into account in the optimization.

The main parts of the Optimal Control are the internal model, the objective function and the constraints, described in Chapters 5.3.1 through 5.3.3.

5.3.1 The internal model

The base version of the internal model is the same model as used in previous studies (van Overloop et al., 2010; van Overloop, 2011). As the optimization has to test an enormous amount of control combinations, this is only feasible using a simplified (low-order) version of the modelled system, called the internal model. The internal model is derived from the Sobek model (LSM) and programmed in Matlab. It is an implicit Saint-Venant model using a large grid size of 20km and an one hour time step. The cross sections of the reaches are on average the same as in Sobek, but a constant bed slope is assumed. The large water bodies, such as the IJsselmeer are modelled as reservoirs with a level-area description. The model consists of 36 nodes and 40 reaches and is presented in Figure 7. Included in the base model are 11 structures which can be controlled (Table 1), with 5 proposed new structures which have been added (Table 2). These structures are modelled by flows derived from their Q-h relation. The same boundaries as used in Sobek are implemented. Lateral in- and outflows are snapped to the nearest node. The time horizon over which the internal model simulates is 240 hours, using a 6 hour time step. Over this horizon forecasts for the river discharges and water levels at sea will be used. In this research, perfect forecasts are used consisting of the same values used as input for the model. This will result in an indication of the upper bound of the performance of the Optimal Control. For model calculations where the Maeslantkering is set to fail (see Chapter 5.4), the failure will not be considered in the internal Model as in reality this will not be known beforehand.

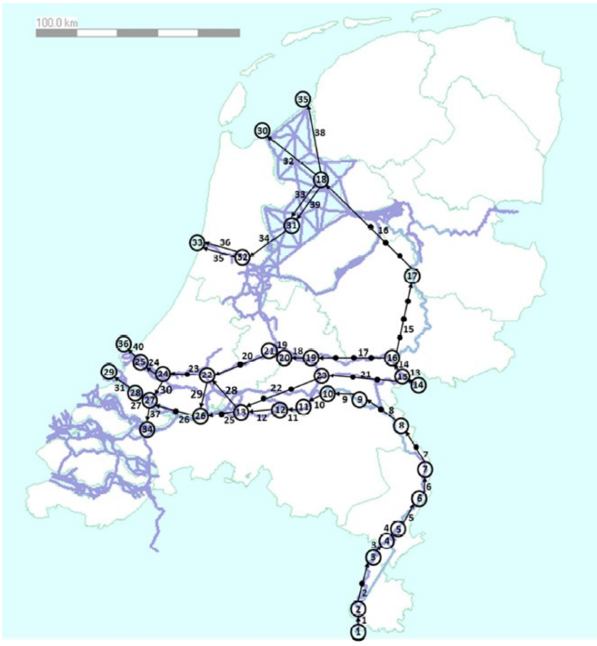

Figure 7: Nodes and reaches in the Matlab model (van Overloop, 2011)

Table 1: Structures in the Matlab model

Location	Type	Width / max capacity
Haringvliet	Gate	960.5 m
Lorentz	Gate	120 m
Stevin	Gate	180 m
Krabbersgat	Gate	36 m
Houtrib	Gate	108 m
Schellingwoude	Gate	9.8 m
IJmuiden	Pump	260 m^3/s
IJmuiden	Gate	36.8 m
Volkerak	Gate	120 m
Maeslantkering	Barrier	-
Driel	Gate	108 m

Table 2: Proposed new structures in the matlab model

Location	Type	Width / max capacity
Spuischuif	Barrier	-
Drechtschuif	Barrier	-
Merwedeschuif	Barrier	-
Pannerdensche schuif	Barrier	-
Afsluitdijk	Pump	1000 m^3/s

5.3.2 Objective function

The general objective function used in the simulations is:

$$J = \sum_{k=1}^{240} \left[W_{\text{h,quad}} \cdot \left(h(k) - h_{\text{ref}}(k) \right)^2 + W_{\text{h,lin}} \cdot h(k) + W_{\Delta\text{Q,quad}} \cdot \Delta Q(k)^2 + W_{\text{Q,lin}} \cdot Q(k) \right] + J_{\text{softconstraints}}$$

where J is the objective function of which its argument needs to be minimized, k is the discrete time index, $W_{h,quad}$ is the weight factor for the quadratic water level deviation from target level, h is the water level, h_{ref} is the target water level (reference or setpoint), $W_{h,lin}$ is the weight factor on the water level, $W_{\Delta Q,quad}$ is the weight factor on the quadratic change of flow of a structure, ΔQ is the change of flow of a structure, $W_{Q,lin}$ is the weight factor on the flow of a structure, Q is the flow of a structure. The latter is used for pump flows. $J_{softconstraints}$ is described in the next paragraph. (van Overloop, 2011)

Locations with setpoints are required to be under backwater, i.e. with a (nearly) horizontal water level, in order to allow control of this water level. Table 3 gives the setpoints at different location and table 4 gives the weights at the different locations and structures.

Table 3: Setpoints used in objective function

Location	h_{ref} (mNAP)
IJsselmeer	-0.4
Markermeer	-0.4
Noordzeekanaal[5]	-0.4

Note that these weights are acquired through an iterative process (trial and error) of changing the weights and evaluating the results, working towards the best suiting weights for this study. For this study an optimum in water safety is sought which results in relatively low weights on the usage (flow through the structure and/or change in flow) of the controlled structures. This entire process can be very time consuming considering the calculation time required for a complete set of calculations and additionally results for all calculations have to be evaluated (see section 5.4). The processor time used in this study totals in an estimated 16000 hours.

[5] The setpoint at Noordzeekanaal is dynamically lowered to -0.6 mNAP if the level of the IJsselmeer exceeds -0.15 mNAP, only in cases where Optimal Control is applied.

Table 4: Weights used in objective function

Location/Structure	$W_{h,quad}$	$W_{h,lin}$	$W_{\Delta Q,quad}$	$W_{Q,lin}$
Hollandsch Diep		1 (1/1)		
Haringvlietsluizen			4e-8 ($1/5000^2$)	
Volkeraksluizen			1e-8 ($1/10000^2$)	1e-2 (1/100)
IJsselmeer	400 ($1/0.05^2$)			
Markermeer	400 ($1/0.05^2$)			
Noordzeekanaal	400 ($1/0.05^2$)			
Lorentzsluizen			1.56e-8 ($1/8000^2$)	
Stevinsluizen			1e-8 ($1/10000^2$)	
Krabbersgatsluizen			1e-8 ($1/10000^2$)	
Houtribsluizen			4.49e9 ($1/15000^2$)	
Schellingwoudesluizen			1e-6 ($1/1000^2$)	
IJmuiden pumps				7.69e-4 (1/1300)
IJmuiden sluizen			4e-8 ($1/5000^2$)	
Driel			1e-4 ($1/100^2$)	

The Maeslantkering is kept out of the objective function, due to its major impact on the robustness of the solvers used. As closing of the barrier is a discrete choice (either open or closed, 0 or 1), no derivative can be determined from such a problem which makes it hard to optimize. One way how this could be implemented is by the use of Time Instant Optimization (Dekens, 2013) though this would not be practical for this research. Instead, the Maeslantkering closes when, in the prediction of 48 hours ahead the water level in the Nieuwe Waterweg at Rotterdam exceeds 3.87 mNAP or the water level in the Nieuwe Maas at Dordrecht exceeds 3.25 mNAP.

Due to time constraints regarding this thesis, the new structures have not been included in the objective function. Instead the Spuischuif, Drechtschuif, Merwedeschuif and Pannerdensche Schuif fully close when the same criteria for the closure of the Maeslantkering are exceeded. The Pannerdensche Schuif is closed and opened 30 hours in advance of the other barriers. The pumping station in the Afsluitdijk is set to full capacity when the water levels in the IJsselmeer or Markermeer exceed 0 mNAP within the prediction horizon of 240 hours. This means these structures, as modelled, do not benefit from the advantages of Optimal Control.

5.3.3 Constraints

The structures have hard constraints for the minimum and maximum flow. These flows are determined in an iterative way in 3 iterations. First from a forward estimation with the local controllers and the second and third time from a forward estimation using the MPC control actions (only applicable when the Optimal control is activated, for the reference calculation only the first forward estimation is used). (van Overloop, 2011)

On the water levels, soft constraints can be applied at certain locations. These become active once a certain threshold is exceeded in the case of a positive soft constraint, or when the water level drops below a certain threshold in the case of a negative soft constraint. Similar to locations with setpoints, location where soft constraints are applied are required to be under backwater. In the current model one soft constraint is applied (Table 5) in order to keep water levels in the area high enough to guarantee navigation.

$$J_{\text{softconstraints}} = \sum_{k=1}^{240} \left[\alpha \cdot W_{\text{h,softconstraint}} \cdot \left(h(k) - h_{\text{softconstraint}}(k) \right)^2 \right]$$

$$\alpha = \begin{Bmatrix} 0, \text{ if } h(k) < h_{\text{softconstraint(postitive)}} \text{ or } h(k) \leq h_{\text{softconstraint(negative)}} \\ 1, \text{ if } h(k) \geq h_{\text{softconstraint(postitive)}} \text{ or } h(k) > h_{\text{softconstraint(negative)}} \end{Bmatrix}$$

Table 5: Soft constraints used in the objective function

Location/Structure	$h_{softconstraint}$ (mNAP)	Type
Hollandsch Diep	0.4	Negative

5.4 Hydra-Zoet

In order to determine the effect of Model Predictive Control on the water system in general, i.e. the effect on the overall system behaviour; considering all possible scenarios, the probabilistic model Hydra-Zoet is used.

To be able to use Hydra-Zoet, the load level on the system has to be known for a lot of combinations of boundary conditions, since these combinations should cover the whole range of circumstances occurring in reality; only circumstances with extremely low probabilities of occurrence, e.g. smaller than 10^{-6} per year, can be left out as being irrelevant (as these don't contribute to the failure probabilities which are relevant for Hydra-Zoet). Usually a few thousand combinations have to be considered for the more complex water systems. A physical model is used to generate water levels corresponding to the combinations of boundary conditions. In the probabilistic part of the model Hydra-Zoet, proper probabilities are assigned to the combinations of boundary conditions, eventually providing the exceedance frequencies of load levels. Actually, the probabilistic calculation is more complicated than this, since the model has to take care of different time scales of the random variables: discharges and lake levels vary at much longer time scales than storms and storm surges. In Hydra-Zoet, a whole range of boundary conditions has to be used as input for (a) physical model(s), where the results of the physical models are then weighed with the proper probabilities of the combinations, at the same time accounting for differences in time scales. Therefore, to be able to use Hydra-Zoet in this thesis, proper input for the Sobek model has been generated (see Chapter 5.5).

After each calculation in Sobek, the maximum water levels at a large numbers of locations are saved in a database to be used in Hydra-Zoet, resulting in 108 water levels per location. However, since this database would normally contain thousands of water levels per location, the database is not filled completely and thus cannot be used by Hydra-Zoet. The results of this limited set are subsequently 'blown up' by means of copying through an intra- and extrapolation routine, in order to create a complete database required to perform a Hydra-Zoet calculation. Hydra-Zoet then uses statistical data belonging to the input data to combine and weigh all the combinations in order to produce water level frequency lines (water levels for a full range of exceedance frequencies) per location, from which the normative water levels can be determined (Figure 8).

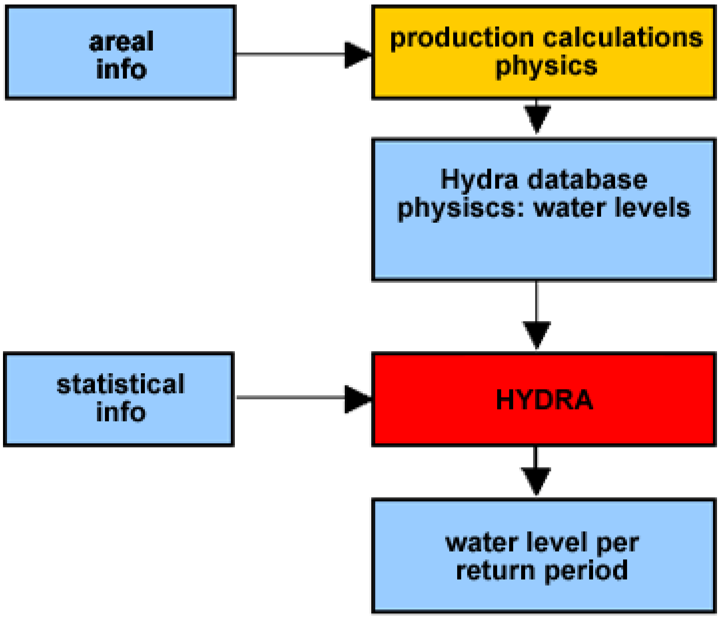

Figure 8: Schematisation of calculation (translation of Geerse, 2011)

Hydra-Zoet is part of the DeltaModel and intended to be used in combination with Hydra databases which are completely filled. These databases contain many thousands of water levels per location, resulting from the complete set of calculations performed by the models in the DeltaModel. Dependant on the water (sub)system, different parameters are considered as random variables and used for the probabilistic calculation, as can be seen in Table 6. These parameters are considered to be independent. Not all parameters are considered to limit the number of calculations, as certain parameters will only have an effect on certain subsystems. (Geerse, 2012) Note that for this study not all random variables will be used (e.g. 'state Ramspol barrier'). For the Maeslantkering, a probability of failure of 1/100 is used, for the combination of the Maeslantkering and a second Maeslantkering, a probability of failure of 1/10000 is used which is considered negligible. In calculations where the Measlantkering is set to fail, the barrier is set to remain opened completely.

Table 6: Random variables used per location in Hydra-Zoet (Geerse, 2012)

	water systems			
	Vecht and IJssel Delta	lake area	tidal rivers	upper rivers
random variables				
Rhine discharge			+	+
Meuse discharge			+	+
Vecht discharge	+			
IJssel discharge	+			+
Lake IJssel	+	+		
Lake Marken		+		
wind speed	+	+	+	+
wind direction	+	+	+	+
sea level			+	
state Maeslant barrier			+	
predictions Maasmond			+	
state Ramspol barrier	+			

5.5 MODEL CALCULATIONS

In order to perform a water safety calculation, in the DeltaModel a varying number, between several hundred and several thousand, calculations are performed per water system in the Netherlands (see Chapter 5.4). These calculations consist of different combinations of boundary conditions for wind speed, wind direction, storm surge level, river discharge level and state of storm surge barriers. Since a single calculation with Optimal Control enabledtakes several hours, even when using a parallel computing platform, on which numerous calculations can be performed at once, this would result in total calculation time which would be vastly too large for this thesis.

As alternative chosen is to use a limited set of 108 calculations determined by previous research of $HKV_{\text{LIJN IN WATER}}$ (Geerse, 2012). This set has been determined for the 'tidal-rivers' subsystem, and is known to perform well when analysing water levels in this subsystem. The set consist of 9 river discharge levels, combined with 6 storm levels and possible (dependant) failing of the Maeslant barrier and Hartel barrier. Wind is coming from one direction (WNW, 292,5°) and wind speed is coupled to the storm level. In contrast to the DeltaModel, where calculations are made with the peak discharge as constant discharge on the river branches, in this thesis discharge waves are used in order to allow better determination of the effects of Optimal Control. Maximum storm setup and river discharge per combination are presented in Table 7 and Table 8.

Table 7: Storm setup with coupled wind speed

Scenario	Storm setup [m +NAP]	Wind speed max. [m/s]
H1	0	3.01
H2	1.29	14.17
H3	2.47	20.42
H4	3.54	24.81
H5	4.57	31.86
H6	5.59	36.03

Table 8: Maximum river discharges

Scenario	Rijn discharge [m^3/s]	Maas discharge [m^3/s]
Q1	600	55
Q2	1000	490
Q3	6000	1156
Q4	8000	1626
Q5	10000	2095
Q6	13000	2800
Q7	16000	3504
Q8	18000	3974
Q9	20000	4444

Dependant on the duration of the discharge peak and the storm duration, both peaks will coincide in only a limited number of locations throughout the system. In this thesis the choice has been made to set up the timing of the peak storm level (Figure 9) and peak of the discharge (Figure 10) in such a way that these peaks coincide at Dordrecht. In this way the most unfavourable situation is created in the transitional area from upper rivers to tidal rivers. (Thonus, 2006) Storm setup and discharge waves are set up according to (Ministerie van Verkeer en Waterstaat, 2007a). In Appendix A graphs for all storm setup scenarios and river discharges are included.

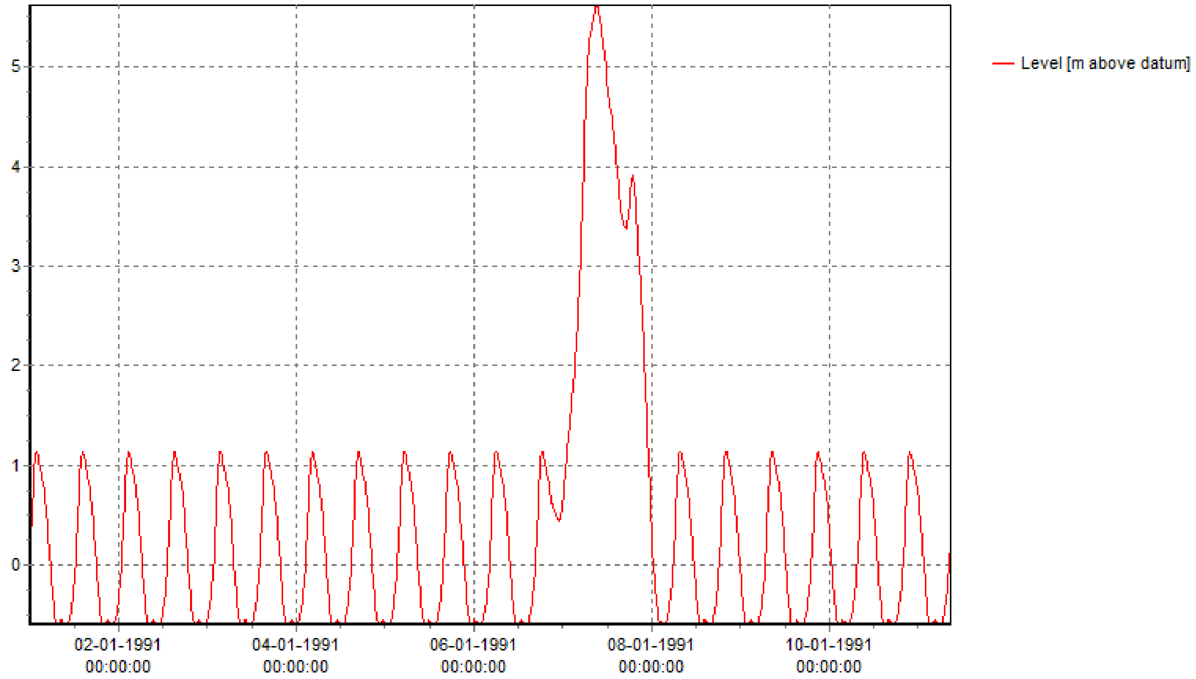

Figure 9: Storm setup at node N_NDB_1 for storm level H6

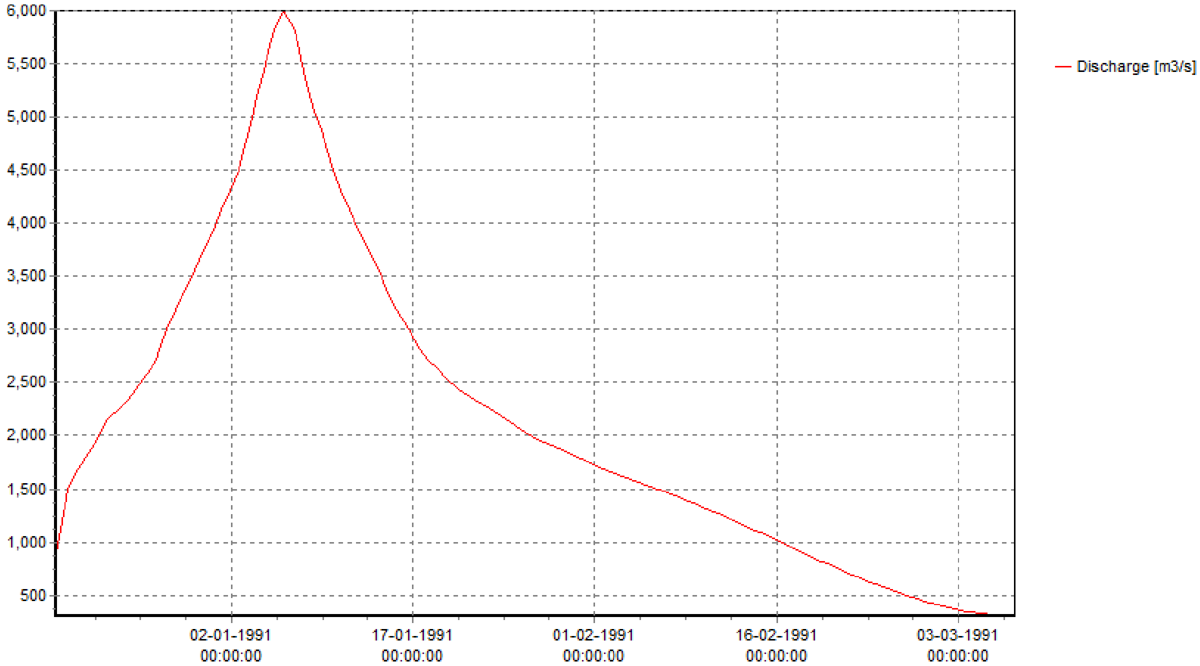

Figure 10: Rhine discharge wave for discharge level Q3 (simulation period is 01-01-1991 01:00 to 11-01-1991 07:00)

6 RESULTS

Below the results from the model calculations are presented. In Chapters 6.1 through 6.3 results in terms of graphs of water levels and discharges for a number of locations are presented per case for a selection of calculations. The combination of locations with water level results and discharge results have been selected to provide a complete picture of occurrences within the complete system, whilst limiting the number of selected locations in sake of clarity. Per location the results for a total of 6 out of the 108 calculations are plotted; a single discharge level is selected (Q7) and the combinations thereof with all storm levels (H1 through H6), additionally for all selected cases the Maeslantkering does not fail. Note that for the discharge graph of 'Zuidwestelijke Delta', a negative flow resembles flow into the delta, the discharge graph of '193' resembles flow through the pumping station in the Afsluitdijk. At Figure 25 that, though it appears a small amount of discharge exists, it has to be noted that no actual discharge to or from the Zuidwestelijke Delta takes place but the displayed result is caused by effects within one reach due to wind.

In chapters 6.4 and 6.5 two comparisons are made; the difference in MHW between the situation with current control and the situation with optimal control and the difference in MHW between the situation with optimal control and the situation with optimal control and the new structures. For each comparison a map (visual overview) is presented with the differences in MHW throughout the Netherlands.

6.1 Results Current Control

In Figures 11 through 26 (Chapter 6.1.1 and 6.1.2) the results are plotted for cases where current control is applied. What can be observed from these results is how the current water system would react under the given boundary conditions. Clearly different parts of the water system are affected by different types of boundary conditions. Locations at Rotterdam, Dordrecht and Hollandsch Diep are affected most by the discharge, and sea water level boundary conditions. Locations at the IJsselmeer are affected by the discharge and sea water level boundary conditions to some extent, but the peak water levels are mainly influenced by the amount of wind. Locations at the Markermeer are only affected by the wind as there is no in- or outflow from this lake when current control is applied.

6.1.1 Results Current Control: Water levels

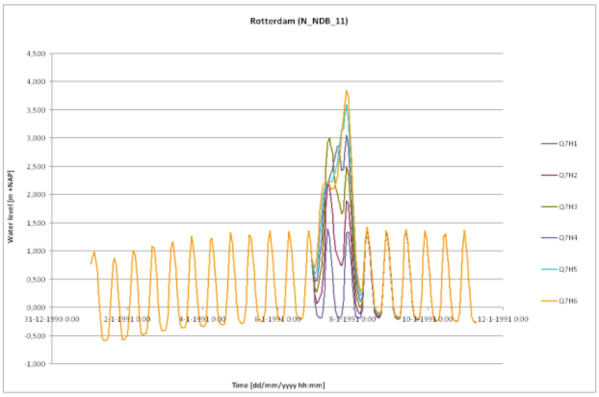

Figure 11: Results at Rotterdam (CC)

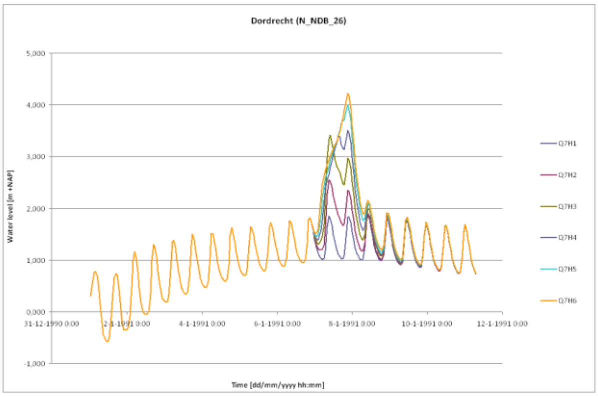

Figure 12: Results at Dordrecht (CC)

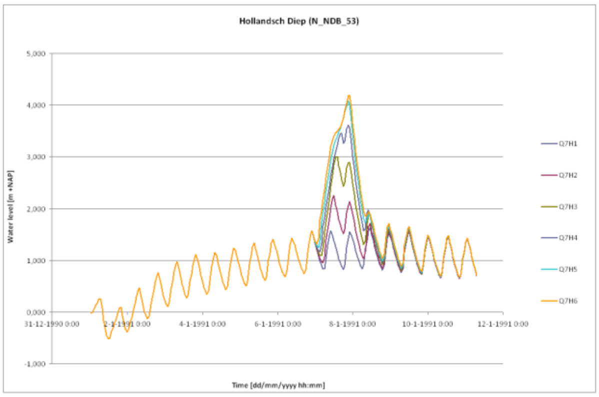

Figure 13: Results at Hollandsch Diep (CC)

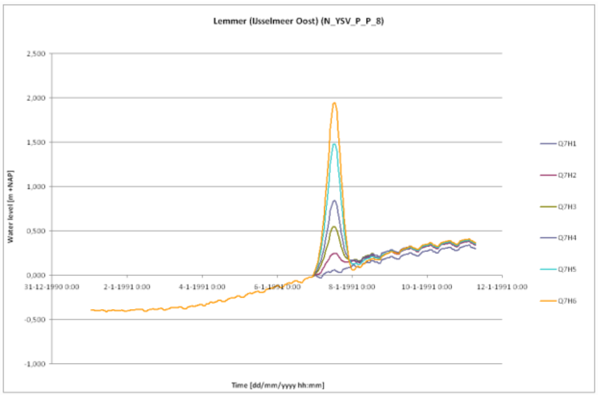

Figure 14: Results at Lemmer (CC)

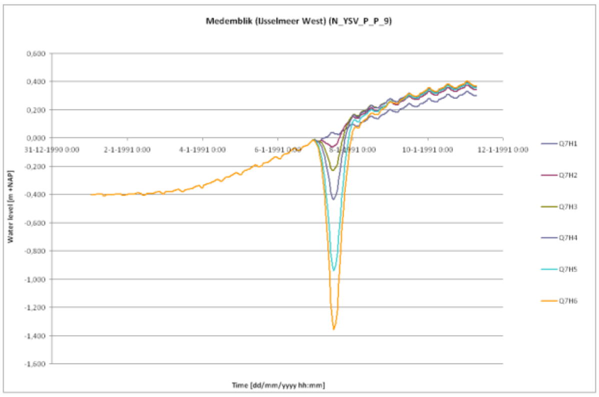

Figure 15: Results at Medemblik (CC)

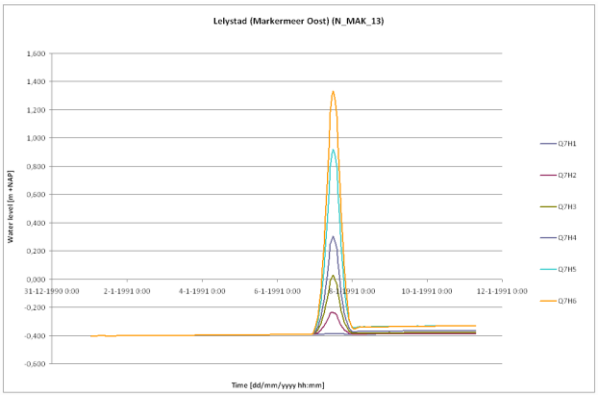

Figure 16: Results at Lelystad (CC)

Figure 17: Results at Rotterdam (CC)

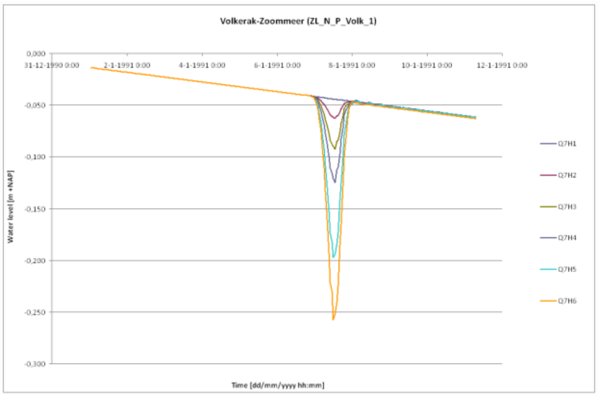

Figure 18: **Results at Dordrecht (CC)**

6.1.2 Results Current Control: Discharges

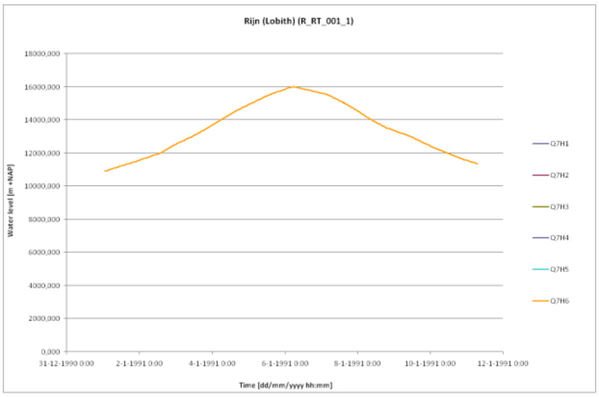

Figure 19: Results for the Rhine (Lobith) (CC)

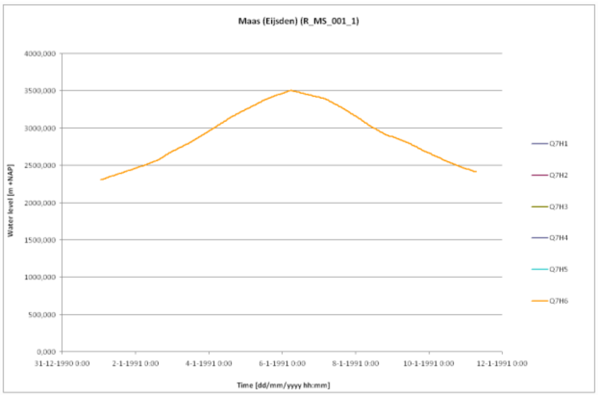

Figure 20: Results for the Maas (Eijsden) (CC)

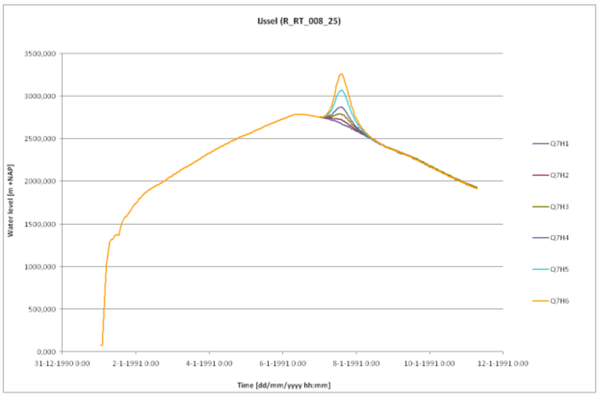

Figure21x: Results for IJssel (CC)

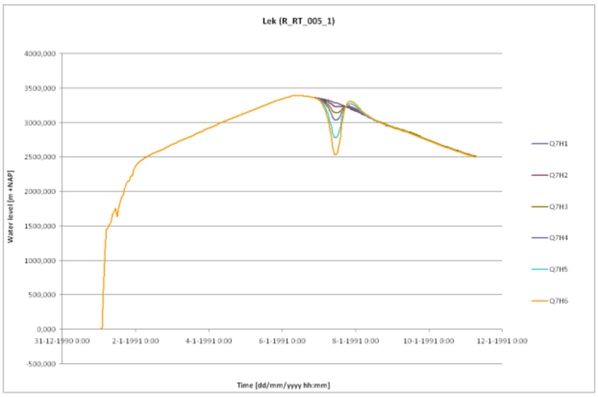

Figure 22: Results for the Lek (CC)

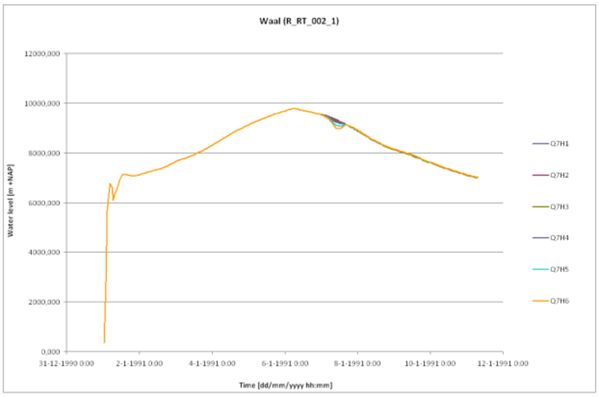

Figure 23: Results for the Waal (CC)

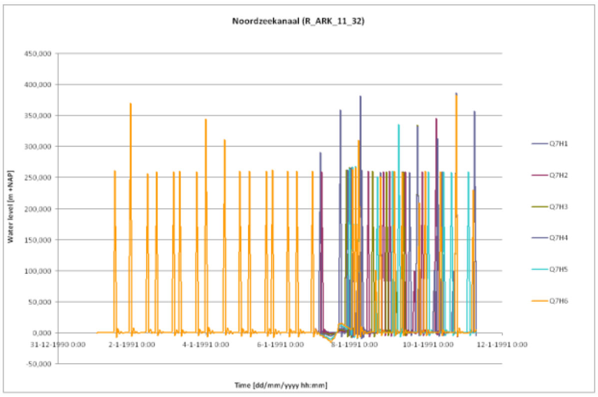

Figure 24: Results for the Noordzeekanaal (CC)

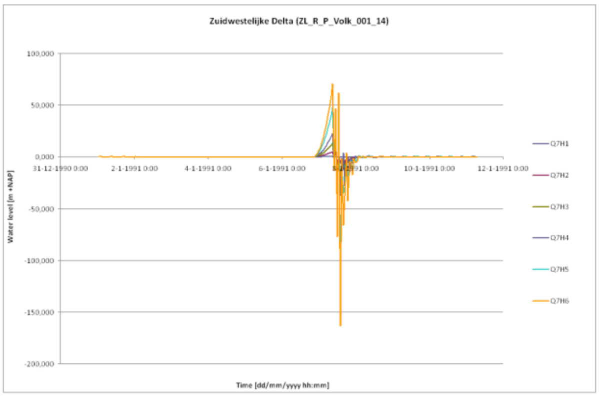

Figure 25: Results for the ZuidWestelijke Delta (CC)

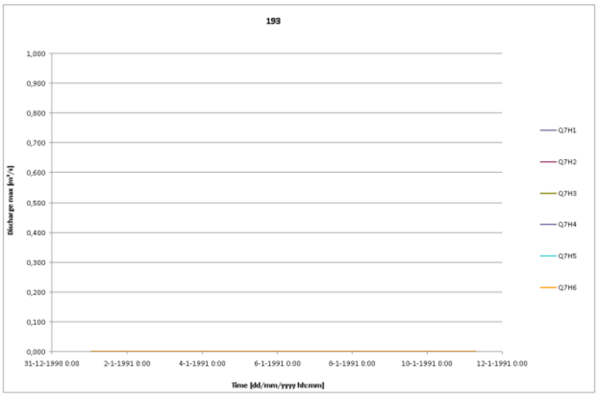

Figure 26: Results for the pumping station Afsluitdijk (CC)

6.2 Results Optimal Control

In Figures 27 through 42 (Chapter 6.2.1 and 6.2.2) the results are plotted for cases where Optimal Control is applied. What can be observed from these results is that there are some clear effects when applying Optimal Control. In certain cases, mostly ones with low river discharge, application of Optimal Control results is higher water levels as the controller tries to keep water levels at their respective setpoints or above soft constraints (in the case of Hollandsch Diep). In more extreme cases the application of Optimal Control can result in (much) lower peak water levels, especially in the ‘Benedenrivieren’ subsystem (Rijnmond area and Hollandsch Diep / Haringvliet). At the IJsselmeer application of Optimal Control yields lower water levels for nearly all scenarios as the Markermeer is now used for storage with a same target level and weight assigned as the IJsselmeer. At the Markermeer and Volkerak-Zoommeer application of Optimal Control always yields higher water levels, since these assets are not used in the current control. Flow through the Noordzeekanaal is greatly increased, though water levels do not increase by much as the flow into the Noordzeekanaal from the Markermeer is limited by the gates at Schellingwoude. It has to be noted that the obtained results are unique for the weights determined in Chapter 5.3.2, by changing these weights, setpoints and soft constraints different amount of water can be directed to different parts of The Netherlands. In this way also different amounts of water can be directed towards the Markermeer and Volkerak-Zoommeer.

6.2.1 Results Optimal Control: Water levels

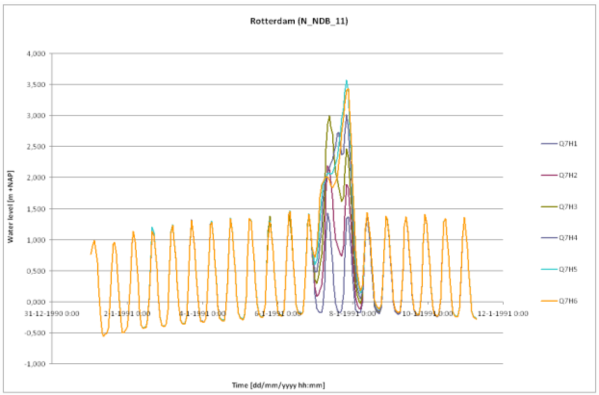

Figure 27: Results at Rotterdam (OC)

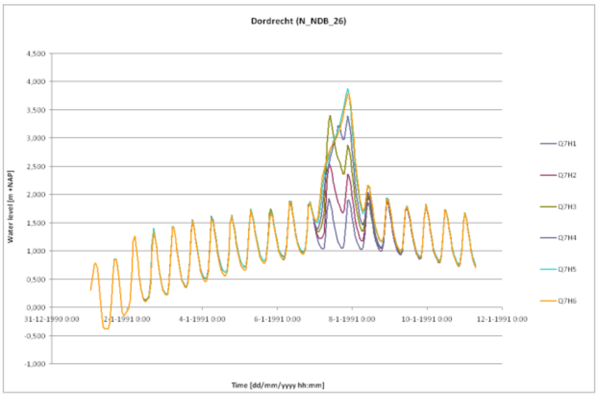

Figure 28: Results at Dordrecht (OC)

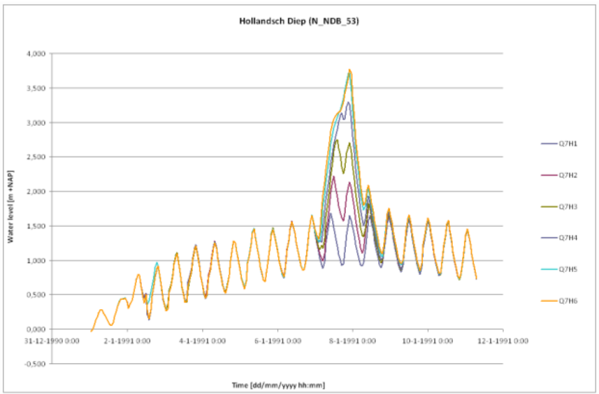

Figure 29: Results at Hollandsch Diep (OC)

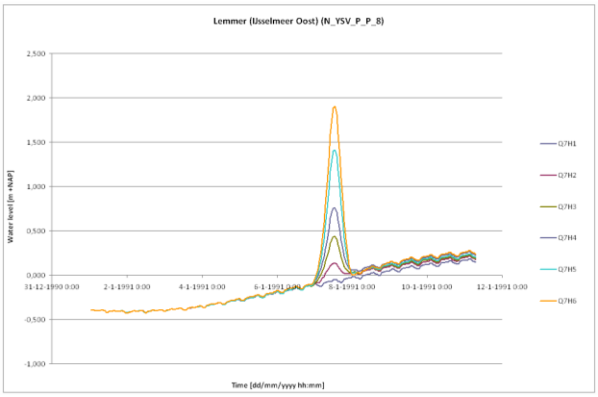

Figure 30: Results at Lemmer (OC)

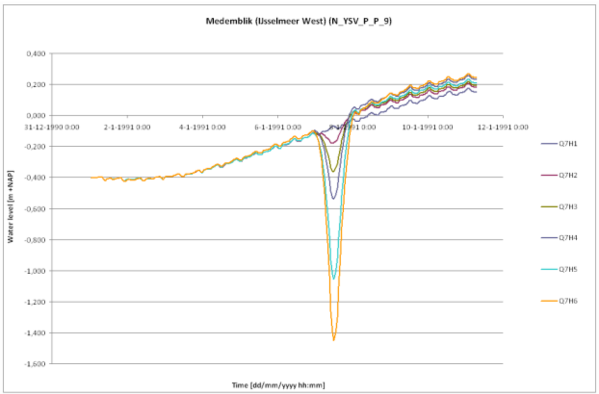

Figure 31: Results at Medemblik (OC)

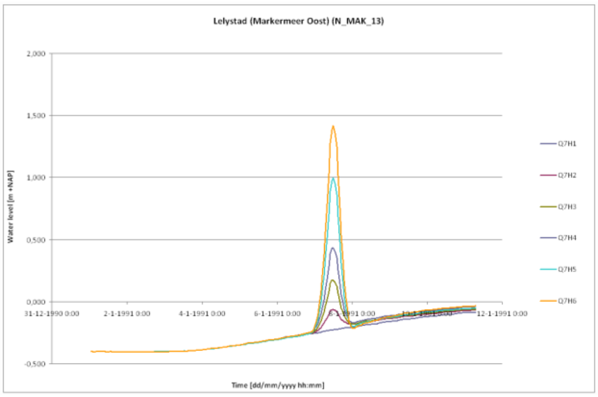

Figure 32: Results at Lelystad (OC)

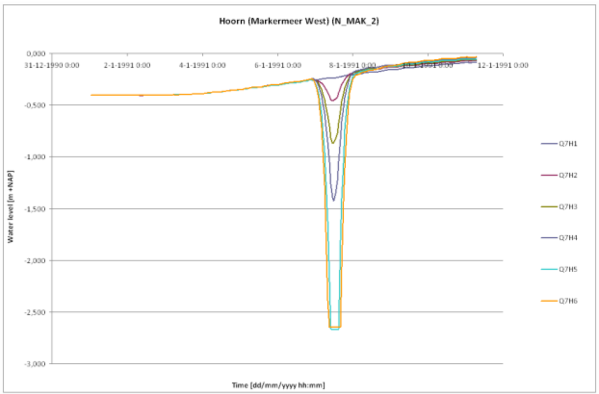

Figure 33: Results at Hoorn (OC)

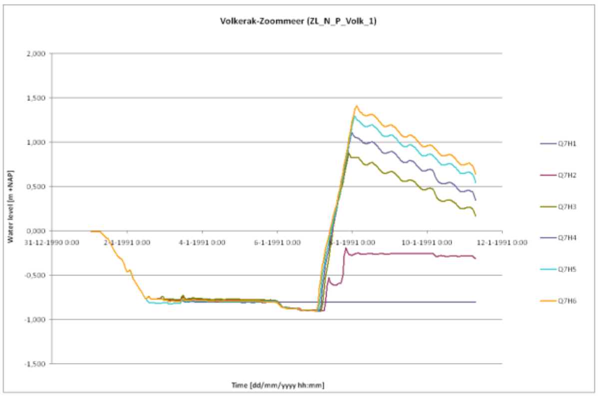

Figure 34 Results for the ZuidWestelijke Delta (OC)

6.2.2 Results Optimal Control: Discharges

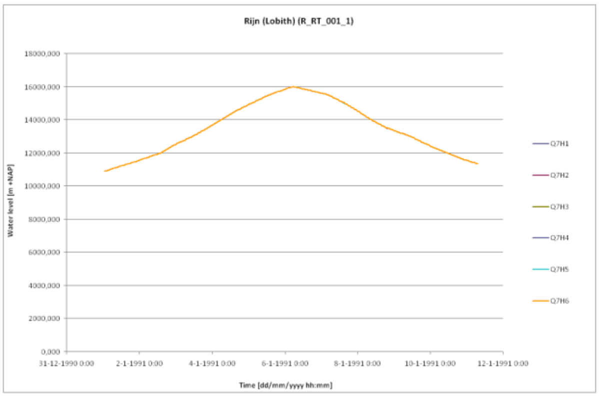

Figure 35: Results for the Rhine (Lobith) (OC)

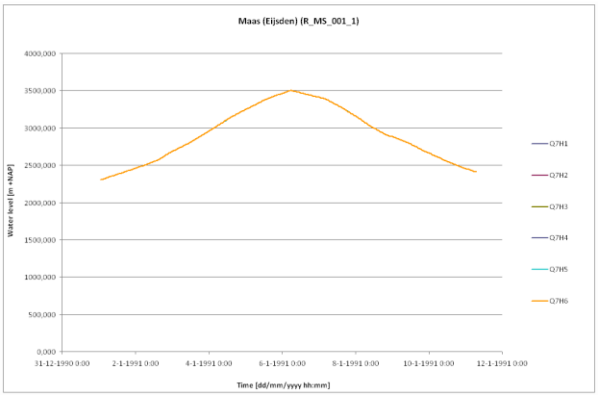

Figure 36: Results for the Maas (Eijsden) (OC)

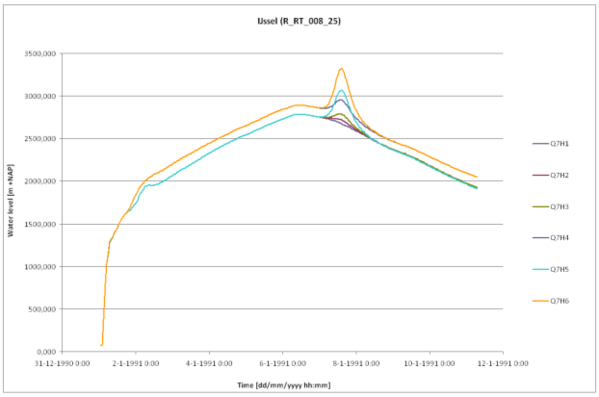

Figure 37: Results for IJssel (OC)

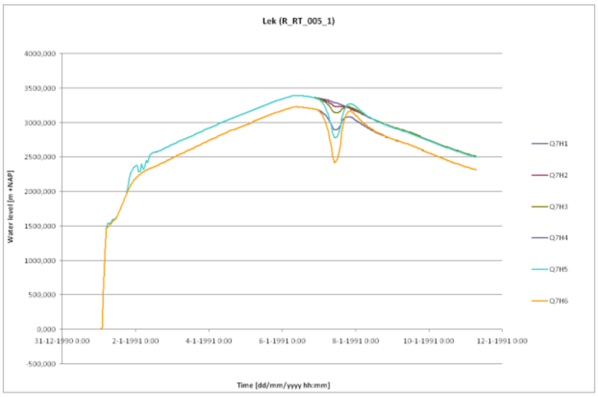

Figure 38: Results for the Lek (OC)

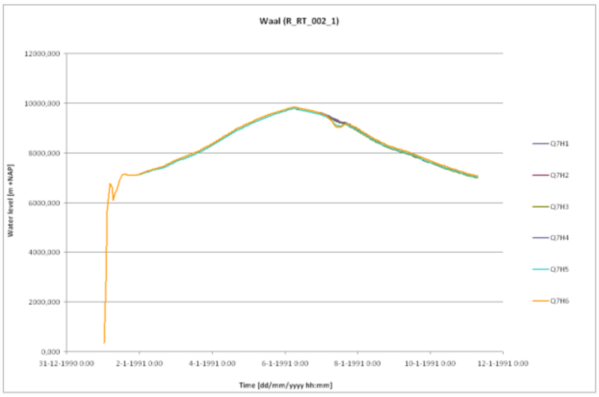

Figure 39: Results for the Waal (OC)

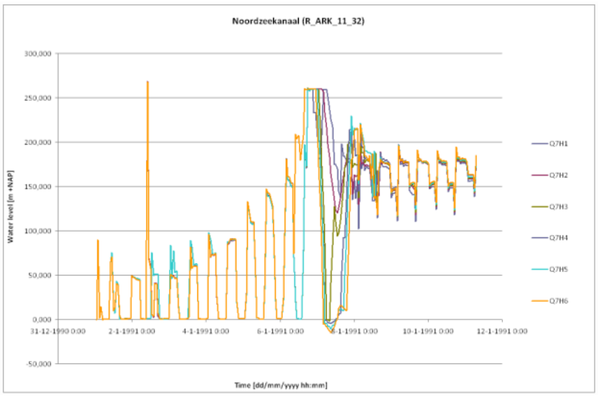

Figure 40: Results for the Noordzeekanaal (OC)

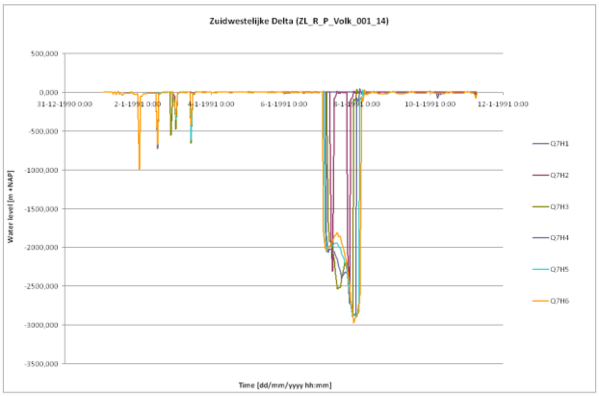

Figure 41: Results for the ZuidWestelijke Delta (OC)

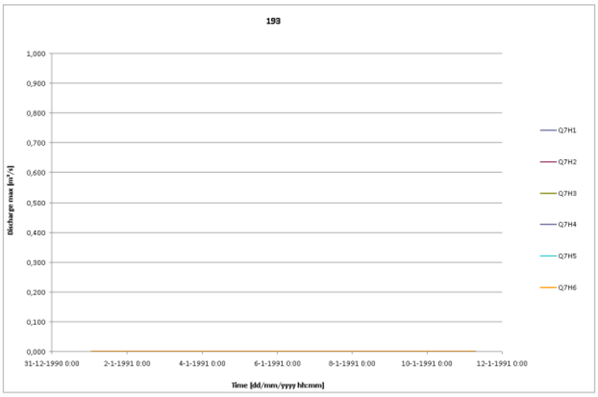

Figure 42: Results for the pumping station Afsluitdijk (OC)

6.3 Results Optimal Control with new structures: Water levels

In Figures 43 through 58 (Chapter 6.3.1 and 6.3.2) the results are plotted for cases where Optimal Control is applied and the new structures are added to the water system. The effects of the new structures, compared with the situation where Optimal Control is applied without the new structures, differs per location. As the four barriers are set keep the water level in the Rijnmond area below a certain threshold, the effects thereof are only visible in the cases where there this threshold is exceeded, i.e. the more extreme cases (combinations with H5 and H6, possibly H4 and H3 depending on the discharge boundary conditions). Where this is the case, peak water levels at Rotterdam and Dordrecht are kept about 1.0 m lower compared to the situation without new structures, and peak water levels along the IJssel and Lek are kept 0.5 to 1.0 m lower. At locations along the Waal, Hollandsch Diep and Haringvliet peak water levels are about 0.5 to 2.5 m higher in these cases. Locations along the IJsselmeer and Markermeer benefit from the reduced inflow from the IJssel in the more extreme cases, and from the new pumping station in all cases where the water level at these lakes would rise above 0.0 mNAP, resulting in a reduction in peak water level of up to 0.3 m. These effects are most clear at the (north-)east side of these lakes, as results at the (south-)west side are influenced most by the wind.

6.3.1 Results Optimal Control with new structures: Water levels

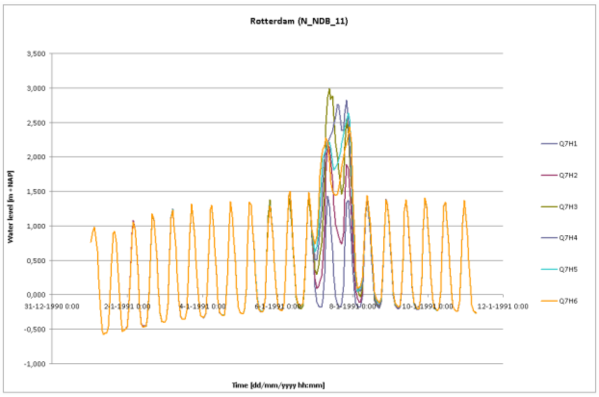

Figure 43: Results at Rotterdam (OC + NS)

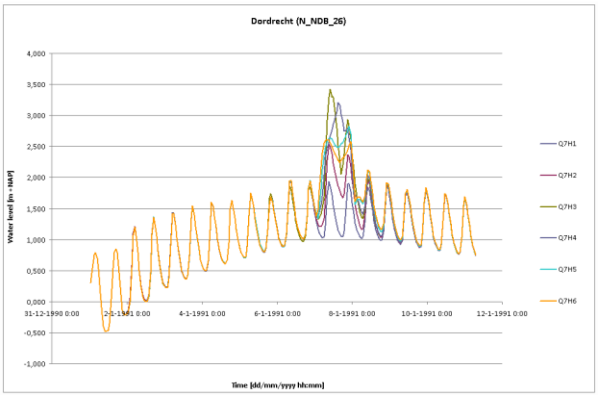

Figure 44: Results at Dordrecht (OC + NS)

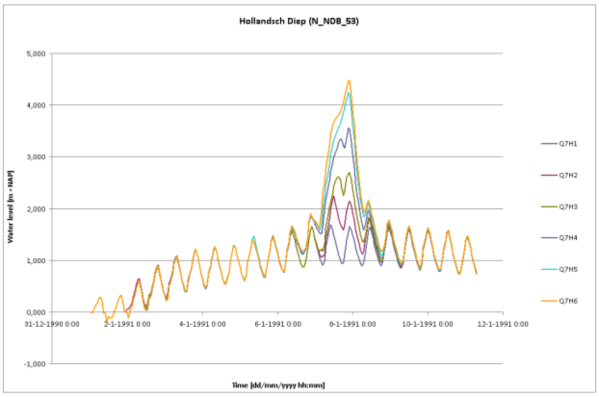

Figure 45: Results at Hollandsch Diep (OC + NS)

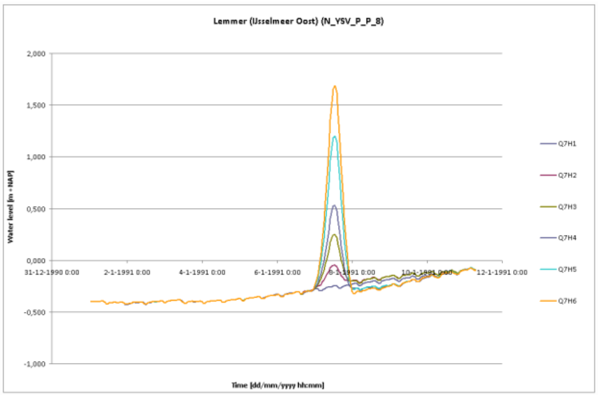

Figure 46: Results at Lemmer (OC + NS)

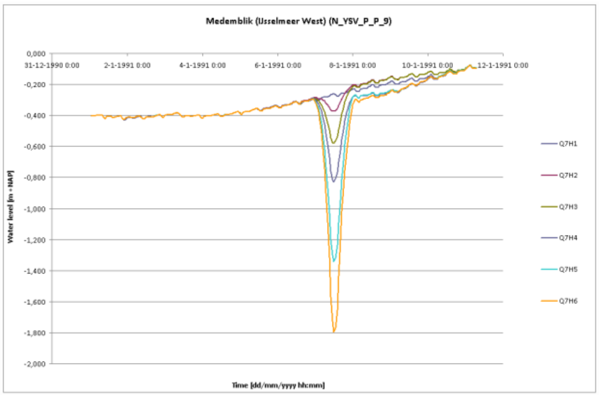

Figure 47: Results at Medemblik (OC + NS)

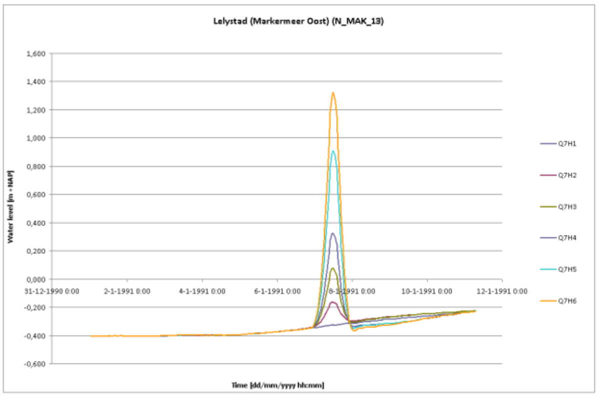

Figure 48: Results at Lelystad (OC + NS)

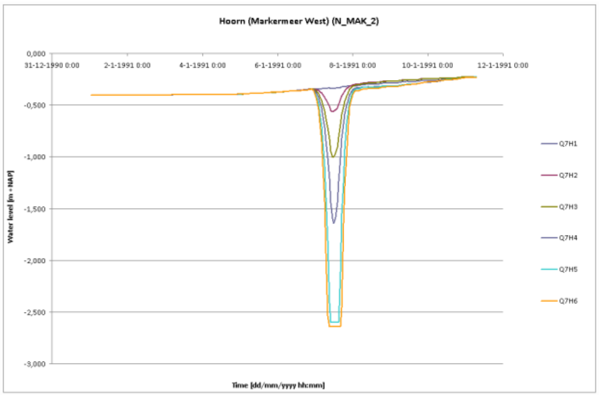

Figure 49: Results at Hoorn (OC + NS)

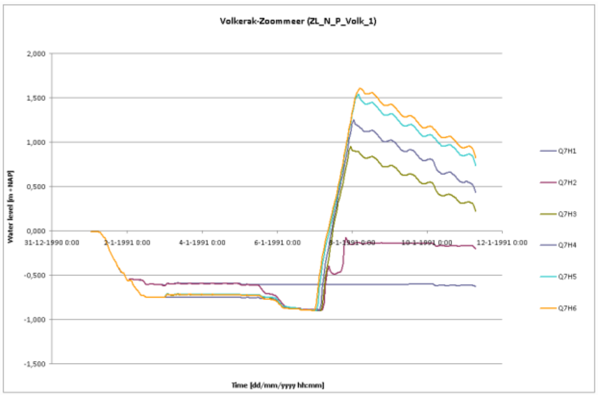

Figure 50: Results for the ZuidWestelijke Delta (OC + NS)

6.3.2 Results Optimal Control with new structures: Discharges

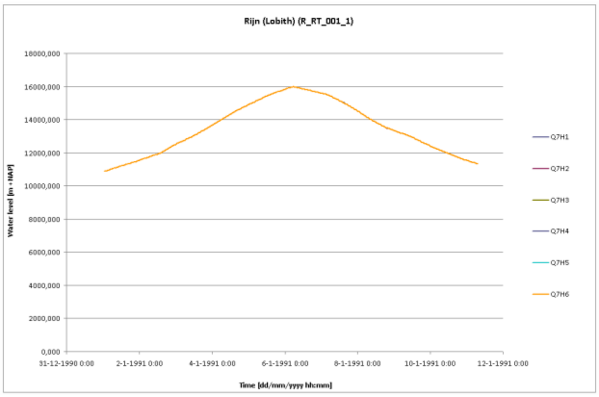

Figure 51: Results for the Rhine (Lobith) (OC + NS)

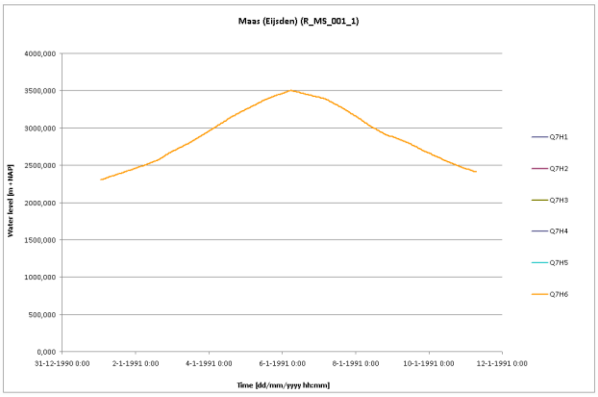

Figure 52: Results for the Maas (Eijsden) (OC + NS)

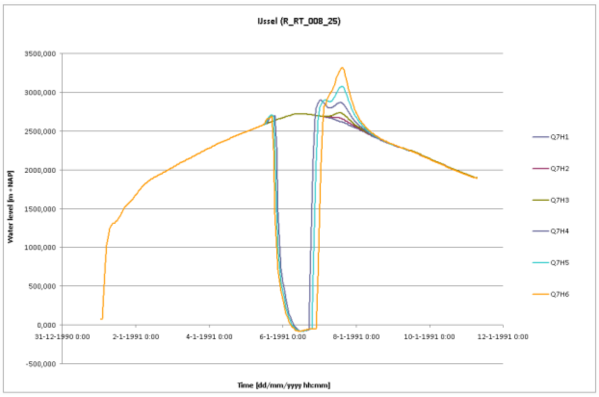

Figure 53: Results for IJssel (OC + NS)

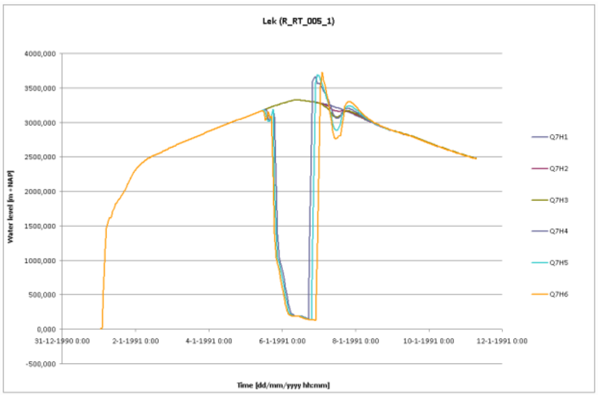

Figure 54: Results for the Lek (OC + NS)

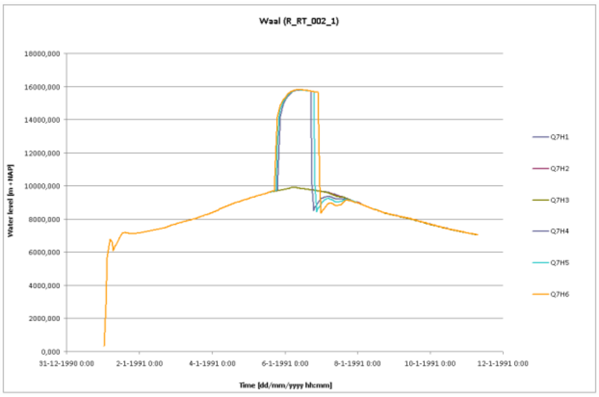

Figure 55: Results for the Waal (OC + NS)

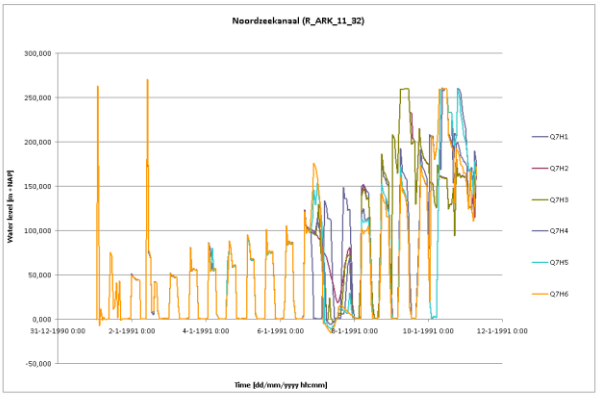

Figure 56: Results for the Noordzeekanaal (OC + NS)

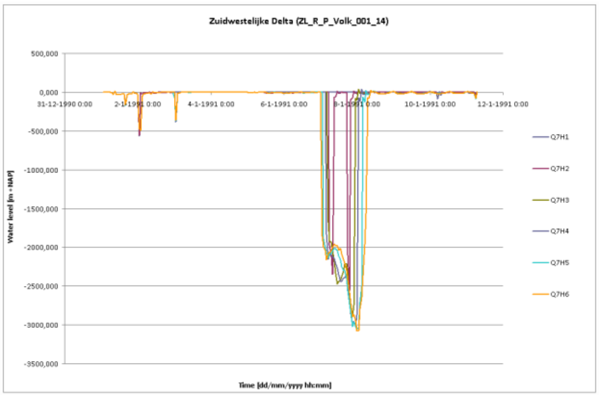

Figure 57: Results for the Zuidwestelijke Delta (OC + NS)

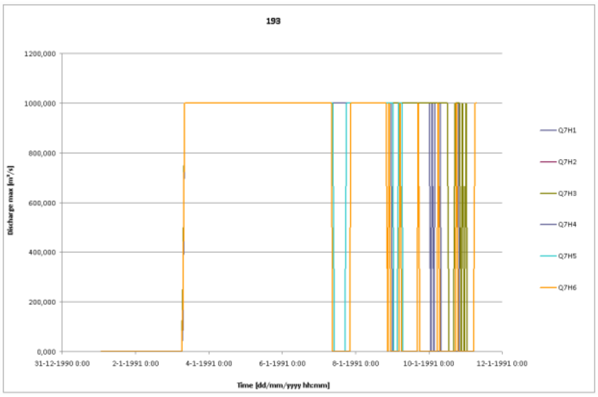

Figure 58: Results for the pumping station Afsluitdijk (OC + NS)

6.4 Comparison Optimal Control – Current control

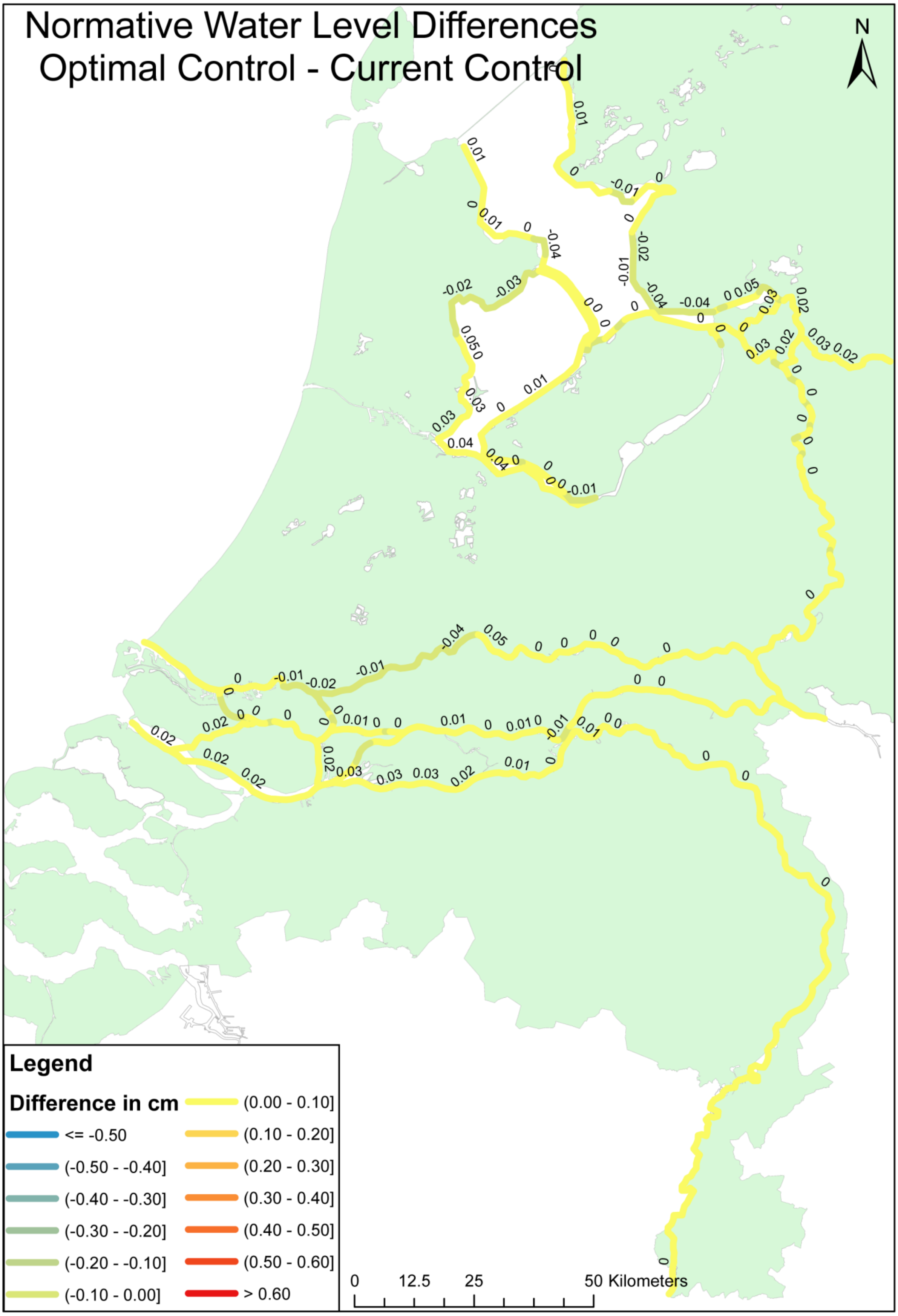

Figure 59: Normative water level differences between Optimal Control and current control

In Figure 60 the differences in results in terms of the normative water levels between the Optimal Control setting and current control setting are visualized. The clear effects of the application of Optimal Control, as described in Chapter 6.5, cannot be recognized in Figure 60. Only on the IJsselmeer water levels are decreased locally by up to 5 cm, where on the Markermeer water levels increase up to 5 cm, otherwise the differences between the two settings are minimal as all scenarios are considered and effects are levelled out.

6.5 COMPARISON OPTIMAL CONTROL WITH NEW STRUCTURES – OPTIMAL CONTROL

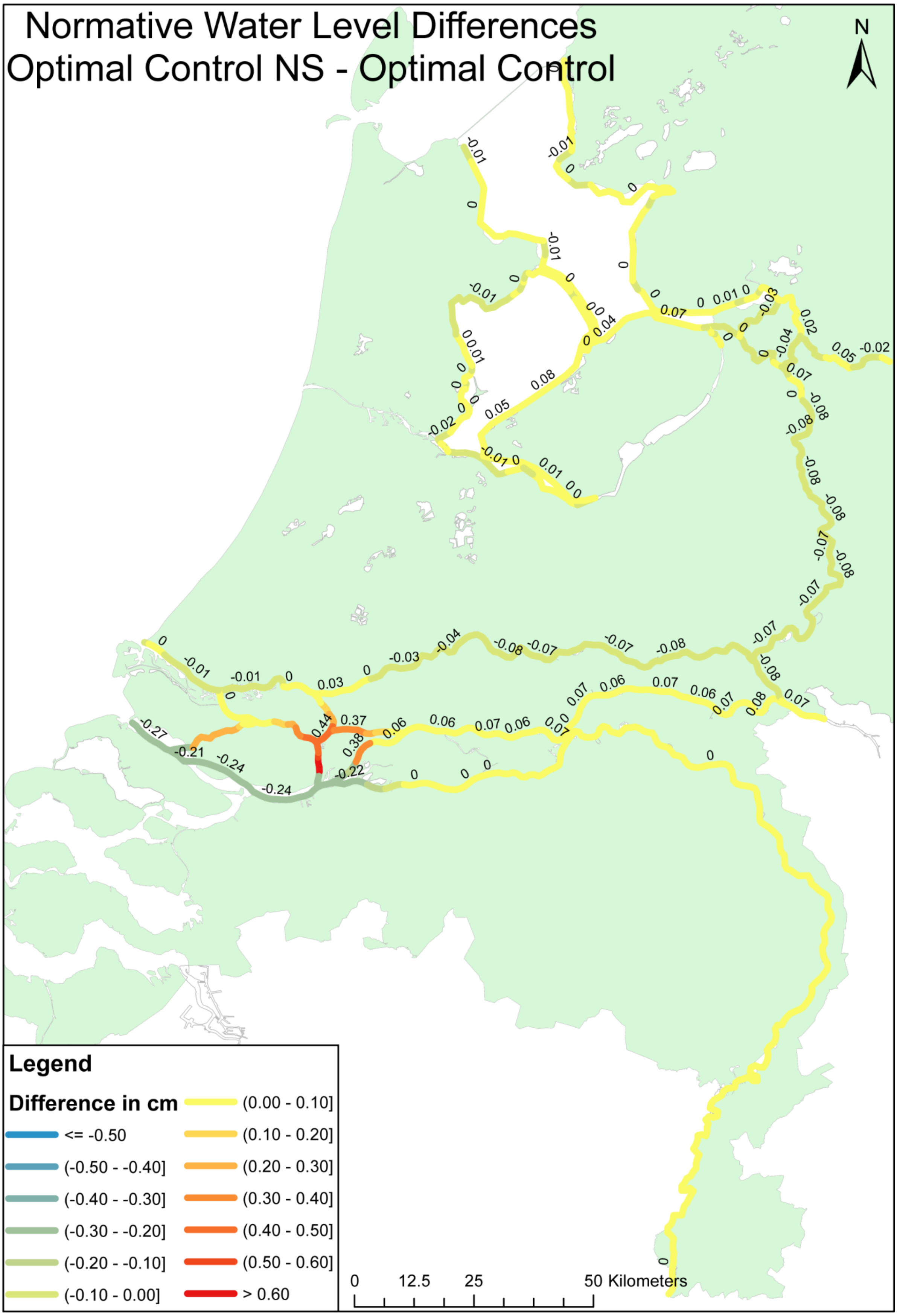

Figure 60: Normative water level differences between Optimal Control with new structures and Optimal Control without new structures

In Figure 61 the differences in results in terms of the normative water levels between the Optimal Control with new structures and Optimal Control without new structures setting are visualized. Now the effects of the new structures are less clear, as is to be expected as all scenarios are taken into account and the effect of the new structures differs per scenario (see Chapter 6.6). Water levels on the IJssel and Lek are lower due to the effect of the Pannerdensche Schuif, and as a result of this water levels on the Waal are higher. However the water levels on the Haringvliet and Hollandsch Diep appear to be lower compared to the case without the new structures. Water levels on the IJsselmeer and in the Rijnmond area do not change much, water levels on the Markermeer increase locally up to 8 cm and water levels near Dordrecht increase up to 44 cm. These effects are not in line with the effects displayed in Chapter 6.6.

An example of results which are more in line with what could be expected is displayed in Appendix B, where the differences in peak water levels from the Sobek calculations for case Q7H6 are visualized. Here the effects of all new structures, as described in Chapter 6.6 are more clear. Water levels in the Rijnmond area, as well as on the Lek, IJssel, Markermeer and IJsselmeer are kept much lower, while water levels on the Waal, Hollandsch Diep and Haringvliet are higher. It must be noted that the results in Appendix B are the result of one single (extreme) case and will therefore not represent the effects on the normative water level, though effects more in line with the results displayed for this calculation should be expected.

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

In this research, a model framework has been set up allowing for a probabilistic water safety analysis of The Netherlands using Model Predictive Control. The setting of this research is to assess the feasibility of the application of Optimal Control, looking at the possibilities enabled by the application of Optimal Control, which is intended to be a part of the measures accounting for safety against flooding. During this research improvements and adaptations have been made to the models used. Using this framework, a probabilistic approach, which is required by Dutch law for any measure in order to be considered a potential solution for safety against flooding, can be followed in order to determine an indication of the effect of the application of Model Predictive Control. One property of the system under consideration, when Model Predictive Control is applied, is that once the optimal settings for the structures have been given, it cannot be determined exactly why the given settings are optimal since these are the result of a minimization of the objective function which includes many locations, structures and goals, as these settings are determined by weights set in the Optimal Control. Therefore an iterative process has been gone through in order to determine a best suiting set of weights to be used in the objective function of the internal model.

What can be concluded from the results is that, when applying Optimal Control, clear effects can be expected in certain cases, while in other cases differences with current control are minimal (Chapter 6.2). As a result, the effect on the overall system behaviour (normative water levels) is minimal as all scenarios are considered and effects are levelled out. In the upper rivers water system no differences can be observed as in this water system (almost) no structures exist to influence the water distribution. These results are unique for the weights determined in Chapter 5.3.2, by changing these weights, setpoints and soft constraints different amount of water can be directed to different parts of The Netherlands where possible

When the new structures are added to the model, more extensive differences can be observed. It must be noted that the implementation of these structures in this research can be considered indicative as these structures have not been included in the objective function (Chapter 5.3.2), therefore a more complete implementation should be considered for further research. The effects of these structures are clear when considering individual cases (Chapter 6.3), however the results in terms of differences in normative water levels are not in line with results obtained from individual cases. More detailed inspection of the results obtained from different parts in the model framework revealed some inconsistencies in the outcomes of the Sobek-calculations, which are probably the cause of the deviating results in terms of normative water levels. Due to the complexity of the model framework and enormous amount of data output such inconsistencies can be easily overlooked. Considering this the results displayed in Chapter 6.5 should not be considered representative for the differences in overall system behaviour when the new structures are added to the system. Possibly some inconsistencies still exist for the calculations with current control and Optimal Control without new structures as well.

A distinct effect of the new structure Pannderdensche Schuif is that, when (fully) closed, at high discharges this structure causes large backwater curves which will be noticeable even in Germany. As this structure closes off the Pannerdensch Kanaal, all water has to flow through the Waal resulting in much higher water levels, both in upstream and downstream directions. These effects are possibly aggravated by the fact that this structure is opened and closed within one hour (one time step in the MPC, one output time step in Sobek), though this still means that before such a structure could be utilized, measures have to be taken to prevent or reduce these effects.

7.2 Recommendations

In this research the focus has been on water safety, and less on the costs associated with the operation of the controllable structures or costs associated with the effects on navigation when utilizating certain structures (Maeslantkering, Volkerakrsluizen). Notably, when Optimal Control is applied, the Maeslantkering is allowed to close multiple times after each other in order to create storage the Rijnmond area. However, this technique to create storage is more successful in certain cases than others , largely depending on the amount of flow through the Nieuwe Waterweg (as the Maeslantkering has to open when the water level at the side of the Nieuwe Waterweg becomes higher than the water level at seaside). Further research to the effect of this technique on the creation of storage, the consequences thereof on navigation and/or further tuning of the Optimal Control is recommended.

In- and outflows from local water systems (Water Boards) have not been considered. Especially for the lakes subsystem the effects from local water systems could prove to be significant and the effects thereof should be investigated further.

Due to the setup of the boundary conditions, it is possible that the discharge capabilities through the structures in the Afsluitdijk (Lorentzsluizen and Stevinsluizen) have been overestimated. For the 'lakes' sub-watersystem, not the current wind- and storm setup of one day should be considered normative but a (less extreme) setup at seaside with a longer duration which limits discharge through these structures. This effect has not yet been investigated in this study.

As mentioned in Chapter 3.1, not all existing structures in the Dutch water system have been considered as the effects of most of these structures for flood prevention are considered to be (very) limited. For low-flow scenarios, however, the effects of structures may have been unjustly neglected, though the effect of this on MWH-level is estimated to be negligible.

In this research the assumption is made that all dikes are able to withstand the imposed water levels. In reality this might not always be the case. To (partly) accommodate for this assumption, the results in terms of peak water levels could be compared to the local dike height. This way, the results can be observed as increase (or decrease) in freeboard, instead of decrease (or increase) of peak water level. This way still the assumption is made that the dikes do not fail due to other failure mechanisms than overtopping.

A number of the new structures (Spuischuif, Drechtschuif, Merwedeschuif and Pannerdensche Schuif) described in Chapter 3.2 are modelled as barriers which are either completely opened or completely closed, and have not been included in the objective function. Substantial improvements are to be expected when these structures are modelled as gates which can take any level instead of completely opened or closed, and are included in the objective function. For this to be realized the internal model has to be modified further and more research is required on which weights are applied to the usage of these structures. Additionally setpoints or soft constraints may have to be added for further improved results. Furthermore, in the current model the Pannerdensche Schuif completely closes off the Pannerdensch Kanaal, which would cause problems for navigation and cooling water availability. A solution should be sought for a type of control which would allow a minimum amount of discharge to go through the structure to prevent these problems.

In this research, a rough estimation was made for the costs of the realisation of new structures and required additional measures. In order to determine if these measures provide viable solutions, further research is required to provide more detail about these costs and benefits. Furthermore, these measures would require enough political support, which has not been considered in this research.

For the structure Pannerdensche Schuif; instead of a structure which limits the flow through the Pannerdensch Kanaal, a solution can be sought in the direction of a structure or other measures which can dynamically increases the capacity through the Waal, or a combination of both. This way large positive backwater curves can possibly be avoided. Another option could be to move the Pannerdensche Schuif further downstream, similar to the 'Afsluitbaar Open Rijnmond' variant (Stijnen et al., 2010) however this way a portion of the benefit of this structure is lost since (a part of) the Lek and IJssel are no longer protected, and this may require the realization of a new canal or other storage accommodating measures.

In this research, when the Maeslantkering is set to fail, only the failure mode where the barrier remains completely opened is considered. In reality however, additional failure modes exist, e.g. 'partial failing' where only one of the two barrier arms closes or both arms close but cannot be sunk, which have varying effects on the water levels in the Rijnmond area (Botterhuis et al., 2012). These failure modes have not been included in this research and should be investigated further.

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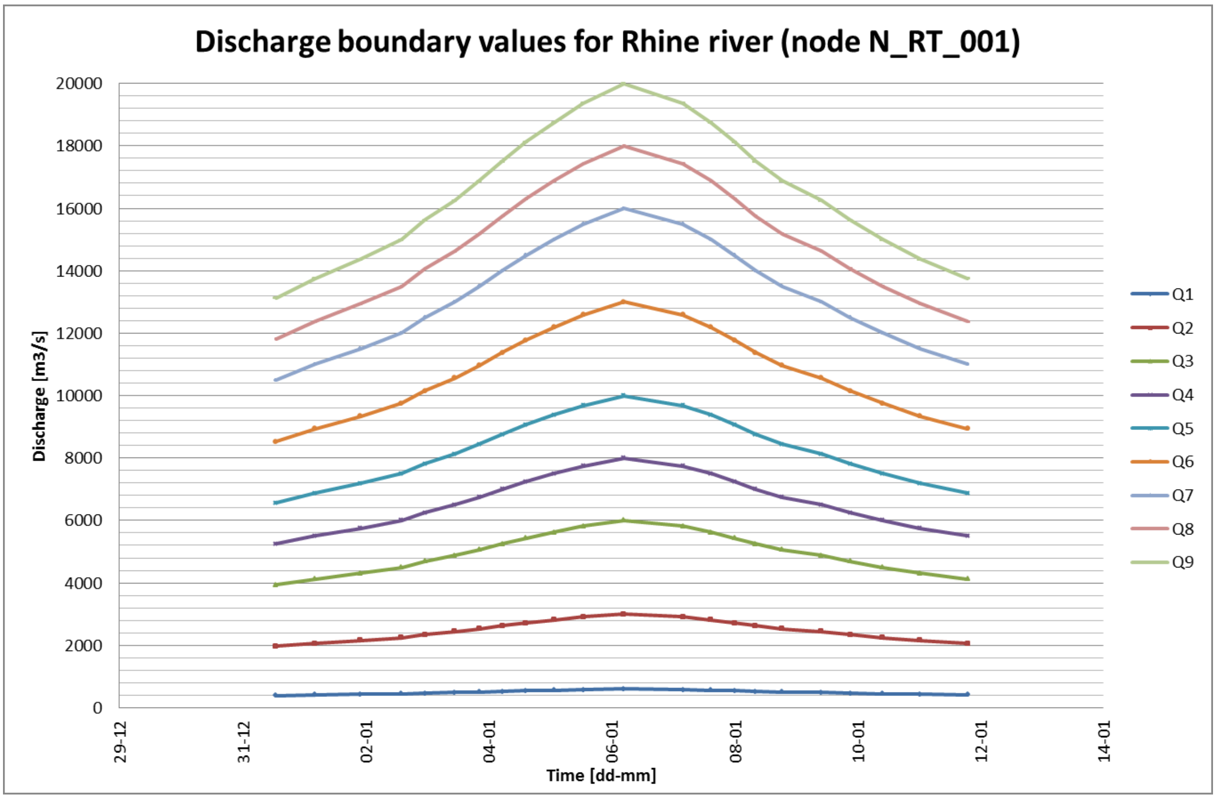
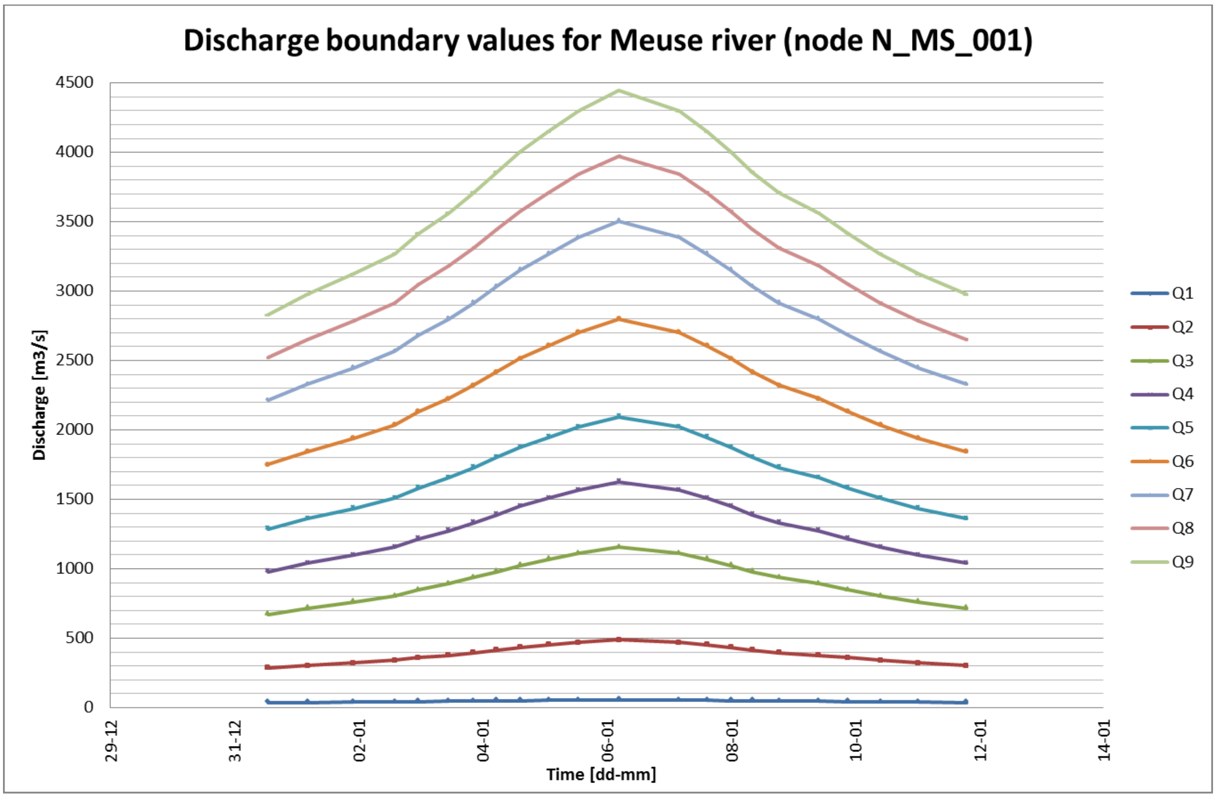
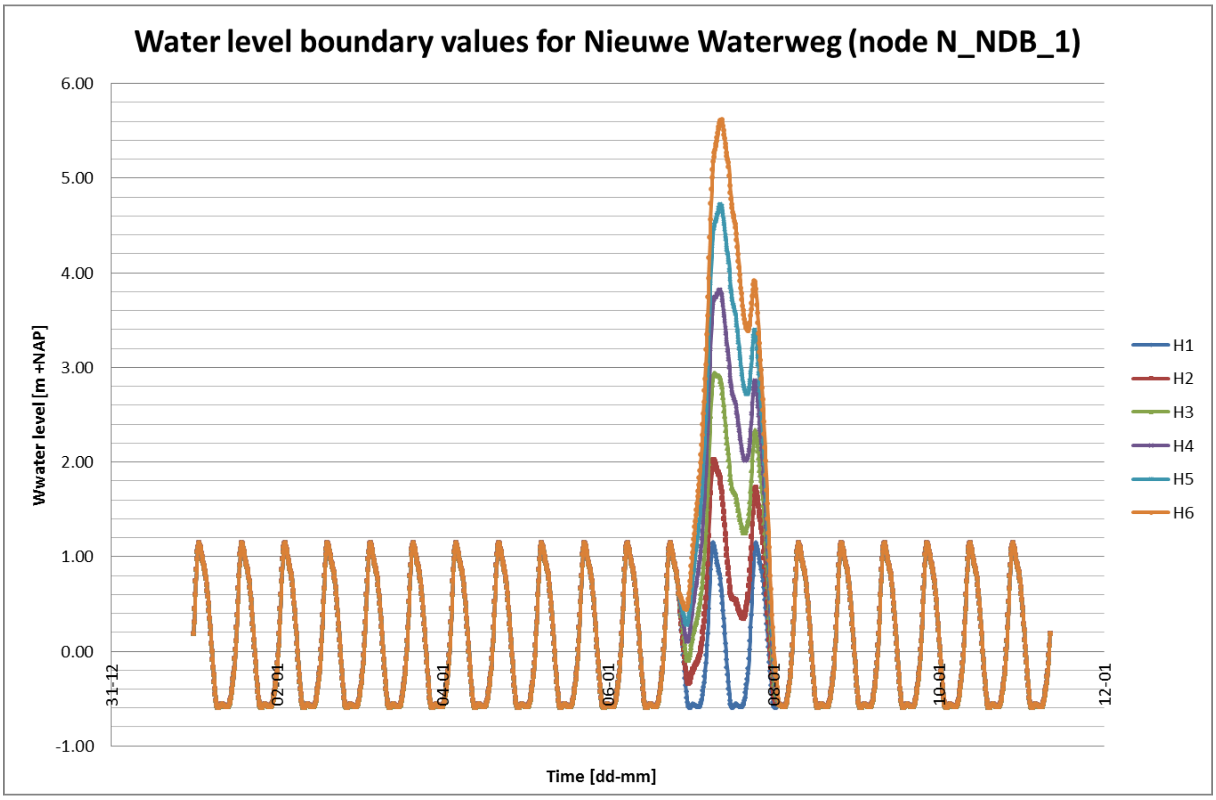
APPENDICES

Appendix A – Boundary conditions







		H1	H2	H3	H4	H5	H6
Date	Time	Wind speed [m/s]					
6-1-1991	5:40:00	0.00	0.00	0.00	0.00	0.00	0.00
6-1-1991	20:40:00	0.00	0.00	0.00	0.00	0.00	0.00
7-1-1991	9:10:00	2.91	13.72	19.78	24.03	30.85	34.94
7-1-1991	10:50:00	3.00	14.13	20.36	24.74	31.76	35.93
7-1-1991	11:10:00	3.01	14.17	20.42	24.81	31.86	36.03
7-1-1991	11:30:00	3.00	14.13	20.36	24.74	31.76	34.94
7-1-1991	13:10:00	2.91	13.72	19.78	24.03	30.85	2.91
8-1-1991	1:40:00	0.00	0.00	0.00	0.00	0.00	0.00
9-1-1991	0:00:00	0.00	0.00	0.00	0.00	0.00	0.00

APPENDIX B – WATER LEVEL DIFFERENCES CASE Q7H6