Deltares

Investigation of PID controller performance in 1D DflowFM (D-hydro)

Introduction

This report focuses on the Tenryū river in Japan, which originates from Lake Suwa and discharges into the ocean at the city of Hamamatsu. More specifically, the case is focused on the reach between the towns of Hiraoka and Funagira. During the 20th century, several dams have been built along this river, which are operated by a Japanese power supplier. Over the reach considered, these dams along with reservoirs are located at Sakuma, Akiba and Funagira, ranked from upstream to downstream. Along these reservoirs, sediment is being accumulated, which especially causes problems downstream of the dams, such as coastal erosion at the river mouth. The client already partly takes care of the problem by extracting sediment from the reservoirs and by dumping it in the river downstream of the dams. However, this strategy is not optimal yet and for investigation of the problem, the client contacted Deltares for advice.

Deltares has already investigated several sediment management strategies using a 1D-model to investigate the behavior of the system. This has been done by implementing the software D-FlowFM, which uses a flexible mesh grid. However, the hydrodynamics of the system has not been adequately modeled yet, since the operational behavior of the dams is not known. Fortunately, some historical data is available in the form of inflow, spillway and generator discharges and reservoir water levels from the years 1998 up to 2007. The aim of this report is to find a way to model the operational behavior of the dams such that the behavior is as close to the observed behavior. In their turn these findings can be used to model additional scenarios for the dam behavior, in order to find the most optimal dam behavior in combination with the proposed sediment strategies.

Theory

For modeling the behavior of the three dams according to the observed data, several methods can be applied. One of these methods is the PID-rule, which controls an output parameter (i.e. a spillway gate opening) and uses different coefficients accounting for various effects. It is described by the following equations:

$$e(t) = x_{sp} - x(t)$$

$$f(t) = K_P e(t) + K_i \int_0^t e(\tau)\, d\tau + K_d \frac{de(t)}{dt}$$

In which:

- $e(t)$ represents the error between the setpoint (x_p) and the observed or modeled value ($x(t)$). In the Tenryū case, the values for x_{sp} will be the observed water level time series. The values for $x(t)$ will be the calculated values by the model. Thus, after every modeled time step, the error will be calculated.
- $f(t)$ is the output parameter (i.e. a weir crest level or a pump discharge).
- K_p, K_i and K_d are the respective gain factors. K_p is a factor that converts the error between the setpoint and observed value of the control parameter to the output parameter (i.e. weir crest

level). The K_i and K_d parameters account for the cumulative effect and the rapid change of the parameter respectively.

When discretizing this set of equations for modeling, the following formulations are obtained:

$$e^n = x_{sp} - x^n$$

$$f^n = f^{n-1} + K_p(e^n - e^{n-1}) + K_i \Delta t_n e^n + K_d \frac{e^n - 2e^{n-1} - e^{n-2}}{\Delta t}$$

The factors K_P, K_i and K_d are not known beforehand and thus will have to be estimated. To get a sense for the sign and magnitude of these parameters, equations are investigated that give relations between discharges, water levels and gate openings. Since the spillway gate will have adjustable gates, the following relations apply:

$$Q = \begin{Bmatrix} B\,\mu\,d_g\sqrt{2g(h_{up} - z_s - \mu\,d_g)}, & if\ h_{down} < z_s + d_g \\ B\,\mu\,d_g\sqrt{2g(h_{up} - h_{down})}, & otherwise \end{Bmatrix}$$

Since all three dams have downstream water levels that are lower than the crest level, $h_{down} < z_s + d_g$, the upper relation applies. As can be derived from the equation, the gate opening height should increase when h_{up} increases, in order to keep the water level from rising any further.

Using this knowledge, the sign for K_p can be found. Neglecting the K_i and K_d terms and using the same setpoint at two consecutive timesteps yields the following expression:

$$f^n = f^{n-1} + K_p(x^{n-1} - x^n)$$

As pointed out before, the gate opening should increase when the water level increases to keep the water at setpoint level. Let's say f is the gate opening height, x is the water level and that the reservoir water level rises over time, so $x^n > x^{n-1}$. In this case, f^n should be larger than f^{n-1} since more excess water needs to be spilled. Hence, K_p should be negative in order to make f^n larger than f^{n-1}.

Since the cumulative integral of the error is quite large (due to the large timestep of 86400 seconds), K_i should be relatively small compared to K_p. The differential effect also depends on the magnitude of the time step. The differential part is small due to the large timestep and thus K_d should be relatively large to make its effect noteworthy.

Methodology

The real case consists of the Tenryū river with the three dams. The behavior of the river is simulated in D-FlowFM using the Deltares Integrated Model Runner (DIMR). Data about reservoir levels and generator discharges is available for the years 1998-2007. To find the calibration constants for the dams (K_P, K_I and K_D), the following plan is followed, according to the D-RTC user manual.

- K_i and K_d are set to zero, while the value of K_p will be gradually increased until the solution starts to oscillate. The sign of K_p depends on the output of the control parameter. Hence the sign of K_p should be positive.
- When the solution of K_p starts oscillating, K_p should be divided in half and K_i should be gradually increased, until the solution starts to oscillate again.
- Finally, K_d should be increased until the solution matches the desired water level.

Generally, this plan is followed, however, to achieve the most optimal fit additional simulations are done to see what the effect will be of different K_p, K_i and K_d configurations.

Since the period considered is quite long (10 years), the calibration is done over shorter periods that can be modeled in a shorter time span. In this way, calibration constants can be adjusted quickly enough to achieve a desirable result. When a desirable result is obtained (i.e. an error of +/- 10 cm at max compared to the observed water levels), the simulation time is upscaled. Finally, some tweaks are done to keep water levels in between the error range.

It is also necessary to notice that the controlled water levels of the upstream dam (Sakuma) will influence the modeled water levels and discharges of the other dams. Hence, calibration will be done beginning at the upstream dam.

To quickly assess whether a simulation is good or not, the standard deviation of the error between observed and modeled water levels is calculated, which is defined as:

$$\sigma = \sqrt{E[e^2] - (E[e])^2}$$

With:

$$e = x_{modeled} - x_{observed}$$

The closer this deviation is to zero, generally the simulation will qualify as good. However, a downside of this approach is that values that have a certain offset, but are still relatively close to each other, the standard deviation will also be relatively low. Thus, the simulation will also qualify as good. This approach should thus be handled with caution.

Post-processing of the results is done by using MATLAB scripts.

Results & discussion

In the timespan set for this project, it was not possible to converge to the error range that was desired. Especially the calibration of the Sakuma dam behavior gave difficulties due to its highly varying reservoir water level. Hence, it was decided to calibrate the other dams (Akiba and Funagira) regardless, such that water levels and discharges in the downstream reaches would be correct. However, the Akiba dam also took many simulations to converge, and thus, simulations have been stopped. In total, 456 simulations are done in order to find the right parameters which are all documented in overview files, along with their calculated standard deviations.

The observed water levels for all three dams along with the modeled water levels are displayed in Figure 1. In this figure, the simulations that were found to be the best are used for displaying the modeled water level.

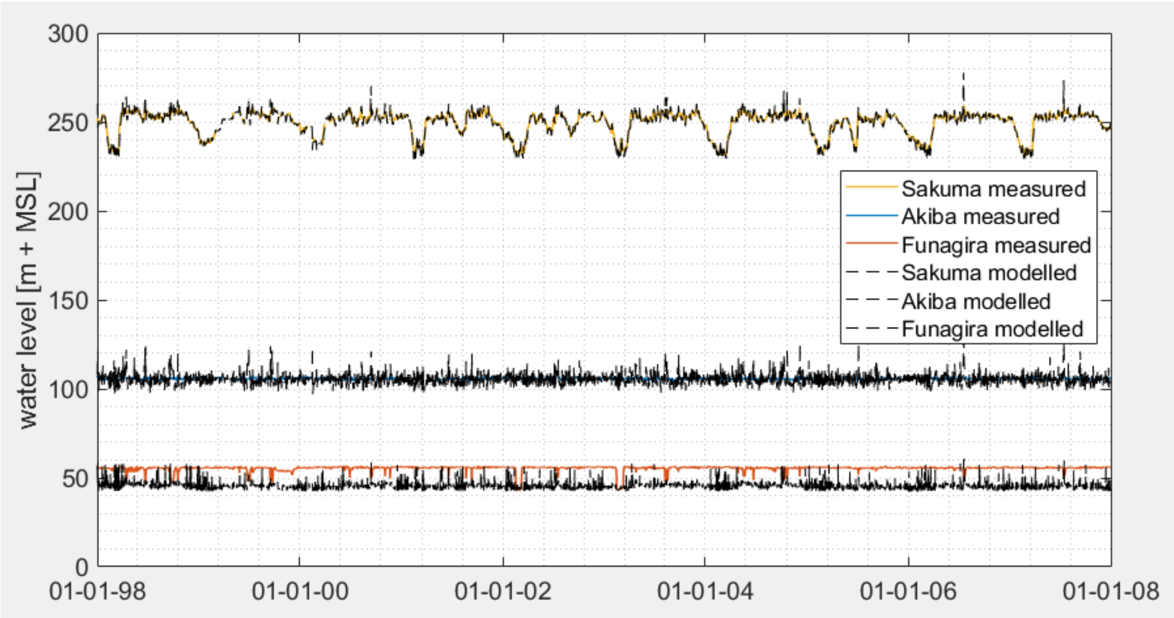

Figure 1: Modeled water levels by using the PID-controller

At first glance, the modeled water level at Sakuma seems to be following the observed water level relatively well. For Akiba, the solution seems grassy. For Funagira, the modeled water levels are off since simulations were halted. For illustrational purposes, in Figure 2, a zoomed in random section of Figure 1 is displayed for both the Sakuma dam and the Akiba dam.

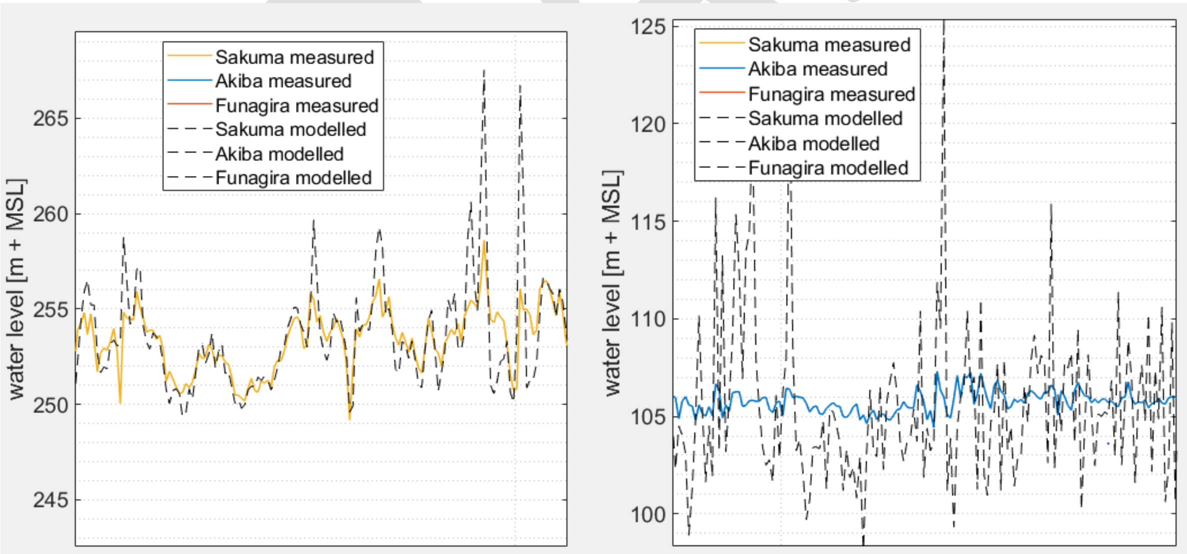

Figure 2: Zoomed in sections of the Sakuma water level (left) and the Akiba water level (right)

As can be seen, for the Sakuma dam, the modeled water level follows the measured water level relatively well, however, still errors could be observed of 5 meters or more. For the Akiba dam, the situation is worse, since it is difficult to observe whether the modeled water levels follow the trend and water levels are generally far off. The observed errors for all dams are displayed in Figure 3.

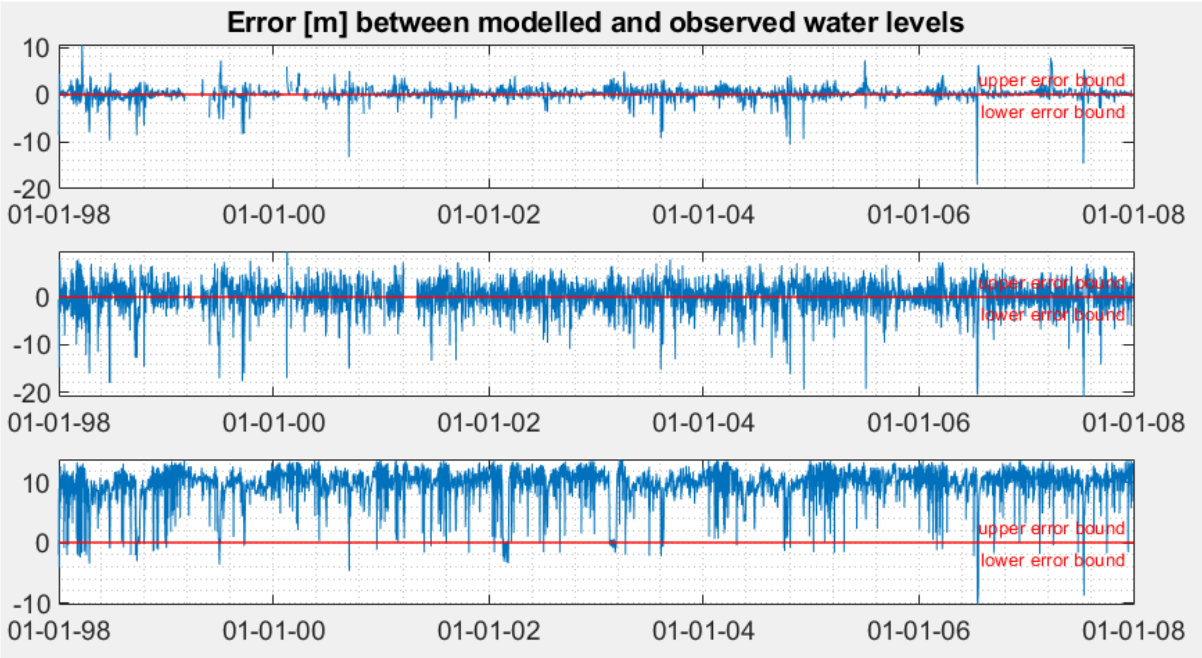

Figure 3: Error between modeled and observed water level for all three dams

In this graph, the upper and lower error bounds are displayed as red lines. It can be observed that none of these simulations lie in between these bounds, which was the goal of this project. Hence, additional calibration should be done in the future.

In Table 1, the used K_p, K_i and K_d values for the dams are displayed.

	Sakuma	Akiba	Funagira
$\boldsymbol{K_p}$	-0.035	-0.0325	-0.075
$\boldsymbol{K_i}$	-0.000002	-0.00000012	0
$\boldsymbol{K_p}$	-20	0	0

Table 1: Most optimal PID-parameters, as found at the end of the project

Structure behavior

A different way of finding the operational behavior of the dams was by simply using the structure formula from the Theory-section. With this formula, the gate opening heights can be predicted by using the data from 1998-2007 in order to find the operational behavior of the dams. After this, a comparison is made between the gate openings predicted by the formula and the gate openings modeled by using the PID-controller, which is displayed in Figure 4 for the year 2007.

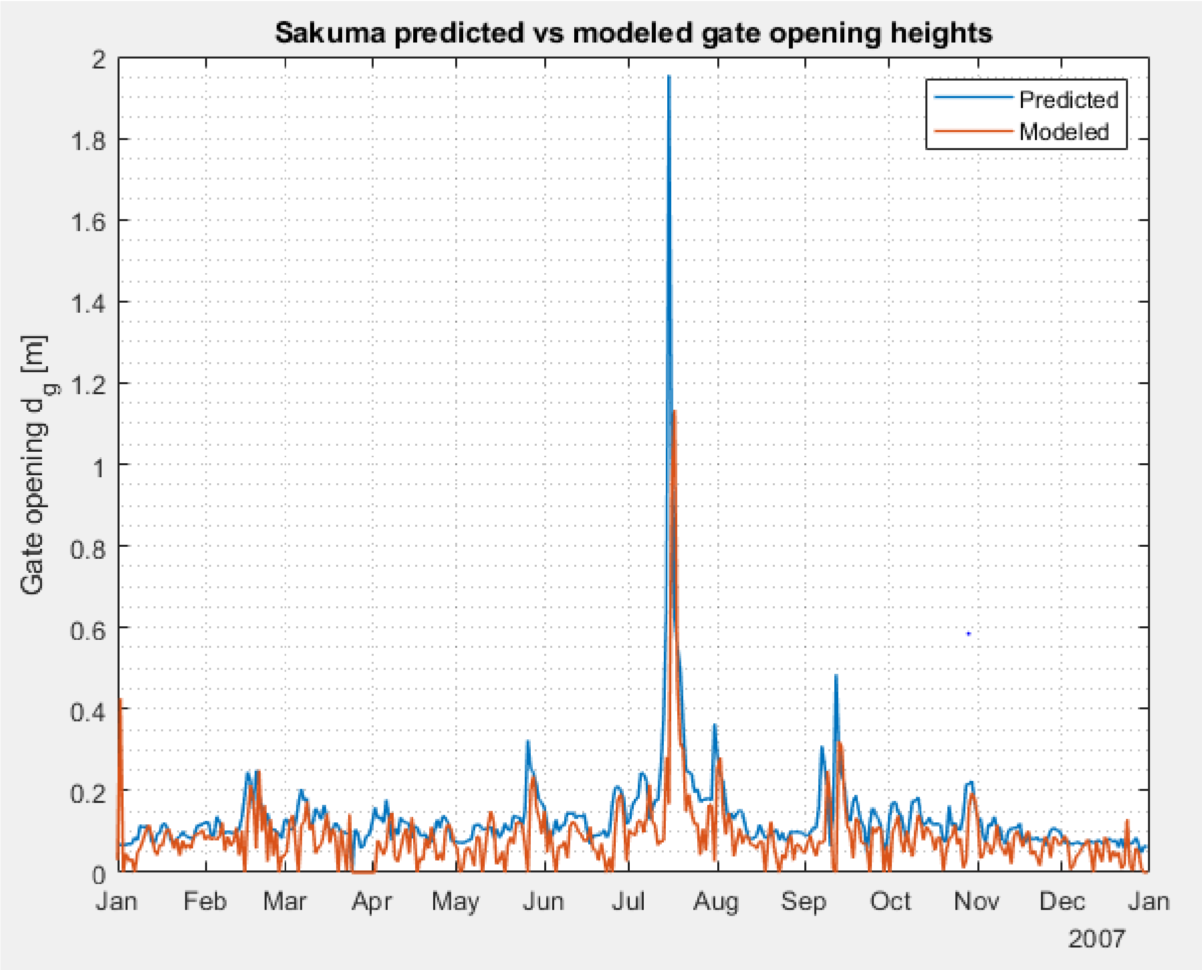

Figure 4: Comparison between the predicted and PID-controlled gate opening height for 2007.

As can be observed from the graph, the PID-controller seems to underestimate the predicted structural behavior slightly when compared to the predicted behavior. It can also be observed that the gate opening height modeled by the PID-controller shows slightly more fluctuations than the predicted gate opening height.

The predicted behavior can also be implemented in the model by using a time-rule, which uses a time series of these gate openings to control the dams. The results of this implementation for the year 2007 are shown in Figure 4.

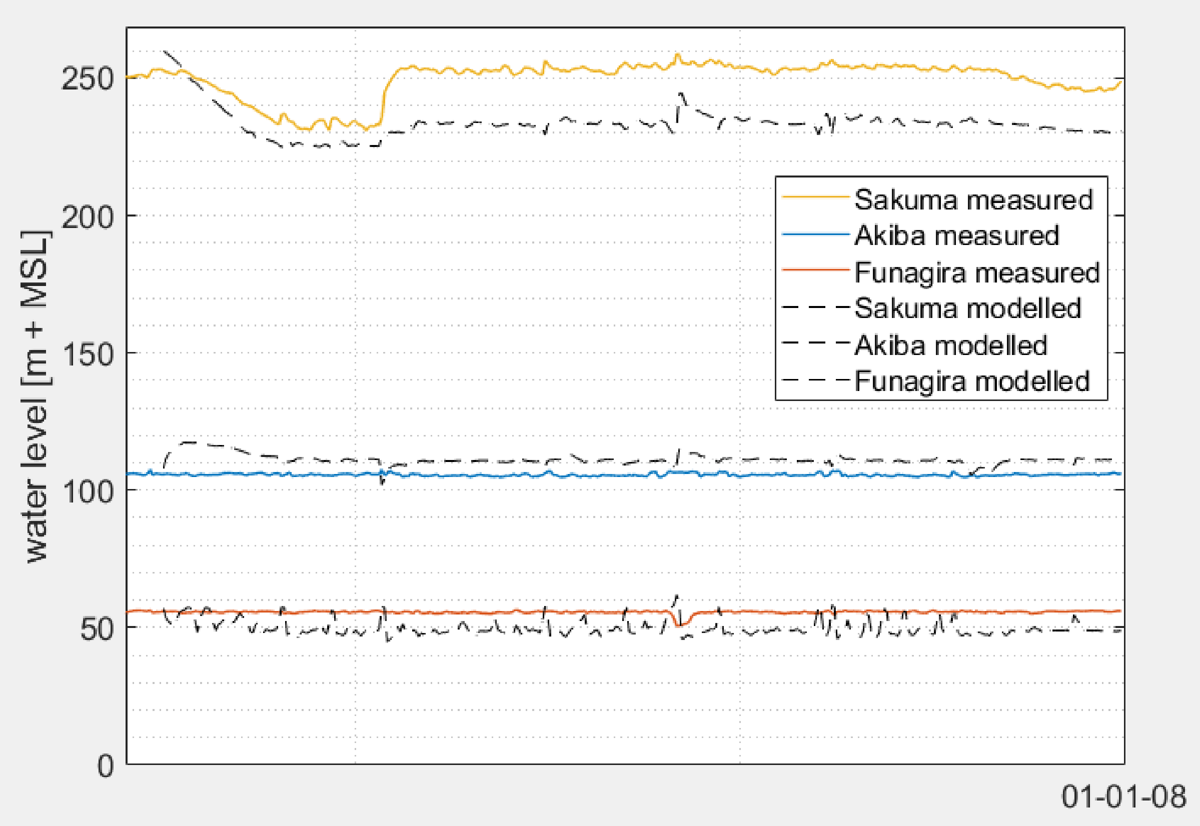

Figure 5: Observed and modeled water levels by implementing the predicted gate openings as a time-rule

As observed from Figure 5, the predicted water levels by using the time-rule are not accurate at all, implying that the structural behavior cannot be modeled by simply implementing the predicted gate openings by using the hydraulic structure relation. However, as stated before, the operational behavior of the Sakuma dam influences the behavior of the downstream dams, thus, it cannot be concluded whether this method of modeling is wrong for these dams. When comparing the time-rule with the PID-rule, the PID controller gave a better result for the Sakuma dam, as observed in Figures 1 and 2. For the other dams this should be investigated when the Sakuma dam hydrodynamics are calibrated properly due to the reasons mentioned before.

Conclusion & recommendations

When calibrating the reservoir water levels, generally two problems arose that continuously kept calibration from converging to the desired results:

- The first problem encountered for the Sakuma dam was the reservoir filling speed. The observed reservoir filling time was found to be quicker than the filling time that can be modeled by adopting the PID-controller. Prone to filling, the PID-controller closed the gate, which remained closed for the filling time, however the observed water level still rose quicker than could be modeled. This caused a large error between modeled and observed water levels that could not be eliminated.

- The second problem arose with the flood discharge peaks. The PID-controller tends to overshoot these peaks, as the large value in the discharge boundary data could not adequately be damped by the controller.

Since it was not possible to converge to the desired outcome (i.e. an error range of about 10 cm) in the timespan for this project, recommendations are made to converge to this outcome for future investigations. This implies for the several dams:

- Especially for Sakuma, when trying to find optimal combinations of K_p, K_i and K_d, it helped by adopting a matrix in which different combinations of these parameters are displayed. An example of this is displayed in Figure 6. The standard deviation of the error between observed water level and modeled water level are shown for different parameter combinations as a measure of how good a simulation is. The smaller this value is, the better the simulation.

Worksheet for finding optimal Kp-Ki-combination Sakuma										
		Kp								
		0.02	-0.0225	-0.025	-0.0275	-0.03	-0.0325	-0.035	-0.0375	-0.04
Ki	-1.00E-07	2.1827				1.8102			1.704	
	-1.50E-07		1.951	1.8638	1.8315			1.6786	1.6633	
	-2.00E-07	1.9721			1.7515	1.7101	1.6801	1.6611	1.6674	
	-2.50E-07		1.8303	1.7705	1.7259	1.6936	1.6732	1.6726	1.7021	
	-3.00E-07	1.8681	1.8027	1.7533	1.7164	1.6928	1.6934	1.7093	1.7668	
	-3.50E-07		1.7889	1.747	1.7205	1.7222	1.7354	1.7652		
	-4.00E-07	1.8336	1.7842	1.7539	1.7574	1.7574	1.7741			
	-4.50E-07		1.7918	1.7975	1.7887					
	-5.00E-07	1.8329	1.8393	1.8315	1.8463	1.8904				
	5.50E-07									
	6.00E-07									
	6.50E-07									
	7.00E-07									

Figure 6: Example of a table for finding the optimal Kp-Ki-combination

- For Sakuma, an arbitrary crest height (220 m) is used to model the gate behavior, well below the lowest observed water level in the reservoir, to make sure water would always be able to flow through the structure. In reality, the crest height of the spillways for this dam is a lot higher. For both Akiba and Funagira, a crest height of 94,5 m and 42 m are used respectively, according to the real-life situation.
- Especially for the Akiba dam, the approach to quickly assess results using the standard deviation of the error should be handled with caution, since for this dam, the seemingly best solutions (having the lowest standard deviation of the error) were in fact quite far off. Hence, it's always a good thing to additionally investigate the output by means of graphs to get a sense for what is happening.
- Increasing the K_d was generally found to smoothen/dampen out the solution. However, increasing the K_d value too much gave an oscillatory behavior of the modeled water level. Furthermore, for the Sakuma dam, K_d was found not to have a major effect on the outcome. Thus, overshoots of 2 meter (and the flood waves) could not be eliminated by increasing K_d. Hence, as a recommendation, the solution should be sought in finding an optimal K_p-K_i-combination, before increasing K_d. It should be noted that the optimal K_d parameter varies for various K_p-K_i-combinations, as well as how much it affects the solution.

- During modeling, generally a warning message was found in the model log-files that the crest widths of both the Akiba dam and the Funagira dam were changed into a decreased value. This problem has to do with the fact that flows may be too low and prevents values as 1 mm water depth on crest of the structure (which is the case in this project). Whether these crest widths persist during the entire modeling operation is not known, however, for future modeling, this should be considered. A way to solve this is by making the crest widths smaller such that the spillway, which normally consists of multiple gates, is representative for just one of these gates, say 20 meters. Hence, water levels on the top of the crest will be larger and more representative.

References

D-RTC user manual
RTC-Tools manual