Application Feed Forward Controller on Delta Mendota Canal

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December 2004

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CanalControl

Preface

Control on water systems (especially Real Time Control) is a very attractive aspect of Water Resource Engineering, since its goal is to solve practical Water Management problems with theory from control engineering. It is a challenge to find the balance between the performance of the water system and the complexity of the control system, on the one side and on the other side the economic consequences and feasibility to implement the control system in reality. The control system that is best suitable for the situation is determined by political, practical and economical boundaries and depends on the operator's experience and familiarity with controlled systems of and his or her knowledge of the water system. Therefore, the boundaries of this study go far beyond theoretical knowledge on controlled systems and pure water management, which makes this field of study so interesting.

This MSc-thesis deals with a specific water delivery problem, concerning reservoirs in Californian irrigation systems. The researched system is the Delta Mendota Canal in "the Valley" of California. The research is carried out at the Irrigation Training and Research Center in San Luis Obispo, California and at the Department of Water Resources Engineering of the Delft University of Technology in The Netherlands.

My visit to San Luis Obispo the first quarter of 2004 was only possible due to the great willingness of Dr. Charles Burt and ITRC to have me in their office and to the never ending enthusiasm of Peter-Jules who put a lot of effort in the preparations for my research abroad.

I would like to thank my substitute mom in California, Maria Burt, for having me in her house for three months and for all the delicious meals and beautiful hikes. I am also thankful about the way you dealt with the dents and scratches that I made when I borrowed your car (or haven't I told you about that yet?).

I would like to thank my Dutch mother, Emilie, for having me in the house for so long (it is about time I get a place of my own) and for providing an atmosphere that made my time as a student very easy.

Not last, but definitely not the least thanks go to Saskia, my friends in Delft and (now in) Zimbabwe for making my time at the Delft University an interesting and amusing period.

Finally, my financial thanks go to the Irrigation Training and Research Center (ITRC), Het College van Bestuur Fonds, Het Universiteitsfonds and Het Lammingafonds. Without them it would have been a very expensive experience!

Marcel Bruggers

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December 2004

Summary

Introduction

The presence of water makes life possible on this planet. Our existence and our economical activities are very dependent on this resource. Proper management of the available water is therefore essential.

This research is part of the operational level of quantitative water management, which concerns the daily operation of the water system infrastructure (sluices, gates, pumping stations, etc.). It is done in collaboration between the Delft University of Technology and the Irrigation Training and Research Center in San Luis Obispo, California.

The research object is the southern part of the Delta-Mendota Canal (DMC) between the O'Neill Forebay and the Mendota Pool and flows through the western part of the San Joaquin Valley, California. These 75 km of the DMC are divided by nine check structures, creating eight reaches with an average length of 9.4 km. The canal was completed in 1951 and is mainly essential for irrigation supply.

The upstream controlled structures consist of three parallel radial gates and two short crested weirs. At the downstream end of each reach an irrigation district has its turnouts located, to distribute water from the canal to the users.

Problem statement

The controlled irrigation system cannot respond fast enough to large downstream water demands, resulting in water level deviations in the downstream reservoir. With a faster response, the size and duration of the water level deviations can be reduced.
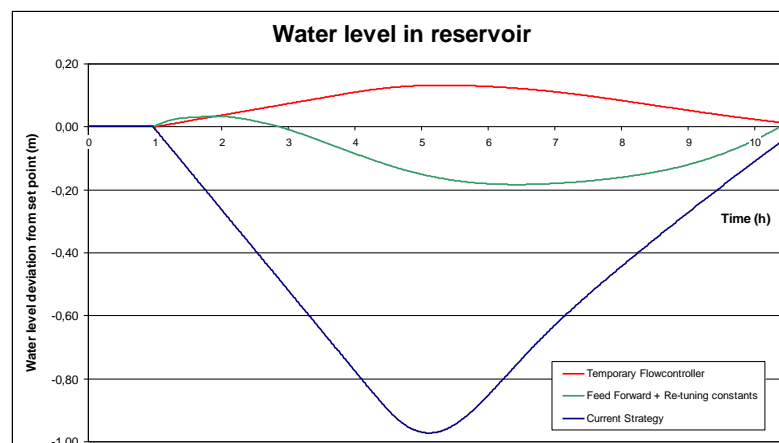
Requirements

- The solution has to be an additional component to the controller so that the robustness of the present control system is not affected.
- Only the measuring points upstream of each check structure may be used.
- The communication frequency is limited to once every 5 minutes.

Recommendations

One solution is found in a feed forward controller in combination with re-tuned constants. An alternative solution is formed by a temporary flow controller, but special attention is necessary when dealing with varying turnout flows.

The figure shows the water level of the reservoir for the present control strategy and for the two recommended strategies.

Water level in reservoir

Contents

1. Introduction

1.1 Water Management

The presence of water makes life possible on this planet. Our existence and our economical activities are very dependent on this resource. Proper management of the available water is therefore essential. Apart from the need for qualitative good water, it is also important to have sufficient water. Hence, Water Management is divided in a qualitative and a quantitative component.

Four levels can be recognized in those two components: the strategic level, the tactical level, the operational level and the technical level. Concerning the quantitative component the next definitions apply. At the strategic level objectives are determined concerning the water distribution from the various sources over the users. In other words: it determines *what* the user will get. In the tactical level these objectives are formulated in measurable and manageable quantities. The operational level concerns the daily operation of the water system infrastructure, which means: the operation of sluices, gates, pumping stations, power stations and other operational devices. This level determines *how* the user will get his water. The fourth level concerns the exact technical design of the system. [Brouwer, 2001] Figure 1.1 demonstrates the foregoing.

LEVELS OF DECISION MAKING

STRATEGIC LEVEL

LONG TERM OBJECTIVES, POLICIES

water quality, water quantity, risks, safety, …

TACTICAL LEVEL

SYSTEM BEHAVIOR, PERFORMANCE

set points, flows, error margins, dead bands, …

OPERATIONAL LEVEL

SYSTEM KNOWLEDGE, FUNCTIONAL DESIGN

proportional controller, decoupled, feed forward, …

TECHNICAL LEVEL

COMPONENT KNOWLEDGE, TECHNICAL DESIGN

engine power, type of steel, scada, hardware

Figure 1.1 – Levels of decision making

1.2 Framework thesis

This report fits in the operational level. It is done in collaboration between the Delft University of Technology and the Irrigation Training and Research Center in San Luis Obispo, California. It strives for a solution for a delay time problem in an inline canal system in California. An inline canal system is an irrigation system where all pools are in series with each other. Chapter 2 gives an elaborate view on the problem. Information about the irrigation system can be found in paragraphs 1.4, 1.5 and annex 1 provides the exact dimensions. The most important objective of an irrigation water delivery system is to provide a specific amount of water to the users. The water distribution in early irrigation systems used to be arranged manually or by hydro-mechanical controllers, such as the Begemann gate and the Vlugter gate. Figure 1.2 shows a similar hydraulic structure that automatically controls the upstream water level. The presented hydraulic structure is a flap gate.
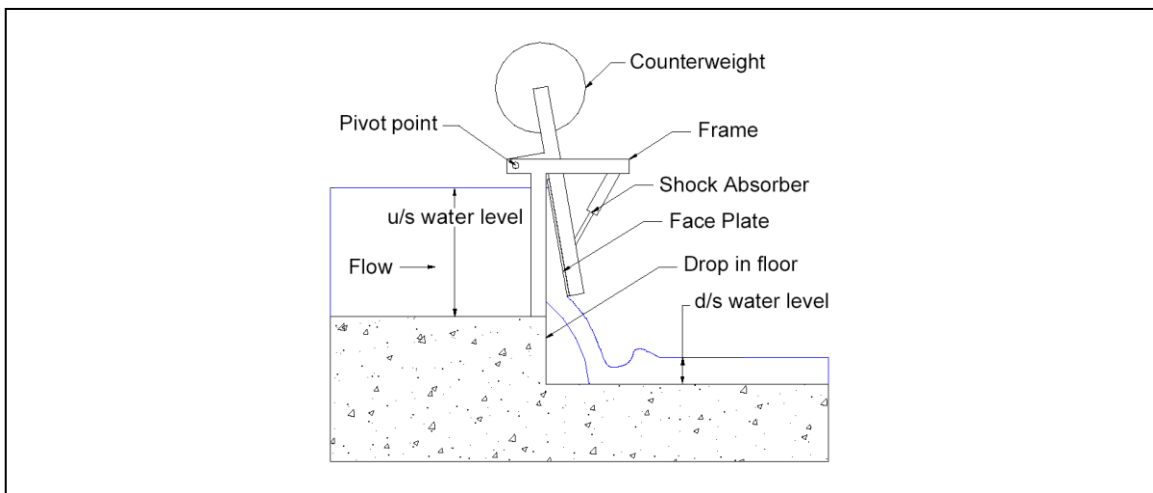
Fig 1.2 – Begemann gate; providing automatic upstream water level control

Adapted from source: Burt, 2000

These control structures are still widely used in developing countries, but particularly in western countries a vast amount of water delivery systems are now controlled with computed automated control. Computed automated control provides a faster, more accurate and flexible water supply so that the users can use the water more effectively. Two other grounds for applying computed automated control[1] are the reduction of operating costs and the increase of the system capacity. [Burt & Plusquellec, 1990]. Since the research subject is a Californian irrigation system and already having automated controlled gates, no further attention is given to hydraulic control structures.

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[1] From this point the term 'computed' in combination with 'automated control' is left out. In this report 'automated control' implies 'computed automated control'.

Figure 1.3 shows a automated radial gate as can be found in the Delta Mendota Canal. The radial gate can be recognized as the metal curved plate behind number 1 in the water, the controlling device for the three gates is located left of number 2 in the gray case. One of the driving devices (an electric motor) is located on the right side of number 3. In the following an electric device together with a gate is referred as 'the actuator'. Some distance further, approximately 100 meters, a pressure transducer to measure the water level is mounted in the canal and provides real time water levels. Number 4, the pressure transducer, is not to be seen in the picture.

Figure 1.3 – Automated radial gate; 1: gate, 2: controller, 3: actuator, 4: pressure transducer

Location: Delta Mendota Canal, Check structure 14, January 2004

1.3 Operational Water Control

Quantitative control actions normally take place by applying feedback control. This type of control is based on an error between the desired water level or discharge and the actual value of those parameters. A schematic representation of a feedback controlled system is shown in figure 1.4. The input of this system is the desired system behavior, the output represents the actual values of the controlled variables. The most left box symbolizes the controller, the part where the control action to be taken by the actuator is calculated. The actuator, for instance a pump or a gate, carries out the control action. The output of the water process is compared with the desired values, which completes the loop. This kind of control is often referred as 'closed loop feedback control system'. Figure 1.4 shows a single-input-single-output (SISO) feedback system.
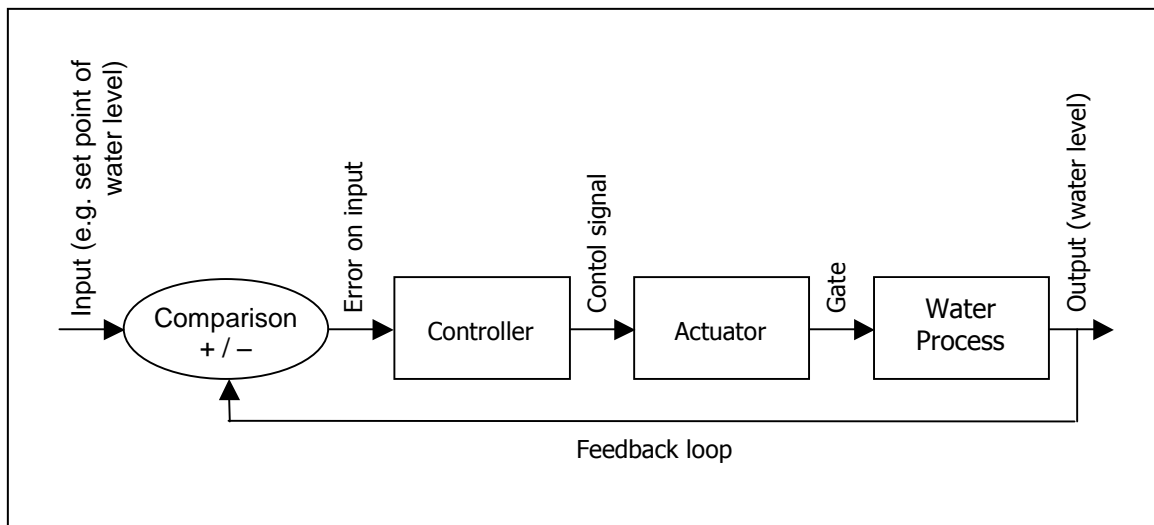
Figure 0.4 - Feedback control system

Dependent on the system, one or more parameters are used to meet the objectives. The next examples from Dutch origin will clarify this. Note that the decision or control parameters may differ in a Californian perspective. Example 1: in a broad, shallow river with a large discharge it is not useful to base the control actions on discharge, however it is important to maintain a certain water level in the river to guaranty safe navigation. Example 2: in a drinking water distribution system water levels are not leading, but it is of the utmost importance to have a proportional distribution over the users. Example 3: an irrigation system often needs to be controlled on both parameters. The water levels in all the canals of the irrigation system need to be above a certain level in order to allow water extraction. For instance water pumps need a minimum water level to prevent air coming in the inlet pipe. On the other hand the water distribution need to be arranged in such a way that enough water will flow in every branch of the irrigation system. Figure 1.5 shows a system where three variables are necessary to meet the system objectives, this type of control is categorized with MIMO (multiple-input-multiple-output).

Input

Output

Comparison
+/-

Controller

Actuator

Water
process

Feedback loop
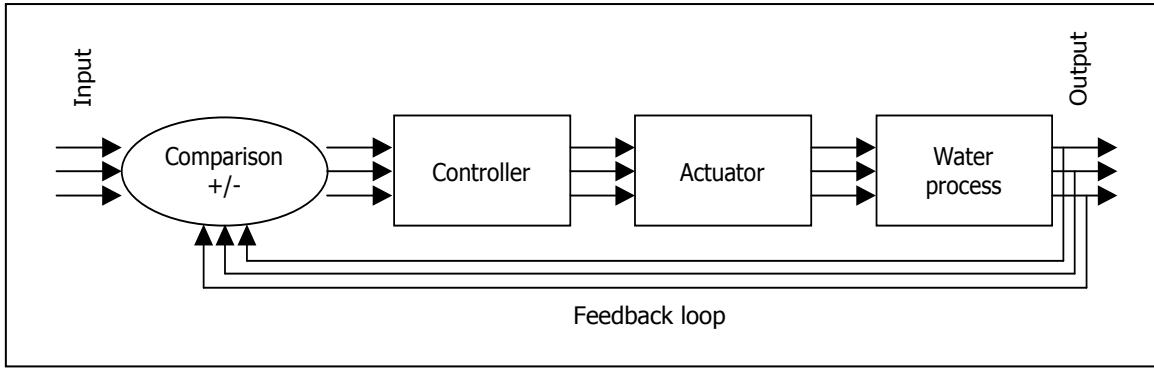
Figure 0.5 – Multivariable feedback controlled system

1.4 Desired control method

When considering water control systems in irrigations canals, a distinction can be made between:
- Self managing systems
- Non self managing systems

In self managing systems variations in the demanded amounts of water are automatically transmitted to upstream (diversion) structures, where the supplied quantities are adjusted. Downstream control[2] is a type of a self managing system. In non self managing systems these variations will result in water level deviations downstream, as with upstream control[2].

On the other hand a distinction can be made between:
- Automated control systems, such as:
    o Hydraulic automated control
    o Computed automated control
- Manual control systems

Regarding manual control systems the control structures are adjusted manually, with automated control systems the water levels or discharges are maintained by automatic hydro-mechanical structures or by computer controlled structures.

The typical Californian layout of a controlled water delivery system for irrigation purposes is shown in figure 1.6. It is composed of a reservoir followed by an irrigation canal, in which the water levels are automatically maintained by radial gates, sluice gates or flashboard gates. At the downstream end of each reach a certain irrigation district or water district has its turnouts located to distribute water from the canal to the agricultural area and users. The control on water levels is done manually using local upstream target levels, which means that the adjustments of each gate are based on the water level just upstream of this structure. The measuring of water levels is done using pressure transducers as water pressure is proportional to water depth. When a control system is able to maintain the water levels at, or close to set point during operation, the inflowing water is automatically matched to the demand (any mismatch will result in a deviation from set point on which the controller will react).
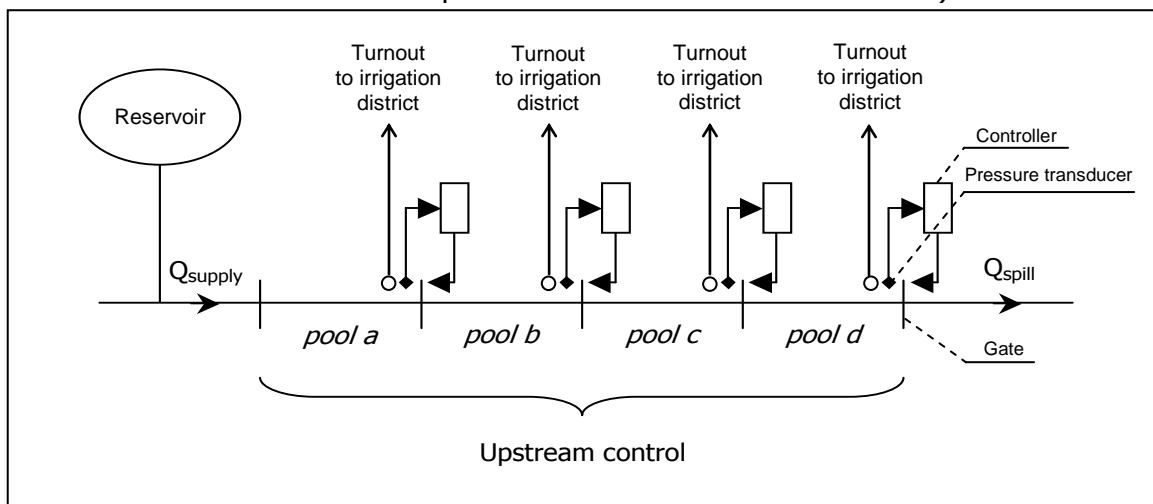
Figure 1.6 – Upstream controlled water system

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[2] The reader is referred to paragraph 3.3 for a description of the control methods

One of the consequences of upstream control is that the supplied quantity at the (upstream) reservoir outlet (Q_{supply}) is not affected by measurements in the system (i.e. by the transducers in pool a, b, c or d). To prevent water deficit, which is considered worse than water abundance, the supplied quantity into the canal is deliberately larger than the demand. The inevitable result is spillage, regarding these systems spillage is considered as an unavoidable given.

It is very difficult to supply the exact amounts of water in advance that are necessary to meet the needs of the users. It is not unusual to have delay times (lag times) of six hours in the system, which make it necessary to supply six hours in advance. Errors in measuring devices and discrepancies between control structures, illegal water extraction, rainfall run-off water, evaporation make 100 percent accurate estimations of the needed quantities practically unfeasible. Above that, limitations in the control frequency make digitizing of the estimations necessary, resulting in less accuracy.

The approach of the Irrigation Training and Research Center (ITRC) in San Luis Obispo towards canal automation is based on providing service to the customers. Service is divided into three categories: equity, flexibility and reliability. The categories in focus are flexibility and reliability. The equity category has already been taken care of by ITRC and will not be regarded any further. [In consultation with Dr. C.M. Burt, 2004] Flexibility can be defined by the degree of 'on-demand delivery', in other words the degree of freedom to change the rate, frequency and duration of the water deliveries or extractions. [Wahlin, 2002] Reliability is the ability of a water delivery system to supply the promised or desired quantities. It also means that the control system is designed properly that no canal banks or structures are damaged. [Burt and Plusquellec, 1990] Flexibility and reliability can be achieved in any irrigation system when the next four – arbitrarily listed – requirements are present:

- An annual water surplus; the demanded amounts of water may not exceed the maximum capacity of the source.
- Information on the expected water demand to anticipate on the required volumes.
- A buffer or reservoir to prevent significant water level fluctuations due to lag times and unforeseen water demands or extractions.
- Controllers and actuators that maintain water levels at all times within a small margin and without causing damage. [Lobbrecht, 1997]

Naturally, besides aiming to provide the best service possible, minimization of water spillage is also a main objective. Based on the preceding, the desired irrigation system is as schematized in figure 1.7. It consists of a number of pools divided by automatically controlled radial gates. Again, on the downstream end of the pools the irrigation districts have their turnout's located (not drawn in figure). One of the pools in the canal is connected to a reservoir, which provides in a buffer function. At the upstream end of this pool the upstream control method is applied, on the downstream side downstream control is applied. The supplied quantity of water that is needed is estimated beforehand and is shown in the picture as 'Q_{supply}'. On the left side the Main Regulator (MR) discharges this flow. On the right side 'Q_{spill}' can be seen, which represents the surplus of the flow and can be regarded as lost. As can be seen in the scheme, the water level in *pool e* is not controlled by a gate in the canal. It is controlled by a pumping station, which takes care of the water transport between reservoir and

pool e. So *pool e* gets water from the reservoir in case of water deficit and in case of water surplus the pumping station draws water into the reservoir.

The reason for the control configuration, with both upstream and downstream control, is that a surplus of water will result in a rise of the water level in *pool e* and a water deficit will result in a water level drop. The explanation here fore is that in the upstream controlled part the water deficit or water surplus is 'transported' in downstream direction, since the upstream water levels are to be maintained (Figure 1.7). An analogue reasoning applies to the downstream controlled part. That part 'transports' the water deficit (or surplus) in upstream direction, as the downstream water levels are controlled. In the figure this effect is illustrated with 'direction of disturbance'.

The question remains: why not to apply downstream control in the whole canal, since with downstream control the upstream check structure is adjusted. The answer to this question is that downstream control does not always provide the best performance, the highest robustness and is not always suited for the situation. [In consultation with Dr. C.M. Burt, 2004] The reader is referred to the last section of paragraph 3.3 'Advantages and disadvantages of the control methods' for further details.

For this thesis research the Delta-Mendota Canal[3] in California is used to provide real system information, since this canal has a similar structure as shown in figure 1.7. With these kinds of irrigation systems the reservoirs form a substantial part in the construction costs. This thesis aims at a solution to reduce the necessary reservoir size.
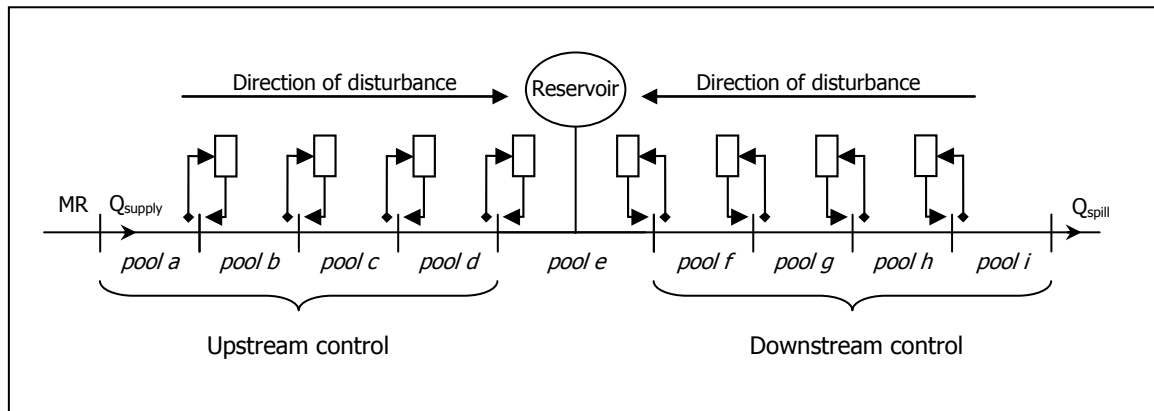
Figure 1.7 – Desired irrigation system lay out

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[3] Data on the Delta Mendota Canal is extracted from documents [USBR, 1992] and [ITRC, 2003] as mentioned in '6. References'.

1.5 Description of the system investigated

Background information

The Delta-Mendota Canal (DMC) is located on the west side of the San Joaquin valley. The general direction of the DMC is northwest to southeast and measures about 200 kilometers. The canal, which was completed in 1951, is essential for irrigation supply as part of the San Luis Unit and the Central Valley Project Delta Division. The canal has its head southwest of Stockton (about 100 km east of San Francisco), where it receives water pumped at the Tracy Pumping Plant from the Sacramento-San Joaquin River Delta. It flows through the western part of the San Joaquin Valley to the Mendota Pool, approximately 50 km west of Fresno. From the Mendota Pool it flows via diversion structures to other irrigation canals and to the San Joaquin River. The initial diversion capacity is 4,600 cubic feet per second (approx. 130 m³/s), which gradually decreases to 3,211 cubic feet per second (approx. 91 m³/s) at the terminus. The Delta-Mendota Canal delivers nearly 4 billion cubic meters of water per year within the service area. Seventy-five percent of this amount is delivered to agricultural lands and the remainder goes to municipal and industrial uses and to wildlife refuges. The agricultural lands and the wild life refuges cover about 8,500 square kilometers within the San Joaquin Valley, the San Benito and Santa Clara counties. The DMC is controlled by the San Luis and Delta-Mendota Water Authority (SLDMWA) since 1992, which is mainly responsible for operation and maintenance of the canal. Figure 1.8 illustrates the allocation of the DMC.

Figure 1.8 – Map of Delta Mendota Canal; scale 1 : 2 500 000

Description of the research system

The picture below shows the model of the last 75 km of the DMC drawn in a map with the adjoining water districts and irrigation districts. The research is carried out only on the last 75 km as was proposed by ITRC. In the model the upper boundary is located just upstream of check structure 13, in the O'Neill Forebay and at the downstream end of the canal, in the Mendota Pool, the lower boundary is located. The map in figure 1.9 is rotated over approximately 45 degrees counter clockwise in comparison with the map shown in paragraph 1.5.
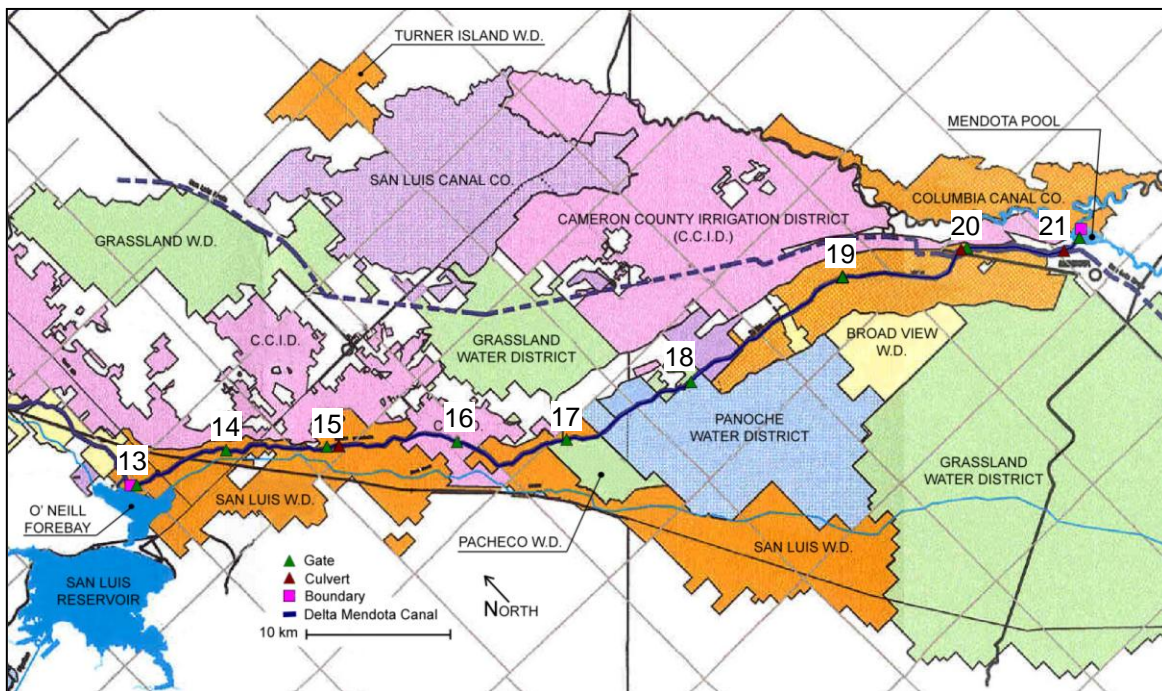
Fig 1.9 – Delta Mendota Canal from O'Neill Forebay to Mendota Pool

Source figure 1.8 and 1.9: sldmwa, 2004

These last 75 km of the DMC are divided by nine check structures, creating eight reaches with an average length of 9.4 km. The first structure (no. 13) is located a little over 90 km downstream of the begin of the DMC and consists of three parallel radial gates and short crested weirs. The radial gates are manually controlled and the weirs (flashboard gates) are manually adjustable. Figure 1.10 shows a photo of check structure 13.

Most of the check structures in the Delta Mendota Canal are composite structures, combining the advantages of short crested weirs and radial gates. With overshot gates the water surplus is immediately discharged over the sill and in case of water deficit the discharge reduces to zero. Considering orifices, the water has to be 'squeezed' through the gate opening, making the undershot gate less sensitive to water level deviations. Hence, the main advantage of weirs (overshot gates) is that they maintain the water level by nature, in contrast to orifices (undershot gates).
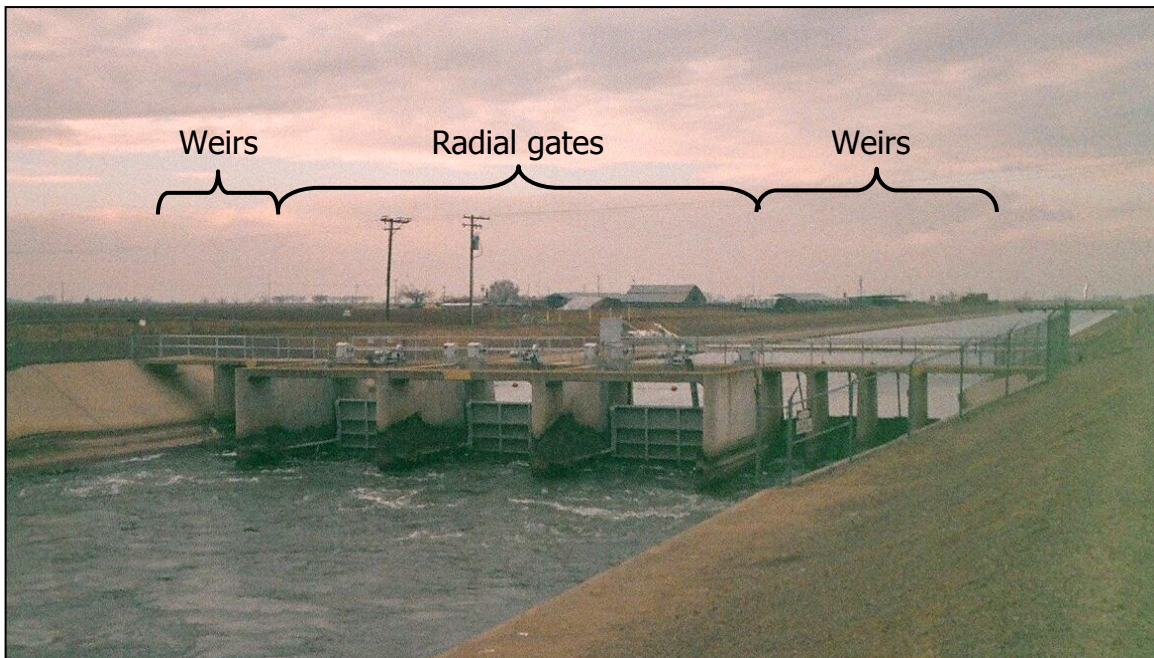
Figure 1.10 – Check structure 13

Besides maintaining the water level it is also important to control the discharge accurately. On this point the undershot gates are better suitable, in view of the fact that the discharge of undershot gates is less sensitive to water level deviations than with overshot gates. The schematization of composite gate no. 13 is presented in figure 1.11.
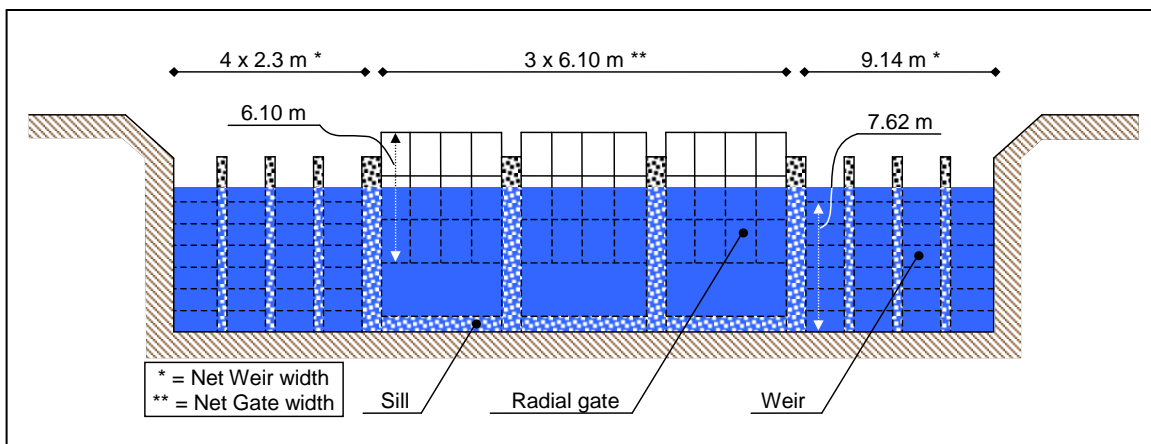
Figure 1.11 – Schematization check structure 13

The photo on the next page (figure 1.12) shows the first flow through the Delta Mendota Canal. This black and white picture gives an indication of the size and the construction of the irrigation canal. The concrete lining is clearly visible.

Figure 1.12 – Historic photo of Delta Mendota Canal

The cross section varies along the length of the canal, according to the necessary capacity. The next schematization (figure 1.13) displays dimensions of the cross section in reach 1.
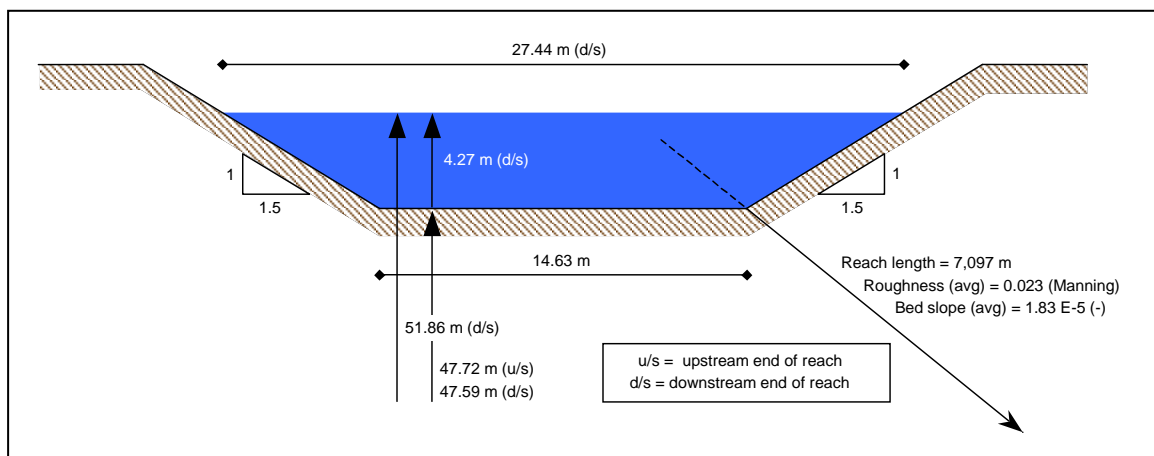
Figure 1.13 – Schematization cross section reach 1

Figure 1.14 shows a longitudinal section of the DMC. The sill heights above Mean Sea Level are presented in rectangular boxes, the relative distances to check structure 13 are printed under the profile and the downstream set points can be found in the profile. The reader is referred to Appendix 1 for further details.
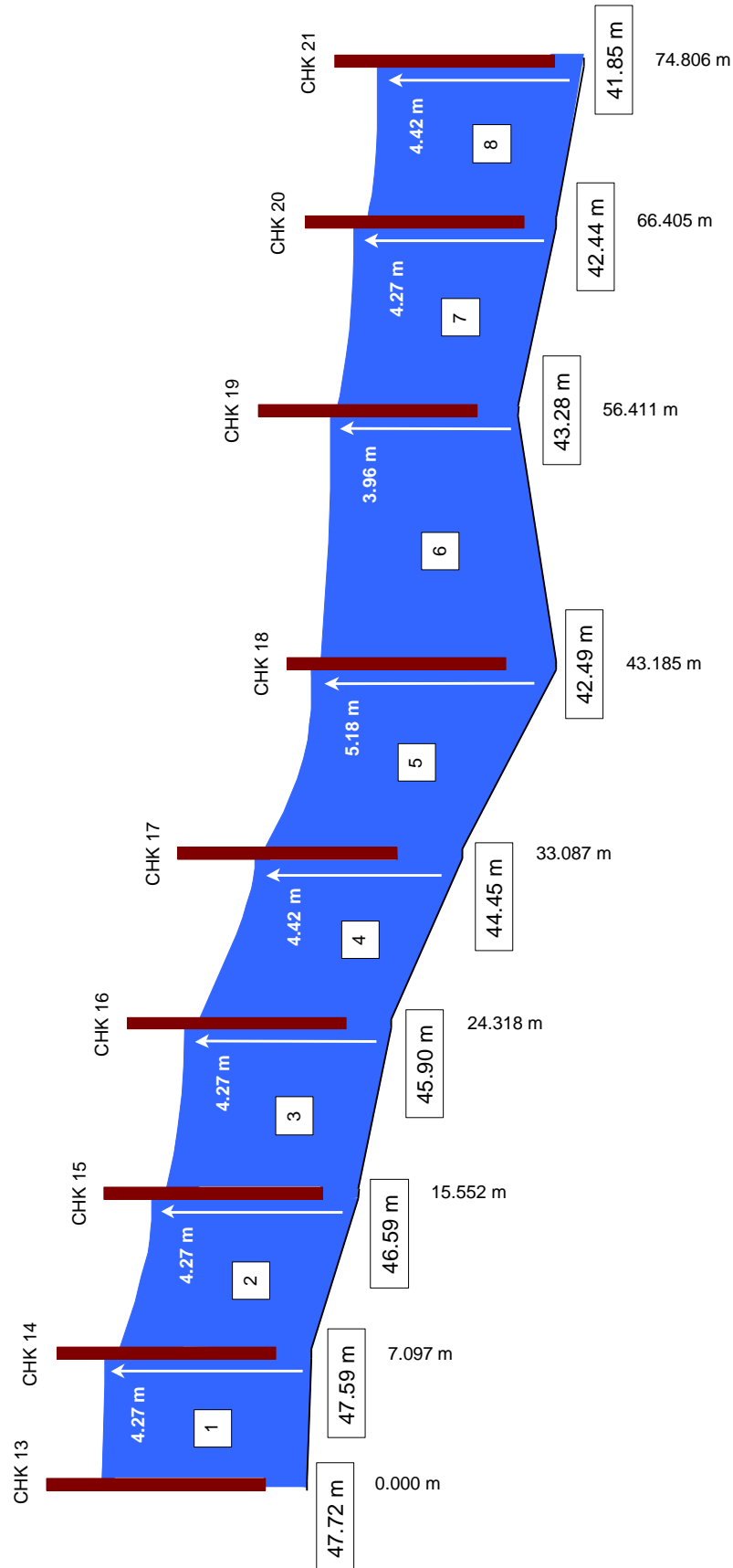
Figure 1.14 – Longitudinal section Delta Mendota Canal

CHK 21 — 41.85 m — 74.806 m
CHK 20 — 42.44 m — 66.405 m
CHK 19 — 43.28 m — 56.411 m
CHK 18 — 42.49 m — 43.185 m
CHK 17 — 44.45 m — 33.087 m
CHK 16 — 45.90 m — 24.318 m
CHK 15 — 46.59 m — 15.552 m
CHK 14 — 47.59 m — 7.097 m
CHK 13 — 47.72 m — 0.000 m

4.42 m — 8
4.27 m — 7
3.96 m — 6
5.18 m — 5
4.42 m — 4
4.27 m — 3
4.27 m — 2
4.27 m — 1

2. Theory on control systems

2.1 Introduction

With the help of various control methods water levels and discharges in irrigation systems, rivers and polders are controlled. Depending on physical circumstances or strategic decisions upstream or downstream control is applied. In some cases an upstream or a downstream feedback control system is not enough to keep the water levels within an acceptable range around set point. Then it is necessary to add extra 'control components' to the control system, such as a feed forward controller or to apply decoupling. The next paragraphs provide the characteristics of the most relevant control methods and control algorithms.

2.2 System Approach

"A system is a collection of equipment and operations, usually with a boundary, communicating with its environment by a set of inputs and outputs." Those inputs and outputs are no material nor energy streams, they are respectively 'things that cause' and 'things that respond'. [MIT, 2003] A cause can be a control action, for instance the closure of a gate or it can be a disturbance in the form of precipitation or turnout.

In the figure (3.1) below the disturbances are added to the scheme that was already presented in paragraph 1.3. Note that the influences of movements of the specific gate are not incorporated in category 'Disturbances' since in this scheme the gate opening is deliberately adjusted. In case another, not by this control scheme controlled, gate would move and it would affect the water process in the scheme, the influence of that gate would be categorized under 'Disturbance'. If a disturbance is known it is possible to make use of that knowledge and to pre-act on that knowledge. This is represented by the feed forward loop in the scheme.
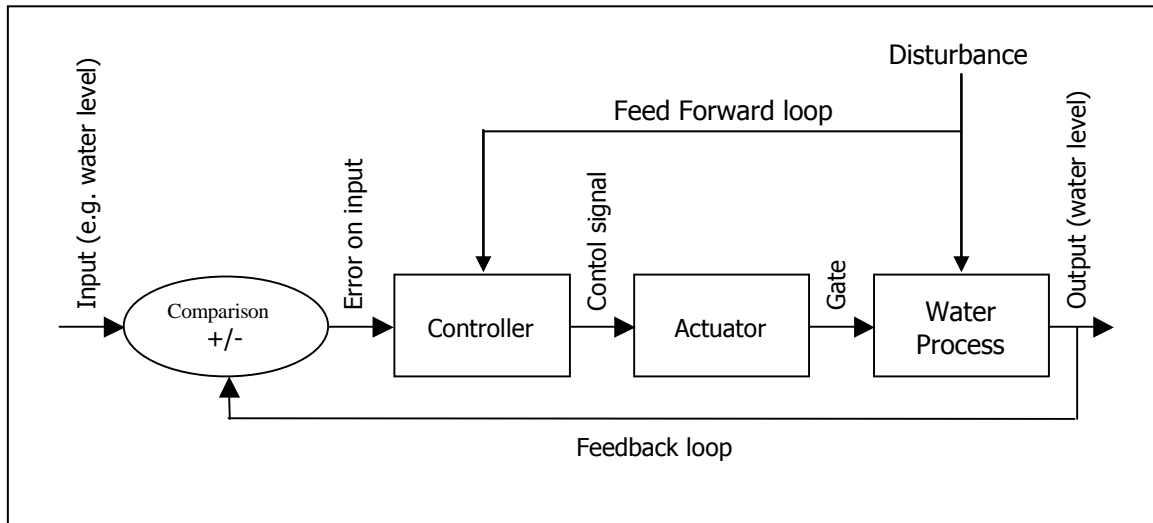
Figure 3.1 – System approach to water control

2.3 Control methods

Introduction

Computer controlled automated systems can be categorized as centralized and decentralized. Decentralized control, or local control, is the type of control where the control actions are based on measurements near the structure. With centralized control, also measurements from other locations can be used for determining the control action. Centralized control can potentially provide higher performance than decentralized control, since more information can be used [Schuurmans, 1999; Wahlin, 2002]. A drawback of centralized control is that it requires more hardware, increasing the chance of system failure. Particularly communication links can easily be damaged, for example by cable cuts or radio interference [Overloop, 2002]. The next sections deal with decentralized control and the simplest forms of centralized control.

Local upstream control

When using local upstream control the water level in the reach is controlled by a structure at the downstream end of that reach. In other words, one can speak of upstream control when the movement of the structure is based on a water level at the upstream side of the gate. [Burt 1987] The term local indicates that the water level to be controlled is near the check structure. Therefore the transducers are drawn in figure 3.2 just upstream of the check structures. Control methods that are not 'local' have their transducers remotely located, or have their measurements gathered to a central (master) controller. The last section of paragraph 3.3 covers the (dis-)advantages of the different control methods.

Due to the location of the structure in the reach, a water deficit in the reach can only be solved by closing the (downstream) structure. This means that the upper reaches have priority – concerning water use – over the downstream reaches and therefore are the users located at the upstream reaches in the advantage. In case of water abundance the water surplus is transported to the lower reach(es) with the possibility to cause inherent problems. To make this control method work the main regulator has to be set, such that the downstream reaches get enough – but not too much – water. The main regulator is the most upstream check structure, where the water is diverted in the irrigation system. This upstream control is usually applied where there is an imposed allocation of the available water or when downstream control cannot be applied.
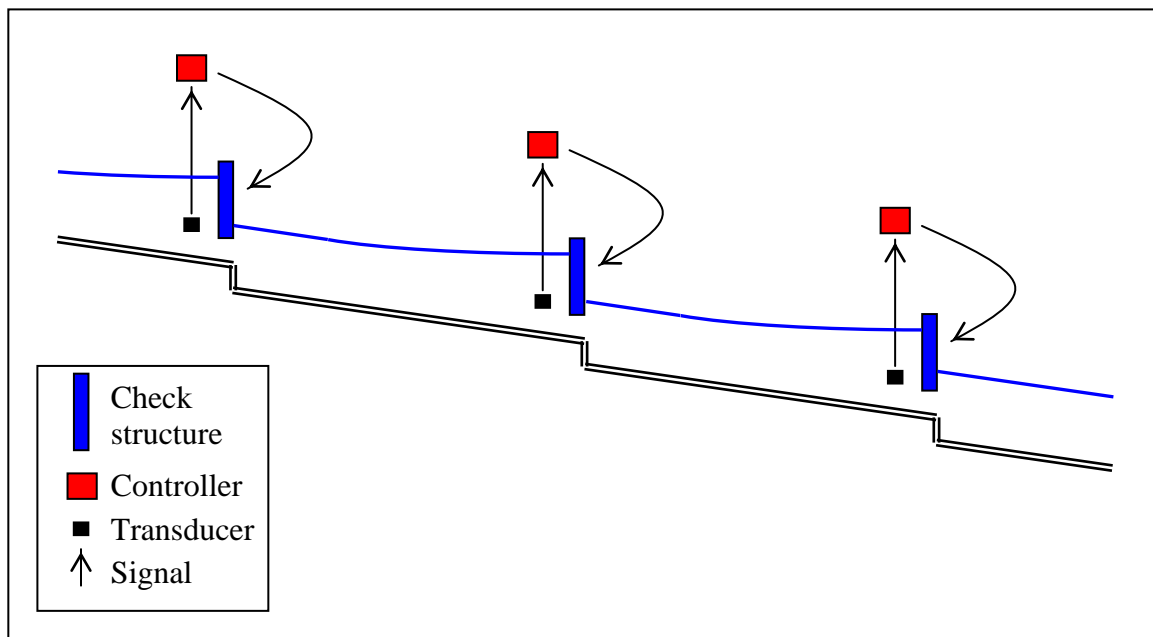
Figure 3.2 – Local upstream control

Remote downstream control

With remote downstream (figure 3.3) control the control structure is located at the upstream end of the reach and the water level measurements are done at the downstream end of the reach. Downstream control can also be applied with water level measurement devices located near the check structure (local downstream control), but it does only work properly when in regular operating circumstances the whole reach is affected by back water. It is therefore only applicable in short or leveled reaches. The same argumentation is applicable for remote upstream control. Above that, measuring at the downstream end of a pool or reach usually provides more adequate control as the turnouts are often located at the downstream end. In general the target levels are set with the intention to allow optimal water extraction by the turnouts, thus maintaining the water levels at that point is in most cases essential.

With remote up- and downstream control the communicating distance can sometimes cause some problems or additional expenses in comparison to local upstream control. Data transmission is particularly in hilly areas or with long reaches (and thus long communication distances) a weak link in the control system.

Water deficit in a reach will open the upstream located check structure. This results in a water level drop in the upstream reach and eventually the main regulator will open. This method is often used when enough water is available at the main regulator and the intake by the users is not restricted. With this control method the water users near the lower reaches do not have to deal with flooding problems as they would have with upstream control. Here the 'downstream' users benefit from their position in the system.
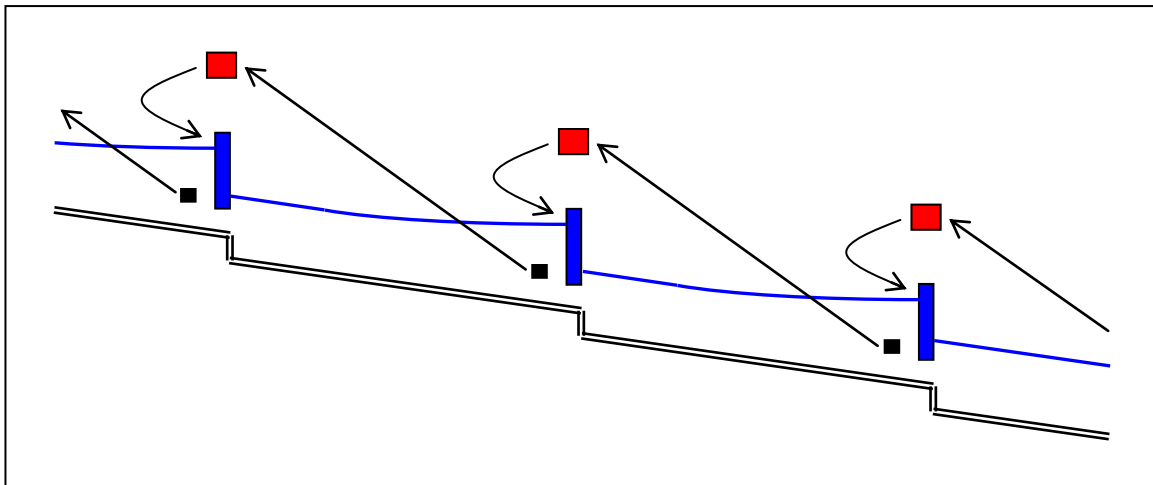
Figure 3.3 – Remote downstream control

Feed Forward and decoupling

Upstream and downstream control can be extended with a decoupler and/or a feed forward controller. Both additions have the objective to send information about disturbances to a controller that is unaware of the coming disturbance. With this information the receiving controller can take an anticipating action to minimize water level deviations.

A decoupler (see fig 3.4) is a direct or indirect data connection between two consecutively controllers in the system. The response to a disturbance of one controller is directly – or with a delay – transmitted to the next, such that the disturbance does not lead to severe water level deviations in the intermediary reach, as the receiving controller had the advantage to anticipate.

Something similar is a feed forward controller (see fig 3.4). A feed forward controller is an extra input in the controller and provides information about expected disturbances, such as scheduled off takes or expected inflow due to rainfall. The main purpose of a feed forward controller is to anticipate on those influences by making corrective control actions beforehand [Astrom and Wittenmark 1997]. Since the influences are not yet in the system the moment the controller (pre-)acts on those influences, the control actions cannot be calculated by using the water level error. A feed forward controller is therefore an open-loop controller and requires good knowledge of the processes or process model [Astrom and Wittenmark, 1997]. By anticipating on the expected disturbances the water level fluctuations will be attenuated and the flow will stabilize more quickly. [Clemmens et al. 1998a]

Figure 3.4 – Feed forward control (1) and a decoupler (2)

Advantages and disadvantages of the control methods considered

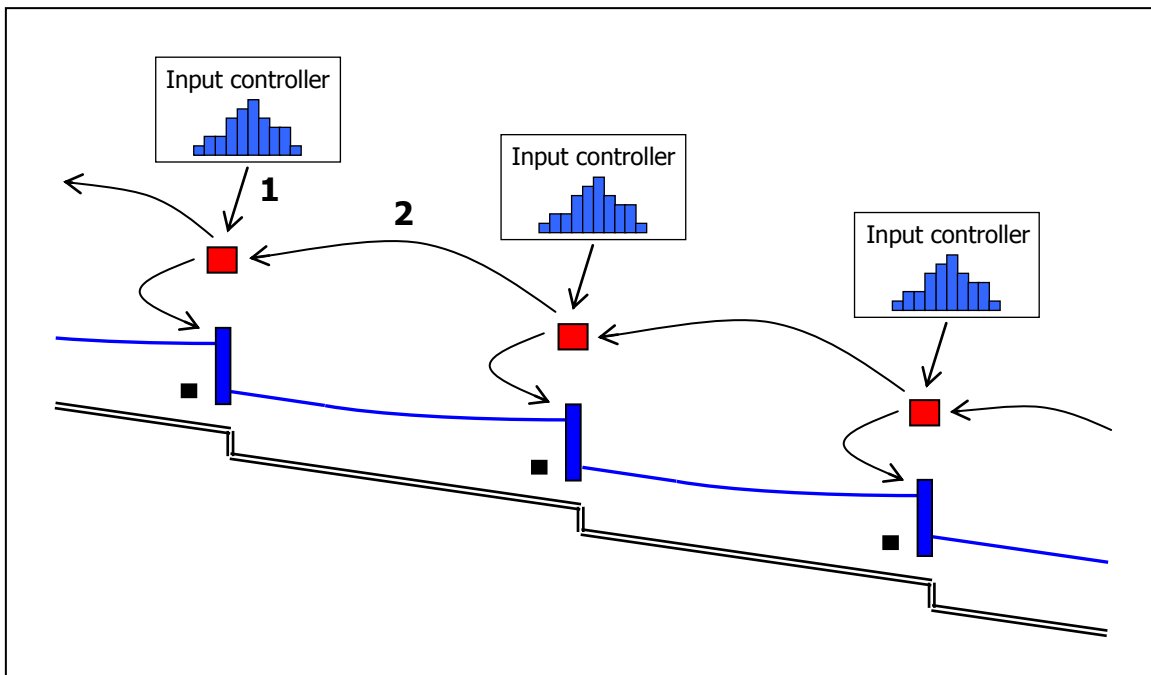Table 3.1 shows the differences between four control methods that can be applied on the Delta Mendota Canal. It does not provide a complete list of all positive and negative aspects of the control methods, however with this quick scan the reader can get (more) acquainted with these control methods. The decoupler and the feed forward controller are additions to a control system and are not control systems by themselves. They are in the table to illustrate their impact when added to one of the four control systems. The rating below is displayed with the symbols --, -, o, +, ++ and *n/a*; this is respectively from poor to good and 'not applicable'. The three categories that are mentioned in paragraph 1.4 can be found in the table. The categories 'performance' and 'costs' are added to complete the quick scan.

Table 3.1 – Quick scan of the control methods.

	Local upstream control	Remote upstream control	Local downstream control	Remote downstream control	Decoupler	Feed forward
Equity control system						
E1 In case of water deficit in the system	-	--	+	++	++	*n/a*
E2 In case of water surplus in the system	+	o	o	+	++	*n/a*
Flexibility control system						
F1 Suitability for 'on-demand delivery'	-	--	+	++	++	++
F2 Suitability for 'imposed allocation'	++	+	-	--	*n/a*	*n/a*
Reliability control system						
R1 Supplied quantities according to demands	-	--	+	++	+	++
Performance control system						
P1 Measurement location near turnout	++	--	--	++	*n/a*	*n/a*
P2 Possibility to control the volume in the reach	*n/a*	*n/a*	*n/a*	*n/a*	++	+
Costs control system						
C1 Construction, maintenance and operational	++	--	++	--	--	--
C2 Embankment requirements	++	--	--	++	++	++

The following will comment and explicate the ratings given above.

E1_1   Upstream control is rated worse than downstream control, since the supplied quantity cannot be adjusted automatically.

E1_2   Remote upstream and local downstream control use measurements at the upstream end of a reach and 'notice' water level deviations later than with measurements at the downstream end. Generally speaking the results of these control methods are less accurate. When controlling an irrigation system that is not always and not totally under backwater conditions, these methods are not recommended.

E1_3   A decoupler speeds up the recovery of the system, as it transmits the local control measures to the controller upstream.

E2_1   Both upstream and downstream control can deal with too much water in the system. The main difference is that with upstream control the surpluses are spilled, where with downstream control the supplied quantity is decreased.

E2_2   Comment E1_2 and E1_3 are also applicable.

F1_1   With a feed forward controller known off takes can be scheduled and can be anticipated on. Feed forward yields therefore better performance. (F1_1)

F1_2   Comment E1_1, E1_2 and E1_3 are also applicable.

F2_1   In case of imposed allocation the quantities of water to be used per district (i.e. per reach) are pre-calculated. Transporting the water trough the system for the downstream users is therefore more important than adjusting the supplied quantity. Thus upstream control provides the best performance.

F2_2   Comment E1_2 is also applicable.

R1_1   When the estimations of the amounts of water to be supplied are accurate, the delivery by an upstream control system can be satisfying. In practice, these estimations are practically never 100% accurate. [Burt, 2004] The timing of the supply is also a weak link. In a upstream system the choice has to be made between spillage and deficits.

R1_2   Comment E1_2 is also applicable.

P1_1   Only when measuring near the turnout in the reach the leading parameter regarding the supply is measured. Thus measurements at this location in the reach yields the best performance.

P2_1   A decoupler controls the volume in a reach, as it transmits the local control measures to the controller upstream. A the discharge imposed by a feed forward controller is based on a schedule and makes no use of actual measurements. The signal of the decoupler is based on actual measurements and is therefore more accurate.

C1_1   The costs of remote control systems are higher, due long distance communication.

C2_1   Reaches fill up at the downstream end, since that is where the water level is the lowest. When controlling the downstream water level the requirements for embankments are equal or lower than when controlling the water level at the upstream end.

2.4 Feedback control algorithm(s)

Introduction to the PID algorithm

Whatever the nature of the process, there will be some desired point or range of operation and some difference from that desired level. Controllers try to minimize these errors using measurements of the current situation (feedback) and sometimes knowledge about the future (feed forward). Concerning the feedback component there are three, generally accepted, different ways of responding to disturbances. The first is to respond proportionally to the error (P), the second is to respond proportional to the speed with which the error changes (D) and the last is to incorporate a time integral such that the response can be made proportional to the integral of offset (in centimeters) over time (I).

These three responses can be realized with a PID (proportional-integral-derivative) control algorithm. When controlling water levels or flow the derivative part is often not used as the advantages are small and the extra problems regarding stability are big. [Lecture notes TUD, operational water management, 2001] The control algorithm of a PID controller is given by the transfer function:

$$Q_{desired}(t) = K_P \cdot e(t) + K_I \cdot \int e(t) \cdot dt + K_D \cdot \frac{de(t)}{dt} \qquad (3.1)$$

in which:

      U: the control action to get the water level back to set point (often: discharge)
      K_p: the proportional control constant
      K_i: the integral control constant
 K_d: the derivative control constant
      e(t): the time dependent error on the controlled variable; e.g. the water level

The figures in section 'Assessment of the proportional, integral and derivative part' show the response to a disturbance signal. With these responses the influences of the three different components are evaluated.

Definition of terms describing response behavior

In the next section certain terms are used to describe the behavior of the controllers. With figure 3.5 these terms are clarified. Figure 3.5 shows a response of an arbitrarily (feedback-) controlled system to a step in turnout flow (inset graph).
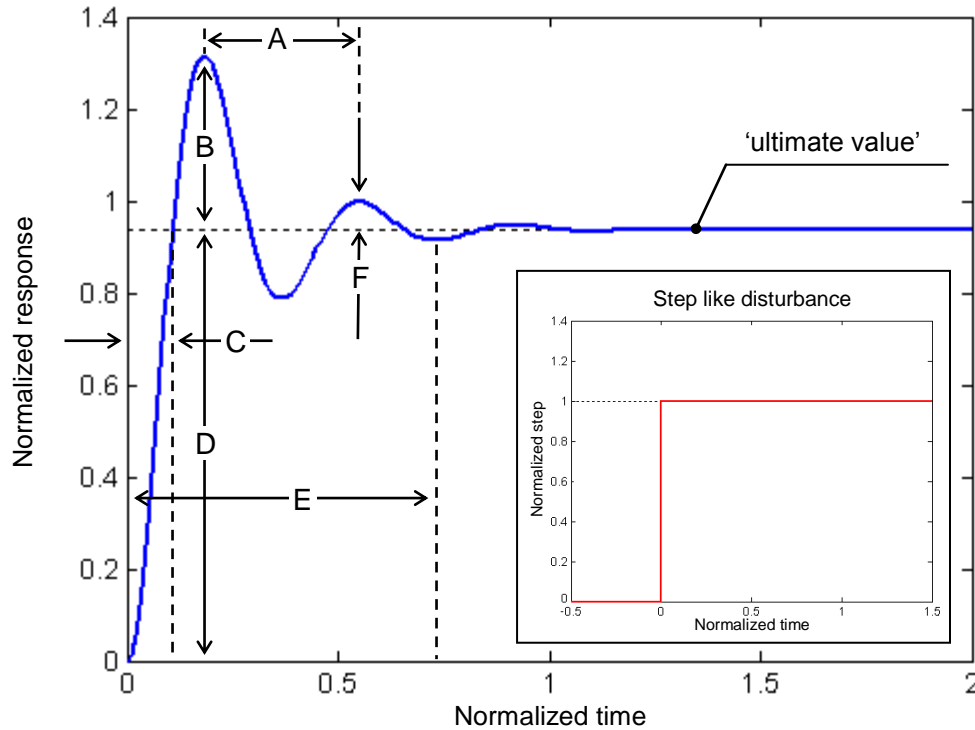
Figure 3.5 – Response behavior [Coughanowr and Koppel, 1965]

The letter A in the figure points out the time interval between two consecutive peaks and is equal to the time interval between alternate crossings of the ultimate value. This time interval is defined as the *period.* In this case the response is subject to damping. Damping influences the amplitude of the oscillations and lengthens the period. In a system without damping the period is called *natural period.* A step disturbance will cause an undamped system to oscillate perpetually about its ultimate value.

The *overshoot,* represented in the figure with arrow 'B', is the magnitude with which the response exceeds the ultimate value. In order to have a better insight in the response behavior the overshoot is often expressed as the ratio B/D or as a percentage of the *ultimate (or final) value* (D).

The time interval, assigned by arrow 'C', is defined as *the rise time.* It is the time needed for the response to reach the ultimate value for the first time, regardless of the further behavior. In case of an over-damped system (without oscillations), the rise time equals the response time.

The time until the response remains within a 5% margin of the ultimate value is called the *response time* or the *settling time*. It is an indicator for the stability of the controlled system. The response time is indicated in the figure with arrow 'E'.

The *decay ratio* is the ratio of the amplitudes of successive peaks above the ultimate value. In the figure above the *decay ratio*, also known as the *damping ratio,* equals the quotient F/B.

## Assessment of the proportional, integral and derivative part

Proportional control

        First a proportional controller is applied. By increasing the gain (i.e. the K_p value, also known as 'the proportional control constant') the system responds stronger to water level deviations. If this gain factor is chosen too big the system will oscillate, if it is chosen too small the water levels will be off set point. In figure 3.6 a proportional controller is subjected to a step like disturbance. The formula of a proportional controller is given by:

$$Q_{P\_desired}(t) = K_P \cdot e(t)$$

        One can see that the proportional controller has a steady state error; it does not make the controlled variable reach the target level. In the presented example the target level has the value 1. Lets assume an upstream controlled water system consisting of one reach enclosed by two gates of which the downstream one with a proportional controller. With a constant base flow (Q_{Base}) both gates have a non-zero opening height (O_{Base}). When a disturbance enters the system (precipitation, etc.) the water level rises in that reach. The downstream gate responds proportional to the water level deviation, but this extra opening height might not be enough to stop the water level rise. One control time step later the water level has risen further, leading to an extra opening of the gate. Lets now assume that this extra gate opening stops the water level rise and that the inflow equals the outflow exactly. The next control time step the error remains the same and thus will the gate opening not be altered. The result is a steady state with an offset from target level. This offset is named the steady state error (see figure 3.6). The value of the steady state error depends on the K_p value and the disturbance.

        When the disturbance has passed the flow through the gates equal Q_{Base} again. The proportional controller brings the controlled variable (the water level) back to target level (not in figure).
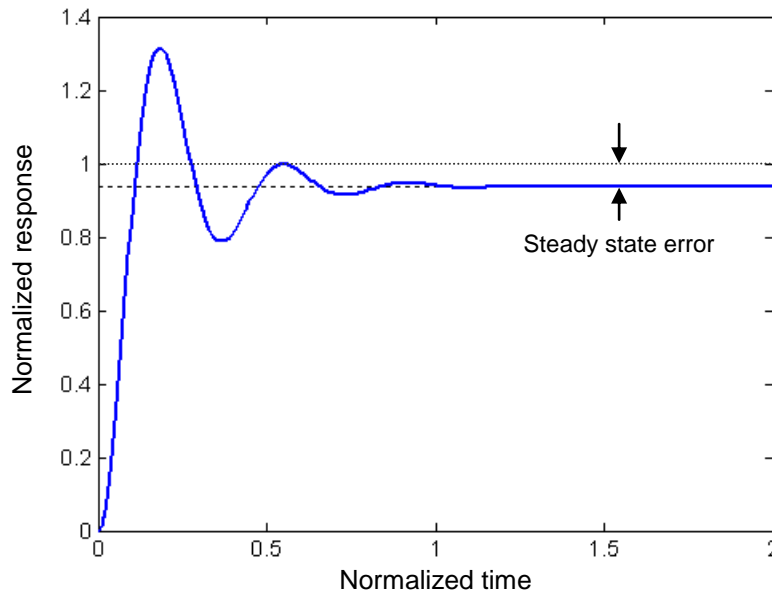
Figure 3.6 – Response of a proportional controller

Proportional-Integral control

In the next picture (3.7) the response is drawn of a PI controller to a step disturbance. The main task of the integrator is to eliminate the steady state error and in some cases it can make the response to the disturbance stronger and it can shorten the rise time.

The integral action is determined by the integral of the error over time; in practice the time interval (dt) is equal to the calculation time step. The formula of the integral part is:

$$Q_{I\_desired}(t) = K_I \cdot \int e(t) \cdot dt$$

This integral eliminates the steady state error of the proportional controller since the output of the integral increases (absolutely seen) during the time the water level is off set point. The graph in the figure below shows the response to the same step disturbance as used previously.

The biggest disadvantages of the added integral part are that the integral action makes the overshoot bigger and that it can cause instability. Two specific problems occurring with controllers incorporating an integral action are wind up and disturbance amplification, they are described later in this paragraph.

The main goal of the integral part can be described as follows: the objective of the integral part is to realize the final step to the target level and not to enforce big discharges; the proportional part will do that. When regarding block like disturbances (consecutively step 'up' and step 'back') a proportional controller can suffice, without introducing problems as disturbance amplification or wind up. However, long lasting deviations from set point, as a result of the steady state error may lead to unacceptable situations
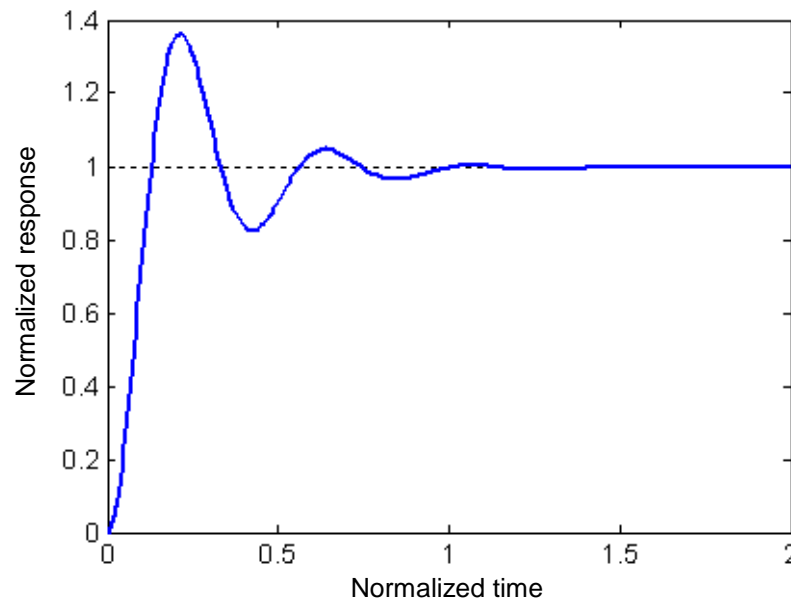
Figure 3.7 – Response of a proportional integral controller

Proportional-Integral-Derivative control

In very few cases a derivative action, also know as pre-act, is used in the controller. As mentioned previously the derivative action depends on the speed with which the error changes. Therefore problems are especially expected with changing set points, fluctuations around set point and with high frequent disturbances, since the derivative action then approaches infinity. The next formula represents the pre-act.

$$Q_{D\_desired}(t) = K_D \cdot \frac{de(t)}{dt}$$

In practice, the majority of control systems are time discrete, thus infinite values of the derivative action are not likely to occur. However, filtering the influences of step like disturbances and high frequent oscillations of the measured variable is often necessary. When assuming that all disturbances, and therefore the errors, are sinusoidal oscillations:

$e(t) = A \cdot \sin(\omega \cdot t)$

the derivative action then is:

$K_D \cdot de/dt = \omega \cdot K_D \cdot A \cdot \cos(\omega \cdot t)$

Hereby it is clear that the amplitude of the derivative action is determined by the frequency of the disturbance. Filtering of disturbances for tuning the derivative action is outside the scope of this thesis and will not be regarded any further.

When the mentioned problems can be avoided the derivative action has a positive influence to the response. Using the pre-act the strength of the response increases and the response time decreases. With figure 3.8 is tried to illustrate a possible response behavior.
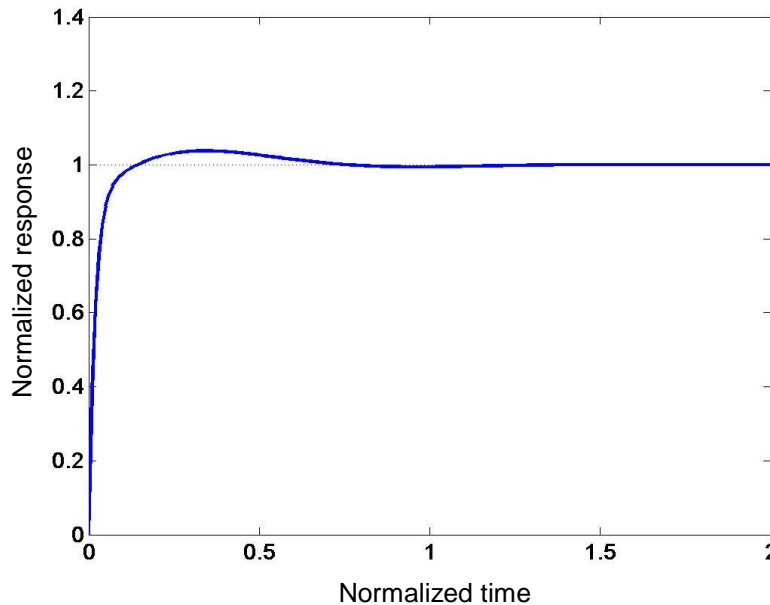
Figure 3.8 – Illustration of the response of a Proportional Integral Derivative controller

Summarizing response analysis

The differences of the three controllers is shown by plotting their responses to a step like disturbance, see figure 3.9.
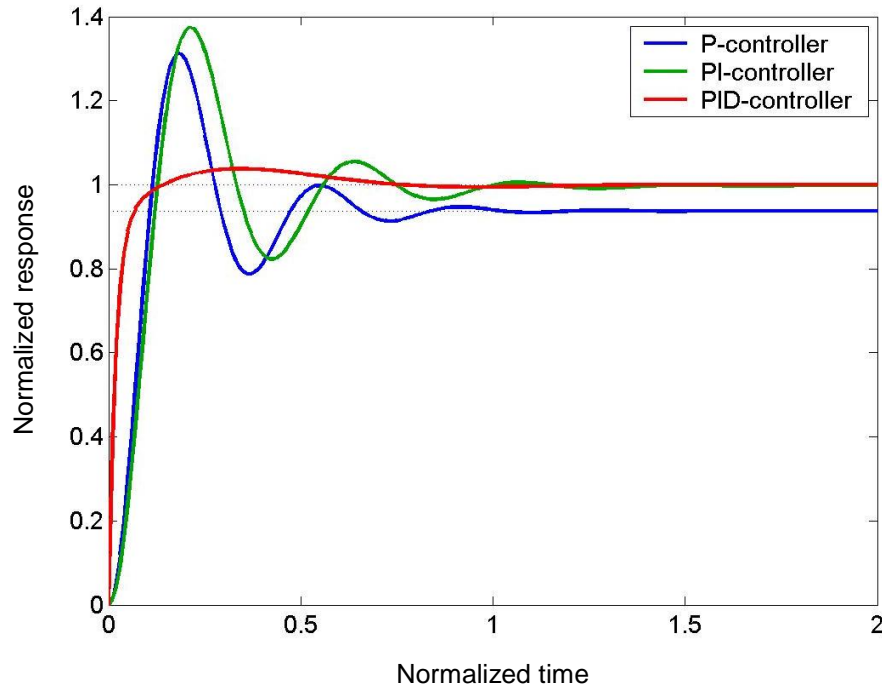
Figure 3.9 – Response comparison of the three presented controllers.

The next table (3.2) summarizes the effects of the three components, as they can be noticed in the figure above.

Table 3.2 – Influences of the proportional, integral and derivative part

Closed loop response	Rise time	Overshoot	Settling time	Steady state error
K_p	Decrease	Increase	Increase / Decrease	Decrease
K_i	Decrease	Increase	Increase	Eliminate
K_d	Decrease!	Decrease	Decrease	No Influence

Proportional-Integral controller with a Filter (PIF)

Resonance

Not only with the derivative action a filter is sometimes necessary, also with proportional and proportional-integral based controllers filtering of the measurements can be required. In leveled reaches where the bed slope is small, waves can easily travel up and down in the reaches [Schuurmans 1997]. When a control system reacts on those waves, the system can get into resonance behavior. Properly defined: resonance can occur when there is a phase-shift of about 180 degrees (π radians) between the measured water level and the gate movement and where the gate influences the water level at the measurement location. A phase-shift of 180 degrees means, that when the water level is *relatively* high at the measurement location, the gate is (partially) closed as if the water level were *relatively* low. Figure 3.10 shows the phase shift for three different situations.
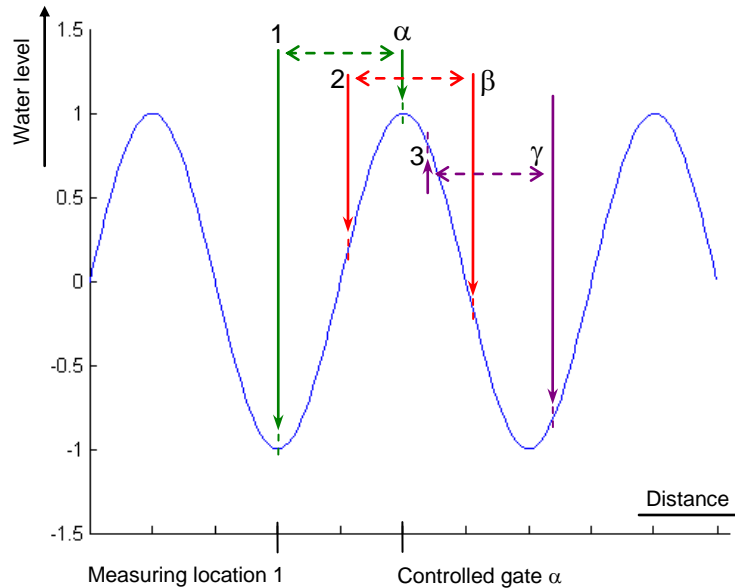
Figure 3.10 – Phase shift criterion for resonance

Assume that a resonance wave in the canal reach can be represented by a sinusoidal shape. In the figure the blue line represents the water level in the reach for a specific moment. At that moment measurement 1 (see figure) is done and the water level at the gate is represented by α. This means that the control system reacts as if the water level is relatively low although – near the gate – it is relatively high. The phase-shifts 2 - β and 3 - γ demonstrate the phase-shift for two other moments in a reach. The sensitivity to resonance waves is a property of a reach, but the occurrence I caused by external influences. Those influences, such as turn outs, precipitation, wind and other disturbances, induce waves in the reach that are by nature transformed into waves that travel through the reach with (a multiple of) the ground resonance frequency.

Figure 3.11 demonstrates the preceding for a remote upstream controlled system. The top figure shows the water level is above set point due to a disturbance. Therefore the pressure transducer signals the controller to open the gate. Half a period later the water level is below set point, resulting in a 'close' signal to the controller.
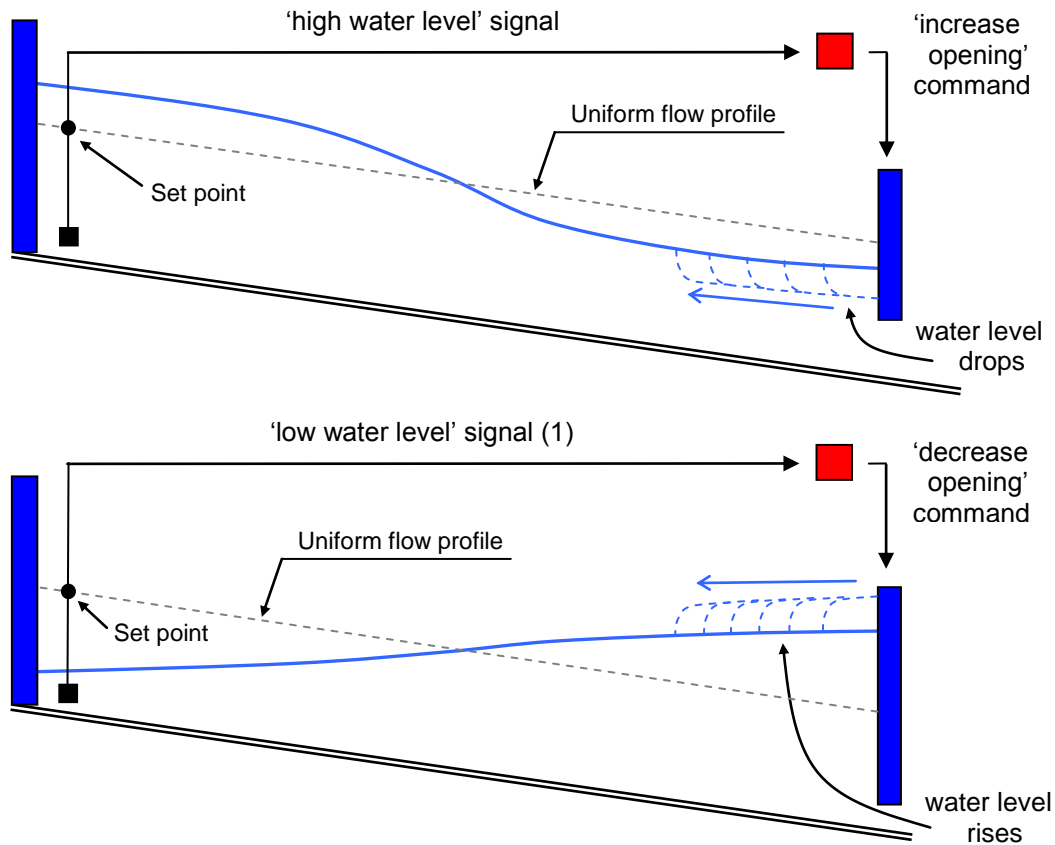
Figure 3.11 – Resonance behavior

The control system reacts on the resonance wave with a wave (the control wave) of the same frequency. Amplification occurs when the control wave exceeds the damping influence of friction on both the resonance wave and the control wave.

The frequency with which the reach starts to oscillate depends on the wave speed, the length of the reach and the measuring location. In figure 3.12 below, the reach oscillates with three times the ground resonance frequency.
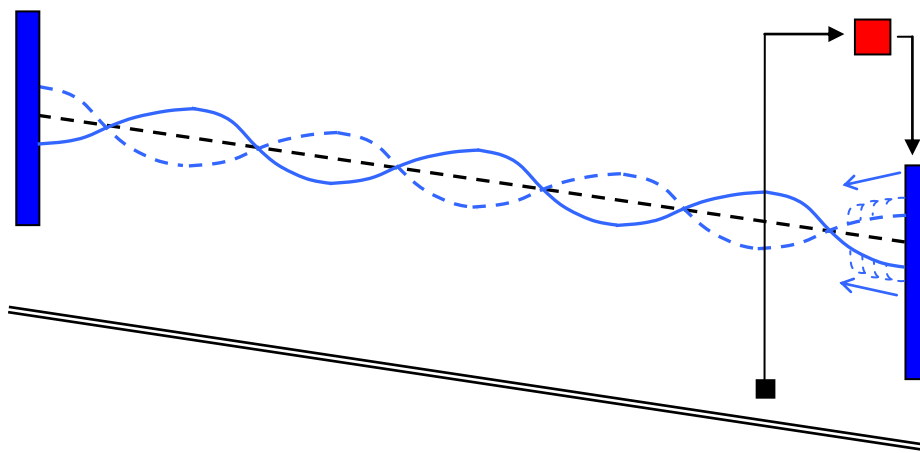
Figure 3.12 – Higher resonance frequency due to local control

Filtering

Because of the sensitivity to resonance waves, controllers of structures in short or leveled reaches have limited strength to prevent instability. Weaker controllers result in performance loss and are therefore not desired. After determining on what frequencies the system gets into resonance behavior, it is possible to reduce the influences of waves with those frequencies by applying a first order filter.

The formula for a filter that is applied on the error is:

$$E_{filtered,\ new} = F_{coefficient} \cdot E_{filtered,\ previous} + (1 - F_{coefficient}) \cdot E_{measured} \text{ use continuous filter}$$

$$E_{f,new} = $$

In which:

$E_{filtered,\ new}$: the filtered error of the current time step
$E_{filtered,\ previous}$: the filtered error of the previous time step
$E_{measured}$: the unfiltered measured error of the current time step
$F_{coefficient}$: the filter coefficient

When applying a low filter coefficient the highest frequencies are filtered out and with a high filter coefficient also the lower frequencies are filtered out. Depending on the measuring location a high or low filter coefficient is desired. The filter coefficients and the gain factors are determined with a Multiple Model Optimization Algorithm [Schuurmans and Overloop 2004]. The gain factors can be increased when applying a filter, resulting in a faster response of the control system. Figure 3.13 shows the response of a control system with a filter and without a filter to an oscillating disturbance [Schuurmans 1997]. The frequency of the oscillations is chosen such that there is a phase difference of 180 degrees between the water level at the measuring location and near the structure. The figure shows between 0 and 500 seconds that the PIF controller allows those oscillations, but does not respond to them. If the controller would respond to the oscillations the system would become unstable. The PI control constants are significantly weaker, so that the response to the oscillations does not make the control system unstable. The consequence is that the performance decreases, as figure 3.13 clearly shows.
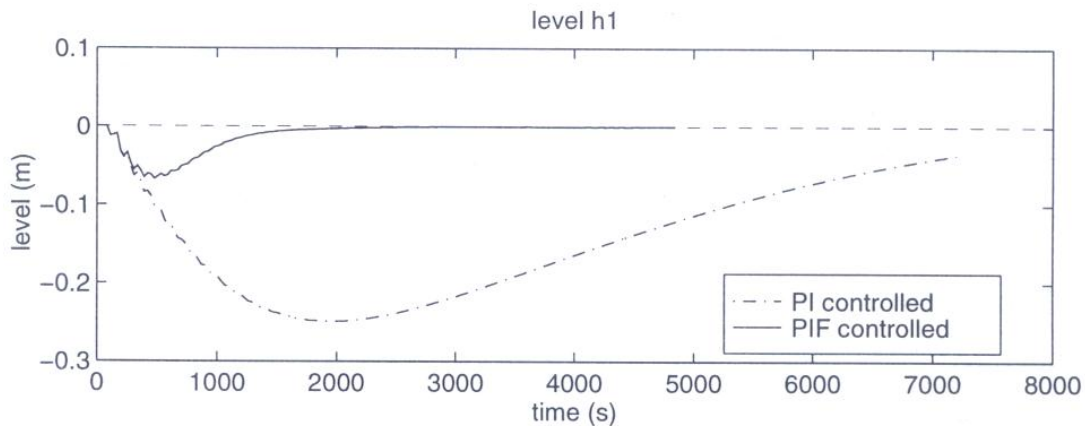
Figure 3.13 – Influence of a filter

Wind up

Wind up, also known as "integrator wind up", can cause long lasting and large deviations from set point. Wind up is caused by saturation of the actuator. In practice, saturation of the actuator means that the actuator cannot meet the control output (actuator input).

Integrator wind up can occur with (time-discrete) controllers that have integral action. The integral part of the controller can become very big when the water level to be controlled is out of the control range of the actuator.

Wind up can take place when e.g. the water level in an upstream controlled reach is too low, causing the downstream gate to close and the upstream gate is also closed due to water deficit in the upstream reach. Then the inflow is zero and the outflow is also zero, with the water level below set point. In this situation the integral part will continue to increase, until the inflow in the reach makes the water level go above set point. At that moment the integral can have become very big and can cause significant and long lasting water levels above set point. Figure 3.14. shows the water system on set point at $t=t_0$. Between $t=t_1$ and $t=t_2$ both the upstream and downstream gate close due to water deficits, leaving the reach below set point. At $t=t_2$ the upstream gate opens since the upstream water deficit is no longer and the reach starts to fill toward set point. When at $t=t_3$ the water level is back on set point the integral part is still big and thus enforcing wind up. After $t=t_4$ the system stabilizes and reaches steady state at $t=t_5$. This problem is called wind up. In the worst case wind up can cause the system never to reach set point.

To prevent wind up, the operating range of control elements should be limited to the range of the devices they are driving. Basically, this solution comes down to stop updating the integral when the actuator is out of its control range. This provides instant recovery when the control error changes sign. Reducing the gains can also prevent wind up in some cases, but it will result in a reduced speed of the response to the disturbance. The application of the last strategy is therefore not desirable.
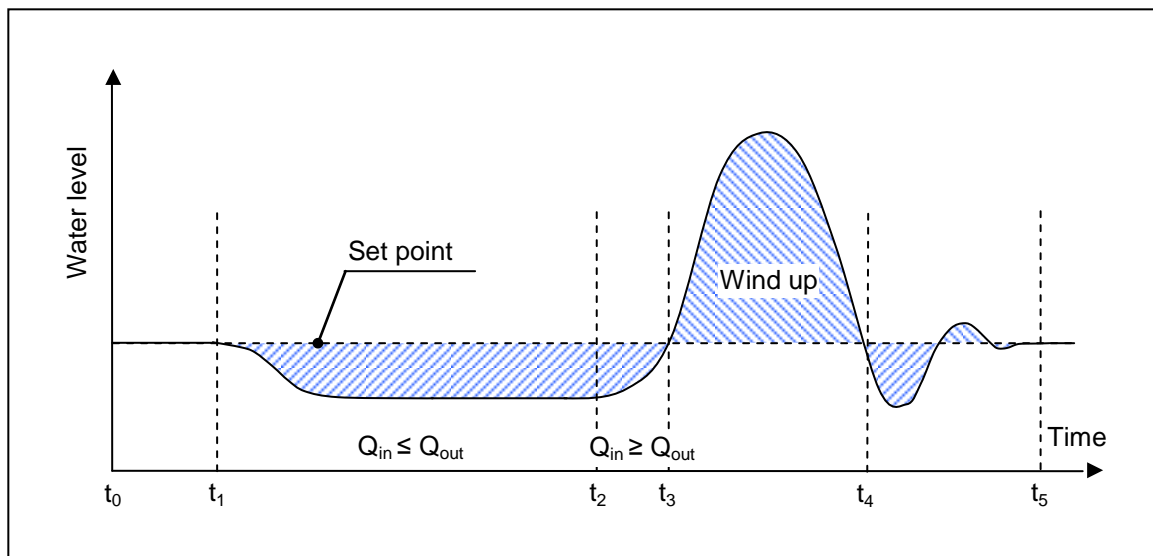

Figure 3.14 − Wind up phenomena

Disturbance amplification

When a disturbance on the water level is amplified due to the interaction of a controller in, for instance, an irrigation system one can speak of disturbance amplification. The scientific criterion for disturbance amplification is as follows. "When there is a pole at zero in the Nyquist diagram that was made with the transfer function of the controller the appearance of disturbance amplification can be predicted." [K.J. Åström and B. Wittenmark, 19xx] For more information about Nyquist diagrams the reader is referred to *Computer Controlled Systems* by K.J. Åström and B. Wittenmark.

Disturbance amplification is caused by the fact that the discharge through a gate is not exactly and not simultaneously reproducible at the next gate. The next gate can either be the upstream gate or the downstream gate, depending whether the system is upstream or downstream controlled. In general, disturbance amplification is the result of not being able to maintain the volume water exactly constant in a reach. The controller can also be tuned such that no overshoot takes place, however then the performance will degrade. The next scenario illustrates disturbance amplification.

When considering an upstream controlled system the disturbance is amplified in downstream direction, as the upstream water level is controlled. See paragraph 1.4 for 'direction of the disturbance'. Let there be a disturbance (D), for example due to rain, on the upper reach (R1), see figure 3.15. Gate 1 (G1), located at the downstream end of this reach, starts to react to this disturbance when at the measuring location in reach 1 the water level is no longer on target level. Assume that the discharge can be characterized as Q1 in the figure.
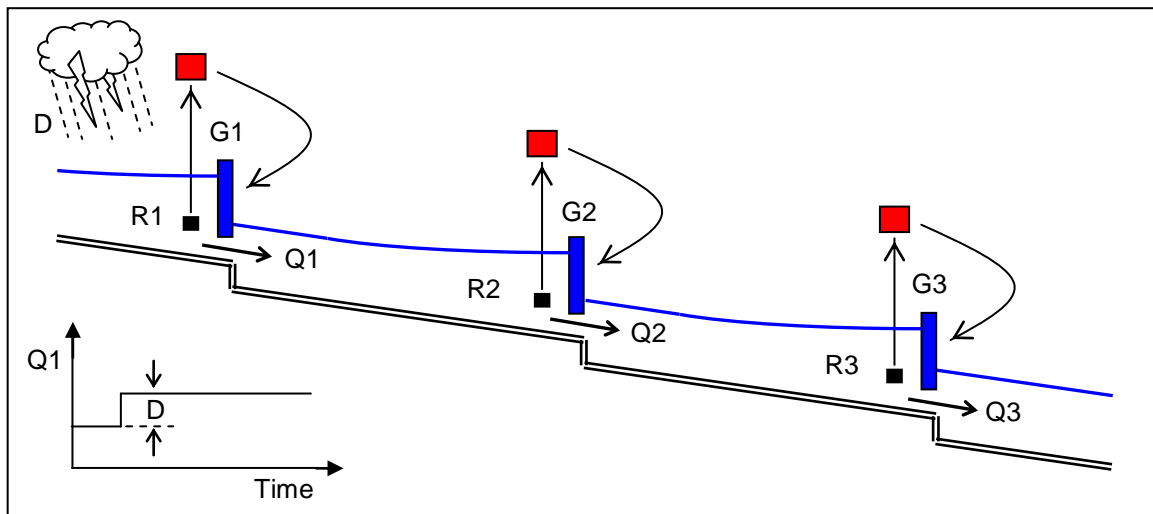
Figure 3.15 – Disturbance amplification scenario

Let's say that the reach is very short and that delay times (lag times) can be neglected. Although the reach is very short the step wave that is released at G1 will always show deformation over the length of the reach, see figure 3.16.
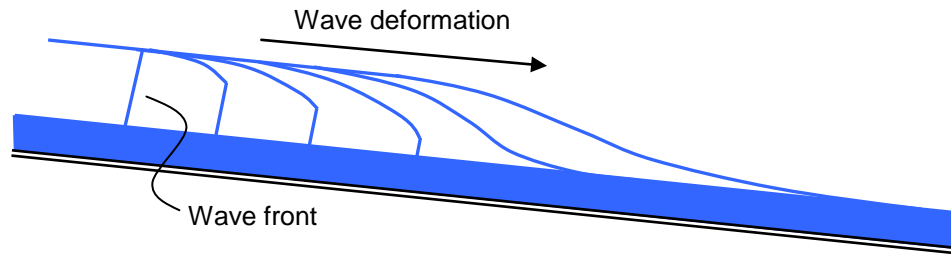
Figure 3.16 – Wave deformation

Adapted from : Schuurmans, 1997

The wave deformation prevents gate 2 (G2), which is also local upstream controlled, to produce the same step like change in discharge as G1. Therefore, on the moment assigned in figure 3.17 with "X", when the discharge through G2 equals the discharge through G1, the integral part is not zero. During the *rise time* (see figure 3.17) the integral part has grown, since the change in volume in the reach caused an error on the water level. The output of the integral part can cause an additional increase of discharge with the size of the *overshoot*. The next gate (G3) will respond to a further deformed wave with a larger overshoot. This is one cause of disturbance amplification.
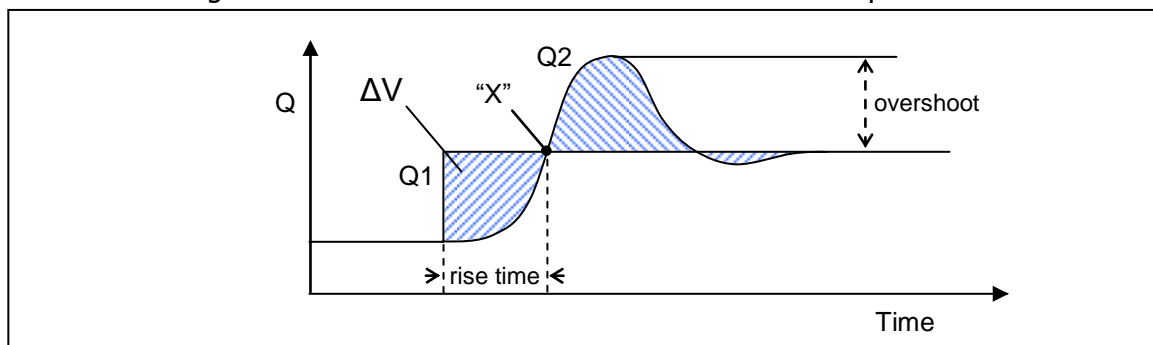
Figure 3.17 – Response Gate 2 to step disturbance; no delay time & imperfect response

Now assume that it is possible to reproduce the step like change in discharge, but that the reach is longer and that therefore the delay time is significant. Figure 3.18 shows the increase in volume (ΔV) in the reach that causes the overshoot. Note that in this case the reach itself behaves as the integral part.
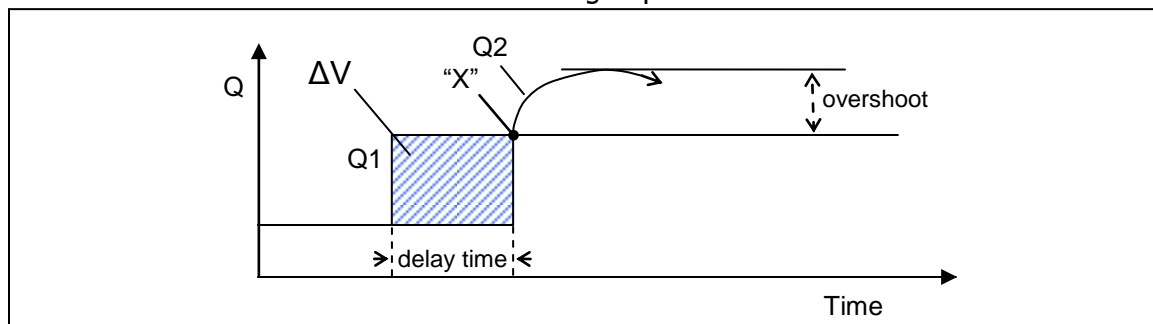
Figure 3.17 – Response second gate to step disturbance; no rise time & perfect response

The increase in volume can result, as in the previous case, in overshoot. The figure is not realistic and has only an indicative value.

2.5 Control system to be implemented

Currently, the control on the water level in the Delta Mendota Canal is done manually. Different operators drive along the canal and adjust the check structures based on their calculations and experience. Because of the advantages automatic control has over manual control, ITRC conducts an investigation how to automate the control. The control system that ITRC designed for the last 75 km is local upstream control. The feedback actions are based on a proportional- integral controller with a first order filter.

The control constants and the filter coefficients are tuned to give an as strong as possible response to the disturbance without becoming unstable. The tuning was done with Multiple Model Optimization [Overloop et. al., 2004]. Using a set of linear models that are based on the discretisation of the Saint Venant equations (shallow water equations), the next cost function is herein minimized:

$$J = \sum_{i=1}^{N} \lambda_i J_i \tag{3.2}$$

where λ_i represents a weighing factor for the i-th linearized model, where N equals the total number of models and J_i is the objective function for the i-th model. Each linear model gives an approximation of the Saint Venant equations for a certain flow range so that the controllers function in all flow conditions without becoming unstable. Normally the number of models is two; one model for low flow circumstances and one for high flow circumstances. The objective function J_i is defined as:

$$J_i = \sum_{k=0}^{\infty} \left[e_{(k)}^T Q e_{(k)} + u_{(k)}^T R u_{(k)} \right] \tag{3.3}$$

The objective function incorporates a water level deviation vector e and a control input vector u. The influences of the two components on the objective function are weighed with separate weighing factors, respectively Q and R. Both e and u vectors are represented twice in the fomula such that product increases quadratic with the increase of e and u.

According to simulations with CanalCAD and Sobek this control system provides satisfying results during regular operating conditions. Figure 3.19 illustrates the control system for the Delta Mendota Canal.
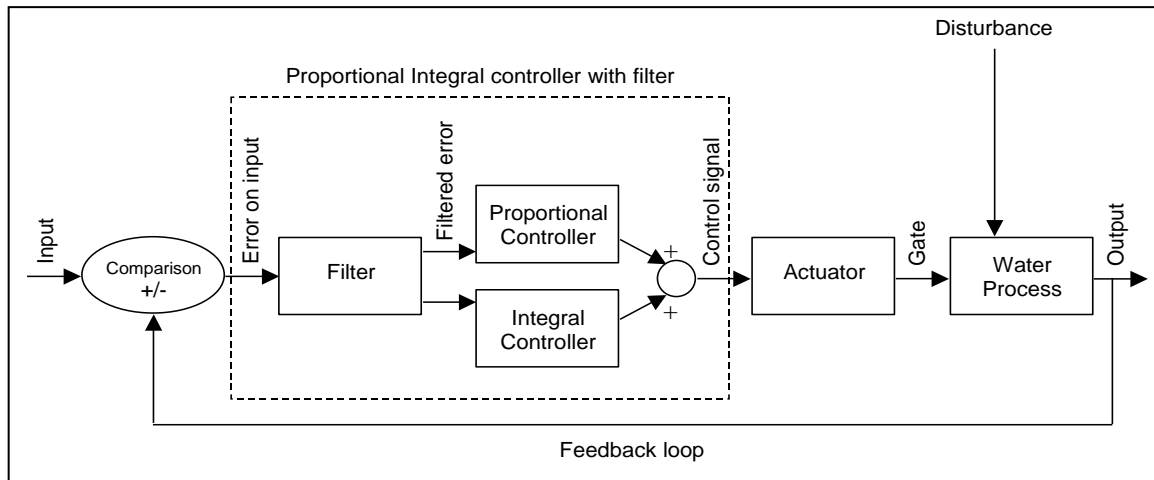

Figure 3.19 – Control system design Delta Mendota Canal

3. Problem Analysis

3.1 Introduction to the problem

Irrigation is a major item in large parts of the world, as farmers cannot always depend on rainfall to accommodate their demands. So, canals have to transport water from large reservoirs or rivers to agricultural land. However, without controlling the water movement the water supply is not efficient.

The efficiency is determined by the amount of water that was actually delivered, relative to the amount requested or intended to be delivered. Therefore, the efficiency depends on the flexibility of the system, for example: if no variation can be made in the flow, waste of water or water shortage is unavoidable, or if no use can be made of the buffering capacity, flooding may occur after heavy rainfall. Flexibility can be achieved through control. This can be done manually, but automatic control has the preference. Automatic control is desired mainly for two reasons. First, manual operation of the actuators in the field is an unattractive job, sometimes the operation has to be carried out in the desert or in dry plains. It can also be an impossible task to operate the structures, when after heavy rainfall the unpaved roads on the dikes are inaccessible. Second, due to optimizing the growth of the crops, the farmers have increasing requirements on the flexibility of the delivery of the water. Which means: frequently varying water demands and unscheduled water extraction.

Several automatic control methods have been developed, such as upstream and downstream control, which can be triggered by a local or remote sensor or can be linked via a data connection. Data communication over longer distances, necessary for remotely triggered or centralized systems, can be of importance for the performance of the system. The advantage of systems with data communication over longer distances is the ability to anticipate on known disturbances, such as precipitation and scheduled water extraction. However, control systems that are dependent on long distance communication are vulnerable, as this is one of the weaker links in the control system. Moreover, data communication over long distances is expensive. Communication over short distances usually causes no problems or high costs. Because of the risks and the costs, local controllers sometimes have the preference, even though the performance is generally lower.

3.2. Problem description

Problem 1 – Matching supply and demand

The Delta Mendota Canal delivers water to a variety of destinations, such as agriculture, municipal and industrial uses and to wildlife refuges. In combination with the sheer size of the delivery area and the high demands per destination – concerning flexibility in water delivery – a 100% accurate calculation of the total needed amount of water is practically unfeasible. Above that, natural influences and inaccuracies within the system make the delivery computations even less precise.

Paragraphs 1.4 and 1.5 indicate the existence of reservoirs in Californian irrigation systems and particularly in the Delta Mendota Canal. The reservoir is an essential part in an irrigation system, however the construction costs are a serious drawback, so it is desired to minimize the size of the reservoir. The bigger the differences are between supply and demand, the bigger the reservoir needs to be and the more expensive the construction of the irrigation system is.

The first question is how to prevent differences between supply and demand and thereby minimizing the time in which the reservoir water level is off set point. Paragraph 1.4 stated that the accuracy of the supplied quantities cannot be improved, hence the solution can only be provided at the side of the demand. Based on this assumption the next possibilities exist:

1) Minimizing differences between supply and demand can be done by having the irrigation districts extract strictly the pre-demanded amount of water (and have the irrigation districts themselves provide the necessary flexibility).
2) The market mechanism can also provide in a solution; in case of water shortage the price of water rises, resulting in a smaller demand.

The next enumeration provides commentary to the (partial) solutions above.

ad 1) Imposing limitations to the users is a non-desirable, though very effective, solution. As mentioned in paragraph 1.4 the basic goal of ITRC regarding canal automation in irrigation systems is 'providing service'. Imposing limitations in any way is not consistent with providing service and can therefore not be part of a solution.

ad 2) Applying the market mechanism is a proven method to balance supply and demand, though it is not desirable in this case. The mentioned suggestion is not in line with the main goal of ITRC, see ad 1.

It appears that within the allowed solution area there is no answer to the question how to prevent differences between supply and demand. This is an unfortunate conclusion, but it does not mean that there is nothing left to do. If water level deviations are inevitable, controlling these deviations (think of: duration and size) is the next best answer. Problem 2 is about controlling duration and size of water level deviations in the reservoir.

Problem 2 – Minimizing differences supply and demand

At ɪᴛʀᴄ research on the possibility of applying automated local upstream control is carried out on the Delta-Mendota Canal. In the CanalCAD model [ɪᴛʀᴄ, 2004] and in the Sobek model [Bruggers, 2004] of this 'on-demand' system the local upstream[4] triggered controllers are able to keep changes in water level small when there is a supply surplus. In practice this is always the case, since the supplied water is partially destined for the downstream controlled part (see figure 1.7). However, the upstream controlled part does not and cannot respond to demands exceeding the supply in the downstream controlled part, which leads to water level deviations in the reservoir. The basic problem with upstream control is that the main regulator is not water level controlled, or at least: the control is not based on a water level in the (irrigation) system.

Without changing the current control method five options are available (see paragraph 2.3) that can offer a solution when the water demands do not match the water supply. The five options are based on circumstances with water deficit in the system; an analogue reasoning can be applied for water abundance.

a) The first is to increase the flow at the source (at the main regulator) and to wait until the water arrives and solves the water shortage.

b) The second solution is to build a bigger reservoir and to make use of this extra buffer so that the demands are always met for both the upstream controlled part and the downstream controlled part.

c) The third is to (partially) open the gate directly upstream of the pool where the water deficit exists and let the water from that pool compensate for the shortage.

d) The fourth is to (partially) close the gate directly downstream of the pool where the water deficit exists and let the water that was meant for the reaches downstream compensate for the shortage.

e) The fifth is to impose limitations to the irrigation district concerning water extraction.

The question remains which of the proposed options is the best, regarding equity, flexibility, reliability, performance and (energy-) costs. The list of options is reduced using the next argumentation.

ad a) In the current situation the tactic described under "a)" is followed in case of water shortage in one ore more reaches. This tactic is not an improvement to the system and is therefore called the 'Base Case'.

ad b) This studies is about finding a method so that the reservoirs can be built smaller. Building reservoirs bigger is therefore not desirable and is excluded for being a solution to the problem.

ad c) Opening the upstream gate is only an option if the reach with the water deficit has a higher priority than the upstream reach. In combination with opening all gates upstream this might be an option, since the water deficit is transported in upstream direction – towards the main regulator – and is therefore temporary present in each (upstream) reach.

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[4] The reader is referred to paragraph 3.3 for an analysis of the different control methods.

ad d) Opening the downstream gate is only an option if the reach with the water deficit has a higher priority than that downstream reach. With this strategy the water deficit is transported in downstream direction and will eventually lead to water shortage in the most downstream reach. The extra water that is discharged at the main regulator needs to travel a longer distance, which means that the time until all reaches are on set point again is postponed compared with the Base Case.

ad e) Here, the same argumentation is applicable as in the evaluation of the solutions in 'Problem 1', see ad 1).

Based on the argumentation above, the research area is narrowed to the way in which the upstream gates have to be opened, in order to optimize the duration and size of the water deficits.

3.3 Boundaries to the solution area

ITRC has set boundaries to prevent solutions being developed that are not feasible to implement in practice. It is very tempting to design a state of the art control system with many measuring points and continuous data communication in all directions (as Model Predictive Control), with a probably higher performance, but the chance that such a control system will be implemented is negligible. Besides the lack of experience with those systems and the undoubtedly high operational and maintenance costs, it is usually wise to try to find a solution that is as simple as possible. Therefore the next requirements apply.

1   Maintain local upstream control
2   Maintain the present measuring points
3   Restrict data communication to a minimum

In practice, these requirements to the solution area impose mean in practice that:

ad 1   the solution has to be designed in such a way that it can be seen as an additional component to the controller and that it operates more or less independent of the present control system.

ad 2   only measuring points upstream of each check structure will be used.

ad 3   the communication frequency is limited as if the data input is done manually. This research keeps a maximum frequency of once every 5 minutes.

3.4 Problem definition and research goal

The problem with the water deficits using the current control method is that the system cannot respond fast enough to downstream water demands. With a faster response i.e. getting the water faster to a downstream reach or reservoir, the size and duration of the water level deviations can be affected positively.

The present challenge is to develop a control strategy, such that the performance improves in the situation mentioned in paragraph 2.2. To ensure minimal decrease of the robustness, the strategy must have the characteristics of an additional, but non-essential component to the existing local control system.

The solution is sought in a feed forward controller. A feed forward controller can improve the handling of sudden large changes. With a feed forward controller the moment of delivery of the required additional water is no longer determined by the delay time of the whole canal, but is limited to the delay time of one reach (at the most). A feed forward controller makes it possible to pass water in all reaches at the same time. With a failing feed forward controller the performance of the system will not decrease to an unacceptable level under 'normal' circumstances, since the feedback controller will always persist in maintaining the water level.

Research Goal

"Research on the possibility to speed up water delivery in an existing local feed back controlled system, by application of a feed forward controller."

3.5 Research Approach

The research is carried out in four stages. In the first stage the Delta Mendota Canal is modeled in Sobek. The programming of the currently applied control method in Matlab forms stage two. In stage three the feed forward controller is designed, programmed and evaluated. The last stage comprises the written and oral report of this research.

Stage 1 – Modeling Delta Mendota Canal

General

The model is based on data of the last 75 km of the Delta Mendota Canal in the San Joaquin Valley in California. The system data is provided by ITRC in the form of CanalCAD output and complimentary documents.

Modeling the canal

The eight pools, of which the research canal consists, are separated by automatically controlled radial gates combined with fixed short crested weirs. In the model the boundary reservoirs will be modeled with fixed water levels. This is acceptable assumption since the reservoirs are relatively large. Since the short crested weirs cannot be controlled – the crest level can only be adjusted manually by removing one or more flashboards – they will not be modeled. By ignoring the weirs the control methods do not present better results than in reality. In fact, by ignoring the weirs controlling the water level is more difficult and will lead to a more conservative estimation of the performance.

The water levels directly upstream and downstream of each check structure are exported to Matlab to calculate the flows through the structures and to determine the water level errors. Cross sections, bed and surface levels and roughnesses will be defined on every upstream and downstream end of each reach, or reach division in case culverts are present.

Turnout (-schedules)

Based on the CanalCAD output the lateral flows will be allocated in the model. The lateral flows represent turnouts, the locations where the users or irrigation districts withdraw their water. The lateral nodes will be instructed to extract water following the schedules as provided in the CanalCAD output or in complimentary documents. The number of turnouts in Sobek may differ from reality, in case they are not significant. This study aims to provide a solution to a problem based on reality, but is not meant to model the exact conditions. By neglecting a few minor influences the model stays clear and the results remain transparent.

Stage 2 – 'Determining solution area'

Solution area

　　　　The investigated solution area has to be in line with the demands of ITRC and has to be applicable with regard to physical restrictions and control theory.

Stage 3 – 'Programming current control method in Matlab'

Data connection

　　　　In Sobek data exchange with Matlab is set up in the settings section. Matlab is used to implement control strategies and to calculate the control actions. The combination with Matlab creates the possibility to apply advanced control techniques and to switch easily between the different control strategies. Another advantage of Matlab is that the presented output can (and must) be programmed manually, providing the highest flexibility. Besides the settings in Sobek two files in the work directory have to be adapted: sbk_loc.rtc and sbk_meas.rtc. In these files the locations and variables are defined that have to be exported to Matlab (sbk_loc.rtc) and the structures to be controlled (sbk_meas.rtc).

Calculation and control settings

　　　　The calculation grid is set to 100 meters and the time step in computation is set to 1 minute to obtain accurate results in combination with short calculation time. The control frequency is set to 5 minutes accordingly to paragraph 2.3.

Programming the control method

　　　　The control files in Matlab can be assigned to five different categories: 1. Import variables from Sobek, 2. Initialize internal variables for usage, 3. Compute the control actions, 4. Plot the desired variables and 5. Export the control actions.

　　　　The files in category 1 and 5 connect Sobek and Matlab, the category 4-files are mostly useful for interpreting the results, for debugging and for creating figures. The main goal in this phase is to create files in category 3, the control files with the control method incorporated.

Validation

　　　　Validation takes place using the CanalCAD output [ITRC, 2004] after building the model, including the lateral flows and applying the current control strategy. When giving similar results in several simulations as the CanalCAD output the model is approved. By finishing the preceding the 'Base Case' with local upstream control is set up.

Stage 4 – 'Programming Feed Forward controller'

Modeling

First a model for the feed forward controller, based on the Integrator Delay Model [Schuurmans, 1997], will be developed. Using this model the feed forward controller will be designed and programmed in Matlab. This feed forward controller opens the check structures sequentially in order to prevent large water level fluctuations and to speed up the water delivery (no reaction of the feedback controller should theoretically be necessary). Two necessary input parameters are required for the Matlab model. The first is the storage area per pool and the other is the lag time per pool. These two parameters vary with the flow conditions in the pools, however, for this model the assumption is made that these are constants. The values of the constants are provided by ITRC and are verified using the Sobek model.

Secondly, to minimize the delay time (lag time) for water to get to a downstream location, a second feed forward controller will be designed. The difference with the first feed forward controller is that this controller does not open the gates one at a time, but all at once. When the need for water exists in a reach, a signal has to be sent to the main regulator (most upstream check structure) containing information on the desired additional flow. From this point the feed forward controller will control the downstream gates up to the reach or reservoir with the water deficit.

Points of interest

The robustness of the feed forward controller is verified with different flow variations at different flow regimes. The robustness of the whole control system is verified after disabling the feed forward controller. When the model is approved, the results will be evaluated and compared to the reference case.

4. Controller design

4.1 Introduction

This chapter consists of different cases with different control strategies, followed by a further analysis of the most suitable control strategy. The test results show the response and the control actions of all the gates after a step-like increase of the main regulator (at t = 01h00). A base flow of 40 m³/s and a step of 10 m³/s is chosen, so that the results are not affected by physical restrictions of the canal and the structures. In this way the simulation results are not specific for this system. Each test case strategy is evaluated individually on three criteria: on change in discharge, on water level deviation from set point and on the variation in crest opening height. In the general analysis the strategies are also compared based on the water level deviations that they cause.

The criteria are presented as graphs with eight lines, representing the discharges through the nine check structures, the water level fluctuations upstream of each structure and the openings of the eight check structures.

The discharge is a criterion since the goal of this research is to increase the flow into the reservoir as quickly as possible. The water level deviation is an indicator for the performance of the control system, and therefore an important criterion. The efficiency or energy use can be evaluated with the movements of the crest. It is also an indicator for the wear and tear of the moving parts of the structure.

Three assumptions concerning the criteria are made.

- To make the water level criterion reasonable, a dead band of 4 centimeters is applied. This means that the water level may fluctuate between 2 cm above and 2 cm below set point, without any negative judgment. The 2 cm is chosen to be the dead band since in practice waves with an amplitude of 2 cm are easily caused by wind or birds in the water.
- Water levels above set point are considered as undesired as water levels below set point. Out of a water delivery thought water levels below set point are undesired and out of a safety point of view water levels above set point are undesirable.
- The opening of a check structure is regarded equally as the closure over the same distance. The speed with which the gates open does not affect the evaluation.

4.2 Controller design and system response

Base case – current control system design

Applied strategy

This case shows the response of the current control method to the step disturbance of 10 m^3/s. Local upstream control is applied with a flow controlled main regulator.

Graphs

The discharge graph (see figure 4.1) shows one step like change in discharge and eight response graphs. The first check structure is the main regulator and is flow controlled, since it has to discharge (supply) a specific amount of water into the canal. The main purpose of a flow controller is – evidently – to maintain a certain flow and not to maintain a certain water level. The shape of the discharge graph is therefore almost 'step like' where the other eight are 'response like' as illustrated in paragraph 3.4.

The same graph also shows a nice example of limited disturbance amplification. The disturbance, here an increase in flow of 10 m^3/s at the main regulator, is amplified with approximately 160% leading to a maximum discharge at check 18 of 16 m^3/s.

The two horizontal dotted lines at +/- 0.02 m represent the dead band in the middle graph. It is a visual aid to remember that waves of this size are not significant. It is clearly visible that with this large disturbance (magnitude is 25%, from 40 m^3/s to 50 m^3/s) the water level deviations are kept within 6 centimeters.

The graph at the bottom displays the nine gate openings from one hour before and 24 after the step in discharge at the main regulator. The different gate openings are caused by differences in flows to be discharged and by other head differences since the checks are identical.

Evaluation

The current control system satisfies in maintaining the water levels in all the eight reaches. However, as can be seen this method is not suited for rapid water transport through the canals. When there is a need for water in the downstream reservoir (see figure 1.7) the operator will increase the opening height of the main regulator. This is simulated in the top figure on the left page at time equals 1 hour. The current control method takes more than three hours after adjusting the main regulator to create an extra inflow in the reservoir. The causes can be found in the canal itself e.g. delay times and the delaying effect of the check structures, and in the control system: local upstream control can only respond to local disturbances and cannot anticipate on remote disturbances.

The question is how to improve the existing control system such that the performance improves, without altering the control method (see paragraph 3.3). It would not be difficult to change the control method to remote upstream control – with the advantage that it would be possible to anticipate on disturbances coming from upstream direction – however remote upstream control has many negative aspects, see table 2.1. Above that, the restrictions to the solutions do not allow other measuring points than the existing. The next case covers the usage of a feed forward controller as suggested in chapter 2.
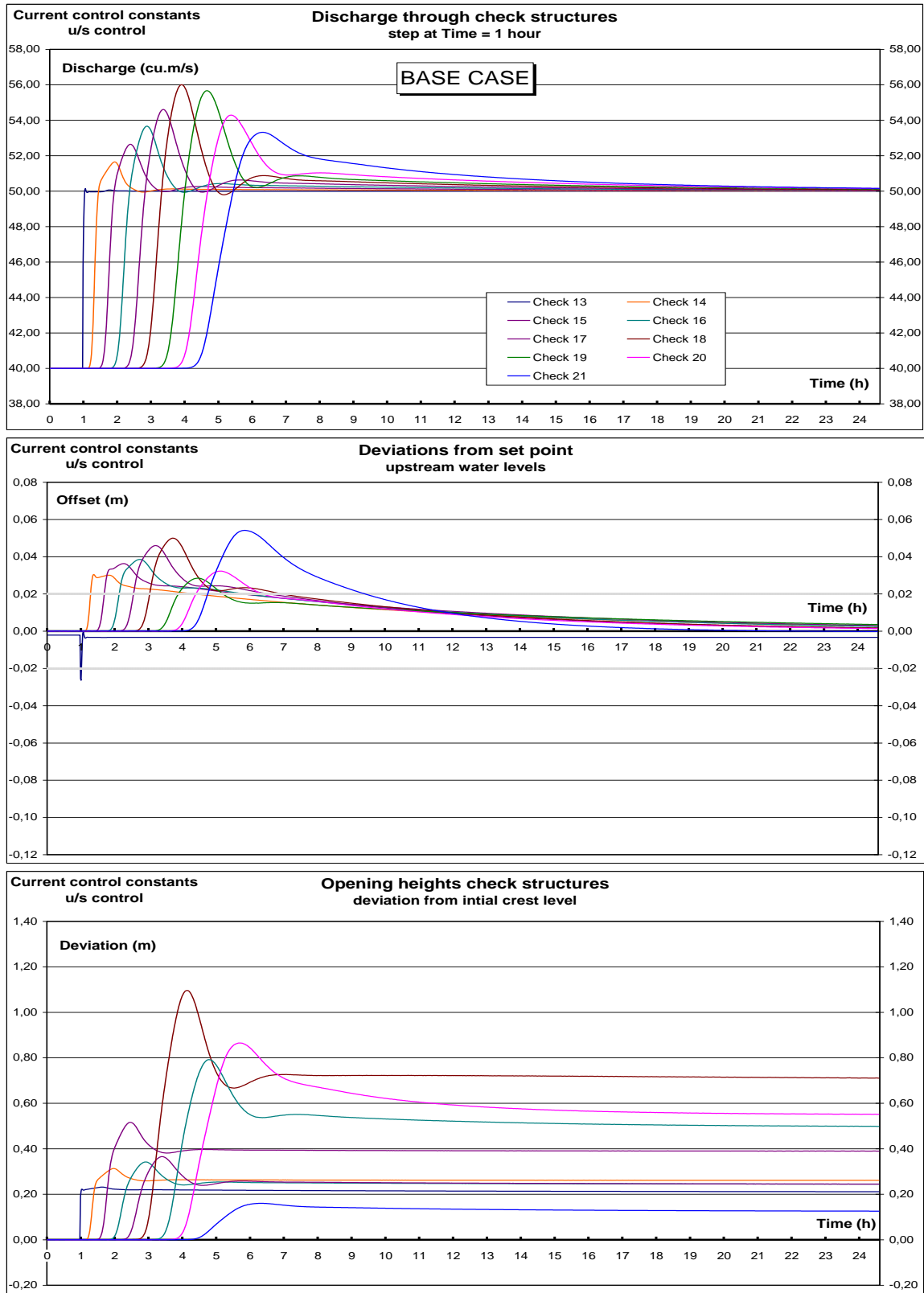
Figure 4.1 a/b/c – Discharge, water level and crest opening Base Case

Case 2: 'Current constants and Feed Forward'

Applied strategy

In addition to the current control method a feed forward controller is incorporated in the control system. The control action of a feed forward controller is based on knowledge or predictions about disturbances, which are not yet in the system. Usually, its goal is to minimize water level deviations by using this information to anticipate on the disturbances. However, in this case a feed forward controller is used with the goal to convey water faster trough the canal system. When at the main regulator the extra discharge is released, this information is directly transmitted to the downstream gates, which will directly increase the opening height to achieve the desired flow rate. Using the feed forward principle in this manner, it is expected that the water levels will fluctuate even more than without this controller, as this feed forward signal is based on discharges and not on water levels. This is a known decrease of performance, but is accepted since the primary goal is to create an extra discharge into the reservoir as quickly as possible. Besides, the effect of this feed forward controller is only noticeable directly after the feed forward action, which will only occasionally be used.

Graphs

The graphs on the right (figure 4.2) show indeed a quick response of all the checks, but the desired step like change in flow at each gate is not produced. When enforcing an extra discharge at a certain gate, prior to the moment on which water – that was released upstream – arrives at that gate, the water level will drop. This water level drop, which was introduced by the feed forward controller, makes the feedback controller respond. Here, the feedback and the feed forward controller have conflicting actions. The initial increase of the discharges is the result of the feed forward controller and the strong response after the water level drop is due to the control action of the tightly tuned feedback controller. The checks start to discharge the desired 50 m^3/s after the water from the reach upstream has arrived. Only the discharge through check 13 is directly as desired, since the controller for this check is not water level based.

Besides the initial water level drop after discharging the extra 10 m^3/s, the water levels are kept within the 2 cm dead band and that is even better than without the feed forward controller.

Evaluation

Even though the last check structure discharges 4 m^3/s extra, directly after the step has been made, the time until the discharge measures 50 m^3/s is comparable to the time needed in the base case. Since the water levels are maintained very well, it may be possible to reduce the influence of the feedback action by allowing larger water level deviations. Reducing the feedback action will not only result in a better approximation of the step change in disturbance, it prevents powerful control actions that consume much energy and cause wear and tear. It is also possible to reduce the feedback action by waiting until the additional discharge from the upstream check arrives, however this delaying effect is not desirable. With re-tuning the control constants and the filter the feedback action will be weakened. This strategy is covered in case 3.
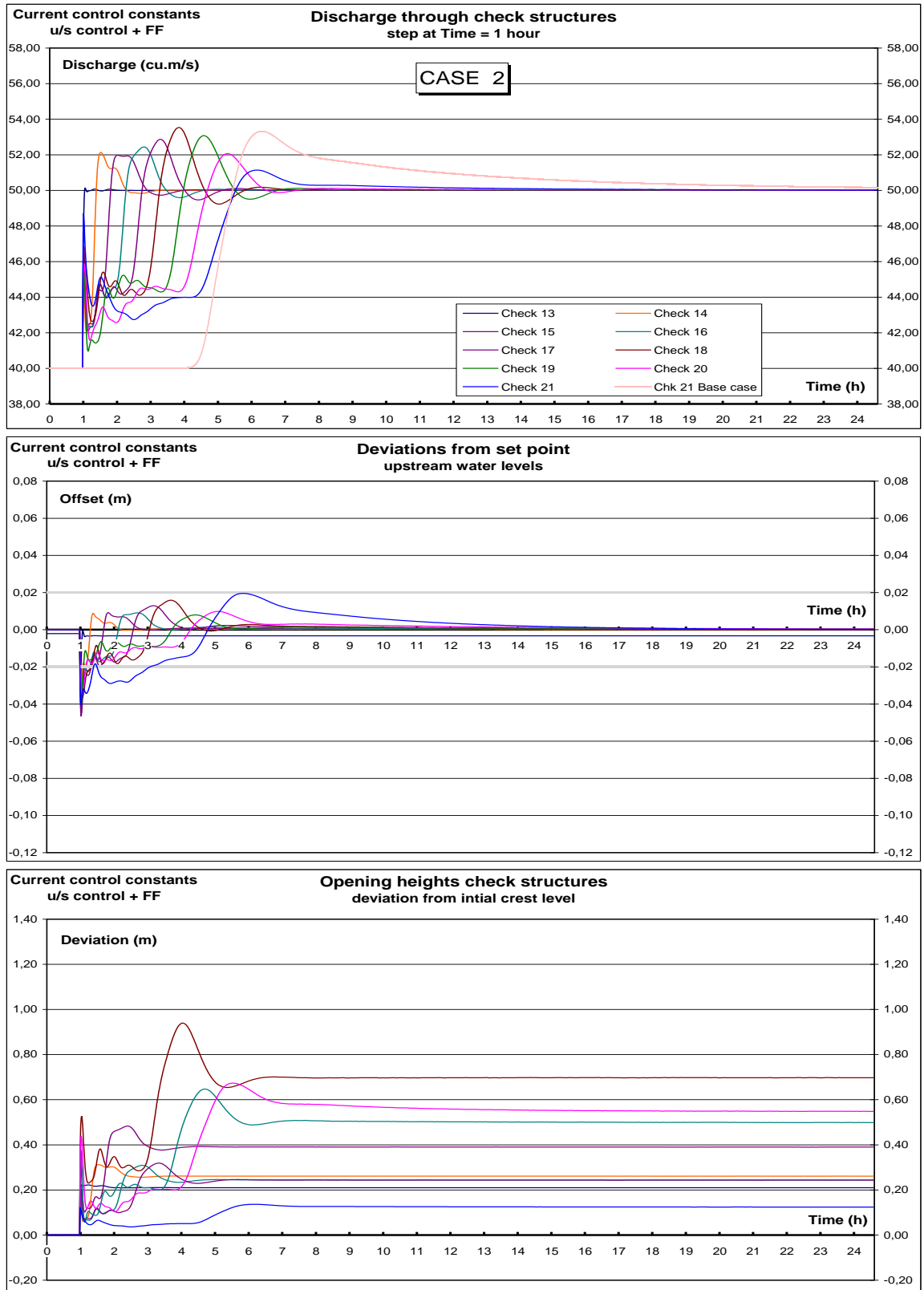
Figure 4.2 a/b/c – Discharge, water level and crest opening Case 2

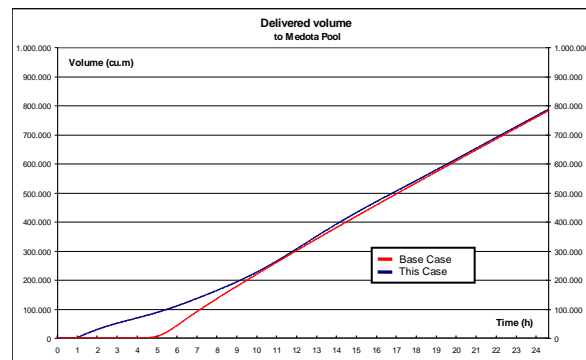## Case 3: 'Re-tuned control constants and feed forward'

Applied strategy

Until now the control constants were tuned for local upstream control. That was done with a Multiple Model Optimization [Overloop, 2004]. The control constants and the filter in this case are also tuned with that algorithm, but now the tuning is based on remote upstream control. When tuning for remote control, the constants are weakened as the lowest resonance frequency of a reach is the most critical one (highest resonance peak). In addition to re-tuning the control constants the filter is re-tuned. The filter constants are optimized with the same algorithm. By doing this, the controllers will respond less powerful to the waves that are introduced by the step of 10 m^3/s. In other words: the feed back action on the step change in disturbance is weakened.

Graphs

The graphs in figure 4.3 a to c show the response of the system with the newly tuned control constants. One can see that the feedback action is still present, but is not affected by the waves traveling through the reach. The waves can be recognized between 01h00 and 03h00 and have a maximum amplitude of approximately 7 cm. The discharge drops after the first – feed forward induced – step to 50 m^3/s, but does hardly show any response to those waves. Usually, a 7 cm water level deviation causes a strong feedback action. The more conservative settings of the control constants can also be recognized in the paths that the crest levels follow.

Evaluation

In terms of performance the re-tuned constants and the feed forward controller have a positive effect. This is clearly visible when comparing the discharge at the last gate of the Base Case, with the discharge at the last gate of this case. The discharge of Base Case is drawn with the dotted line. In the first three hours after increasing the flow at the main regulator the average discharge is 6.3 m^3/s (total volume: 68.000 m^3), whereas with the current control system the average discharge is zero. Hour 4 is chosen to make a comparison, because at that moment check structure 21 starts to increase the discharge towards 50 m^3/s in the Base Case. Figure 4.3 d gives a representation of the delivered volume to the reservoir. It is clear that the improvement in delivery can only be made in the first part where the lag times play a role and that in the second part the discharge at the main structure determines the inflow in the reservoir.
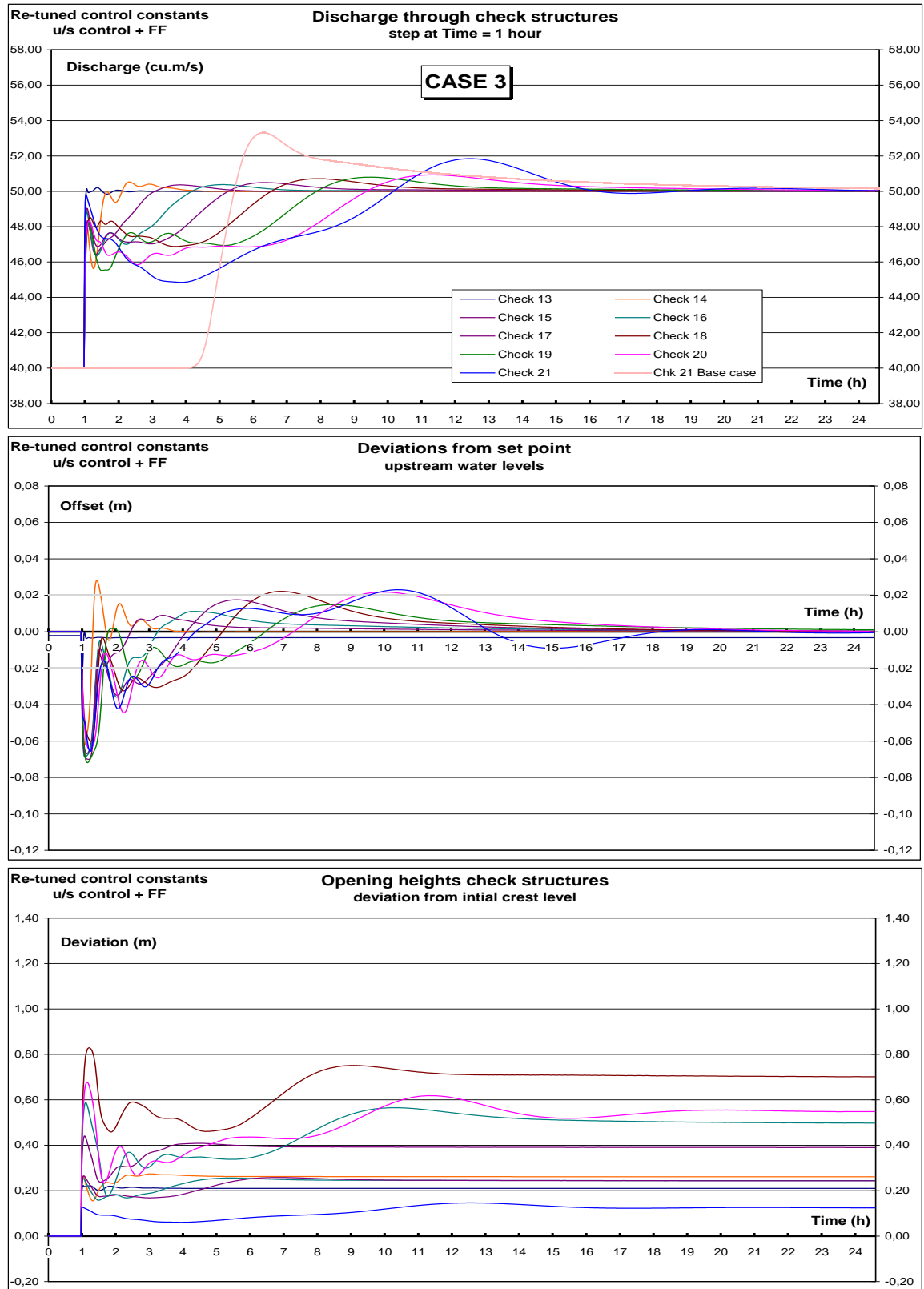

Figure 4.3 d – Delivered volume to the reservoir

Figure 4.3 a/b/c – Discharge, water level and crest opening Case 3

Case 4: 'Target level adjustment'

Applied strategy

In this case the re-tuned control constants were used with the feed forward controller in combination with a target level adjustment. It appeared in the previous case that there was still a feedback action, although it was significantly less than in case 2. The last possibility to cancel out the feedback influence is to alter the target level. Appendix 2 describes the integrator delay model on which this target level adjustment is based. In figure 4.4 the results are shown of a simulation where the target level is ramped down from 01h00 to 01h30 and ramped back up from 01h30 to 02h00. The main argument for the first time interval is that the feedback actions only have a big influence in the first 30 minutes after 01h00. The time interval for the ramp up is also chosen to be 30 minutes to prevent discontinuities, as there will be, in case the target level is set back instantaneously. On the other hand, it is not desired that the ramp back influences the system over a long period.

Graphs

With this target level adjustment the formerly strong feedback actions are almost not to be recognized in the discharge graph. Discontinuities in discharge are noticeable for check structure 14 and 19. The peaks between 01h30 and 02h30, measuring respectively 55.3 m^3/s and 46.6 m^3/s, are likely due to nearby culverts that affect the storage area or to errors in the determination of the target level ramp. Since the overall picture gives proper insight in the effect of the target level adjustment, the cause is not investigated.

During the first hour a maximum deviation of 7.5 cm is reached, which enters the dead band almost within three hours. Hence, the water levels are maintained as well as they are in case 3.

Evaluation

The discharge into the reservoir in this case is bigger than in the previous cases. After three hours the delivered volume is over 78,000 m3 at an average discharge of over 7.2 m^3/s. Compared with case 3 this is an increase in performance of almost 15%. Moreover the crest heights vary a little less than in the previous case, which is beneficial out of an energy perspective.
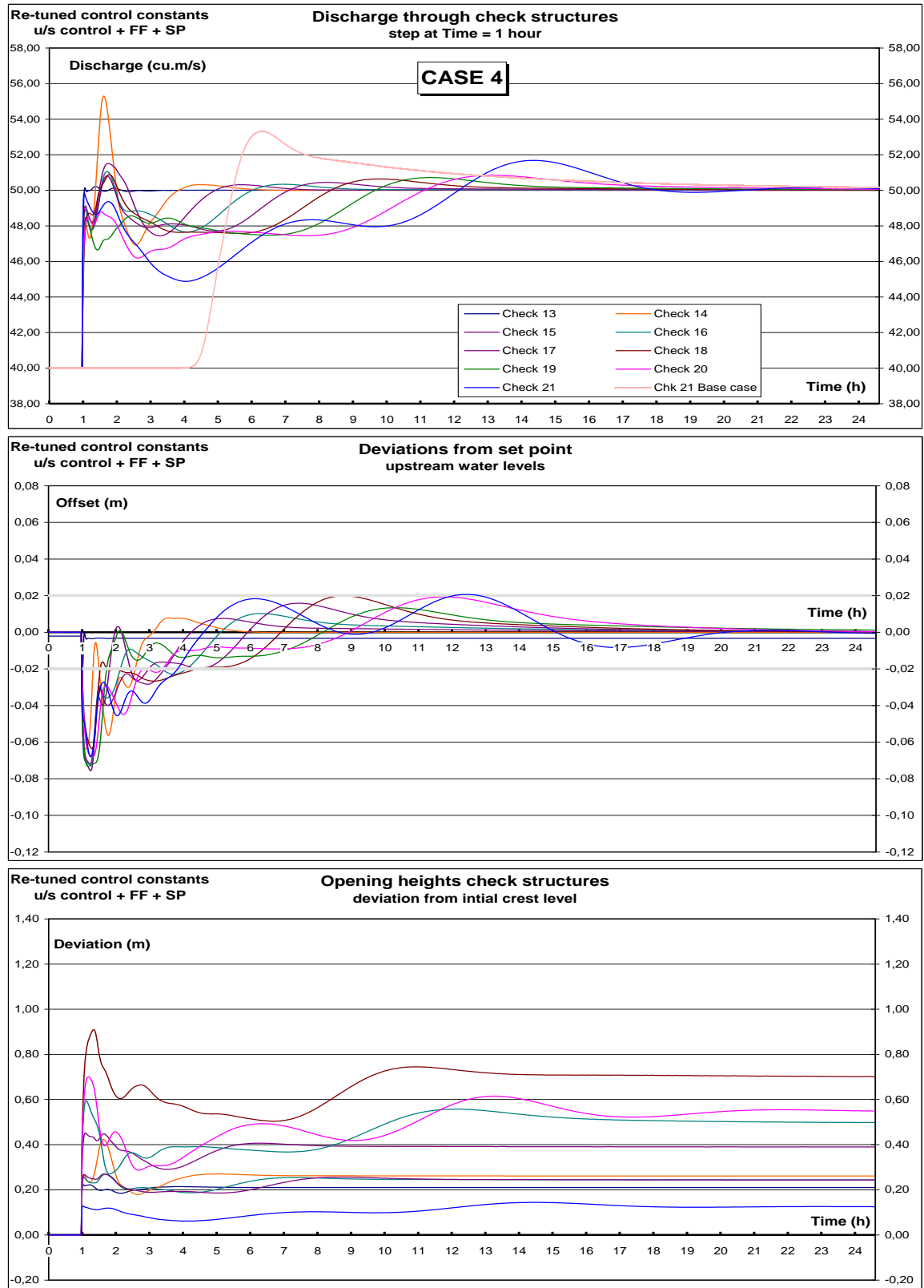
Figure 4.4 a/b/c – Discharge, water level and crest opening Case 4

4.3 General evaluation (quick scan)

The previous pages show responses of control strategies that are produced when applying a feed forward controller, when re-tuning the constants and the filter, and when temporarily adjusting the set point. These are the degrees of freedom when maintaining the current control method, as demanded in paragraph 2.3. This paragraph presents an evaluation of the different strategies and determines which strategy will be analyzed further.

- The preference to add a feed forward to the control system is, out of a delivery perspective, obvious.
- The characteristics of case 2, however, are not without some negative aspects. They concern the wear and tear of the moving parts and the energy consumption.
- The strategy in case 3 provides better results; bigger and faster water delivery to the reservoir, less heavy control actions and the water level deviation stays within the acceptable margin.
- Although adjusting the target levels (case 4) leads to better results, it is not the strategy that will form the solution to the problem. The next list clarifies the preference of the strategy of case 3.
    - The storage area needs to be determined for every flow condition
    - The ramp has to be tuned for each check structure
    - Culverts located upstream (and downstream) of the check structures can hinder the ramping strategy
    - The water system takes longer to stabilize
    - Adding the ramping strategy makes the controller and the implementation more complex
    - Adding the ramping strategy makes the controller less suitable for wide implementation
    - **But above all**, the strategy can be false. In the presented graphs of case 4 this is not visible, however it is clear when examining the simulation results. See figure 4.4 d to k.

The graphs on the next page show a wave traveling in up- and downstream direction that is caused by the opening of the gates. The eight graphs are momentary recordings of the water level in reach 1; reach 1 is exemplary for the other seven reaches. The percentage of the reach that is influenced by backwater (during high flow regime) lies between 70% and 90%. It is clear that the Integrator-Delay model is not applicable, as that model assumes an integral rise of the water level in the storage area.

The strategy with the target level adjustment worked so well, since the ramping had – by accident – the same shape as the wave that was caused by the feed forward controller. The discontinuities, in the discharged flow that are mentioned in case 4, can also be caused by the illegitimate application of the ramping strategy.

The strategy to be analyzed further is: re-tuning the control constants in combination with a feed forward controller (case 3).
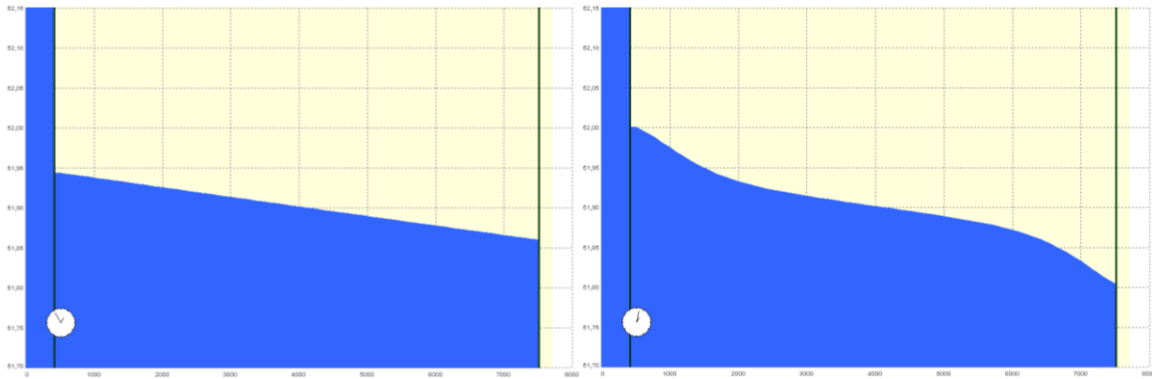
Figure 4.4 d (left) and 4.4 e (right); Water level in reach 1 at time is −00h05 and 00h02
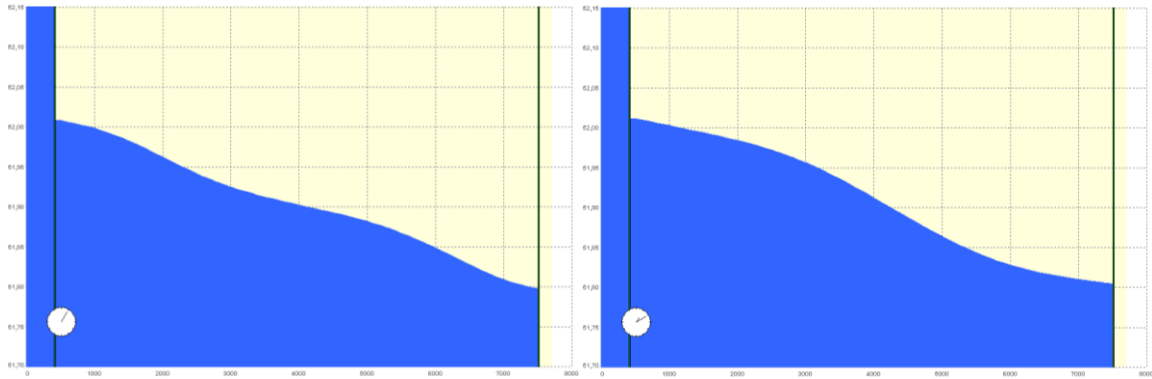
Figure 4.4 f (left) and 4.4 g (right); Water level in reach 1 at time is 00h05 and 00h10
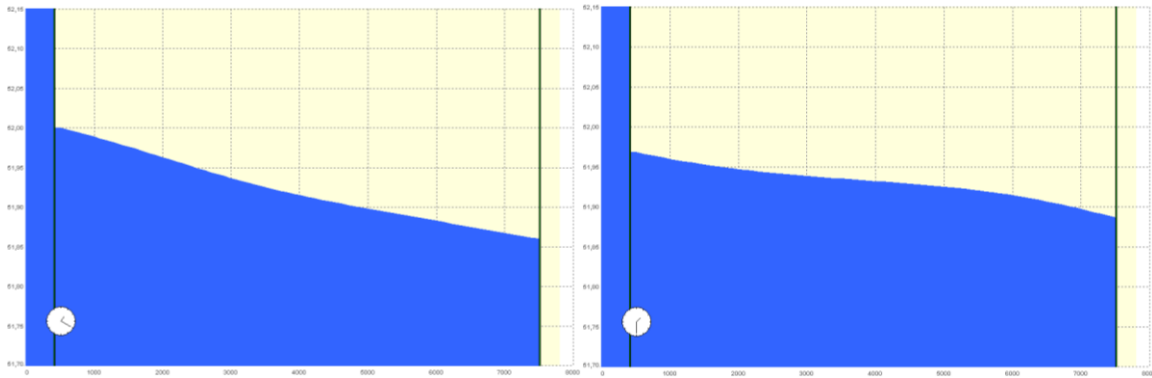
Figure 4.4 h (left) and 4.4 i (right); Water level in reach 1 at time is 00h20 and 00h302
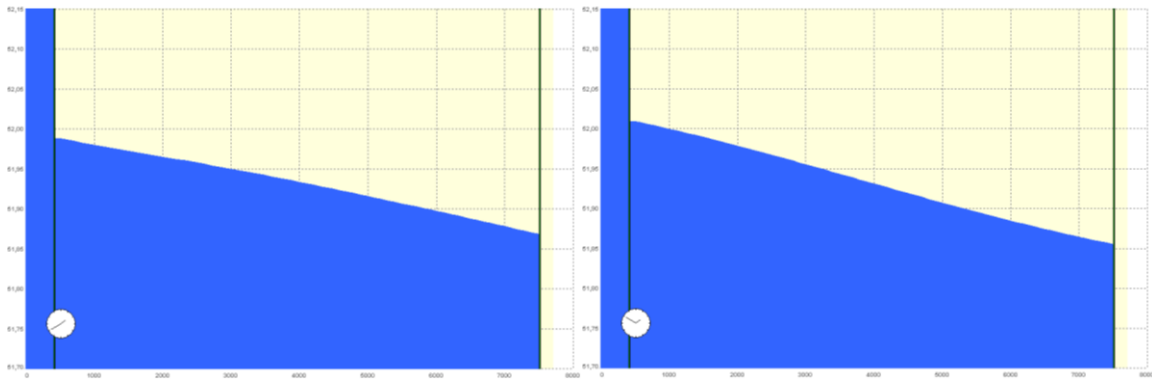
Figure 4.4 j (left) and 4.4 k (right); Water level in reach 1 at time is −00h40 and 00h50

4.4 Analysis and improvements strategy case 3

Case 5: 'Robustness and performance analysis'

Applied Strategy

The application of a feed forward controller in combination with the re-tuned control constants may have given better results than the current control system, but it is also important to verify the performance without the feed forward controller. The control system must maintain the water levels within acceptable margins when the feed forward controller is disabled. This can occur when data communication fails or in case of an error in the programming. The reason why this research turned in the direction of a feed forward controller, was that this type of controller functions as an additional, but non-necessary part of the control system. Meaning that the performance of the control system would not suffer – during normal operating procedures – from the not-functioning feed forward controller. In this case the feed forward controller is disabled, but the re-tuned constants and the re-tuned filter is still in operation. With this strategy not only the robustness is verified, it also gives an insight in the performance of the control system during normal operating circumstances.

Graphs

As expected, the 'weaker' control constants respond slower to disturbances and produce larger deviations from set point (see figure 4.5). When observing the discharge graph, it comes into view that the overshoot of check 21 is of the same magnitude as the step in flow. The maximum discharge is 62 m^3/s, which is 12 m^3/s over the equilibrium discharge.

The water level deviations are also increased; from a maximum deviation of 7 cm in case 3 to over 16 cm in this case. The time span in which the water levels are outside the dead band has increased from roughly three and a half hour in case 3 to roughly 13 hours in this case.

The paths that the crest levels follow are inherently extended.

Evaluation

The performance of the control system appears to be a problem since the maximum water level deviation equals approximately 16 centimeters. The embankment is most likely not the problem, but the change in storage in the reach can result in significant performance loss.

The graphs show water levels that exceed the set points with 10 to 16 cm, hence performance loss is not to be expected here. However, when not a increasing step like disturbance enters the system, but a decreasing step, the water levels will drop below set point. The response of the control system will then be the inverse – by approximation – of the response shown in figure 4.5. The response will not be exactly the inverse due to non-linearities in the relation between discharge and head difference.

Another problem is expected when the maximum discharge of the gates is reached. The maximum discharge in the graphs equals almost 62 m3/s. When this discharge cannot be realized due to physical restrictions, safety restrictions (damage to embankments) or otherwise, the water level deviations will become even larger. In this case the embankment heights can be insufficient.
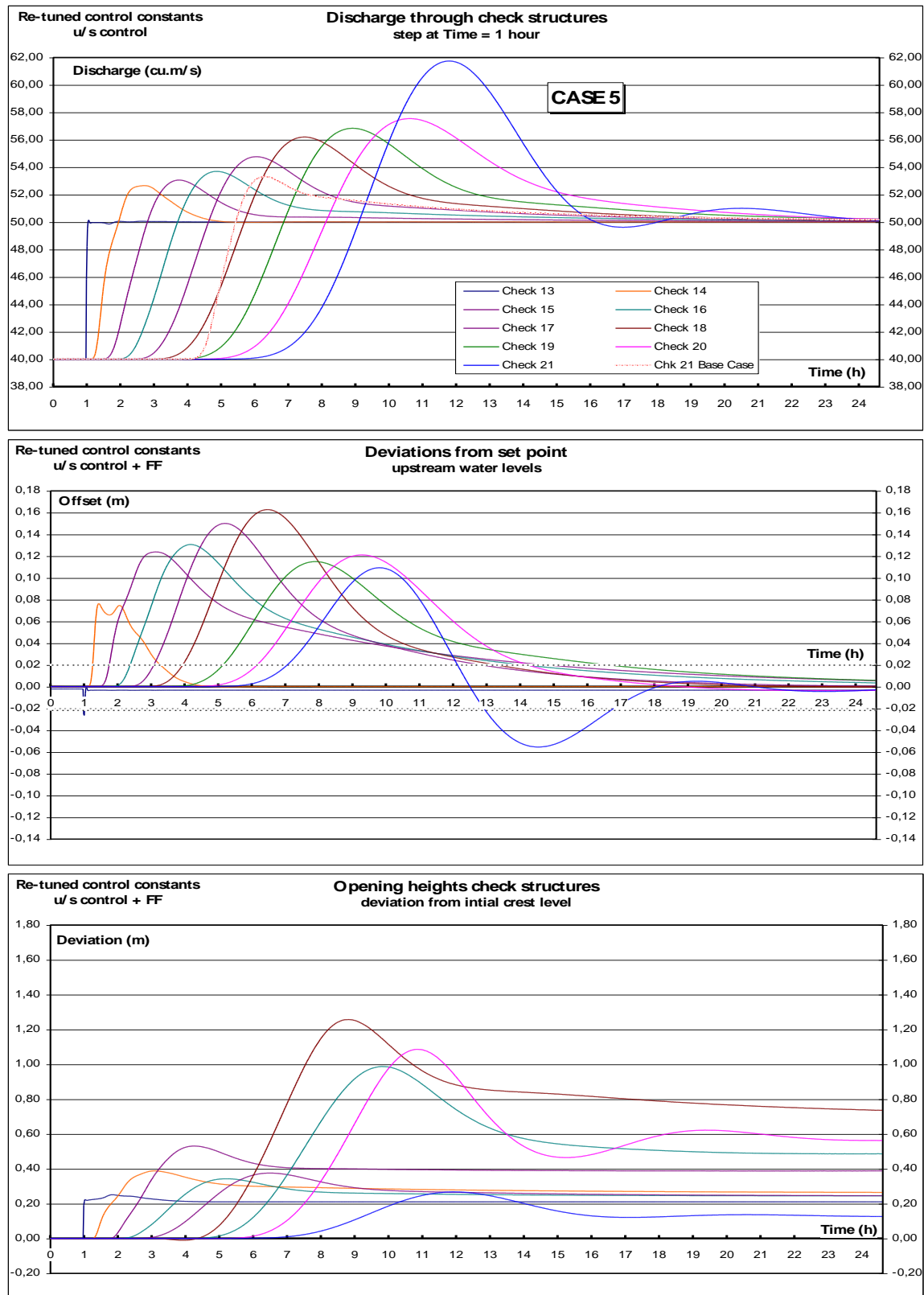
Figure 4.5 a/b/c – Discharge, water level and crest opening Case 5

Case 6: 'Improving performance – switching constants I'

Applied Strategy

The robustness of the control system suffers when applying the re-tuned constants and filter. Not only when the feed forward is disabled and the step disturbance enters the system, also with regular disturbances the water levels will drift from set point. In other words: the performance is insufficient with a failing feed forward and the performance is insufficient under regular circumstances. The strategy to be followed in this case is to limit the time span in which the re-tuned constants and filter are applied.

In this case an analysis is made of the switching between the current and re-tuned constants when the water system is subjected to a block like disturbance. At the moment on which the step up is made, the current control constants are replaced by the re-tuned control constants and when the system is recovered from the step back, the re-tuned constants are again replaced by the current constants. In this way the advantages of the strong responding current constants and the weak responding re-tuned constants are combined.

Graphs

The upper graph in figure 4.6 shows clearly the step up at time is 01h00 and the step back at time is 25h00. Eighteen hours later, at time is 44h00, the second switch is made. This moment is chosen since at that moment the water level deviations from set point are for each reach within 1 cm. The step up is identical to case 3 and the step back shows almost the inverse of the step up response. At time is 44h00 a discontinuity is shown in the graph, however the size is negligible. For reference the discharge through check 21 of the Base Case is drawn with the dotted line. The non-linearity in the relation between head difference and discharge can be recognized best in this graph.

The water level deviations from set point remain as shown in case 3. Also for the step back the deviations have a maximum of approximately 6 cm. At the moment of the second switch almost no water level fluctuation is noticeable.

Evaluation

The graph show that the significant water delivery is speeded up with almost five hours and that the water levels are within the dead band at time is 26h00, which is 6 hours earlier than with the current control method.

The switching of the constants forms a discontinuity, which is undesirable. When switching between the control systems at time is 1 hour that discontinuity is no longer noticeable as at that time the step in discharge is made, which is by itself a large discontinuity

With this strategy the performance loss is reduced to a time span of 43 hours for this system The time span depends on the system since the switches can best be made simultaneously with another discontinuity (step up) or when the water levels are close to set point (step down).
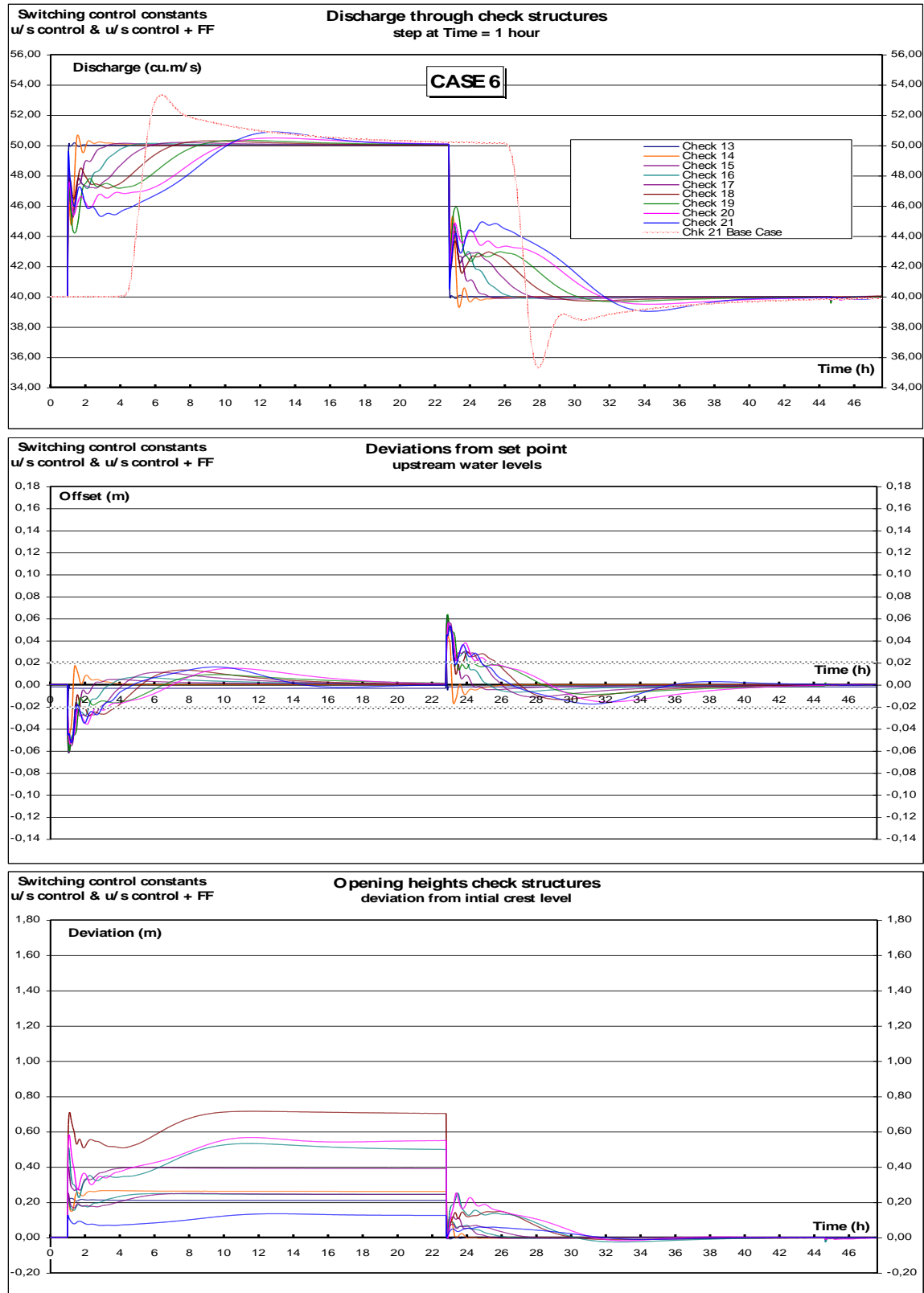
Figure 4.6 a/b/c – Discharge, water level and crest opening Case 6

Case 7: 'Improving performance – switching constants II'

Applied Strategy

The strategy is identical to the previous, with the exception that the second switch is made simultaneously with, or even prior to the step back. This analysis is done to illustrate the effect of the step back with the current control constants. Not only the research argument applies for this analysis, also in case of long lasting deliveries to the reservoir the response of the system is important. In this case it might be desired to make the second switch earlier than 18 hours after the step back, so that the time span with the performance loss does not increase to a great extend.

Graphs

At time is 20h00 the second switch is made and at time 25h00 the constants are switched back. This strategy is chosen to split the effects of the second switch and the effects of the step back. Above that, the results remain comparable with the previous case. The discharge graph in figure 4.7 illustrates the overshoot (undershoot) due to the step back. Note that the undershoot exceeds the overshoot, which is caused by the mentioned non-linearities in the system.

The magnitude, of the water level deviations after the step back, is of the same order as they are in the Base Case, hence within 6 cm. Water level deviations caused by the second constant-switch are negligible.

Evaluation

The performance of this strategy is comparable with the strategy of the previous case, however the robustness is higher. This is the consequence of the step back taking place after the second constant switch. The performance of the current control constants (without a feed forward controller) is better than the performance of the re-tuned control constants with a failing feed forward controller (see figure 4.1 and 4.5). In case the feed forward does work, this strategy is preferred.

Since the second switch does not lead to significant discontinuities, this switch is best to be made as soon as possible. The moment on which the second switch can be made is dependent on the accepted water level deviations or discharge changes. When switching to the current constants, the response to the error on the water level at that time leads to discontinuities. Mostly these discontinuities only consist of changes in discharge and to a lesser extend of water level deviations, but when approaching the moment of the step up the water level deviations may become significant. Concerning water level deviations and stationary flow, it is advisable to make the second switch when the error on the water level is within a few centimeters.
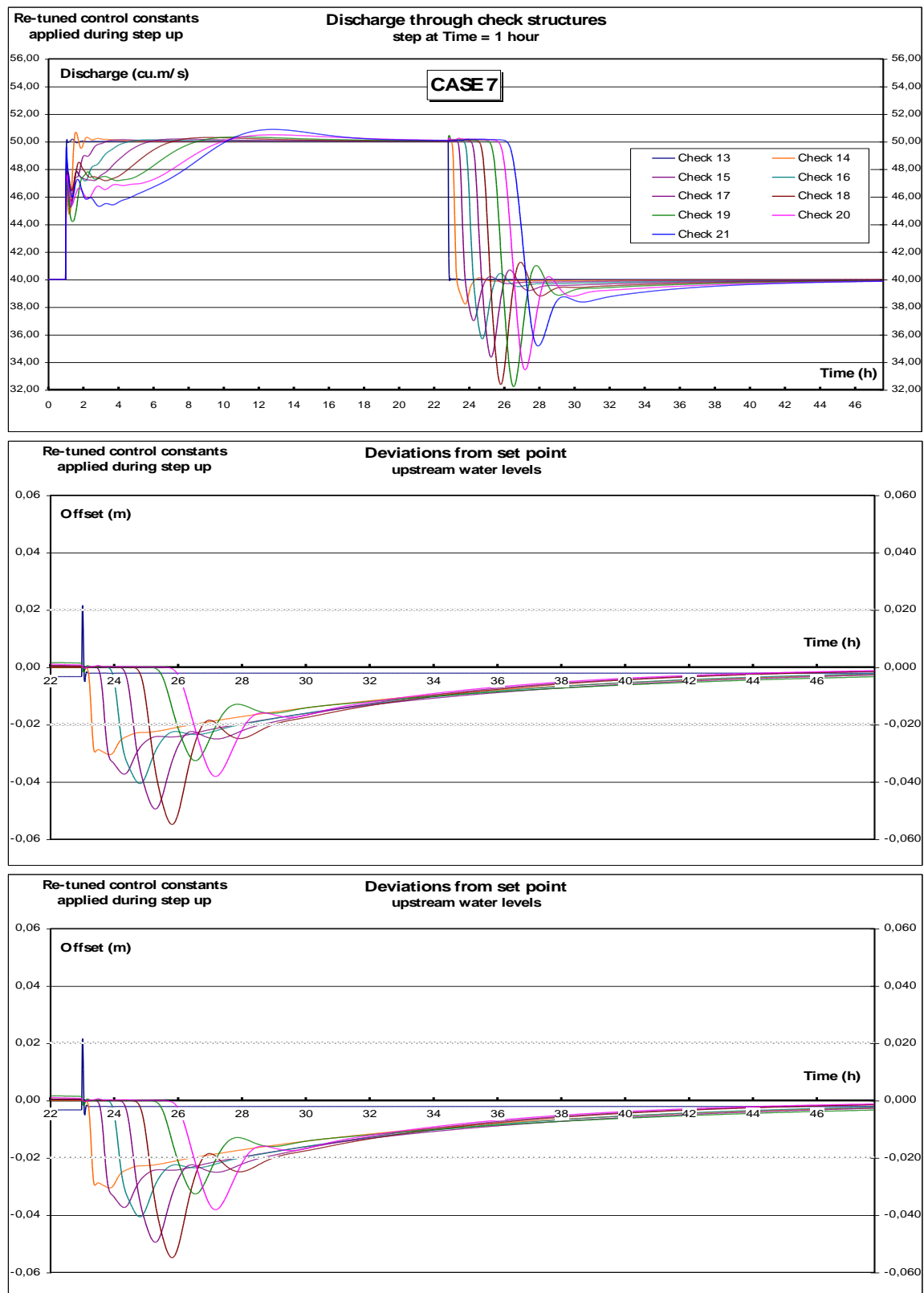
Figure 4.7 a/b/c – Discharge, water level and crest opening Case 7

4.5 Strategy recommendation

The advised strategy (especially in case of long lasting deliveries):

- use under normal circumstances the current control constants
- when making a step up or down (e.g. to fill the reservoir) switch to the (less tight) re-tuned feedback constants
- switch back to the current constants as soon as possible and leave it this way in case the response to the step back of the current constants is satisfying. (highest robustness)
- if not, switch to the re-tuned constants when making the step back and switch back to the current constants when the water level is near set point (best performance)

4.6 Alternative strategy: no water level control

The previous cases were built under the assumption that switching to another control strategy was not allowed, as paragraph 2.3 stated this. However, if it would be allowed to make a temporary switch to an other strategy, one very tempting and simple strategy would come into picture.

The alternative strategy is to switch to a flow controller during the first hours after the step up. In the next case it is researched if a flow controller could be used for the water delivery problem and what the results are. It is clear that a switch to a flow controller, and thus leaving the water level uncontrolled for a certain amount of time, should ring the 'alarm bells'. This strategy is researched because of the simplicity and the economic advantages.

Case 8: 'Flow controller'

Applied Strategy

The figures 4.8 show the response of the system when a flow controller is applied during 4 hours after the step up. Although this strategy does not provide the highest safety, as during the four hours there is no control on the water level, the results are nevertheless provided because of the simplicity of implementation in reality. As shown in the previous cases, control on water level causes unsteady conditions due to the conflicting actions of the feed forward and feedback controllers. With a temporary switch to flow control during the unsteady conditions, the conflicting actions are prevented. Another advantage is that with a flow controller the delivery to the reservoir is almost instantaneously 50 m^3/s. With a flow controller it is possible to make easily use of the volumes of water in the reaches in order to create a fast inflow into the reservoir. A flow controller on the upstream and downstream end of the reaches should maintain the total volume in the reaches and should not lead to water level deviations.

However, water level deviations are likely to occur when the released discharges are not exactly equal to the computed discharges, for instance as a result of measurement errors. Or, when during the four hours of the flow controller, the extracted amounts of water alter. With regard to these issues, it is necessary to define margins between which a flow controller can be applied and outside which the system only operates on water level control.

Graphs

As expected the discharges are not affected by the waves in the reaches. The flow controller maintains the flow at 50 m^3/s during three hours. After t is 04h00 the water level based controllers are again activated. They reduce the flows to fill up the reaches to their set points.

The initial water level deviations are slightly larger than in the case with the re-tuned control constants in combination with a feed forward controller. It lasts significantly longer until the water levels in all reaches are within the dead band.

A consequence of trying to maintain the flow, during the time that the waves are present, is that the crest levels need more adjustments to compensate for the altering head differences.

Evaluation

On first sight this strategy seems to deliver excellent results, but at the expense of safety. As mentioned the flow controller should only be installed in combination with operational margins and with accurate measurements.

The water level deviations due to inaccurate flow control can be as big as 15 cm when the difference between inflow and outflow is 2 m^3/s. This is also an indication for the effects of altering turnout flows that can result in both an increase and decrease of the water levels.

Essential for the performance is that the conditions concerning the turnouts remain stable during 3 hours, or that the altering turnout flows lead to adjustments of the upstream flow controllers. However, this would be an extra uncertainty.
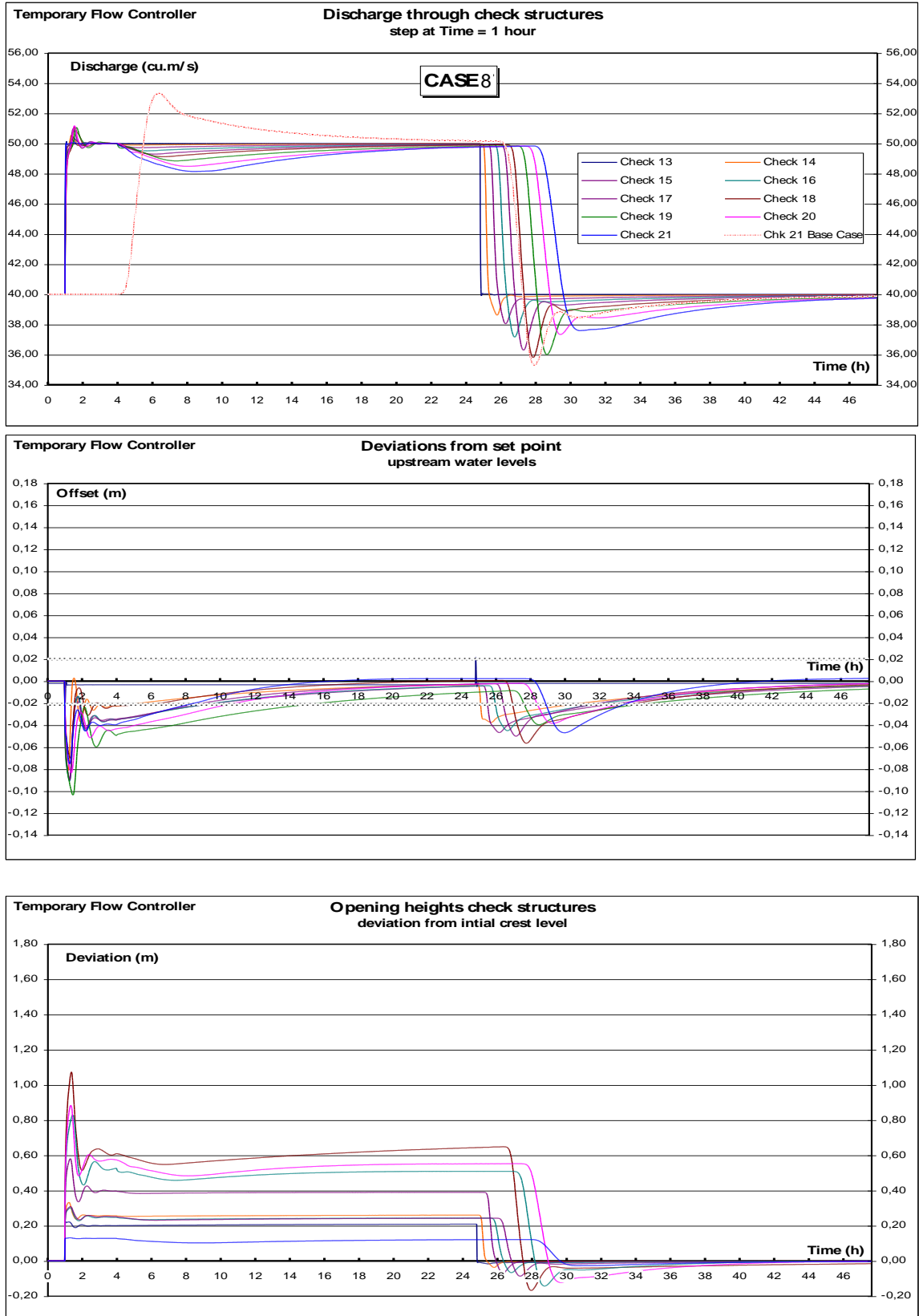
Figure 4.8 a/b/c – Discharge, water level and crest opening Case 7

5. Recommendation

One solution is found in a feed forward controller in combination with re-tuned constants. An alternative solution is formed by a temporary flow controller, but special attention is necessary when dealing with varying turnout flows. The figure shows the water level of the reservoir for the present control strategy and for the two recommended strategies.
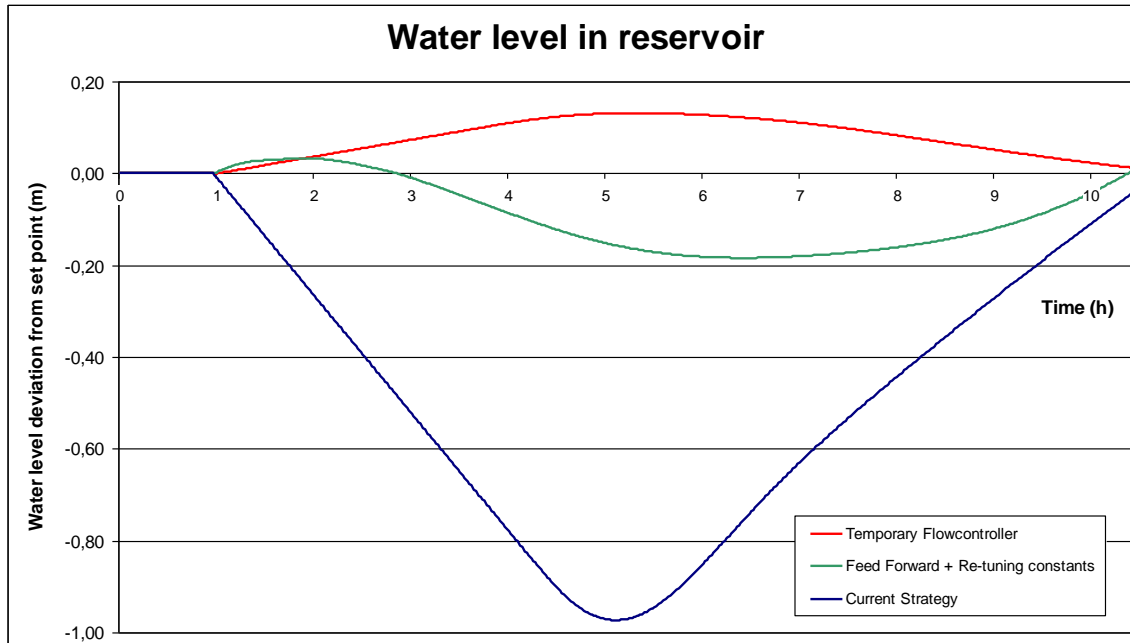
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Appendix 1. Dimensions of the irrigation system

This annex provides the exact specifications of the Delta Mendota Canal and the structures from the O'Neill Forebay and the Mendota Pool.

Table X1.1 Characteristics check structure 1 (13)

Check ID	Type (-)	Width (m)	Height (m)	# Present	Location (m)
CHK 13	Radial gate	6.10	6.10	x 3	92,161 (absolute)
	Weir	9.14	7.62	x 1	0 (relative)

Table X1.2 Characteristics reach 1

Reach ID	**1**	Bed level u/s (m)	47.72
From (m)	92,161	Bed level d/s (m)	47.59
From (m, relative)	0	Bed width (m)	14.63
To (m)	99,258	Side slope (-)	1:1.5
To (m, relative)	7,097	Bed slope, average (-)	1.83 E-5
Length (m)	7,097	Roughness (Manning)	0.023

Table X1.3 Characteristics check structure 2 (14)

Check ID	Type (-)	Width (m)	Height (m)	# Present	Location (m)
CHK 14	Radial gate	6.10	6.10	x 3	99,258 (absolute)
	Weir	9.14	7.62	x 1	7,097 (relative)

Table X1.4 Characteristics reach 2

Reach ID	**2**	Bed level u/s (m)	47.59
From (m)	99,258	Bed level d/s (m)	46.59
From (m, relative)	7,097	Bed width (m)	14.63
To (m)	107,713	Side slope (-)	1:1.5
To (m, relative)	15,552	Bed slope, average (-)	1.18 E-4
Length (m)	8,455	Roughness (Manning)	0.016

Table X1.5 Characteristics check structure 3 (15)

Check ID	Type (-)	Width (m)	Height (m)	# Present	Location (m)
CHK 15	Radial gate	5.49	6.71	x 3	107,713 (absolute)
	Weir	9.14	7.62	x 1	15,572 (relative)

Table X1.6 Characteristics culvert 1 (reach 3)

Culvert ID	**1**	Bed level u/s (m)	37.45
From (m)	107,733	Bed level d/s (m)	37.45
From (m, relative)	15,572	Bed slope, average (-)	0
To (m)	107,862	Cross section (-)	Circular
To (m, relative)	15,600	Diameter (m)	14.33
Length (m)	128	Roughness (Manning)	0.013

Table X1.7 Characteristics reach 3

Reach ID	**3**	Bed level u/s (m)	46.59
From (m)	107,713	Bed level d/s (m)	45.90
From (m, relative)	15,552	Bed width (m)	14.63
To (m)	116,479	Side slope (-)	1:1.5
To (m, relative)	24,318	Bed slope, average (-)	7.98 E-5
Length (m)	8,766	Roughness (Manning)	0.016

Table X1.8 Characteristics check structure 4 (16)

Check ID	Type (-)	Width (m)	Height (m)	# Present	Location (m)
CHK 16	Radial gate	5.49	6.71	x 3	107,713 (absolute)
	Weir	9.14	7.62	x 1	24,318 (relative)

The definition of the dimensions of the check structures is as follows.
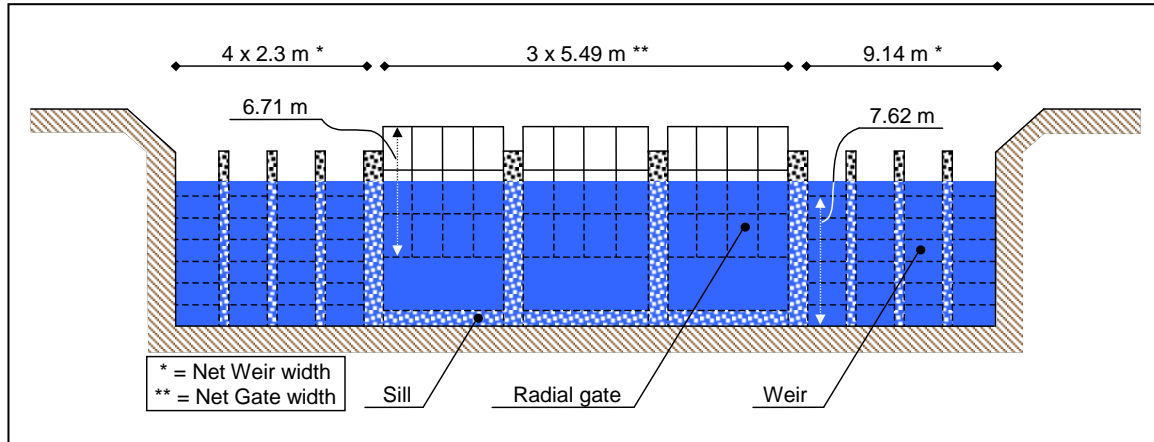
Figure X1.1: Definition dimensions check structures; used dimensions: check 16

Table X1.9 Characteristics reach 4

Reach ID	**4**	Bed level u/s (m)	45.90
From (m)	116,479	Bed level d/s (m)	44.45
From (m, relative)	24,318	Bed width (m)	14.63
To (m)	125,248	Side slope (-)	1:1.5
To (m, relative)	33,087	Bed slope, average (-)	1.65 E-4
Length (m)	8,769	Roughness (Manning)	0.013

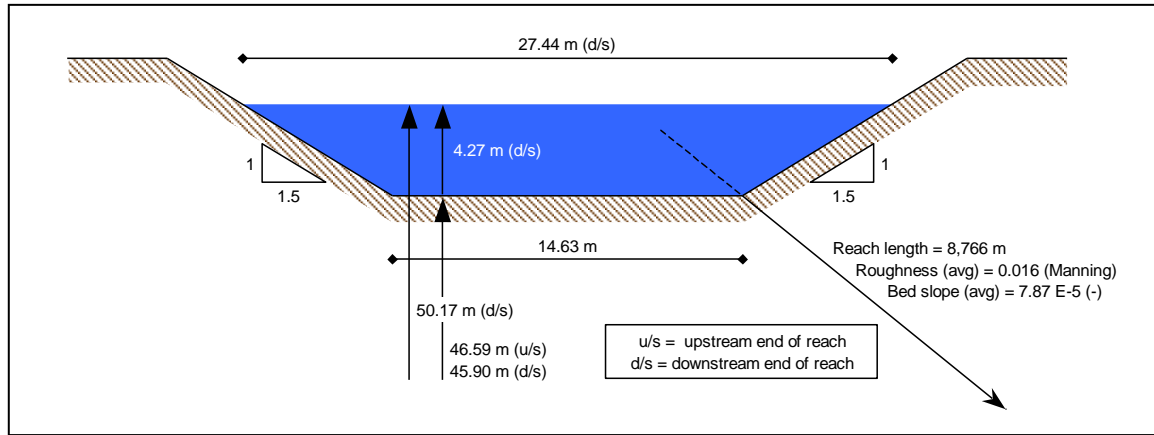**Table X1.10 Characteristics check structure 5 (17)**

Check ID	Type (-)	Width (m)	Height (m)	# Present	Location (m)
CHK 17	Radial gate	5.49	6.71	x 3	107,713 (absolute)
	Weir	9.14	7.62	x 1	33,087 (relative)

Table X1.11 Characteristics reach 5

Reach ID	**5**	Bed level u/s (m)	44.45
From (m)	125,248	Bed level d/s (m)	42.49
From (m, relative)	33,087	Bed width (m)	14.63
To (m)	135,346	Side slope (-)	1:1.5
To (m, relative)	43,185	Bed slope, average (-)	1.94 E-4
Length (m)	10,098	Roughness (Manning)	0.014

The definition of the dimensions of the reaches is as follows.

Figure X1.2: Definition dimensions reaches; used dimensions: reach 5

Table X1.12 Characteristics check structure 6 (18)

Check ID	Type (-)	Width (m)	Height (m)	# Present	Location (m)
CHK 15	Radial gate	5.49	6.71	x 3	107,713 (absolute)
	Weir	9.14	7.62	x 1	0 (relative)

Table X1.13 Characteristics reach 6

Reach ID	**6**	Bed level u/s (m)	42.49
From (m)	135,346	Bed level d/s (m)	43.28
From (m, relative)	43,185	Bed width (m)	14.63 / 18.90 *
To (m)	148,602	Side slope (-)	1:1.5 / 1:2.5 *
To (m, relative)	56,411	Bed slope, average (-)	-5.96 E-5
Length (m)	13,256	Roughness (Manning)	0.016

* change in cross section 2,856 m after begin reach; location 138,202 m

Table X1.14 Characteristics check structure 7 (19)

Check ID	Type (-)	Width (m)	Height (m)	# Present	Location (m)
CHK 15	Radial gate	5.49	6.71	x 3	107,713 (absolute)
	Weir	9.14	7.62	x 1	65,936 (relative)

Table X1.15 Characteristics culvert 2 (reach 7)

Culvert ID	**2**	Bed level u/s (m)	36.39
From (m)	158,097	Bed level d/s (m)	36.39
From (m, relative)	65,936	Bed slope (-)	0
To (m)	158,194	Cross section (-)	Circular
To (m, relative)	66,033	Diameter (m)	8.20
Length (m)	97	Roughness (Manning)	0.013

Table X1.16 Characteristics reach 7

Reach ID	**7**	Bed level u/s (m)	43.28
From (m)	148,602	Bed level d/s (m)	42.44
From (m, relative)	56,411	Bed width (m)	18.90
To (m)	158,566	Side slope (-)	1:2.5
To (m, relative)	66,405	Bed slope, average (-)	8.51 E-5
Length (m)	9,964	Roughness (Manning)	0.015

Table X1.17 Characteristics check structure 8 (20)

Check ID	Type (-)	Width (m)	Height (m)	# Present	Location (m)
CHK 15	Radial gate	5.49	6.71	x 3	107,713 (absolute)
	Weir	9.14	7.62	x 1	73,342 (relative)

Table X1.18 Characteristics culvert 3 (reach 8)

Culvert ID	**3**	Bed level u/s (m)	35.85
From (m)	165,502	Bed level d/s (m)	35.85
From (m, relative)	73,342	Bed slope (-)	0
To (m)	165,534	Cross section (-)	Circular
To (m, relative)	73,374	Diameter (m)	7.92
Length (m)	32	Roughness (Manning)	0.013

Table X1.19 Characteristics reach 8

| Reach ID	**8**	Bed level u/s (m)	42.44
From (m)	158,566	Bed level d/s (m)	41.85
From (m, relative)	66,405	Bed width (m)	18.90 / 18.29 **
To (m)	166,967	Side slope (-)	1:2.5
To (m, relative)	74,806	Bed slope, average (-)	7.04 E-5
Length (m)	8,401	Roughness (Manning)	0.012

** before and after Culvert 3

Table X1.20 Characteristics check structure 9 (21)

Check ID	Type (-)	Width (m)	Height (m)	# Present	Location (m)
CHK 15	Radial gate	5.49	6.71	x 3	107,713 (absolute)
	Weir	9.14	7.62	x 1	74,806 (relative)

Appendix 2 Identification

Integrator Delay Model

The Integrator Delay (ID) Model is a simplification of the Saint Venant equations and provides in a linear response model for flows in open channels. It is a model in which the flow conditions in a reach are strictly separated. The model describes one part called the 'uniform part' with uniform flow and a 'backwater part'. The flow condition named 'uniform flow' is applicable to reaches where the flow profile is parallel to the canal bed. The water depth in this case is called the uniform depth and depends on the bed slope, discharge, cross section, friction and gravity. In contrast to the uniform depth, the backwater depth is not unique for each circumstance. The backwater part is the part where water is stored. Figure X2.1 shows a longitudinal section of a reach that is partly affected by backwater.
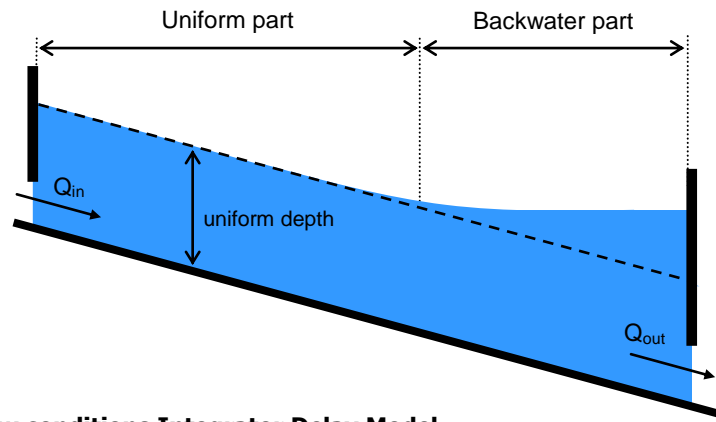
Figure X2.1 – Flow conditions Integrator Delay Model

The backwater part of the reach is considered to behave as a rectangular reservoir with vertical embankments. The flow profile is assumed to be horizontal and delay times are neglected. The water depth in the backwater part is defined by subtracting the bed level from the water level at the downstream end of the reach. The uniform part is considered as the 'conveyance part'; obviously, here the delay times are not neglected. Delay times are important when controlling an irrigation canal. The moment on which the operator has to increase of decrease the supplied quantity is determined by the delay time in the reach of reaches. The delay time is defined as the time necessary for the water to travel from the upstream gate to the downstream gate. The ID Model describes the water level at the downstream end with the next formula:

$$\frac{dh}{dt} = \frac{1}{A_s} \cdot \left(Q_{in}\left(t - \tau\right) - Q_{out}(t) \right) \tag{X.X}$$

in which:
|		
h	–	water level at the downstream end
Q_{in}	–	inflow in the reach
Q_{out}	–	outflow out of the reach
τ	–	delay time
A_s	–	surface area backwater part

Identification

The parameters that need to be determined for the control strategy in Case 4 is the surface area of the backwater part (A_s). For (re-)tuning the constants, as done in Case 3, also the delay time (τ) has to be known.

These two parameters can be determined using a model of the canal in a hydrodynamic modeling package, such as Sobek or CanalCAD. The strategy is as follows. 1) Create a steady flow in the canal where the water level is on set point and the discharge equals the discharge when performing under normal operating circumstances. 2) Increase (or decrease) at the upstream gate the discharge and plot the water level at the downstream end as a function of time. Figure X2.3 illustrates the water level rise after the increase at the upstream gate.
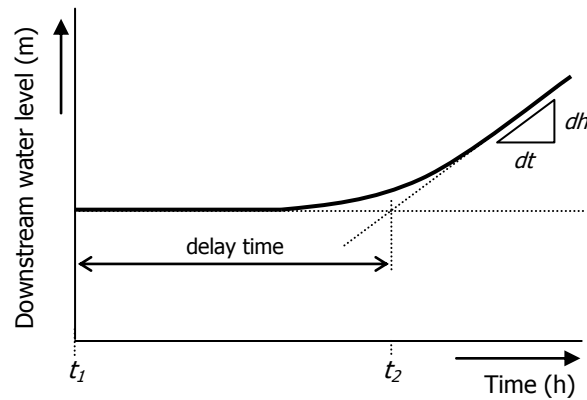
Figure X2.2 – Illustration of identification

With this graph the delay time for remote downstream control and the water level rise per unit of time can be determined. The delay time is defined as the time from the increase at the upstream gate (Time = t_1) until the moment on which the water arrives downstream. For local upstream control the delay time is theoretically zero, since gate movements will directly influence the upstream water level. In practice, the water level starts to increase gradually due to wave deformation (see figure X2.3).
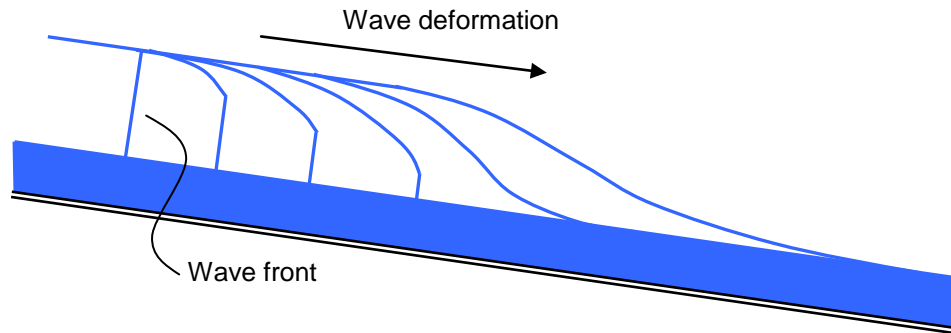
Figure X2.3 – Wave deformation

Therefore the delay time is defined as the time between t_1 and the intersection of the two tangents (t_2) as drawn in figure X2.2. Since the researched system is upstream controlled, the delay time equals the time step in the computation.

The surface area of the backwater part (A_s) can be derived from the water level rise per unit of time, using the following relation:

$$Q_{in} - Q_{out} = dh/dt \odot A_s \qquad\qquad (X.X)$$

This relation can be derived from equation (X.X) when leaving out the delay term. When imposing a certain inflow and outflow, the only unknown parameter is the surface area of the backwater part, A_s.

Identification results for the Delta Mendota Canal

Figure X2.4 shows the results of the identification of the reaches in the Delta Mendota Canal between the O'Neill Forebay and the Mendota Pool. For the identification a base discharge is used of 40 m^3/s and a step in discharge of 10 m^3/s.
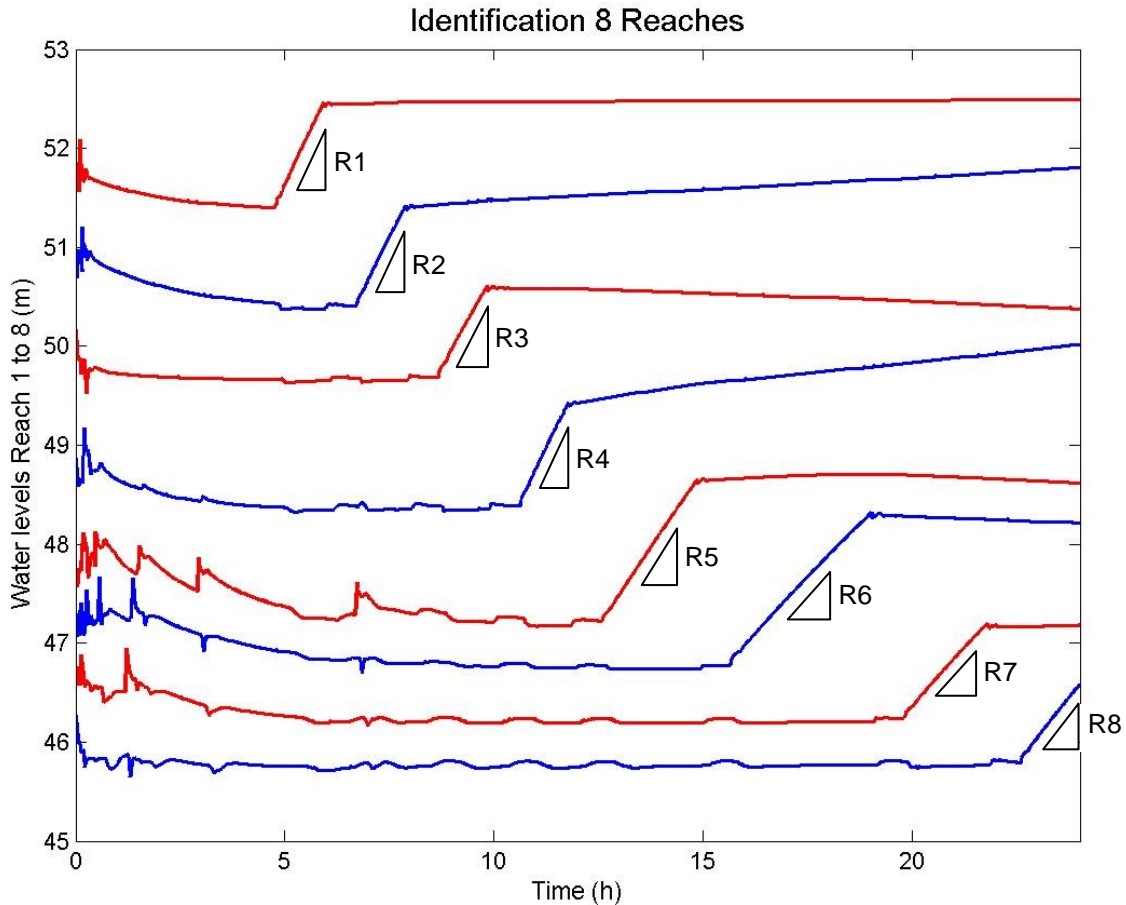
Figure X2.4 – Identification reaches

In figure X2.4 the water level rise is for all reaches clearly visible. The calculated backwater surface areas are presented in table X2.1.

Table X2.1 – Calculated backwater surface areas

Reach 1 (R1)	Area = 198.645 m^2	Reach 5 (R5)	Area = 285.491 m^2
Reach 2 (R2)	Area = 223.835 m^2	Reach 6 (R6)	Area = 482.773 m^2
Reach 3 (R3)	Area = 218.455 m^2	Reach 7 (R7)	Area = 370.383 m^2
Reach 4 (R4)	Area = 228.232 m^2	Reach 8 (R8)	Area = 317.667 m^2