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Hydrological modelling of a recycling facility and landfill

A parsimonious approach

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Abstract

A parsimonious lumped model for simulating a catchment composed of a landfill and recycling facility were developed. The modelling results showed an acceptable conformity with the observed values with NSE of 0.54 and R^2 of 0.56 in hourly simulations, though the results were characterized by a general underestimation. The results of the daily accumulated values showed a greater compliance with the observed values with NSE of 0.69 and R^2 of 0.74, indicating that similar simulations of a dynamically complex catchments can be developed in daily timesteps with a great confidence. The catchment model has been coupled with a reservoir model to simulate the leachate storage ponds downstream the catchment. The time simulated storage of the leachate ponds was aligning with the observed recordings with NSE of 0.96. Simulation of the expected climate change were made to deduce the required expansion of the ponds at the future climate and the required additional volumes were outlined. It is recommended to optimize the leachate ponds storage by improving the treatment plant capacity, or by continuous inflow/outflow control. The future covering of the landfill and subsequent impact on the surrounding area were modelled being a part of the facility future plan for climate adaptation.

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1. Introduction

1.1. Background

As a result of the current environmental laws in regard to municipal solid waste (MSW) management and landfilling practices, many recycling facilities are facing a challenge in assessing the environmental impact arising from their recycling and landfilling activities. The European Union directive on the landfilling of MSW (European Council, 1999), have ruled that a restriction of MSW landfilling is required and set the rules for monitoring the environmental impact from the existing recycling facilities and landfills. The detailed environmental impact assessments for the recycling facilities and landfills are the responsibility of each country and its responsible organizations.

In Sweden, Länsstyrelsen is the governmental body responsible for issuing permits to the recycling facilities and landfills. For each facility, the compliance with Länsstyrelsen rules and conditions is crucial for ensuring renewal of the permits required for recycling and dumping activities. One of the major requirements for permit renewals is that leachate water discharged from the hazardous waste landfills must be collected and stored separately from other water and checked while waiting for final disposal (Nårab Miljörapport, 2020).

The current shift in the global climate and the witnessed consequences of global warming and the subsequent precipitation increase, have made it more critical for recycling facilities to cope with the already changing climate. Subsequently, a paradigm shift in thinking about the expected large and extreme events in precipitation, made it crucial for such facilities to prepare and conduct hydrological assessments for their existing infrastructures, and make plans for future expansion as well.

1.2. Aim

This work generally aims to map the runoff from the various surfaces and make a calculation of the water balance for the entire area, by constructing a computer model with a lumped representation of the catchment with a parsimonious modelling approach. The model is required to be tested against its ability to model a complex semi-urban catchment with a limited set of parameters, and to check its robustness for modelling future climate change events.

The main aim of this work is to model the facility surface water to check the suitability of the existing system during large and extreme rainfall events that may happen at the current and future climate.

The second aim is to study the effect of covering the old landfill and calculate the required amount to be accommodated in the future after closing the landfill. Finally, this work aims to serve as a guidance to the future attempts to approach similar hydrological problems with a parsimonious approach, and to give insights about the possibilities and limitations of surface water modelling.

2. Site description

Norra Åsbo Renhållnings AB, also called Nårab, is a municipal company that handles the cleaning in the municipalities of Klippan, Perstorp and Örkelljunga towns in Sweden southern state, Scania. The company takes care of everything within the collection of waste from households and industries, in addition to the work at the waste facility in Hyllstofta and the management of recycling centers in the area. Nårab waste facility including the landfill will be hereinafter referred to as “the facility”. Figure 1.1. shows an aerial photo of the facility.

Figure 2.1. Aerial photo of the facility (SCALGO LIVE)

The facility in Hyllstofta consists of several different surfaces for handling the waste such as the old landfill, recycling collection, composting area, bio-cells, etc. All of the areas are connected to Nårab's treatment plant, where leachate and surface water from the entire area flows for treatment before it is used for irrigation of surrounding forest areas or pumped to Klippan wastewater treatment plant. The facility treatment plant is old and dimensioned for a smaller area than what exists at its disposal today, and therefore a new assessment of the capacity for the treatment plant is needed to be carried out in the current and future climate.

Figure 2.2. Leachate storage ponds and the treatment plant (top left)

The old landfill which constitutes a major part of the landfill (Figure 2.3.), in which the dumping activities is ceased, and the upper surface is currently composed of permeable material (i.e., gravel). The old landfill is on the process of closing as a future plan by covering the upper surface with a low to non-permeable surface. The proposed covering of the landfill will affect the water cycle within the facility.

Figure 2.3. The old landfill (decommissioned) with the current permeable cover

The facility is composed of different surfaces where surface water is handled in a complex way to prevent the contamination of the surrounding environment by leachate from the landfill and the waste management procedures.

The main goal is to prevent mixing of reject surface water and leachate, in which the water quality is above the required limits, with the surface water from considerably clean surfaces in which the water quality is below the required limits.

Waste management activities are performed at the facility in separate areas for hazardous materials, where each area has its own drainage path and joins the main underground network at a certain point. Drainage is done by stormwater gullies, surface water channels coupled to manholes, and underground perforated drains (Figure 2.4.).

Figure 2.4. Drainage from a storage area within the facility

At the same time, leachate water from the underground drainage pipes below/around the old landfill area is directed towards the leachate collection ponds (Figure 2.2.), where it joins the reject surface water from the hazardous areas, then towards the treatment plant within the facility itself, where the treated water is disposed by infiltration, irrigation to the surrounding area around the facility or pumped to Klippan wastewater treatment plant depending on the effluent quality.

Runoff from asphalted surfaces that is not intended for recycling activities (ex. Office area) is considered to be at a higher quality than the leachate water and therefore mixing of both waters is avoided. The mentioned stormwater is directed to stormwater ponds and the surrounding area through overland flow at designated locations. The stormwater runoff from such surfaces is beyond the scope of this work and it will not be addressed. A schematic diagram of the drainage paths is outlined in Figure 2.5.

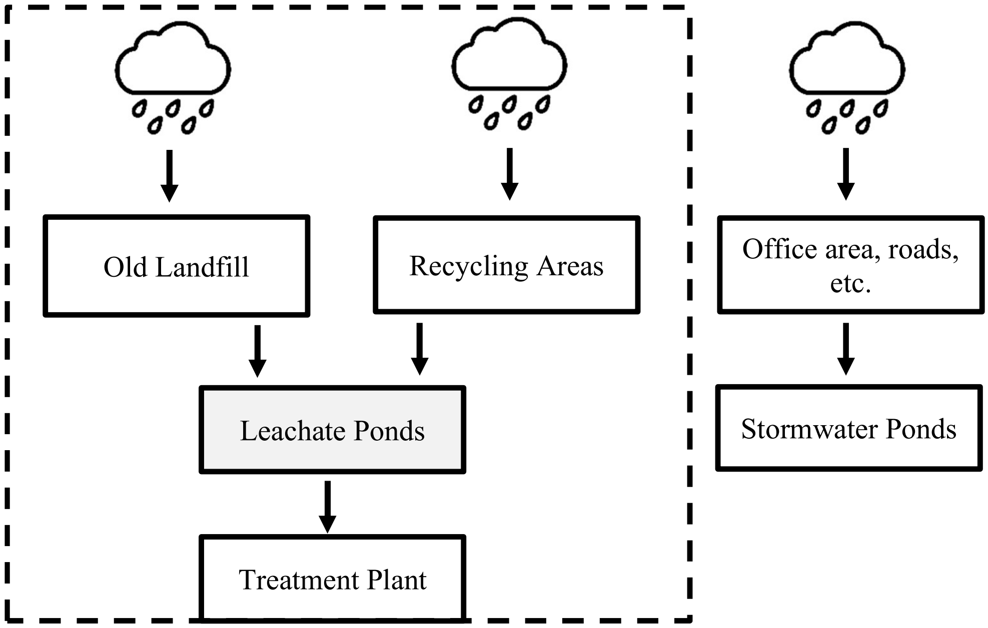

Figure 2.5. Schematic diagram for the flows within the facility outlining the scope of this paper (dashed line).

A water balance is required to be established to ensure meeting the required capacity of the leachate ponds, taking into consideration the maximum capacity of the treatment plant. The future closing of the landfill with non-permeable surface will affect the water cycle and induce a more rapid overland flow from the landfill surface, which is required to be accommodated. The leachate water from below the landfill is expected to continue due to the age of the landfill starting from its commissioning in 1975, as noted by Bengtsson et. al (1994) studying similar old landfills.

The whole assessment is required to be checked against large and extreme rainfall events that is likely to happen in the current and future climate.

3. Theory

Hydrological models for simulating rainfall runoff have been widely used in engineering practice, and many researchers have been keen on predicting different catchment responses by using computer models. In essence, computer models represent the behavior of the catchment by solving mathematical relationships. The level of detail can vary from simple conceptual models to complex hydraulic models (Zoppou, 2001).

The composition of a catchment and the amount of available data usually dictates the type of the model used for simulations. There is a growing trend to use conceptual models for hydrological modelling due to frequent data limitations, and that it provides the advantage of simulating complex soil processes by a simplified relationships with fewer parameters (Willems, 2014). The desired outcome of the model affects the model choice as well. Parsimonious conceptual models emerged as it allows flexibility in changing the model structure from predefined models depending on the catchment in question (Willems, 2014), and that it serves the purpose of the model when the final outflow from the catchment is the main study focus (Coutu et al., 2012).

The approach in this work was by the utilization of a numerical model with a lumped representation of the catchment as an alternative approach to the detailed network modelling (Coutu et. al, 2012). Such simplified approach will allow the simulation of the flows within the landfill with the advantage of decreasing model parameters and computation time.

The main limitation of adopting a conceptual approach is that prior to any input; the structure of the model must be specified, and that usually exits some parameters does not have a physical meaning (Zoppou, 2001; Wagener et al., 2002). This means that the outcome of the model will be greatly dependent on the system understanding by the modeler. In addition, over parametrization and lack of parameter identifiability, represents additional limitation to conceptual models; where combining different set of parameters or different model structures, can produce the same result with each combination (Johnston and Pilgrim, 1976; Uhlenbrock et al., 1999; Wagener et al., 2002).

An urban catchment is different in response to a semi-urbanized or rural catchment. Detailed hydrological modelling of an urban catchment is normally adopted because the mathematical relationships that represents the flow within the catchment can be represented by contemporary hydrology, and the degree of available data is usually more due to available infrastructure data and available utility drawings that was made at the time of construction. This is not normally the case of a semi-urban or rural catchment, due to the complexity of flow processes happening in the soil and the underground layers, and the shortage of reliable data. The case study outlined in this work can be regarded as a semi-urban catchment.

Adding to the complexity, the case considered in this work is composed of a semi-urban catchment where a large section of the area is a municipal solid waste (MSW) landfill and dumping area. The water balance method is normally the adopted technique for approaching hydrological modelling through landfills (Bengtsson et al., 1994; Johnson et al. 2001; Marques and Hogland, 2003), and there exits several modelling techniques for landfills such as the commonly used Hydrologic Evaluation of Landfill Performance (HELP) model (Marques and Hogland, 2003; Berger, 2015; Broichsitter et al., 2018). Several other software noted by Hogland et al. (2003) exists such as PREFLO, MOBYDEC and FILL that can be used for hydrological simulation of landfills. Broichsitter et al. (2018) noted the possibility of using HYDRUS and FEFLOW models to model soil water flow processes normally expected in a landfill. Nonetheless, there were attempts to simplify the flow through landfills to a greater extent by adopting numerical water budget formulas that is based on continuity (Hogland et al., 2003).

Nonetheless, modelling of landfills is normally initiated factoring only precipitation and retained leachate water as the input to the model (Marques and Hogland, 2003). Most of such studies did not factor any additional flows that may contribute to the total flow such as surface and sub-surface flows from the surrounding area, which is the current case in this work. In this case study, the observed flow data from the facility is the total flow from the landfill leachate production and the surrounding area drainage, and there is no sound procedure to isolate each flow due to the existence of one reliable flow meter at the end of the flow line representing the total flow. The objective of this work was also aiming to study the system integrity at the large and extreme flow events, and not in any means aiming to model the flow through the landfill in detail. Thus, it was regarded that a parsimonious lumped model where the landfill represents only a fraction of the flow, is deemed acceptable considering the level of detail required, the available data and resources.

The issue of overparameterization of conceptual models is usually a limitation to the practicality and robustness of any model. In a previous work by Perrin et al. (2003) studying different model configurations, it was noted that a four-parameter model is the optimum complexity in relation to the model output, and that adding additional parameters to a model does not significantly improve the model outcome. Jakeman (1993) also noted the possibilities of using a four-parameter model with two flow components distinctions composed of “quick” and “slow” flow. Subsequently, the choice of the model in this work started from a basic reservoir configuration for semi urban catchment by Coutu et al. (2012), where the model is “directed” towards a distinction of the reservoirs to fall on the “quick” or “slow” categories.

The next section outlines the chosen model in detail and discuss the scientific base behind the model components.

4. Methodology

The approach was to simulate the flow through the catchment by adopting a parsimonious lumped model of storage reservoirs, in which the time-dependent storage of each reservoir is influencing the outflow from the reservoir, which subsequently aimed to simulate the delay of flow within the catchment.

In order to decide on the degree of complexity of the lumped model, a study of the catchment was performed to account for the expected flow types and the degree of influence from the existing infrastructure. The available data did influence the model construction in a great degree. The following sections outline the available data for the model construction.

4.1. Available data

4.1.1. Observed flow

The approach to the model was depending on the available data to calibrate against. There exist several flow meters in the facility where the only reliable data to refer to is the main flow meter downstream the main pump station. The recorded (observed) flow from this flow meter was by automated logging of the flow and recordings were being made with a timestep of 1 hr.

The balance flow meters within the facility (upstream the main pump station) are recorded manually by the facility personnel and on monthly basis, yet there exist several discrepancies in the flow records. Thus, the balance flow meters served only the purpose of validating the observed flow downstream the main pump station.

The obtained records from the main flow meter were from the period of 2019-01-01 to 2021-11-07 (approx. 3 years), with 1 hr. timestep. Thus, the obtained records were a flow value in cubic meter on hourly basis for 25005 timesteps.

The integrity of the observed flow data was checked beforehand, and there exists only six separate missing time steps (6 separate hours) that was solved by interpolation between the previous and the next timesteps.

4.1.2. Precipitation data

The precipitation data was obtained from in-house precipitation gauges within the facility. The obtained data was on hourly basis (similar to the observed flow). Thus, the data was composed of a precipitation timeseries in millimeter with timestep of 1 hr. and 25005 timesteps.

Due to usual discrepancies in precipitation values that arise from measurements taken from one gauge (Villarini et al., 2008); the data received from the facility was checked against the Swedish Meteorological and Hydrological Institute (SMHI) field data from a nearby station located in Helsingborg (Station A), with the same hourly timestep as the facility observations. A comparison between the number of zero-value timestep was made to check for the degree of missing data. It was deduced that the precipitation timeseries from SMHI contained more zero-value timesteps (7972, in year 2020) compared to the data received from the facility (7689, in year 2020). Thus, validating the data received from the facility. It is roughly shown in the below Figure 4.1. that the extremes of the precipitation received from the facility is aligning with the extremes of the observed flow data. Thus, the data received from Nårab is the one used in the model thereafter.

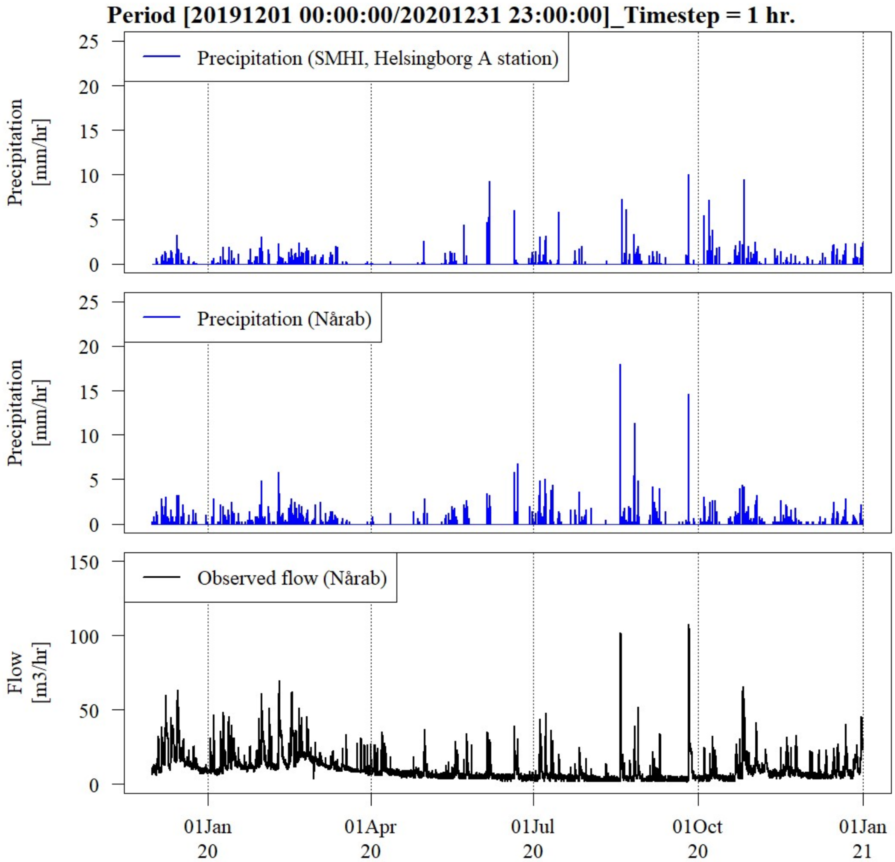

Figure 4.1. Comparison between precipitation data from SMHI and Nårab, showing the observed flow data (Nårab) as a reference between the period of 2019-12-01 to 2020-12-31

4.1.3. Temperature

Air temperature is an additional meteorological that considered in this model, being an active parameter in the Catchment Wetness Index (CWI) equation, accounting for the soil moisture, which will be explained in the following Section 4.2.2.

Since there were no in-house temperature measurements in the facility, the model was supplemented by a temperature timeseries adopted from SMHI online accessed database, measured in the closest station to the facility (42 km) located in Helsingborg city (Station A). The acquired temperature timeseries were on hourly basis for the same period of the observed flow mentioned in Section 4.1.1. which summarized as a temperature timeseries in degree Celsius with hourly basis for 25005 timesteps.

4.1.4. Catchment composition

A subsequent mapping of the area contributing to the observed flow has been performed through field observations, document studies, and by contacting with the facility personnel. The main aim was to locate the flow that is contributing to runoff. The following Figure 4.2. shows the catchment outlining the area contributing to the observed flow records.

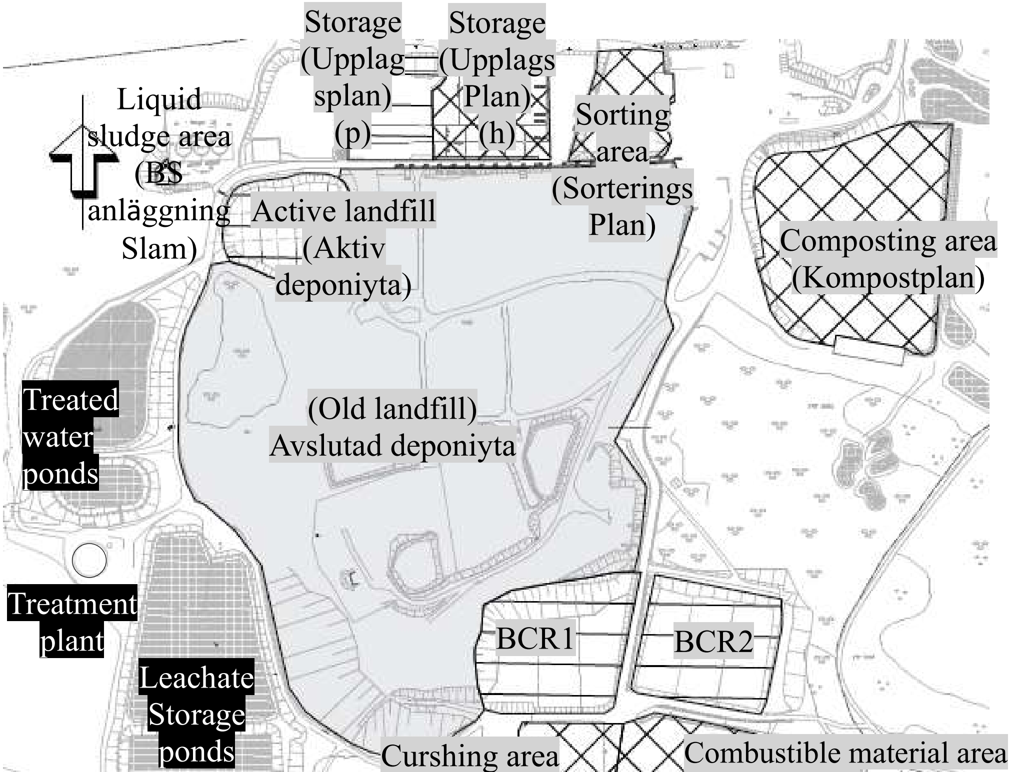

Figure 4.2. Catchment area contributing to the flow (Hard surfaces: cross hatch, Permeable surfaces: horizontal hatch, and the old landfill in transparent grey hatch)

By analyzing the area contributing to the observed flow it is deduced that there exist three distinct differences between each area group as following:

1. Hard (asphalted) surfaces; where drainage of the rainfall runoff is performed by surface flow sloping towards gullies, leading to manholes and subsequent pipes/pumps reaching the main pump station.

2. Permeable (soiled) surfaces; where drainage of the runoff is done mainly by underground perforated pipes, and in some cases surrounding surface ditches following the slope of the area. Both drainage paths are leading to manholes and subsequent pipes/pumps reaching the main pump station.

3. The old landfill; where the drainage is done mainly by percolation of runoff that is mixed with the leachate water and drained by underground perforated pipes, leading to manholes and subsequent pipes/pumps reaching the main pump station.

A schematic diagram of the flow lines from each area is outlined in the following Figure 4.3. It is shown that there exist several structures between each area and the main pump station, which influences the outflow from the catchment. The distinction of each flow types is noted in both figures.

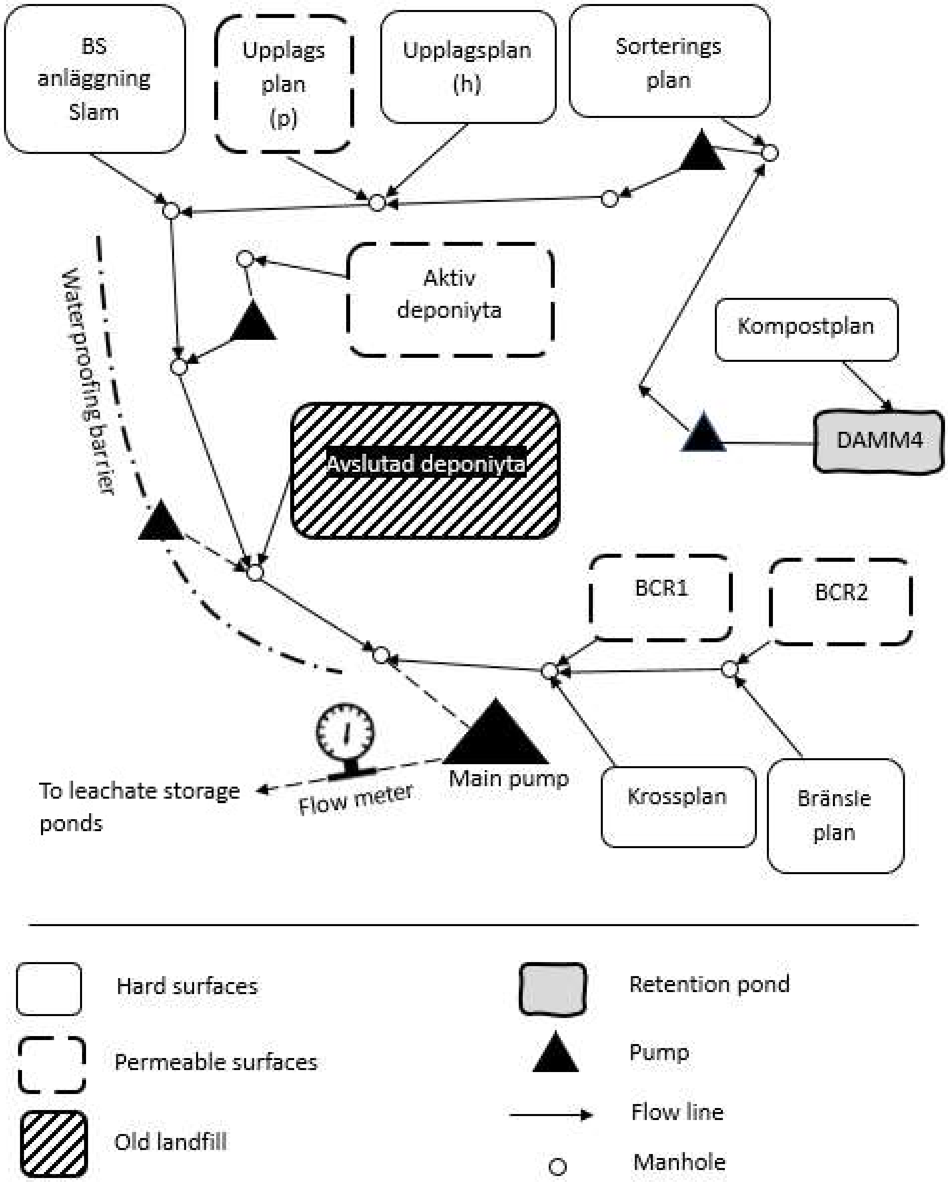

Figure 4.3. Schematic diagram of the flow lines from each area

Some elements must be noted in regard to the above distribution:

- There exist several pumps on flow lines emerging from each area group, which influence the observed flow.

- One particular area lies in the hard surface group designated for dumping and sorting of composted materials (Kompostplan). The drainage from this area flows to a nearby storage pond (Damm 4) and water is being pumped from such pond to the main drainage line leading to the main pump station. It is expected that the runoff from this area is attenuated by the storage within the pond.

- There exists a waterproofing barrier west of the old landfill, between the old landfill and the clean water ponds (Rendamm 1&2), where underground water is being pumped towards the main pump station in order to avoid seepage of such water to the clean water ponds. There is no existing procedure to isolate this flow from the observed flow, as the flow records downstream the pump is on monthly basis and there exists several missing data in the records. Such addition to the observed flow is considered to be captured in the flow from the old landfill, though it is expected that delay of flow from the landfill is considerably greater and that pumping of underground water will result in distinguishable peaks in the incoming flow.

- Since the model is area dependent, a topographical study of the facility has been performed by Autodesk Civil 3D and SCALGO LIVE software to map the roads sloping towards each area, and the addition of certain road areas has been done to reflect the increase of surface area contributing to the runoff.

- A thorough study of the underground infrastructure has been done to ensure that the chosen areas contributing to the observed flow are the only areas contributing to the flow (Figure 4.3.). Any discrepancies in the received drawings from the facility will affect the model outcome to a certain extent depending on the degree of addition/omission of contributing areas.

Thus, three distinctive flow types have been developed in the model, as following:

1. Type h, flow from hard surfaces; characterized by a comparingly quicker flow.
2. Type p, flow from permeable surfaces; assumed slower flow comparing to the flow from hard surfaces (Type h).
3. Type d, flow from the old landfill; assumed a more delayed flow than both the flows from hard and permeable surfaces (Type h & p)

The catchment area of each flow type has been calculated and is summarized in the following table:

Table 4.1. Catchment area for each flow type

Flow type	Area (m^2)
h	42,580
p	31,525
d	105,184
Total	179,289

4.2. Model construction

The catchment is modelled as a set of three flow lines where each flow line is composed of subsequent routing reservoirs representing the three flow types (Figure 4.4.), in which the flow from precipitation represents the input, and the output from each flow line is dependent on the storage in each reservoir. For each flow line, three non-linear reservoirs were used to simulate the flow. Figure 4.4. shows a schematic diagram for the model, indicating the routing reservoirs for each flow type.

Precedent to the routing reservoirs, a snow and soil model were developed to account for snow melt routines and soil moisture processes, respectively.

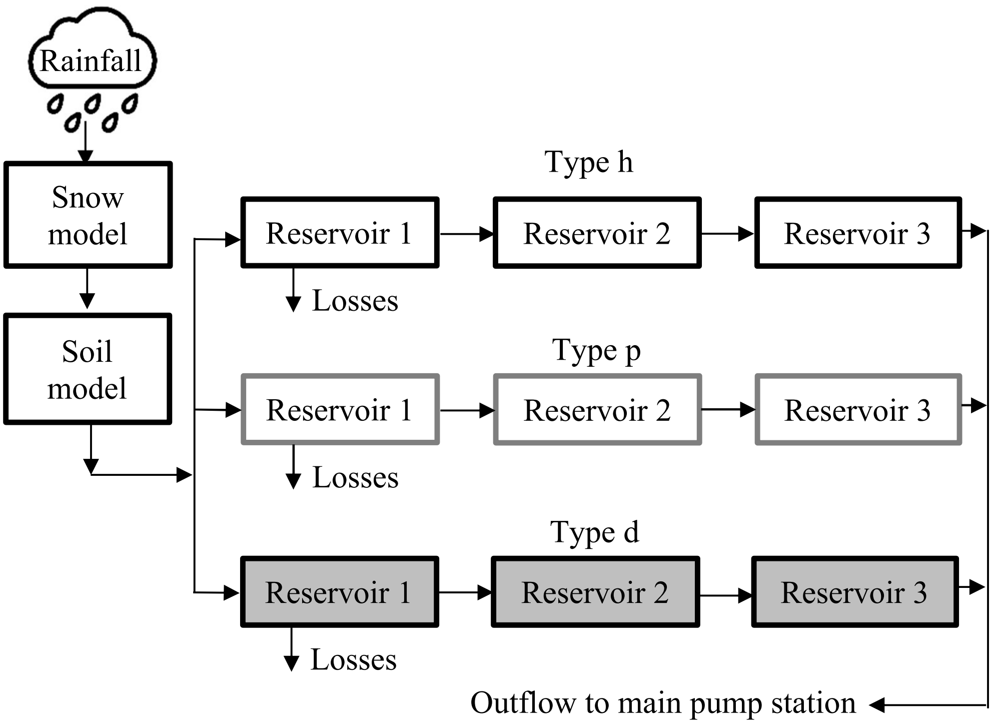

Figure 4.4. Schematic diagram of the main model

The general equation for the flow from each reservoir (Wagener et al., 2002) were as following:

$$\frac{dS}{dt} = Q_{[i+1]} - Q_{[i]} \tag{1}$$

And

$$Q = aS^n \tag{2}$$

Where $Q_{[i]}$ and $Q_{[i+1]}$ is the outflow from the reservoir at timestep i and $i+1$, S is reservoir storage, a is reservoir discharge rate, and n is the discharge linearity.
It can be noticed that a and n are the only parameters to be calibrated in the above equations (1) and (2).

The most common form for reservoir modelling is the linear reservoir (n =1). It has been noted by Wagener et al. (2002) that the transfer from effective rainfall to reservoir outflow is generally represented by a linear reservoir, since the non-linearity is captured in the soil moisture model upstream of the reservoirs. Nonetheless, the flow from the third reservoir representing the outflow from the old landfill (Q_d) could not be reflected by a linear reservoir, indicating that there is complex flow dynamics happening through the landfill where the soil moisture model (Section 4.2.2.) could not capture it in an acceptable manner.

In the first reservoir of each flow type, a fraction of the flow is diverted from the outflow to account for losses such as infiltration or evaporation:

$$L = b(S - S_c)^m \tag{3}$$

Where L is the lost flow, b, and m are discharge rate and reservoir linearity. The losses flow from the reservoir is dependent on the critical storage of the reservoir Sc, in which the losses equation is activated when the reservoir storage goes above the critical storage Sc. which is a model parameter to be calibrated for each first reservoir of the three flow types.

Subsequently, the output flow of the model is the simulated total flow from each flow line, as:

$$Q_{total} = Q_h + Q_p + Q_d \tag{4}$$

Where Q_h, Q_p, and Q_d are the outflow from each flow line for hard, permeable surfaces and the old landfill, respectively.

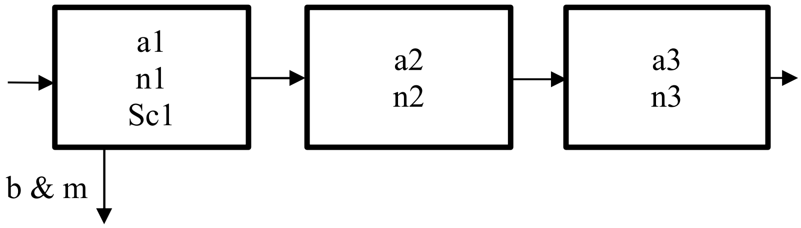

Figure 4.5. Typical routing reservoirs parameters

4.2.1. Snow model

The simulation of the snow routine development was reflected by adopting the degree day method within HBV model (Bergström, 1975) and explained by (Lindström et al., 1997). There exist some attempts to modify the degree day method to be applicable on sub-daily simulations, such as the work done by Tobin et al. (2013), in addition to Rango and Martinec (1995), where the latter noted that it is recommended to couple the degree day method with radiation indexes for sub-daily melt simulations. Though, in this work the model was kept dependent only on the temperature as the only metrological parameter for simplicity. The sub-daily melt was simulated as following:

$$MELT_{[i]} = C_d \, T_{[i]} \tag{5}$$

$$FR_{[i]} = C_f \, T_{[i]} \tag{6}$$

Where *MELT* and *FR* are amount melted or frozen, respectively. Both are dependent on C_d and C_f representing hourly melting and freezing rate respectively. C_d and C_f are model parameters to be calibrated. $T_{[i]}$ represents the air temperature at timestep *i*.

The melt simulation is dependent on the water holding capacity of the snow (*WHC*), representing the amount of water retained in the snow, in which *WHC* is a parameter to be calibrated thereafter.

The detailed algorithm is outlined in Appendix I.

4.2.2. Accounting for soil moisture

The soil moisture was reflected by using an equation reflecting the wetness index of the soil known as Catchment Wetness Index (CWI) (Wagener et al., 2002; Croke and Jakeman, 2008), which calculates the portion of the rainfall that gets translated to an effective rainfall, depending on the wetness index of the previous timestep. The effective rainfall $u_{[i]}$ is hereby calculated as following:

$$u_{[i]} = [c(\emptyset_{[i]} - I_s)]^p \, r_{[i]} \tag{7}$$

Where $r_{[i]}$ is the observed precipitation at timestep *i*. *c*, I_s, and *p* are parameters representing mass balance, soil moisture index threshold and non-linear response terms, respectively. $\emptyset_{[i]}$ is the soil moisture index at timestep *i*, calculated as following:

$$\emptyset_{[i]} = r_{[i]} + \left(1 - \frac{1}{t_{[i]}}\right)\emptyset_{[i-1]} \tag{8}$$

Where $t_{[i]}$ is the drying rate at timestep I, calculated by:

$$t_{[i]} = t_w \, exp^{0.062f(T_r - T_{[i]})} \tag{9}$$

Where t_w is the reference drying rate, *f* is the temperature modulation, T_r is the reference temperature and $T_{[i]}$ is the air temperature at timestep i.

Thus, the parameters to be calibrated in the above equations (7, 8 & 9) is *c*, *p*, I_s, t_w, *f*, and *Tr*.

The original equations described by Jakeman et. al (1990) was meant to directly relate the transfer from rainfall $r_{[i]}$ to effective rainfall $u_{[i]}$ by the parameter c which is thereafter calculated explicitly (i.e., I_s = 0 and p = 1). Nonetheless, the form used in this model is a more general form to allow the non-linear simulation of the moisture indexes through the parameter *p* and the incorporation of a moisture threshold I_s, thus requiring model calibration with the two parameters (Croke and Jakeman, 2008).

4.2.3. Calibration and Validation

The model performance has been assessed by using a set of objective functions, which in essence aggregate the difference between the observed and simulated flow (Wagener et al., 2002). The model was calibrated manually and automatically until it produced the maximum fitting (i.e., lowest residual) according to the current model structure. Preliminary calibration of the model was crucial for the automated calibration to progress in the right direction. Numerous interventions were made after visual inspection of the interim automated calibration results. It was recommended by Willems (2014) that the calibration to be a combination of manual and automatic steps that intertwine some user interventions, which is confirmed in this report.

The results of the calibration were gauged visually and by analyzing the results of several objective functions. The objective functions used for calibration (Wagener et al., 2002; Kalin and Hantush, 2006; Coutu et al., 2012) are as following:

- *Nash-Sutcliffe Efficiency Model (NSE)*

$$NSE = 1 - \frac{\sum_{i=1}^{n}[Q_{obs(i)} - Q_{sim(i)}]^2}{\sum_{i=1}^{n}[Q_{obs(i)} - \overline{Q_{obs}}]^2} \tag{10}$$

Where Q_{obs} and Q_{sim} are the observed and the simulated flow, respectively. $\overline{Q_{obs}}$ is the mean of the observed flow time series. i and n represents the timestep and the total number of time steps, respectively.

- *Normalized Bias (NB)*

$$NB = \frac{\sum_{i=1}^{n}[Q_{obs(i)} - Q_{sim(i)}]}{n\overline{Q_{obs}}} \tag{11}$$

NSE puts emphasis on the extreme flows where NB produces average weighted difference between both sets (Coutu et al., 2012).

- *Coeff. of Determination or Goodness of fit (R^2)*

$$R^2 = \frac{\sum_{i=1}^{n}[(Q_{obs(i)} - \overline{Q_{obs}}) * (Q_{sim(i)} - \overline{Q_{sim}})]^2}{\sum_{i=1}^{n}\left[\left(Q_{obs(i)} - \overline{Q_{obs}}\right)^2\right] * \sum_{i=1}^{n}\left[\left(Q_{sim(i)} - \overline{Q_{sim}}\right)^2\right]} \tag{12}$$

$\overline{Q_{sim}}$ is the mean of the simulated flow time series. R^2 describes the accumulated variances in the observed data that can be explained by the model (Kalin and Hantush, 2006), or simply the fitting of both curves.

- *Deviation of Runoff Volume (DV)*

$$DV = \frac{\sum_{i=1}^{n}[Q_{sim(i)}]}{\sum_{i=1}^{n}[Q_{obs(i)}]} \tag{13}$$

DV represents the difference between the accumulated volume of the observed and the simulated flow during a specific time period.

Thus, the used objective functions and the ideal value of each are summarized as following:

Table 4.2. A summary of the used objective functions with the ideal value

Indicator	Ideal value
NSE	1
NB	0
R^2	1
DV	1

The model has been calibrated during the period of 2019-12-01 to 2021-05-07 (approx. 1.5 year), and subsequently validated for the period of 2021-05-08 to 2021-11-07 (approx. 0.5 year).

Since the obtained data started from 2019-01-01, the period between 2019-01-01 to 2019-11-31 is utilized as a warm-up period for the model, since the model started to produce sufficient results from 2019-12-01. Thus, a schematic of the timeline of the model can be shown in following figure:

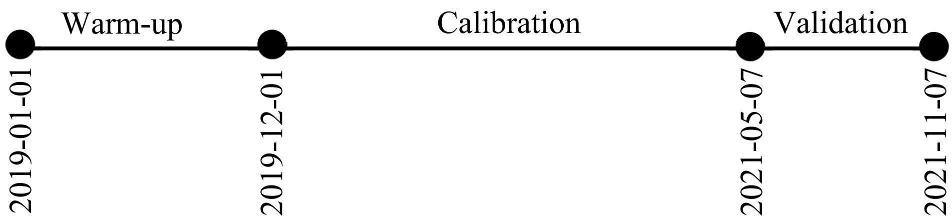

Figure 4.6. Schematic timeline of the model periods

4.2.4. Climate change

A study is conducted to simulate the expected climate change by applying an increase to the precipitation and temperature to simulate the expected changes in the global climate. Subsequently, three scenarios have been studied according to the most recent report by the Intergovernmental Panel on Climate Change (IPCC) (IPCC AR6 WG1, 2021). The three scenarios are outlined as following:

Table 4.3. Climate change scenarios (IPCC AR6 WG1, 2021)

Scenario	Period	Increase in temperature (+ C°)	Precipitation percentage increase (%)
1.0	Near term (2021-2040)	1.5	10
2.0	Mid-term (2041-2060)	2	15
3.0	Long term (2061-2100)	4	25

The increase in precipitation herein is a uniform increase in precipitation from the observed (recorded) precipitation values, and it is not taken in consideration any change in the frequency of the heavy rains (large events) that may arise from the expected global warming as indicated in the IPCC report.

Since the meteorological parameters that the model depend on are precipitation and temperature, the simulation of the future climate will be a balance of increased flow due to precipitation, with increased evaporation due to the elevated temperature.

4.3. Leachate ponds simulation

The flow from the main pump station is directed towards two leachate storage ponds (Lakdamm 1&2) showed in Figure 4.2., where the ponds are currently used for aeration and treatment of the leachate and reject water prior to pumping to the treatment plant. A schematic for the flow through the storage ponds can be outlined as flowing:

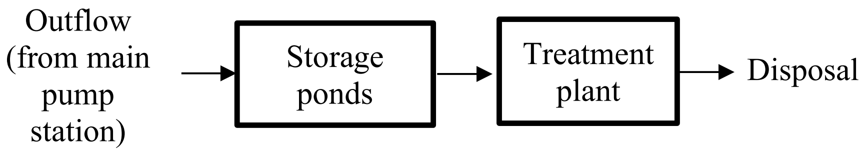

Figure 4.7. Schematic flowline through the storage ponds

In order to simulate the fluctuating volume of the storage ponds, a water balance based on continuity has been developed as following:

$$\frac{dV}{dt} = Q_{in} - Q_{out} \tag{14}$$

$$Q_{in} = Q_{total} + P(A) \tag{15}$$

$$Q_{out} = Q_{treat} + E(A) + Inf \tag{16}$$

Where V is the volume of the storage ponds, Q_{in} is the inflow, Q_{out} is the outflow, P is precipitation (mm/1000), A is the surface area of the ponds (m^2), *Qtreat* is the flow to the treatment plant, E is evaporation (mm/1000), and *Inf* is the infiltration to the sub-ground layers as the ponds were not lined at the bottom at the time of construction.

Qtotal and *P* are two time series of both the total flow from the main model and the recorded precipitation, respectively.
E is the allowance for evaporation in millimeter and was adopted from SMHI online accessed data, representing the yearly average evaporation distributed spatially on Sweden, in which the value for south Sweden were 600 mm/year. The yearly value was thereafter converted to an hourly value by division. Adapting normalized evaporation through the whole year is considered a simplification which was deemed acceptable due to the small value of the evaporation losses compared to the inflow and outflow to the pond by pumping.

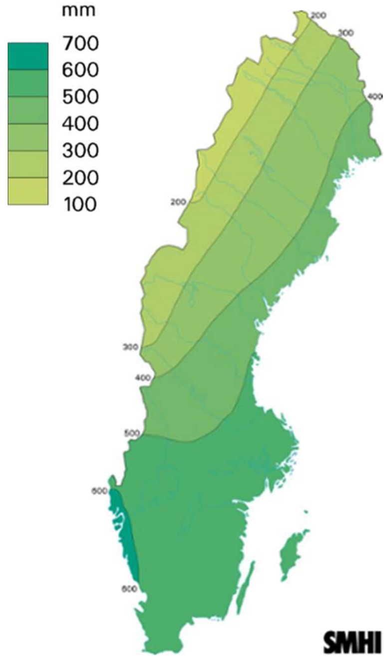

Figure 4.8. Average yearly evaporation in Sweden (SMHI). *https://www.smhi.se/kunskapsbanken/hydrologi/avdunstning/avdunstning-1.30720 (Accessed: 2022-01-22)*

The losses due to the infiltration were adopted from the facility environmental report for two years (Nårab Miljörapport 2019, Nårab Miljörapport 2020). The infiltration was calculated based on the difference in found in the water balance done by the facility. Since the inflow, outflow, precipitation was measured analytically, and that the evaporation is adopted as an average value, the difference in the water balance to achieve continuity was attributed to infiltration. The value mentioned were a yearly average that was normalized to hourly basis as the evaporation previously calculated. Since the infiltration was also in a small magnitude compared to the pumped inflow and outflow to/from the leachate ponds, the simplification deemed acceptable in the simulation.

Thus, the balance parameter in equation (16) is Q_{treat}. The inflow to the ponds is the flow from the main pump station (main flow meter), and the outflow Q_{treat} is depending on the pumping periods which is subsequently dependent on the design capacity of the treatment plant. The pumping to the treatment plant is dependent on the air temperature as there exists a maximum capacity of the heaters within in the treatment plant, where generally during hot periods more pumping is made compared to the cold periods.

According to the facility personnel, the treatment plant practical capacity is 500 m^3/day, the pump flow capacity is 50 m^3/hr and pumps get started at the beginning of each shift at 08:00:00 every day. In addition, it is required a minimum of 12 ºC during the day in order to reach the maximum capacity of 500 m^3/day (i.e., 10 hours of pumping at 50 m^3/hr). The model was built on the daily mean temperature to locate the “warm” days, which has a direct relation to the maximum daily temperature that was noted by the facility personnel. The model was subsequently built on the mean daily temperature of 5 ºC which locates the days where the maximum temperature is 12 ºC and excludes the days of comparingly cold temperature at night.

Subsequently, the flow pumped from the leachate ponds Q_{treat} were simulated as a function of the air temperature at each timestep. The reason to not adopt a normalized average value for the pumping is to preserve the relation between the pumping out from the ponds and the air temperature, since it is expected to have an increased temperature in the future due to climate change in terms of global warming. It was aimed to allow the simulated pumped flow to increase if the temperature get increased in the future. Moreover, the inflow to the ponds is in fluctuation during different periods during the year and keeping time-variant outflow relation will allow the assessment of the ponds capacity at the event of high inflow and low outflow.

Thus, the pumped flow towards the treatment plant Q_{treat} (outflow from the leachate ponds) were simulated as following:

At $T_{m[i]} \geq 5$ ºC, $Q_{treat\ [i]} = 50$ m^3/hr during the hours of [08:00:00 to 17:00:00]

At $T_{m[i]} < 5$ ºC, $Q_{treat\ [i]} = 50$ m^3/hr during the hours [08:00:00 to 10:00:00]

Where $T_{m[i]}$ is the daily mean temperature. It can be noticed in the following Figure 4.9. that the above rule fulfills what has been noted by the facility personnel; where above 12 ºC (max.) the maximum pump flow to treatment is achieved (500 m^3/day), while below 12 ºC the minimum pumped flow to treatment plan is achieved (100 m^3/day). The above rule only applies between March to November, where from December to February (winter) the flow is kept to the minimum value. Figure 4.11. shows a schematic for the above rules.

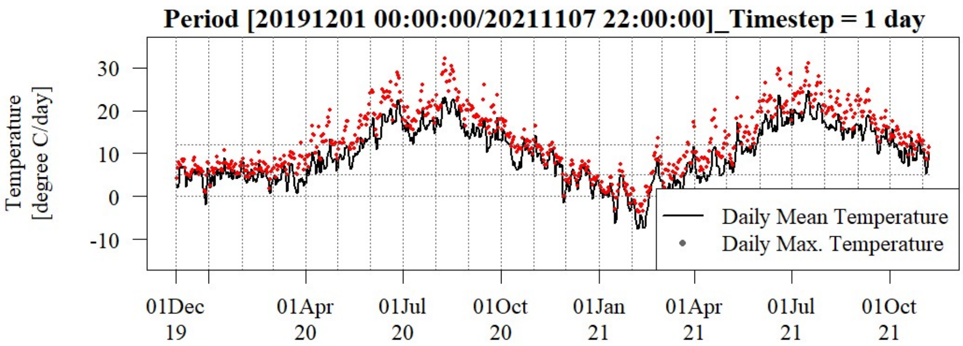

Figure 4.9. Daily mean and max temperature

Additional functions were added to the model to override the above relation and allow the pumping to stop if the ponds have been drained to the bottom level.

Monthly recordings were received from the facility, and it showed the volume of the leachate water contained in the ponds (V) for the whole simulation period. Since the modelling the leachate ponds was on hourly basis (timestep = 1 hr.), the monthly data from the facility were used for validation of the model results by distribution. Since the observed values were specific observations in specific observed time, an interpolation between the observations has been applied to compare the model results with the observations, as showed in Figure 4.10.

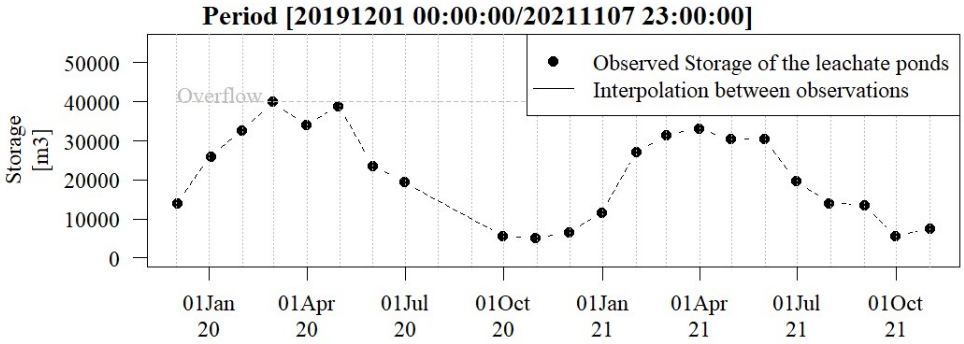

Figure 4.10. Observed storage of the leachate ponds by Nårab

It can be noticed from the observations that the storage has reached very close to the overflow levels two times in March 2020 and May 2020. It was assumed that during these periods the pumping to the treatment plant was increased to the maximum capacity to prevent the overflow of the ponds, regardless of the air temperature, by overriding the pumping sequence by the facility personnel. Thus, the period of year 2020 is the chosen period for the further assessment as it gives better indication on the extreme case of the ponds overflow.

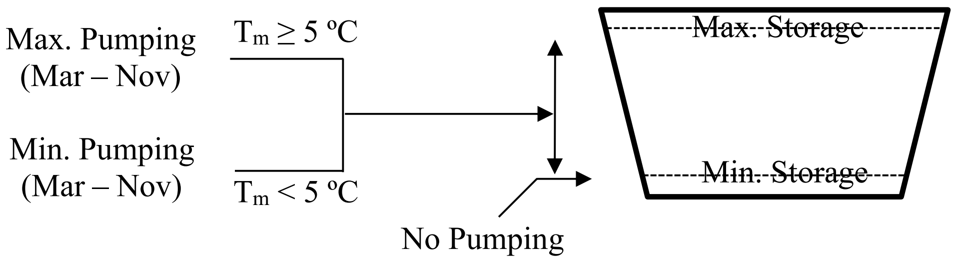

Figure 4.11. Schematic of the rules used in the leachate pond storage simulation.

An analysis of the stage storage of the leachate ponds has been done on Autodesk Civil 3D to relate the stage of the leachate ponds with the storage volume, in order to calculate the maximum pond capacity (at the overflow level). As per the observations, lowest level of the pond is equivalent to volume of 5,054 m^3, which represents the volume of the sedimentation at the bottom of the pond. It has been found that the maximum capacity of the ponds is 40,096 m^3 which is confirming the information in the facility environmental report noting that the volume of both ponds equals “approx. 40,000 m^3”.

Finally, the result of the leachate ponds simulation with the observed flow is outlined in Figure 4.12. It is noticed that the model is showing a similar behavior to the actual observed storage volumes, though there is a degree of error. Since the pumping is initiated manually at the facility, and there is no fixed procedure to incorporate it as additional rules to the model, it was considered that the result is acceptable to proceed further. It is expected also that the shifting from maximum to minimum pumping is done gradually. The methodology is that since the model have reached acceptable result by utilizing the catchment “observed” flow as an input to the model, then the simulation can be repeated later with the catchment “simulated” flow and comparison between both to be made.

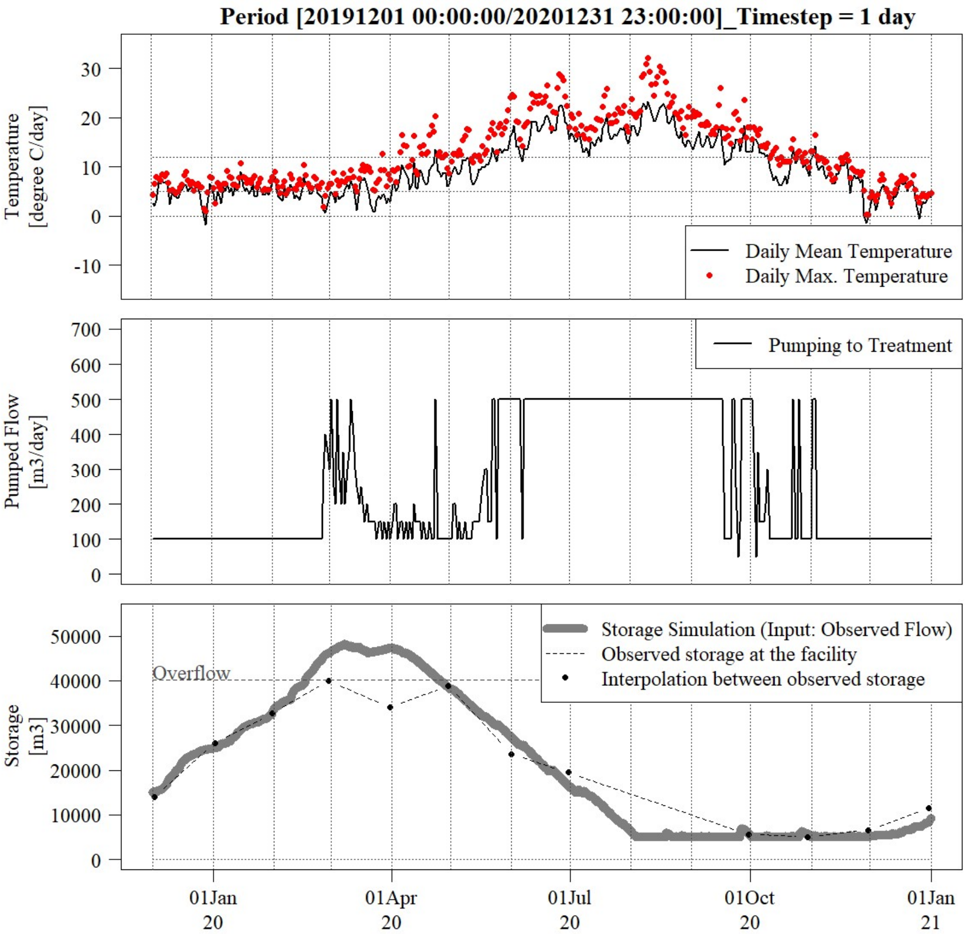

Figure 4.12. Simulation of the leachate ponds storage for the whole catchment simulation period (2019-12-01 to 2020-12-31)

4.3.1. Leachate ponds at the climate change

The model of the leachate ponds is tested with the design inflow adjusted for climate change with the scenarios mentioned in Tables 4.5., in which the precipitation and temperature have been increased to simulate the expected climate change.

Since the model has the temperature as a parameter in the calculation of the outflow as well, the outflow was expected to increase at warmer climates, which allowing the personnel at the facility to pump additional water to the treatment plant. Thus, at the periods of higher temperature and higher precipitation, the inflow is increased (more runoff from the catchment) and also the outflow is increased (more pumping towards the treatment plant). The balance of both inflow and outflow is going to affect the contained volume within the leachate ponds.

4.4. Simulation of the landfill permanent closing

The simulation of closing landfill permanently is expected to occur as per the information received from the facility. The cover of the landfill is yet to be decided. Yet, it is expected that the old landfill will be covered with a material of low hydraulic conductivity (Marques and Hogland, 2003). Such cover with low permeability will allow the precipitation to flow overland and prevent percolation of rainfall to the center of the landfill, thus inducing a quicker response of overland flow flowing from the surface of the landfill. Such overland flow will follow the terrain slopes to reach the sides of the landfill. Figure 4.13. shows the facility watersheds and the depressions around the landfill where the overland flow will accumulate.

It is expected that after capping the old landfill, drainage ditches will be constructed around the area to channel the runoff that will eventually lead towards a stormwater pond. This paper will aim to find the required storage of the stormwater pond that will contain the maximum daily peak from the design rainfall events at the current and future climate change.

The underground flow from the landfill is expected to continue flowing even thought that the landfill will be covered with a low permeable cover preventing percolation of the rainfall. Though, the leachate production is expected to decrease with a decay curve that is yet to be known. It is recommended to assume that the same leachate production towards the leachate ponds as the rate of leachate production future decrease is not known.

As mentioned, since the future cover of the landfill will induce quick overland flow comparing to the current condition, the simulation will be done by using the parameters of the routing reservoirs that is designed for the hard surfaces, and simulated by the area of the old landfill instead by the catchment area of the hard surfaces. Thus, the required volume will be calculated by the maximum daily peak of the simulated flow.

Figure 4.13. Watershed map of the facility (SCALGO LIVE) (Dashed line shows the extent of the landfill, arrows show the slope of the terrain).

5. Results and Discussion

This section outlines the results of the developed model as explained in Section 4 and attempts to clarify the model outcome.

5.1. Catchment simulation (main model)

The calibration of the model has been performed between the period 2019-12-01 to 2021-05-07 which is equivalent to 532 days, with timestep = 1hr. The results of the calibration for this period are summarized in the following table:

Table 5.1. Results of the model calibration by simulation at timestep = 1 hr.

Indicator	Calibration value	Ideal value
NSE	0.54	1
NB	0.01	0
R^2	0.54	1
DV	0.99	1

Accumulating the hourly timeseries to daily values, the results of the calibration are summarized as:

Table 5.2. Table 4.1. Results of the model calibration by accumulating the hourly timeseries to daily

Indicator	Calibration value	Ideal value
NSE	0.69	1
NB	0.01	0
R^2	0.69	1

Subsequently, the model was validated between the period 2021-05-08 to 2021-11-07 which is equivalent to 183 days, with timestep = 1 hr. The model results of the validation period are summarized in the following table:

Table 5.3. Results of the model validation by simulation at timestep = 1 hr.

Indicator	Validation value	Ideal value
NSE	0.54	1
NB	0.09	0
R^2	0.56	1
DV	0.90	1

Table 5.4. Results of the model validation by accumulating the hourly timeseries to daily

Indicator	Validation value	Ideal value
NSE	0.69	1
NB	0.01	0
R^2	0.74	1

Moriasi et al. (2015) noted that the models are considered acceptable if NSE > 0.50 and R^2 > 0.60 for watershed-scale models that is daily simulated. It can be shown that the model has produced acceptable results for the daily accumulated and even "fair" for hourly simulations.

It must be noted that that the complexity of the catchment did not allow capturing all the flow dynamics for hourly simulations, since the catchment includes multiple pumping stations that must influence the flow. Monthly records from the facility shows that such pumping stations operate for certain hours each month and there is no sound way to track or record the changes to modify the model accordingly. Moreover, the dynamics in the outflow from the old landfill will not normally be captured in a parsimonious model.

Nonetheless, since the main objective is to simulate the leachate storage ponds that falls downstream the observed/simulated flow, and the expected attenuation from the flow through the ponds that will "damp" the residuals between the observed and the simulated flow; the results was deemed acceptable to continue further.

It must be noted that the ground water contribution to the outflow is not included as a model parameter. The groundwater data from the Geological Survey of Sweden (SGU) through their open access databases; shows that the groundwater flow below the facility is considered "low", with groundwater flow of 4.2 m^3/hr in a well situated at the center of the landfill when the landfill was relatively young (1996) where there was possibility to construct an observation well at the center of the landfill. Though, since the routing through the old landfill reservoir in the model is based on several parameters that characterized by identifiability, adding the groundwater sub-flow (4.2 m^3/hr) did not represent a major change in the outcome as the reservoir parameters can be adapted to produce the same result with different input flows, especially if it was in a comparingly small magnitude. The additional groundwater input which is almost constant over time did result in adjustment of the model parameters to delay the outflow from the reservoir in order to be able to produce the same result. Thus, it was decided to omit the groundwater inflow to the model and consider that the groundwater is captured in the lumped representation of the catchment.

Since the location of the flowmeter that records the observed flow lies downstream the main pump station, the main pump station was investigated to make sure that the pump maximum capacity did not influence the observed flow, meaning that if the incoming flow is higher than the maximum pump capacity, an accumulation of the flow will happen upstream within the pump pit and the connected drainage network. It was found that the main pump station is operating by two pumps where each pump can pump 45 L/s (≈160 m^3/hr), which is much more than the maximum peak flows in the observed time series. Thus, it is considered that the pump has no effect on the observed flow unless there were manual interruptions by the facility personnel.

5.1.1. Hourly and Daily accumulated flows

In addition to the comparison by the objective functions; the visual inspection is considered an acceptable method to gauge the model results (Willems, 2014). The following figures shows the simulated flow against the observed flow for the full simulation period (calibration and validation), for every 3 months, as following:

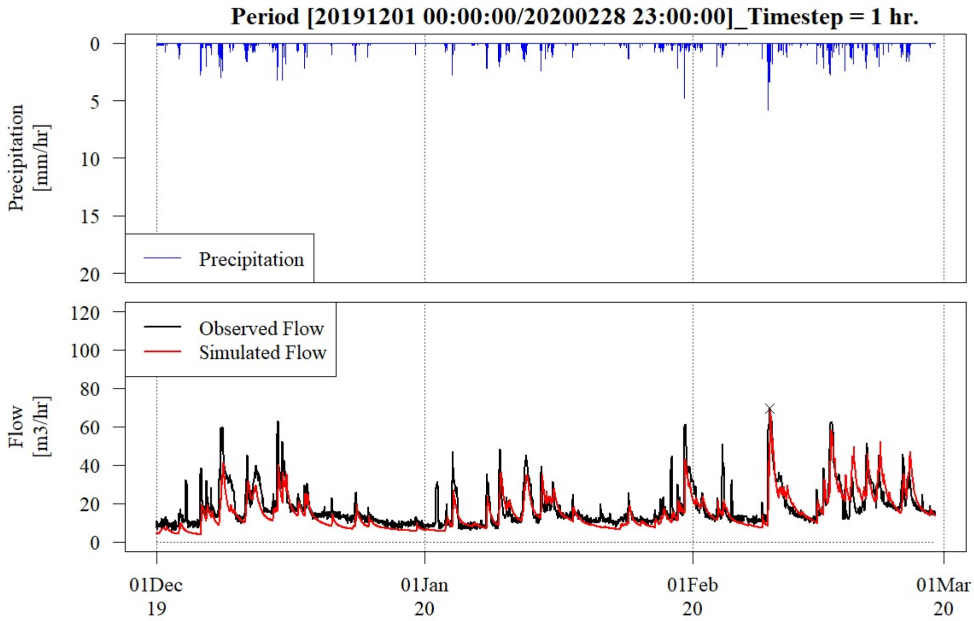

Figure 5.1. Main simulation result between 2019-12-01 to 2020-02-28. The cross shows the maximum of the observed flow during the period.

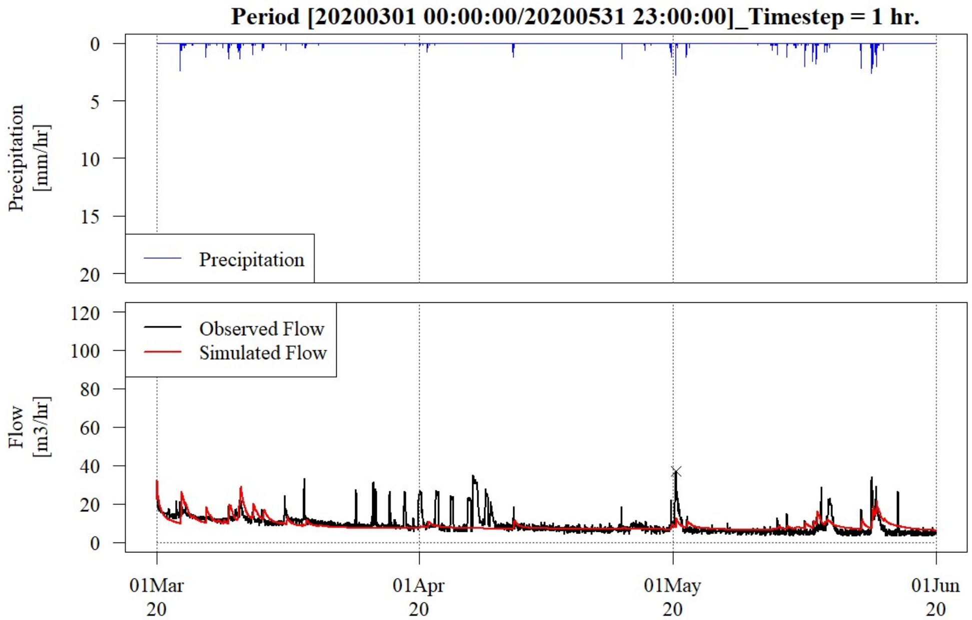

Figure 5.2. Main simulation result between 2020-03-01 to 2020-05-31. The cross symbol shows the maximum of the observed flow during the period.

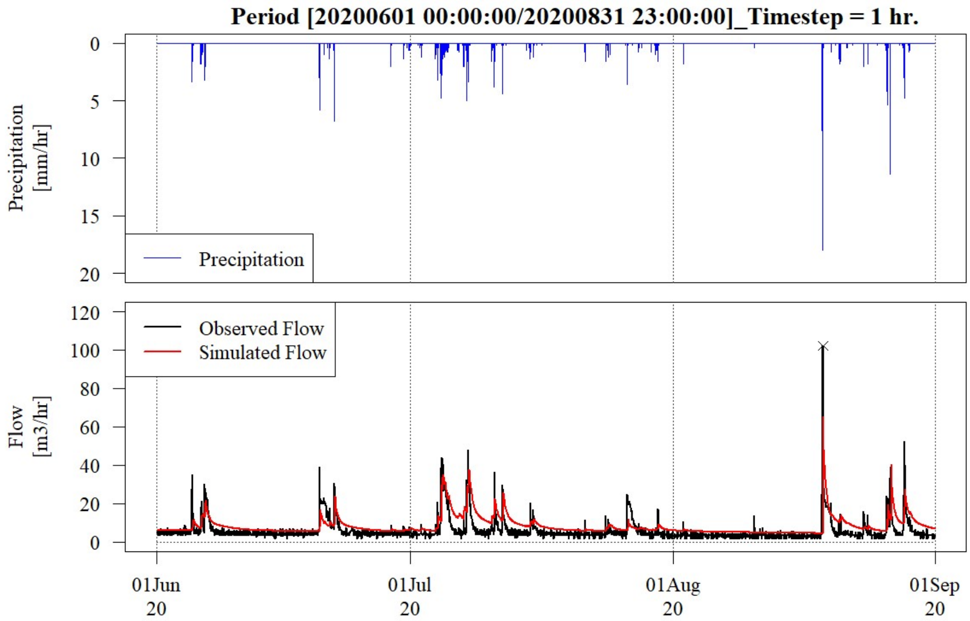

Figure 5.3. Main simulation result between 2020-06-01 to 2020-08-31. The cross symbol shows the maximum of the observed flow during the period.

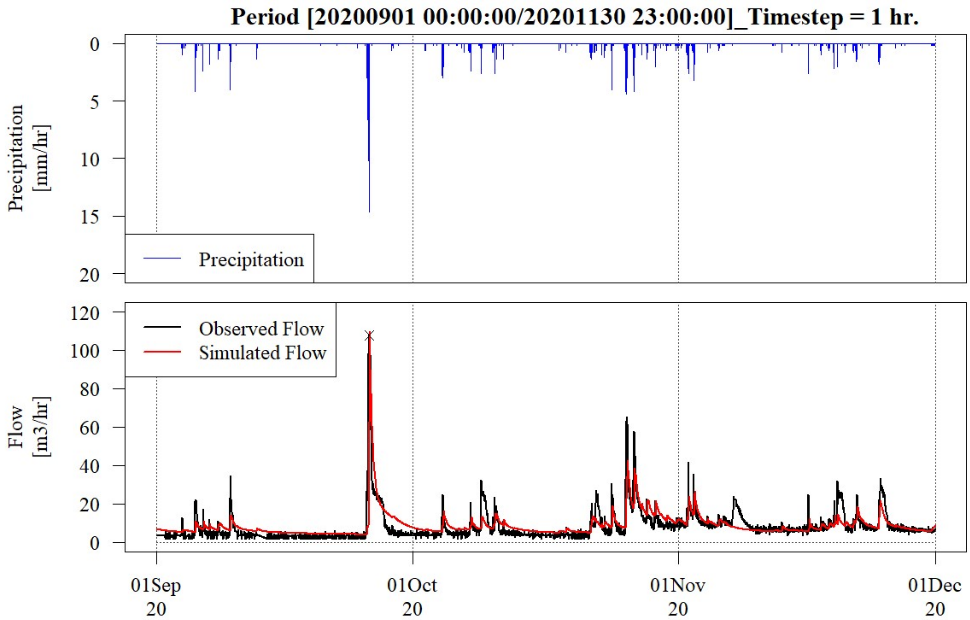

Figure 5.4. Main simulation result between 2020-09-01 to 2020-11-30. The cross symbol shows the maximum of the observed flow during the period.

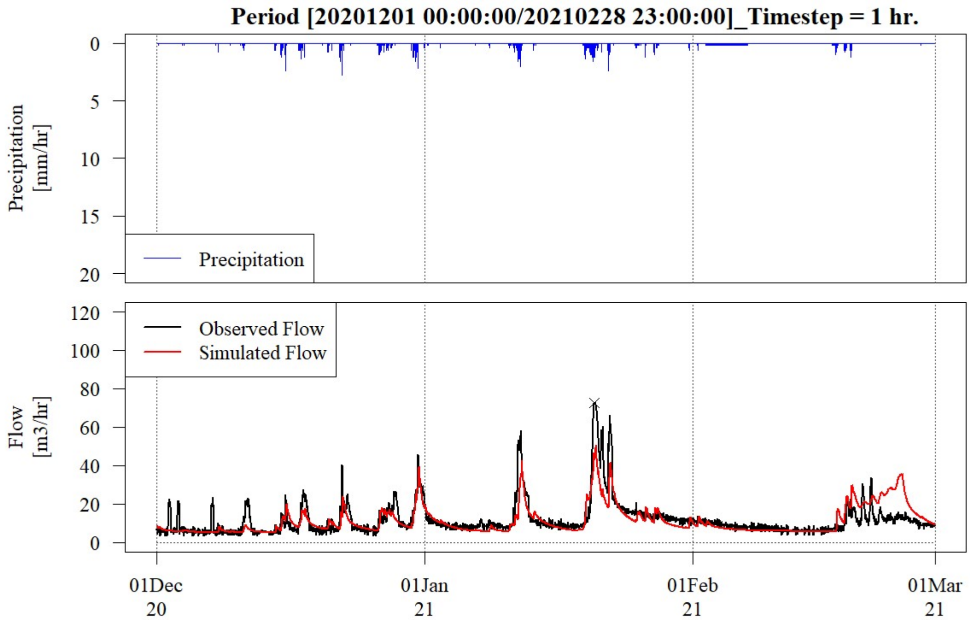

Figure 5.5. Main simulation result between 2020-12-01 to 2021-02-28. The cross symbol shows the maximum of the observed flow during the period.

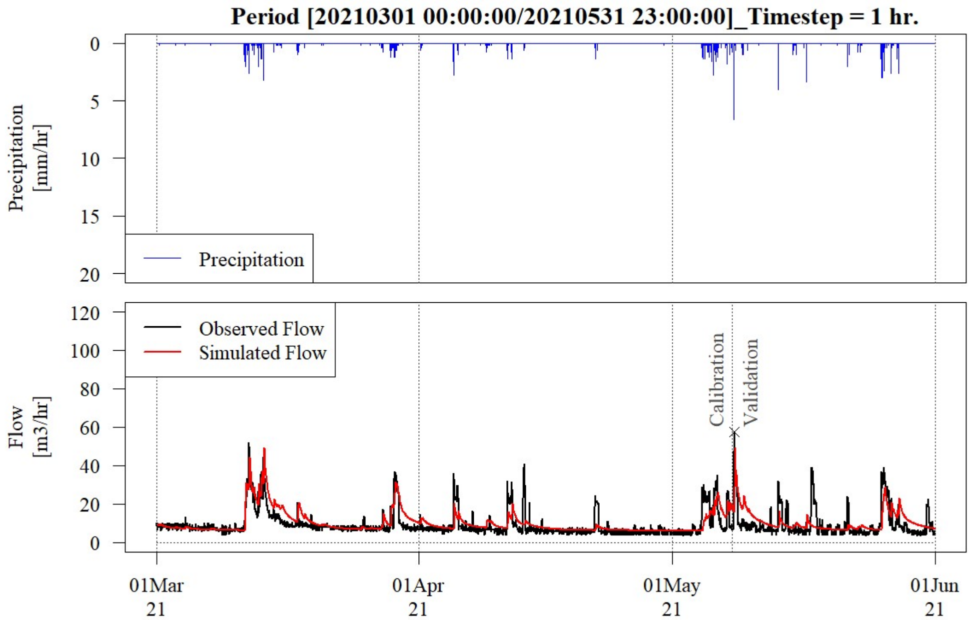

Figure 5.6. Main simulation result between 2020-03-01 to 2021-05-31. The cross symbol shows the maximum of the observed flow during the period.

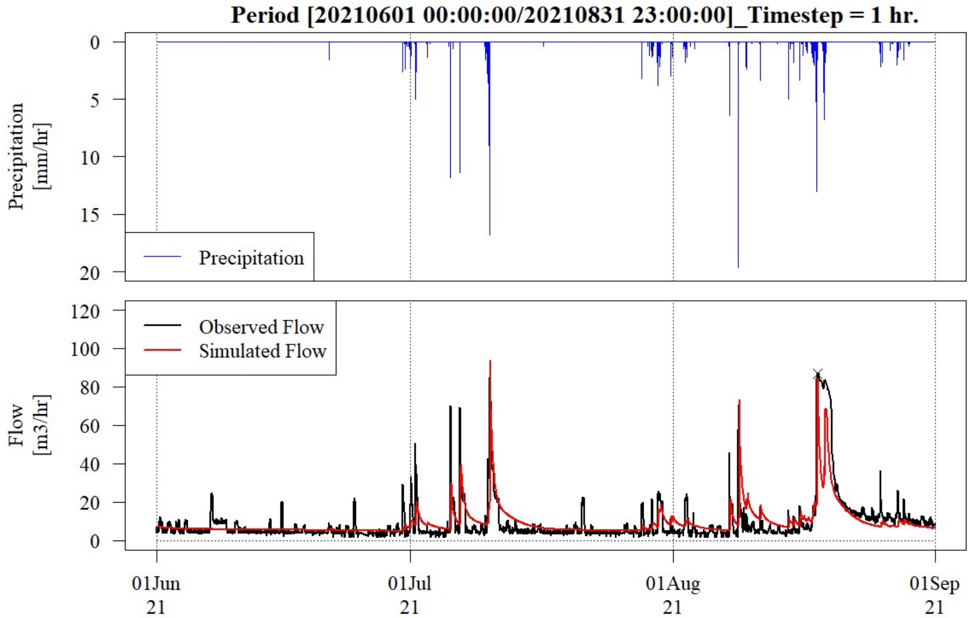

Figure 5.7. Main simulation result between 2020-06-01 to 2021-08-31. The cross symbol shows the maximum of the observed flow during the period.

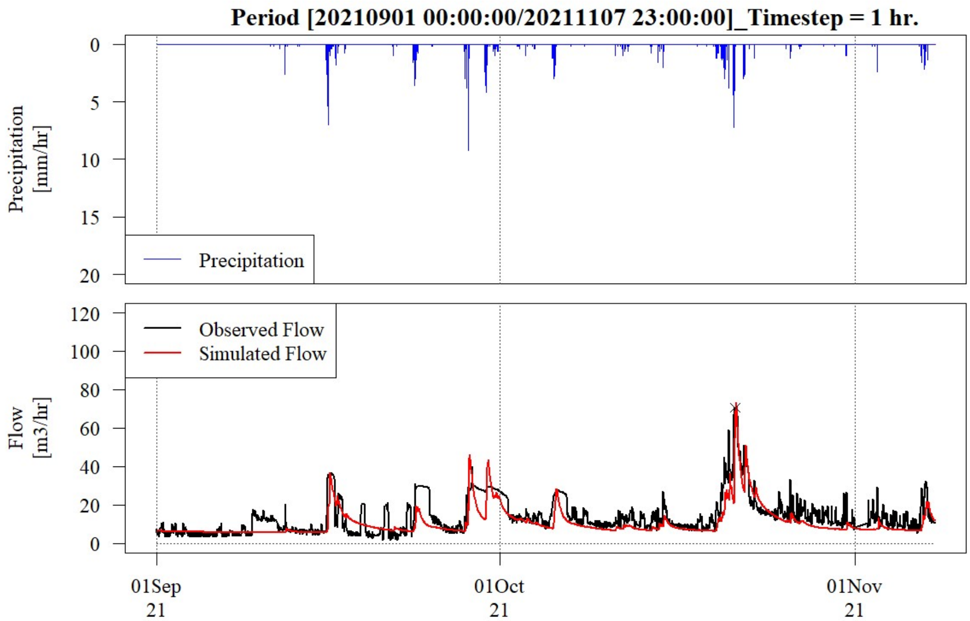

Figure 5.8. Main simulation result between 2020-09-01 to 2021-11-07. The cross symbol shows the maximum of the observed flow during the period.

The following Figures 5.9. & 5.10. shows the observed and simulated flow after accumulating the hourly timeseries to daily values.

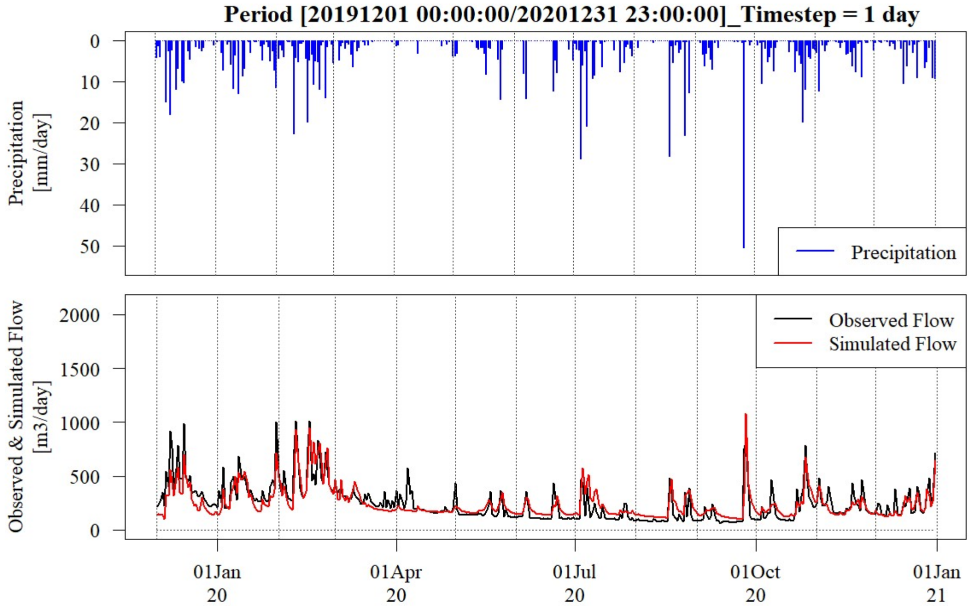

Figure 5.9. Plot of the main simulation result (daily accumulated) between the period 2019-12-01 to 2020-12-31

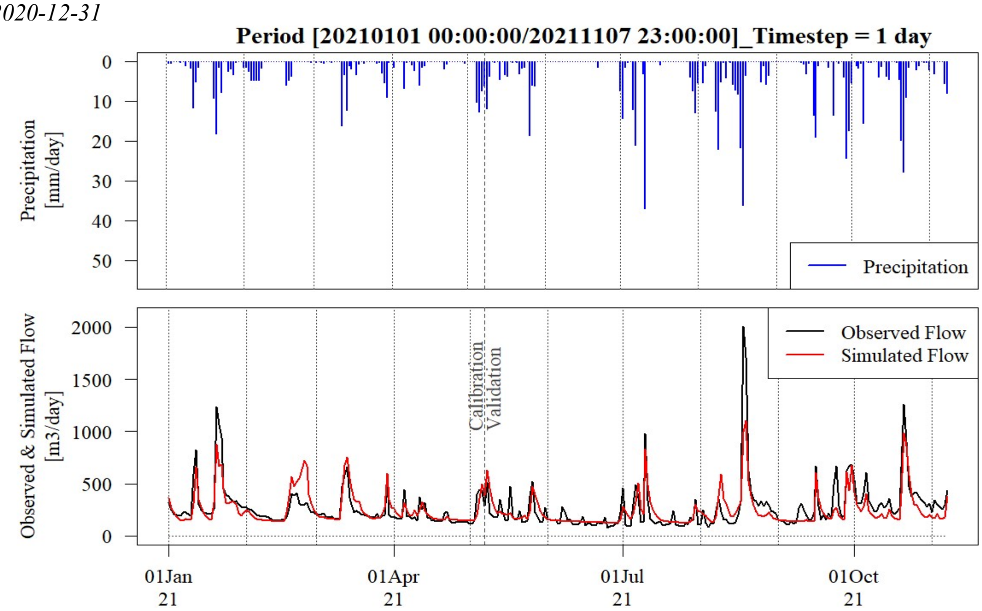

Figure 5.10. Plot of the main simulation result (daily accumulated) between the period 2021-01-01 to 2021-11-07.

It can be noticed that the daily accumulated values are aligning to a greater extent with the observed values comparing to the hourly simulation. This is more likely attributed to "damping" of the discrepancies between the simulated and observed flow which is not captured by the hourly model. As mentioned earlier, the existence of several pumping stations and underground pumping at the waterproofing layer, are the main reasons which influence the variation between the simulated and observed peak flow.

The comparison between the peak flows of the simulated and observed flows shows that there is acceptable fitting of both curves. The only period that shows a significant error between both flows is between the period of 2021-08-18 and 2021-08-21 (3 days), outlined in Figure 5.11. and 5.12.

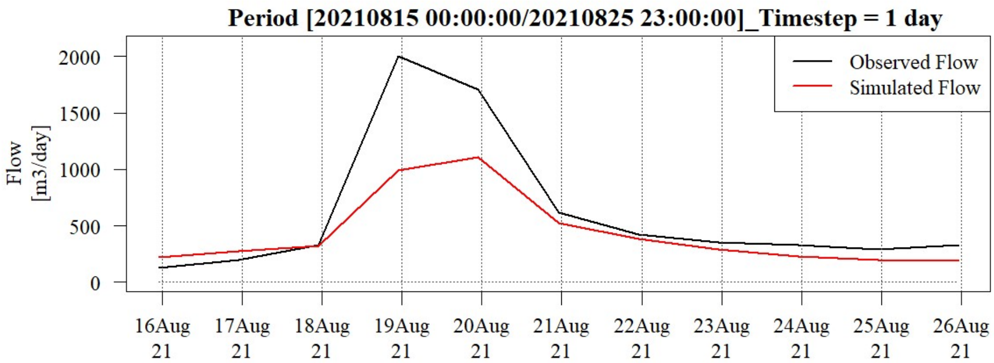

Figure 5.11. Plot of the main simulation result (daily accumulated) between the period 2021-08-15 to 2021-08-25.

By referring to Figure 5.11. that shows the daily accumulated values of the observed and simulated flow, with Figure 5.12. that shows the same period in the hourly simulation, it is expected that the observed values were not correctly recorded during this period given that there is an obvious discrepancy between the observed flow and the precipitation data. The precipitation data shows two rain events with two peak flows, while the observed flow shows a continuous peak that is lasting for 3 days with the same magnitude (almost double any other peak in the observation time series). Thus, it was considered that this period is not accurately recorded and thereafter it has been excluded from the further assessments.

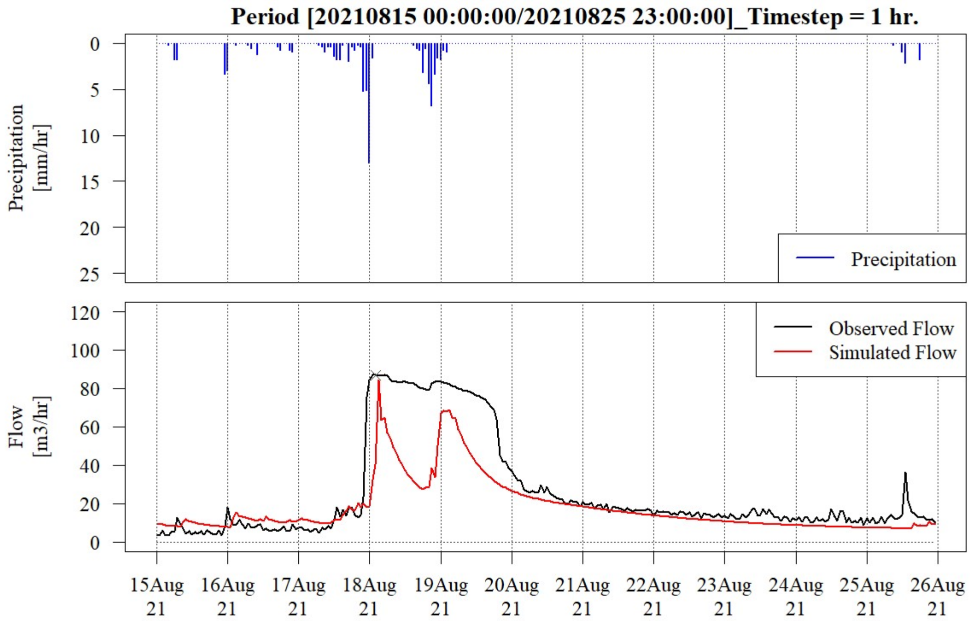

Figure 5.12. Plot of the main simulation result (hourly) between the period 2021-08-15 to 2021-08-25.

Moving back to the daily accumulated values, the results from the validation period shows similar model performance as the calibration period, which was considered acceptable (excluding the period of 2021-08-18 and 2021-08-2 mentioned earlier).

In order to reveal if the model error is dependent on time, a plot of the residual of the observed flow from the simulated flow versus time (Wagener et al., 2002) has been made to reveal any long-term effects from the model, and to verify the model applicability to simulate future flows. It is noted that plotting of the residual flow (Figure 5.13.) did not show any significant time dependence or long-term effects of the model (excluding the 3-day period), indicating that the model will continue to provide similar results with longer simulation times.

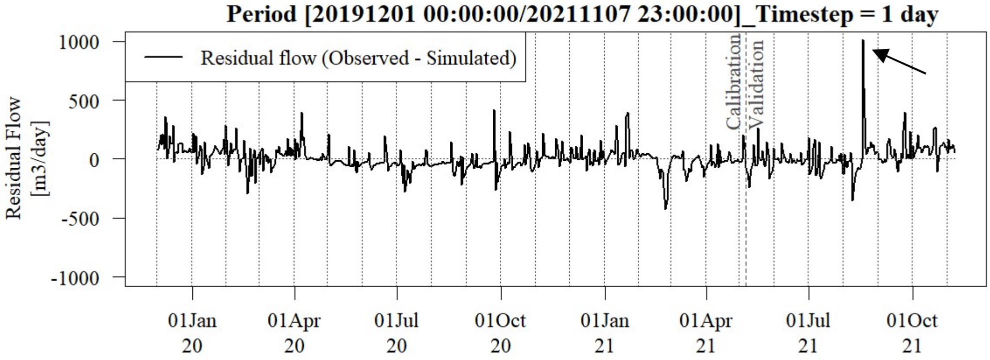

Figure 5.13. Residual flow (Observed - Simulated) between the period of 2019-12-01 to 2021-11-07 for daily accumulated values. Calibration and validation periods are noted on the figure. The excluded section is noted with a black arrow.

5.1.2. Sensitivity Analysis

The sensitivity of the model has been studied to comparatively find the parameters that influence the model to a great degree (Figure 5.14.). The results showed that the linearity (*n*) of the routing reservoirs have a significant impact on the model. At the same time, discharge coefficients (*a*) sensitivity is greater for the old landfill reservoirs due to the difference in the catchment area from other flow types. The model is also sensitive to the soil moisture indexes model (t_W, *f*, *Tr*, *c*, and *p*), but not sensitive to the snow model (*Cd*, *Cf*, and *WHC*) due to the local climate of the facility that drops below zero degrees in a limited period of the year.

Figure 5.14. shows the degree of sensitivity of the model parameters in which each parameter was scaled between 0 and 100, with the corresponding change in the model outcome represented by Nash-Sutcliffe (NSE) and Normalized Bias (NB) values.

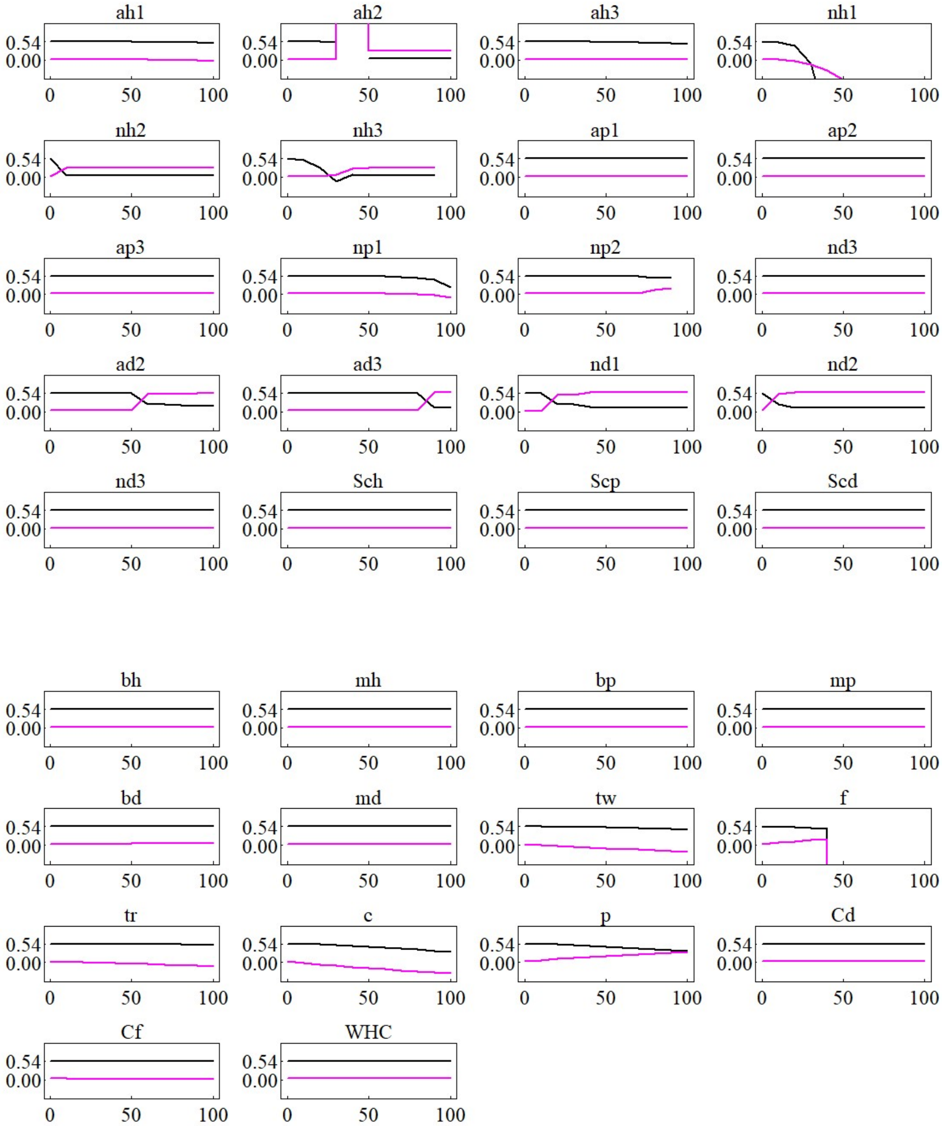

Figure 5.14. Sensitivity analysis (NSE: black, NB: magenta) (all parameters are scaled between 0 and 100)

5.1.3. Assessment of the snow and soil model

In order to assess the contribution of the snow and soil model to the model output, the fraction of the precipitation that is retained by the snow and soil models were assessed by plotting the base precipitation, with the passed fraction from the base precipitation for the snow and soil model, respectively. The following Figure 5.15. outlines the assessment. It can be noticed that the snow model did not show any effect on the model, which was attributed to the local climate of the catchment being in a weather that drops below zero degrees in a limited period of the year, thus no significant snow to activate the model. It is also attributed to that during the periods of below zero degrees, the precipitation was not of a great magnitude to form a snow cover for the model to perform. It was decided to keep the snow model to allow for the possibility for future simulation in colder climate, with indication of that the snow model is yet to be tested as a major component of the main model.

In contrast, the soil model did show a greater effect on the model outcome based on the degree of retraining a fraction of the base precipitation and the passed fraction of the rainfall to the routing reservoirs.

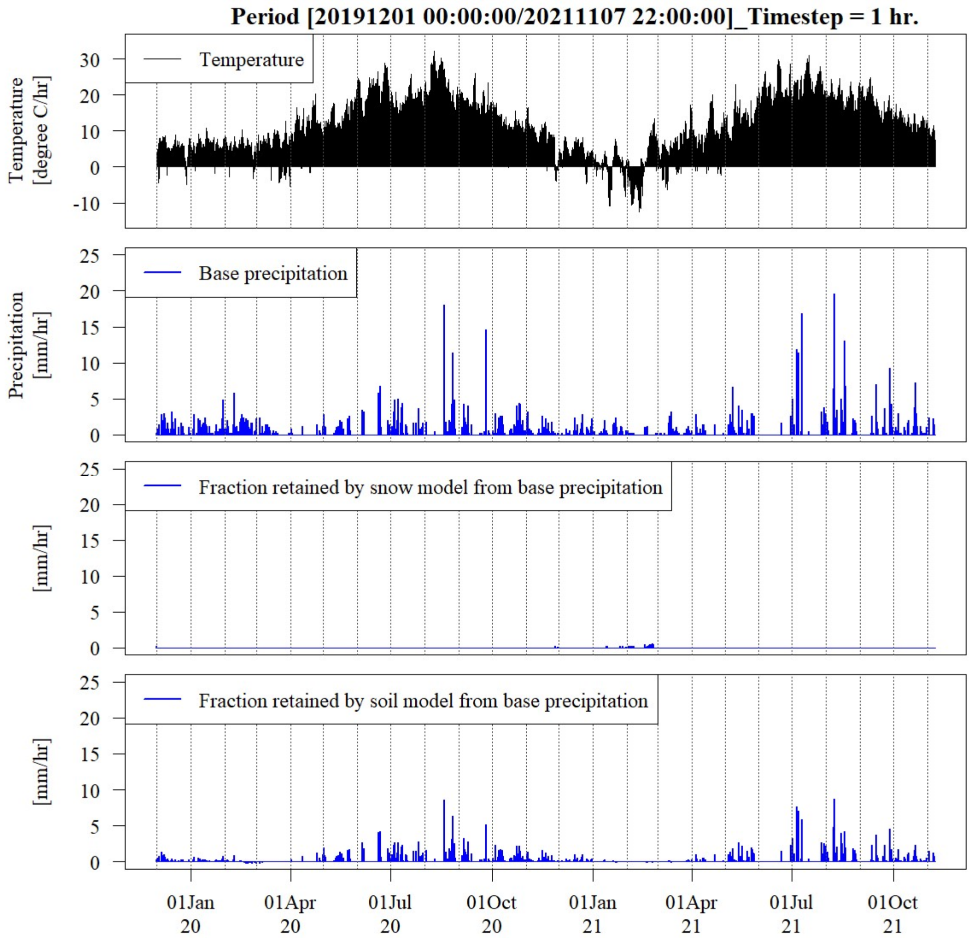

Figure 5.15. Assessment of the snow and soil model

5.1.4. Assessment of the routing reservoirs

In order to assess the flow distinctions (flow types) mentioned previously in Section 4.2. (hard surfaces, permeable surfaces, and old landfill); a separate plot has been made for each reservoir to assess the model degree of matching to the assumptions made about the flow types, as following:

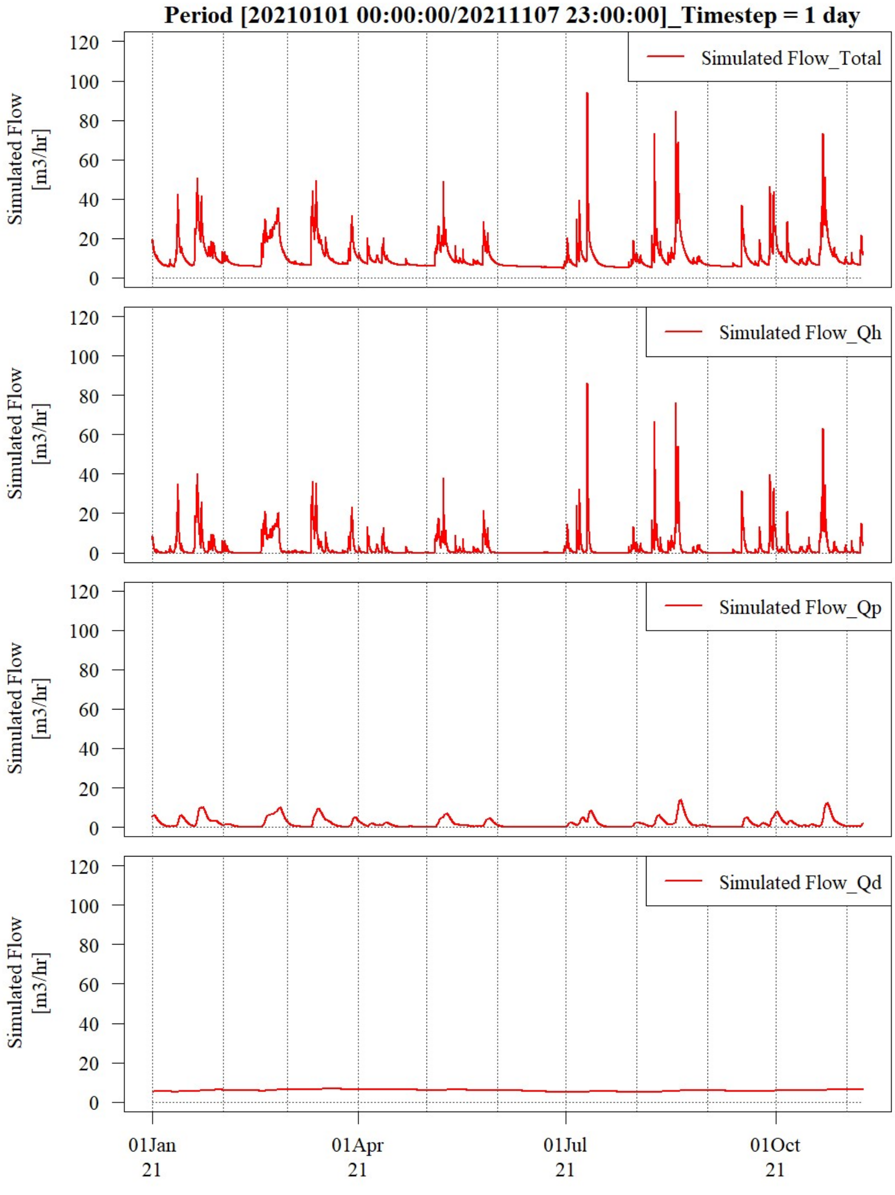

Figure 5.16. Simulated flow from each flow type routing reservoir between the period of 2021-01-01 to 2021-11-07

The previous Figure 5.16. shows that the outflow from each reservoir is matching the preliminary assumptions in terms of the flow delay. The hard surfaces are characterized with quicker response for peak flows, while the permeable surfaces are delayed as expected from drainage through underground perforated pipes. The flow from the old landfill is considered stable through the simulation around 6 m^3/hr, which confirming the work done by Bengtsson et. al (1994), who noted that the landfills of considerable old age are characterized by a constant flow over time.

5.2. Catchment simulation at future climate

In order to simulate the climate change in terms of increased precipitation and global warming within the catchment, the model has been initiated with the scenarios outlined in Section 4.2.4. The result of the simulation was not significantly higher than the main simulation results (Section 5.1.1.). The following Figures 5.17. and 5.18. shows the 3-month period of maximum peak in the previous simulation and a zoomed in plot on the 4 days isolating the peak flow for clarity.

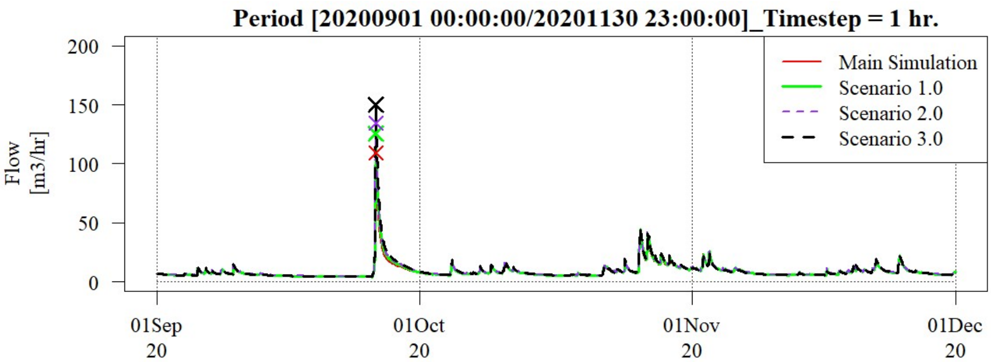

Figure 5.17. Climate change simulation of the catchment (P and T increase) between the period of 2020-09-01 to 2020-11-30

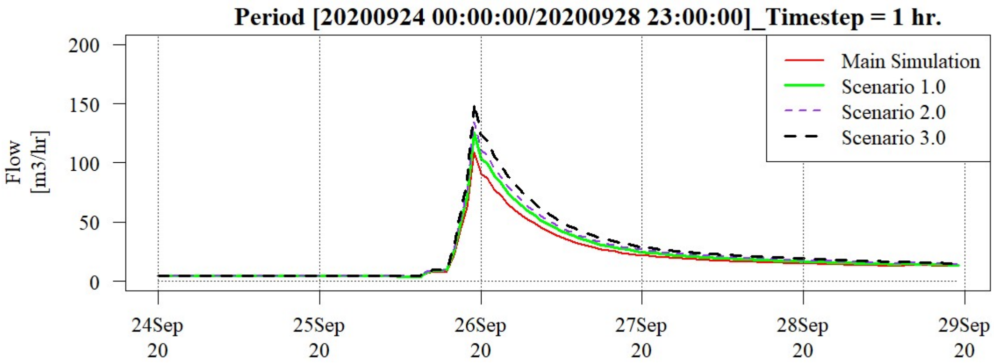

Figure 5.18. Climate change simulation of the catchment (P and T increase) between the period of 2020-09-24 to 2020-09-28

It is noticed that the increased precipitation and temperature have resulted in higher peaks compared to the main simulation.

5.3. Leachate ponds simulation

The catchment simulated flow (Section 5.1.) has been utilized as an input to the model simulating the leachate ponds storage, to verify that the simulated flow will induce same pond storage with time, as following:

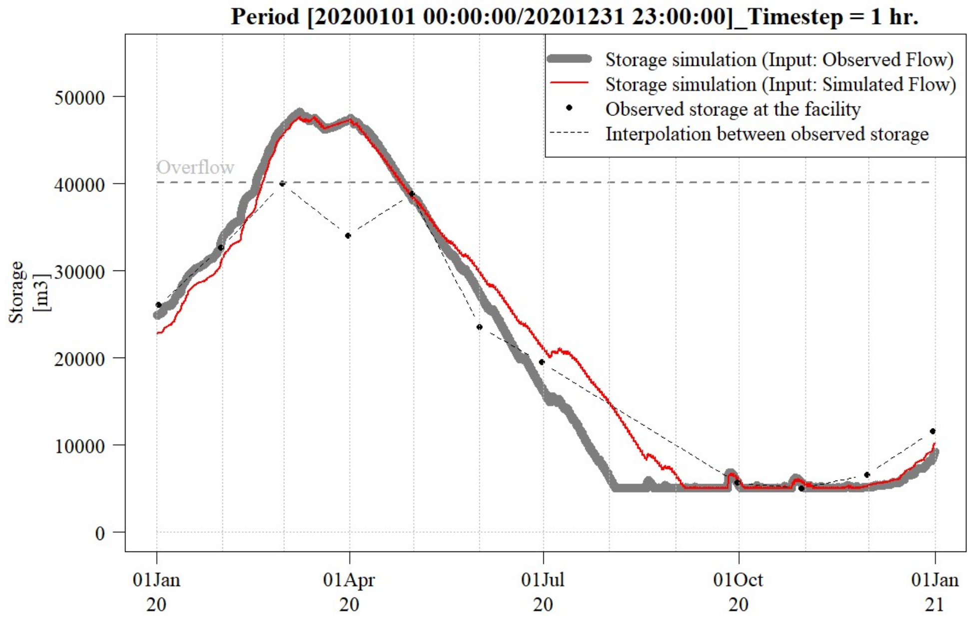

Figure 5.19. Leachate pond storage base simulation

The figure shows that the simulation with the catchment simulated flow did produce a similar result to the simulation with the observed flow, with a NSE of 0.96 comparing to the simulation done with the observed flow. This is indicating that the model can be used to simulate design rainfall events or the expected changes from the future climate change.

The peak above the overflow volume is attributed to that the output from the ponds (pumping) to the treatment plant, were adjusted in the facility to reach maximum pumping (maximum outflow) when the ponds were on the verge of overflowing, in which the observations from the facility showed that it has occurred between March 2020 and May 2020. It was decided to further simulate the climate change with the period of 2020 to allow the storage to reach beyond the overflow level, to get an indication if the ponds will overflow, and as more conservative approach, only if the pumping rules were kept solely dependent on the temperature. It is then understood that the simulated volume above the overflow volume is the amount of water that was “avoided” and that the ponds would have definitely overflown during this period unless the temperature rule were overridden.

It must be noted that since the ponds reached maximum capacity between March 2020 and May 2020, and that the facility personnel needed to override the temperature rule of pumping the outflow; it is considered that the storage of the ponds is already "not enough", where the current pond volume (40,000 m^3) is needed to be increased with an additional 7,800 m^3 to avoid overflow of the ponds, provided there is no override to pump larger flow during the cold periods. The subsequent analysis (the calculation of the required expansion) for the future climate will be based on the required increase from the current pond volume without any modifications (40,000 m^3), to accommodate the increased flow in the future climate.

The information from the facility shows that it is recommended to keep the relation between the temperature and pumping to avoid having problems in the required heating of the treatment plant, which is directly related to the effluent quality. Subsequently, it is decided to adopt the period of 2020 for further simulations as a more conservative approach.

5.3.1. Leachate ponds and the future climate

The leachate ponds storage has been simulated with the expected increase of precipitation and temperature as per the climate change scenarios mentioned in section 4.2.4. The input flow to the model is the output from the catchment simulation at the 3 scenarios. At the same time, the increased precipitation is affecting the ponds directly via direct precipitation, which is subsequently increased. Since the outflow to the treatment plant is dependent on the temperature, the pumping is subsequently increased with the increase of temperature. Thus, the inflow and outflow to/from the ponds are increased to some extent depending on the increased precipitation and temperature. The analysis of the required additional volume (future expansion of the ponds), is hereby based on the additional storage from the simulation done at the future climate, comparing to the storage simulated at the current climate.

Figure 5.20. shows the effect of the 3 scenarios on the storage (contained volume of the leachate ponds):

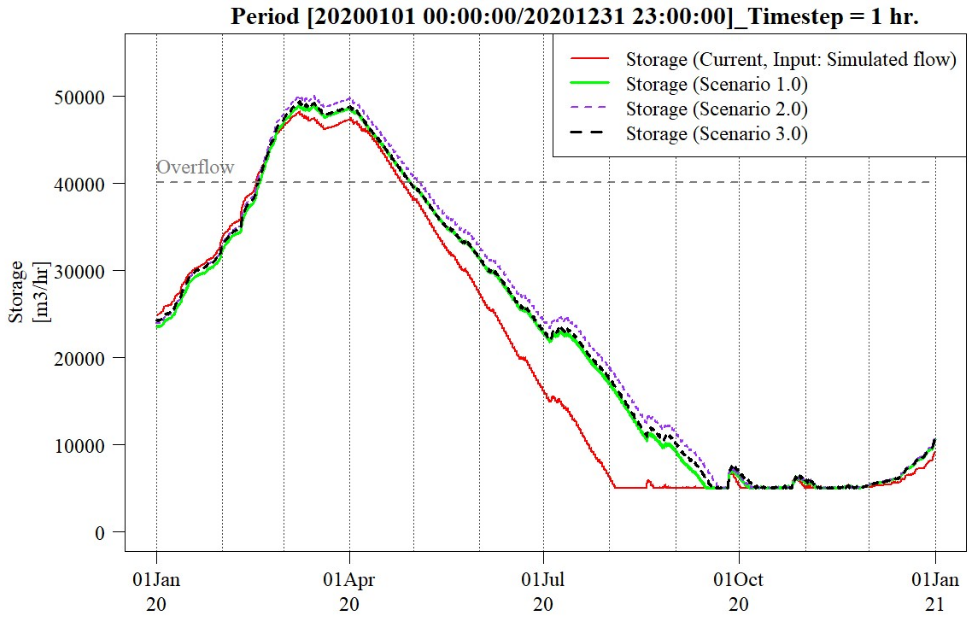

Figure 5.20. Climate change simulation of the leachate ponds

The figure shows that the peak storage is increased for each of the 3 scenarios, though Scenario 3.0 have showed a decreased peak than Scenario 2.0, in which the latter is characterized by less precipitation. The reason for such decrease is that the increase of temperature from 2 to 4 degrees have induced increased outflow (pumping from the ponds to the treatment plant). It can be noticed that the simulation is dependent on the pumping periods and the sequence of starting and stopping the pumping. An analysis of the above Figure 5.20. done by the calculation of the residual from the maximum peak of the main simulation (current climate) and the maximum peak of each of the 3 scenarios (future climate). Thus, the expected required additional volume of the leachate ponds to contain the highest peak are summarized in the below Table 5.5. and represented in Figure 5.21.

Table 5.5. Required additional volume to contain the daily peak flows for the leachate ponds.

Scenario	Period	Increase in temperature (+°C)	Precipitation percentage increase (%)	Current volume of the leachate ponds (m^3)	Additional required volume of the leachate ponds (m^3)
1.0	Near term (2021-2040)	1.5	10	40,000	9,100
2.0	Mid-term (2041-2060)	2	15		10,100
3.0	Long term (2061-2100)	4	25		9,500

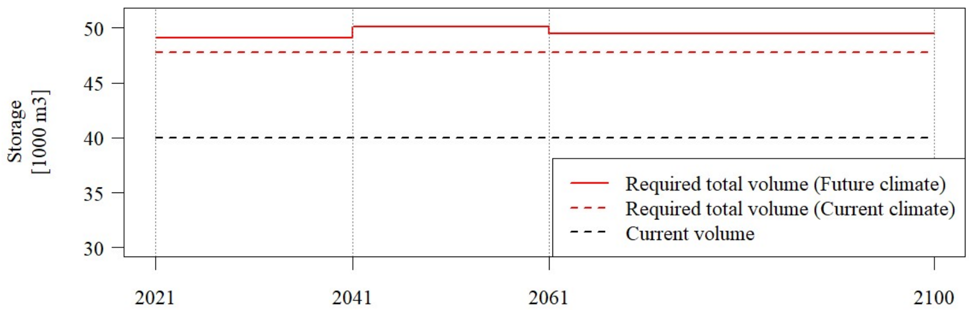

Figure 5.21. Required total volume of the leachate ponds (current + additional required volume)

It must be noted that since the required volume is dependent on the pumping from the ponds (towards the treatment plant), any change in the pumping sequence or the rule of temperature, will affect the values in the above table to a great degree. For example, if the pumps are discontinued during the peak flows for maintenance reasons, the contained volume of the pond will increase to the extent it will overflow. Several pumping scenarios have been studied and it has been noticed that careful coordination between the volume of the leachate ponds and the pumping to the treatment plant can keep the ponds from overflowing. Since the pumping is dependent on the temperature, and that there is no current plan from the facility to improve the capacity of the treatment plant in terms of additional heating equipment to allow for more pumping; the current storage (ca 40,000 m^3) will definitely overflow even at the events that is considered large and not only the extreme events. Since also it is noticed that the rule of temperature has been overridden (Figure 5.19), that the facility has faced the extreme event already that could have caused the ponds to overflow, and the only reason that prevented the overflow was to increase the outflow in addition to decrease the inflow from the catchment (which have been reported by the facility personnel during the same peak period).

It is up to the facility to choose the preventive measure for overflow of the ponds, in terms of increasing the capacity of the ponds (future expansion) or improve the treatment plant itself. It has been reported by the facility that there is a current plan to increase the capacity of the ponds by the construction of an additional connecting pond to allow more storage. This solution is considered acceptable if only the expansion is as per the calculated volumes mentioned in the previous Table 5.5. depending on the desired lifetime of the expansion.

It is noted in this work that the improvement of the treatment plan can greatly affect the storage of the leachate ponds, to the point that the expansion of ponds might not be needed. Yet, since there is no intention from the facility to improve or expand the treatment plant, the assessment of the required improvement is beyond the scope of this work.

In addition, a monitoring system is highly recommended to monitor the water level of the leachate ponds (i.e., volume of leachate contained) and synchronize the monitored volume with the outflow pumping to the treatment plant. Careful monitoring of the storage with synchronizing the pumping outflow is expected to optimize the leachate ponds to a great extent.

5.4. Modelling of future covering of the landfill

The old landfill is planned to be covered with a low-permeable cover that will affect the percolation of the rainfall to the landfill sub-layers. The precipitation that will not percolate, is expected to flow overland to the sides of the landfill. The analysis of the overland flow has been done by using the routing reservoirs in the model that represents the hard surfaces, with the catchment area of the old landfill to check for peak flows at the current and future climate. As mentioned in Section 4.4., it was decided to use the routing reservoirs of the hard surfaces and not the permeable surfaces in order to simulate the most extreme case of peak flows. This has been decided based on the steep slope of the landfill that will induce quicker runoff, and that the hard surfaces that exists in the landfill are dumping grounds for permeable materials that retain (delay) the runoff, so that the hard surfaces are not behaving as pure hard surfaces (ex. Roads), but with a degree of delay to the runoff.

The simulation was intimated with the full period of the main model simulation (2019-12-01 to 2021-11-07), considering the current (recorded) rainfall event, and modified as per the three scenarios of the climate change.

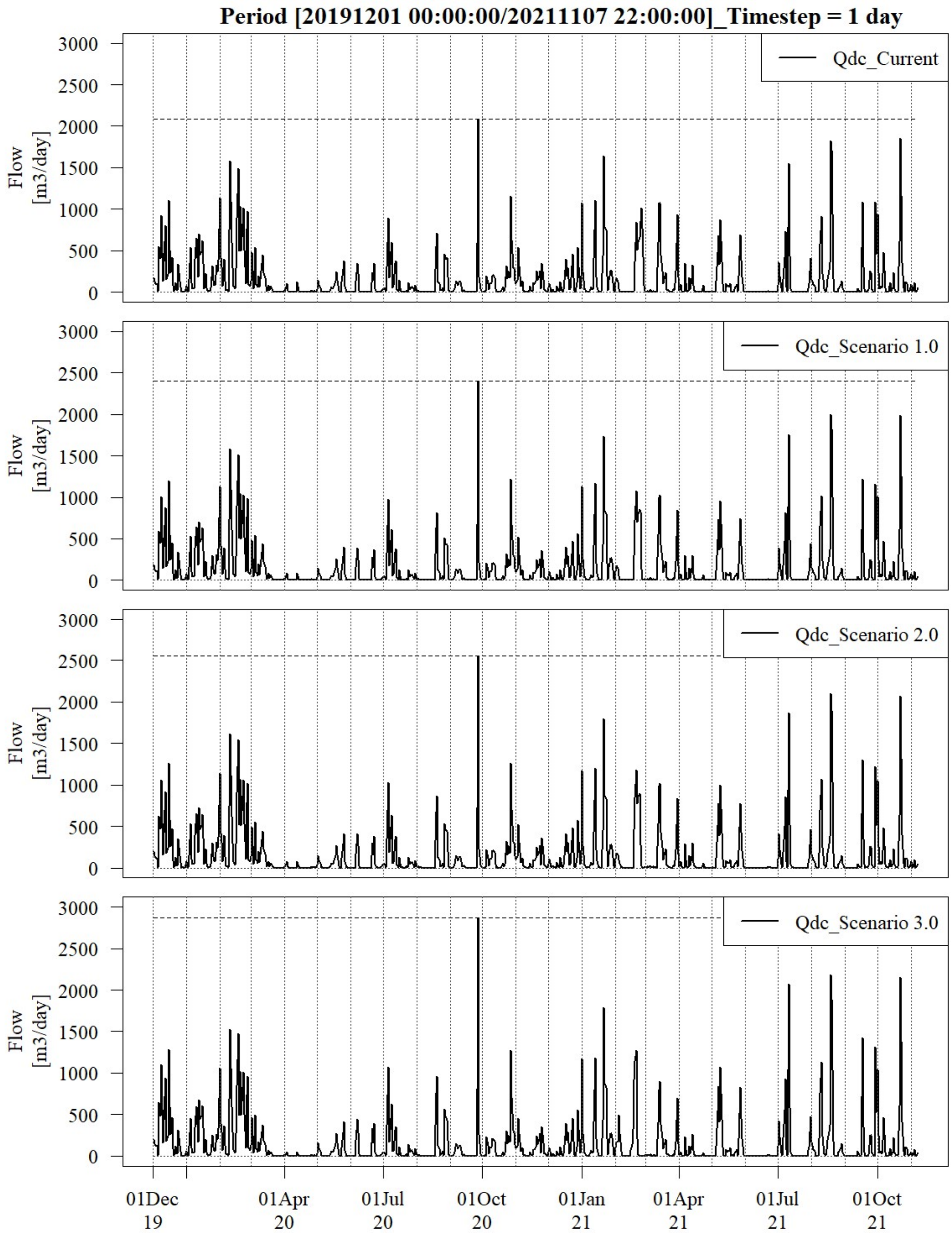

Figure 5.22. Landfill closing simulation

The results of the simulations are outlined in Figure 5.22. as accumulated daily values in order to give indication of the maximum daily peaks which the storm water ponds will be designed on accordingly.

It can be noticed from the above Figure 5.22. that the daily peaks of the expected runoff from the surface of the landfill are behaving with an increasing trend depending on the future climate scenarios, since the precipitation is increasing with each future scenario. As mentioned earlier in Section 4.4., it is expected that drainage ditches will be constructed around the old landfill leading to a stormwater pond for retaining the runoff. The calculation for the capacity of the stormwater ponds is hereby done by noting the maximum daily peaks from each scenario, summarized in the below table:

Table 5.6. Stormwater pond required volume to contain the overland flow from the surface of the landfill after permanent covering.

Scenario	Period	Increase in temperature (+°C)	Precipitation percentage increase (%)	Required volume for the stormwater pond (m^3)
1.0	Near term (2021-2040)	1.5	10	2,397
2.0	Mid-term (2041-2060)	2	15	2,559
3.0	Long term (2061-2100)	4	25	2,874

Thus, a 3,000 m^3 volume of the stormwater pond is expected to be sufficient as per the simulation conducted for a long term change up to the year 2100. Any alternative measure deemed acceptable by the facility other than constructing a stormwater pond should accommodate the abovementioned runoff volume to account for the future climate change.

6. Conclusion

The parsimonious lumped model constructed to simulate the catchment composed of the recycling facility and landfill did produce acceptable results in terms of the fitting of the simulated flow with the observed flow. It has been noted that the hourly model did not successfully capture all the peak flows comparing to the observed values and that there is a general degree of underestimation of peaks. Nonetheless, the daily accumulated values of the simulated flow showed an acceptable fitting of peak flows with the daily accumulated observed flow timeseries. Thus, combing the drainage network complexity with the intertwining hydraulic structures in a lumped representation can open possibilities to simulations of the same nature when the available data is not sufficient or when there no available resources to pursue a detailed hydraulic modelling.

It must also be noted that the simulation of a complex system such as old age landfills is proved to be possible by lumped conceptual models, and that the degree of complexity in the flow dynamics through the landfill layers can be "damped" when utilizing a lumped representation of the landfill itself and the connected drainage network. The simulation of the landfill showed a considerable stable outflow with very delayed fluctuations. The simulation of the old landfill with the connected semi-urban catchment that is usually surrounds landfills within recycling facilities can be simulated in a lumped conceptual model and produce a "fair" hourly result and an acceptable daily values that is matching the observations to a great degree.

The analysis for the routing reservoirs did match the expectations in terms of the degree of delay from each flow type representing the flows from the hard, permeable surfaces, and the old landfill. That is indicating that system understanding is considered a very important factor in the construction of similar models.

The nature of the conceptual models in terms of parameter indefinability have been noted during the simulation since the model could produce the same outcome with different parameter combinations. The addition or omission of an additional flow of a considerable small magnitude (ex. groundwater) does not have a significant effect on the model outcome, since the parameters can be adapted to delay the increased inflow to a greater extent.

A mixture of manual and automated calibration is recommended for similar cases of model structures. The manual intervention is important to ensure the automated calibration to converge and to “guide” the parameters to their optimum value. It is also noted that the simplest model structure that can produce the same result is preferable in order to decrease complexity and computational time.

The catchment simulation is coupled with a reservoir model for the leachate storage ponds downstream the catchment outflow, and the simulation have showed a great degree of compliance with the observed values with NSE of 0.96. It must be noted that the attenuation effect of the storage ponds did “flatten” the residuals between the observed and simulated flow that resulted to this conformity.

Subsequent simulations of the future expected climate change for the short-, mid-, and long-term expected changes showed that the storage ponds are insufficient and that the expected increased precipitation will cause the leachate ponds to overflow unless an expansion is made as per the values recommended in this study. The recommended additional volume on top of the current volume (40,000 m^3) is 7,800 m^3 for the current climate to avoid overflow, 9,100 m^3 for the near term and 10,100 m^3 for the mid- and long-term effects of climate change. The recommendations are based on the extreme flow in year 2020 and increased precipitation and temperature representing future climate. No statistical analysis was done in order to verify the return period of the extreme flow in 2020. The need for additional volume in the storage ponds is also affected by the capacity of the treatment plant. An additional capacity would allow additional outflow from the ponds. In addition, by incorporating a system for online monitoring of the ponds storage while synchronizing with the pumping periods, storage in the ponds could be optimized.

An additional simulation has been initiated to simulate the expected future covering of the landfill with a low-, or non-permeable cover that will limit the percolation to the centre of the landfill and will induce a quicker overland flow to the base of the old landfill surface. The model has been developed using routing reservoirs of a more quicker flow type with the catchment area of the landfill to simulate the resulted peak flows. The analysis has been conducted for the current and the future climate change. The results of the analysis showed the need for a retention stormwater pond to contain the maximum daily peak with a recommended volume of 3,000 m^3 to detain the runoff from the landfill surface in the current and the future scenarios.

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Appendix I

Parameters

```
tw <- 1171.633
f <- 3.664
Tr <- 2
c <- 0.012
p <- 0.514

Cd <- 0.042
Cf <- 0.021
WHC <- 0.1

ah1 <- 0.986
ah2 <- 1.443
ah3 <- 0.096

ap1 <- 0.043
ap2 <- 0.127
ap3 <- 0.027

ad1 <- 0.739
ad2 <- 0.109
ad3 <- 0.114

nh1 <- 1
nh2 <- 1
nh3 <- 1

np1 <- 1
np2 <- 1
np3 <- 1

nd1 <- 0.240
nd2 <- 2.099
nd3 <- 0.732

Sch <- 3.036
Scp <- 5.553
Scd <- 4.727

bhinf <- 0.001
mhinf <- 0.010

bpinf <- 0.003
mpinf <- 0.029

bdinf <- 0.202
mdinf <- 0.021
```

Main model (catchment)

```
#setting timezone
Sys.setenv(TZ = "Europe/Stockholm")

#loading xts package
library(xts)

#Start time of simulation
start_time <- Sys.time()

#data import and conversion
obsflow <- read.csv("Observed flow.csv", header=T, sep=";")
precip <- read.csv("Precipitation.csv", header=T, sep=";")
airTemp <- read.csv("Temperature.csv", header=T, sep=";")

#identify time and date format
tobs <- as.POSIXct(strptime(obsflow$Index,"%Y-%m-%d %H:%M:%S",
tz="Europe/Stockholm"))
tprecip <- as.POSIXct(strptime(precip$Index,"%Y-%m-%d %H:%M:%S",
tz="Europe/Stockholm"))
ttemp <- as.POSIXct(strptime(airTemp$Index,"%Y-%m-%d %H:%M:%S",
tz="Europe/Stockholm"))

#convert observed flow to xts
flow <- xts(x= obsflow$m3, order.by = tobs)

#convert precipitation to xts
I <- xts(x= precip$mm, order.by = tprecip)

#convert temperature to xts
Temp <- xts(x= airTemp$degree_C, order.by = ttemp)

#create identifier with 1 and 0 for the observed flow data
ones <- replicate(length(tobs), 1)
eliminator <- xts(x = ones, order.by = tobs)

#eliminate the periods of no data (6 days)
eliminator['2019-09-02 00:00:00/2019-09-02 23:00:00'] <- 0
eliminator['2020-09-01 00:00:00/2020-09-01 23:00:00'] <- 0
eliminator['2020-09-13 00:00:00/2020-09-13 23:00:00'] <- 0
eliminator['2020-12-13 00:00:00/2020-12-13 23:00:00'] <- 0
eliminator['2021-06-25 00:00:00/2021-06-25 23:00:00'] <- 0
eliminator['2021-11-01 00:00:00/2021-11-01 23:00:00'] <- 0

#elimination of zero observed flow values
eliminator[index(flow[which(flow==0)])] <- 0

#fill zero values of the observed flow by interpolation
flow <- na.approx(replace(flow, flow == 0, NA))

#assigning area sqm for each flow type (h=hard surface, p=permeable, d=old landfill)
Ah <- 42580
Ap <- 31525
Ad <- 105184
```

```
A <- Ah+Ap+Ad

#calibration period
period <- "20191201/20210507"
t_period <- index(I)

#climate change
precip_change_frac <- 1
Temp_change_inc <- 0

I <- I * precip_change_frac
Temp <- Temp + Temp_change_inc

#for each reservoir: dS/dt = Q(i+1)-Qi where Q=a*S^n
#a: reservoir discharge rate, n: reservoir linearity
#binf: losses discharge rate, minf: losses linearity
#Sc: reservoir critical storage
#flow types: h=hard surface, p=permeable surface, d=landfill

#initiating montecarlo loop sequence
#k: number of montecarlo runs

for (k in 1:1) {

  #multiplicators
  MULT1 = 0.95
  MULT2 = 1.05

  #snow model parameters
  Cd <- runif(1, best_Cd*MULT1, best_Cd*MULT2)
  Cf <- runif(1, best_Cf*MULT1, best_Cf*MULT2)
  WHC <- runif(1, best_WHC*MULT1, best_WHC*MULT2)

  #soil model parameters
  tw <- runif(1, best_tw*MULT1, best_tw*MULT2)
  f <- runif(1, best_f*MULT1, best_f*MULT2)
  Tr <- runif(1, best_Tr-1, best_Tr+1)
  c <- runif(1, best_c*MULT1, best_c*MULT2)
  p <- runif(1, best_p*MULT1, best_p*MULT2)

  #catchment model parameters
  ah1 <- runif(1, best_ah1*MULT1, best_ah1*MULT2)
  ah2 <- runif(1, best_ah2*MULT1, best_ah2*MULT2)
  ah3 <- runif(1, best_ah3*MULT1, best_ah3*MULT2)

  ap1 <- runif(1, best_ap1*MULT1, best_ap1*MULT2)
  ap2 <- runif(1, best_ap2*MULT1, best_ap2*MULT2)
  ap3 <- runif(1, best_ap3*MULT1, best_ap3*MULT2)

  ad1 <- runif(1, best_ad1*MULT1, best_ad1*MULT2)
  ad2 <- runif(1, best_ad2*MULT1, best_ad2*MULT2)
  ad3 <- runif(1, best_ad3*MULT1, best_ad3*MULT2)

  nh1 <- 1
  nh2 <- 1
  nh3 <- 1
```

```
np1 <- 1
np2 <- 1
np3 <- 1

nd1 <- runif(1, best_nd1*MULT1, best_nd1*MULT2)
nd2 <- runif(1, best_nd2*MULT1, best_nd2*MULT2)
nd3 <- runif(1, best_nd3*MULT1, best_nd3*MULT2)

bhinf <- runif(1, best_bhinf*MULT1, best_bhinf*MULT2)
mhinf <- runif(1, best_mhinf*MULT1, best_mhinf*MULT2)

bpinf <- runif(1, best_bpinf*MULT1, best_bpinf*MULT2)
mpinf <- runif(1, best_mpinf*MULT1, best_mpinf*MULT2)

bdinf <- runif(1, best_bdinf*MULT1, best_bdinf*MULT2)
mdinf <- runif(1, best_mdinf*MULT1, best_mdinf*MULT2)

Sch <- runif(1, best_Sch*MULT1, best_Sch*MULT2)
Scp <- runif(1, best_Scp*MULT1, best_Scp*MULT2)
Scd <- runif(1, best_Scd*MULT1, best_Scd*MULT2)

#snow model

I <- I

mf <- c()
SWE <- c()
Wcurr <- c()
Wmax <- c()
W <- c()
q <- c()

SWE[1] = 0
W[1] = 0

for (i in 2:(length(I))) {

  #mf
  if (Temp[i] >= 0) {
    if (SWE[i-1] > 0) { mf[i] = Temp[i]*Cd } else { mf[i] = 0 }
  } else {
    if (I[i] > 0) { mf[i] = Temp[i]*Cf } else { mf[i] = 0 }
  }
  #SWE
  if (Temp[i] > 0)
  {
    if ( ( SWE[i-1]-mf[i] ) > 0 ) {
      SWE[i] = SWE[i-1]-mf[i] } else { SWE[i] = 0 }
  } else {
    if ( ( SWE[i-1]-mf[i]+I[i] ) > 0 ) {
      SWE[i] = SWE[i-1]-mf[i]+I[i] } else { SWE[i] = 0 }
  }
  #Wcurr
  if (Temp[i] > 0)
  {
```

```
    if ( ( mf[i]+I[i]+W[i-1] ) > 0 ) {
      Wcurr[i] = mf[i]+I[i]+W[i-1] } else { Wcurr[i] = 0 }
  } else {
    if ( ( mf[i]+W[i-1] ) > 0 ) {
      Wcurr[i] = mf[i]+W[i-1] } else { Wcurr[i] = 0 }
  }
  #Wmax
  Wmax[i] = SWE[i]*WHC
  #W
  if(Wmax[i]<=Wcurr[i]) { W[i] = Wmax[i] } else { W[i] = Wcurr[i] }
  #q
  q[i] = Wcurr[i] - W[i]

}

q_xts <- xts(x = q, order.by = tprecip)
q_xts[is.na(q_xts)] <- 0


#soil model (CWI)

I <- q_xts

tk = c()
sk = c()
uk = c()

tk[1] = tw * exp(0.062 * f * (Tr - Temp[1]))
sk[1] = I[1] + (1-(1/tk[1])*0)
uk[1] = I[1] * ((c * (sk[1]-1))^p)


for (i in 2:(length(I))) {

  tk[i] = tw * exp(0.062 * f * (Tr - Temp[i]))
  sk[i] = I[i] + ((1-(1/tk[i]))*sk[i-1])
  uk[i] = I[i] * ((c * (sk[i]-1))^p)
}

uk_xts <- xts(x = uk, order.by = tprecip)
uk_xts[is.na(uk_xts)] <- 0


#runoff model (catchment)

I <- uk_xts
I <- I*A/1000
Ih = I*Ah/A
Ip = I*Ap/A
Id = I*Ad/A

#flow from hard surfaces, type h

#first reservoir
Qh1 <- c()        #volume runoff first reservoir
Sh1 <- c()        #storage first reservoir
```

```
Sh1[1] = 0        #storage at time t = 0
Lh <- c()         #losses
for (i in 1:(length(Ih) - 1)) {
  if (Sh1[i] < 0) {
    Sh1[i] = 0
  }
  if (Sh1[i] < Sch) {
    Qh1[i] = 0
  } else {
    Qh1[i] = ah1*(Sh1[i] - Sch)^nh1
  }
  Lh[i] = bhinf*(Sh1[i]^mhinf)
  Sh1[i + 1] = Sh1[i] + Ih[i] - Qh1[i] - Lh[i]
}
if (Sh1[length(Ih)] < Sch) {
  Qh1[length(Ih)] = 0
} else {
  Qh1[length(Ih)] = ah1*((Sh1[length(Ih)]-Sch)^nh1)
}

#Second reservoir
Qh2 <- c()         #volume runoff second reservoir
Sh2 <- c()         #storage second reservoir
Sh2[1] = 0         #storage at time t = 0
for (i in 1:(length(Ih) - 1)) {
  Qh2[i] = ah2*(Sh2[i]^nh2)
  Sh2[i + 1] = Sh2[i] + Qh1[i] - Qh2[i]
}
Qh2[length(Ih)] = ah2*(Sh2[length(Ih)]^nh2)

#third reservoir
Qh3 <- c()          #volume runoff third reservoir
Sh3 <- c()          #storage third reservoir
Sh3[1] = 0          #storage at time t = 0
for (i in 1:(length(Ih) - 1)) {
  Qh3[i] = ah3*(Sh3[i]^nh3)
  Sh3[i + 1] = Sh3[i] + Qh2[i] - Qh3[i]
}
Qh3[length(Ih)] = ah3*(Sh3[length(Ih)]^nh3)

#flow from permeable surfaces, type p

#first reservoir
Qp1 <- c()       #volume runoff first reservoir
Sp1 <- c()       #storage first reservoir
Sp1[1] = 0       #storage at time t = 0
Lp <- c()        #losses
for (i in 1:(length(Ip) - 1)) {
  if (Sp1[i] < 0) {
    Sp1[i] = 0
  }
  if (Sp1[i] < Scp) {
    Qp1[i] = 0
  } else {
    Qp1[i] = ap1*((Sp1[i] - Scp)^np1)
  }
```

```
  Lp[i] = bpinf*(Sp1[i]^mpinf)
  Sp1[i + 1] = Sp1[i] + Ip[i] - Qp1[i] - Lp[i]
}
if (Sp1[length(Ip)] < Scp) {
  Qp1[length(Ip)] = 0
} else {
  Qp1[length(Ip)] = ap1*((Sp1[length(Ip)]-Scp)^np1)
}

#Second reservoir
Qp2 <- c()         #volume runoff second reservoir
Sp2 <- c()         #storage second reservoir
Sp2[1] = 0         #storage at time t = 0
for (i in 1:(length(Ip) - 1)) {
  Qp2[i] = ap2*(Sp2[i]^np2)
  Sp2[i + 1] = Sp2[i] + Qp1[i] - Qp2[i]
}
Qp2[length(Ip)] = ap2*(Sp2[length(Ip)]^np2)

#third reservoir
Qp3 <- c()          #volume runoff third reservoir
Sp3 <- c()          #storage third reservoir
Sp3[1] = 0          #storage at time t = 0
for (i in 1:(length(Ip) - 1)) {
  Qp3[i] = ap3*(Sp3[i]^np3)
  Sp3[i + 1] = Sp3[i] + Qp2[i] - Qp3[i]
}
Qp3[length(Ip)] = ap3*(Sp3[length(Ip)]^np3)

#flow from the old landfill, type d

#first reservoir
Qd1 <- c()       #volume runoff first reservoir
Sd1 <- c()       #storage first reservoir
Sd1[1] = 0       #storage at time t = 0
Ld <- c()        #losses
for (i in 1:(length(Id) - 1)) {
  if (Sd1[i] < 0) {
    Sd1[i] = 0
  }
  if (Sd1[i] < Scd) {
    Qd1[i] = 0
  } else {
    Qd1[i] = ad1*((Sd1[i] - Scd)^nd1)
  }
  Ld[i] = bdinf*(Sd1[i]^mdinf)
  Sd1[i + 1] = Sd1[i] + Id[i] - Qd1[i] - Ld[i]
}
if (Sd1[length(Id)] < Scd) {
  Qd1[length(Id)] = 0
} else {
  Qd1[length(Id)] = ad1*((Sd1[length(Id)]-Scd)^nd1)
}

#Second reservoir
Qd2 <- c()         #volume runoff second reservoir
```

```
Sd2 <- c()         #storage second reservoir
Sd2[1] = 0         #storage at time t = 0
for (i in 1:(length(Id) - 1)) {
  Qd2[i] = ad2*(Sd2[i]^nd2)
  Sd2[i + 1] = Sd2[i] + Qd1[i] - Qd2[i]
}
Qd2[length(Id)] = ad2*(Sd2[length(Id)]^nd2)

#third reservoir
Qd3 <- c()          #volume runoff third reservoir
Sd3 <- c()          #storage third reservoir
Sd3[1] = 0          #storage at time t = 0
for (i in 1:(length(Id) - 1)) {
  Qd3[i] = ad3*(Sd3[i]^nd3)
  Sd3[i + 1] = Sd3[i] + Qd2[i] - Qd3[i]
}
Qd3[length(Id)] = ad3*(Sd3[length(Id)]^nd3)

#convert the flow from each last reservoir (Q#3) to xts

Qh_xts <- xts(x = Qh3, order.by = t_period)
Qh_xts[is.na(Qh_xts)] <- 0

Qp_xts <- xts(x = Qp3, order.by = t_period)
Qp_xts[is.na(Qp_xts)] <- 0

Qd_xts <- xts(x = Qd3, order.by = t_period)
Qd_xts[is.na(Qd_xts)] <- 0

#combining all flows to one

Q_all <- Qh_xts + Qp_xts + Qd_xts

#hourly performance indicators
error <- eliminator[period] * (flow[period] - Q_all[period])
error_mean <- eliminator[period] * (flow[period] - mean(flow[period]))
NSE = 1 - sum(error^2)/sum(error_mean^2)

error_mean <- eliminator[period] * (flow[period] - mean(flow[period]))
errorsim_mean <- eliminator[period] * (Q_all[period] - mean(Q_all[period]))
error2_mean <- eliminator[period] * ((flow[period] - mean(flow[period]))^2)
errorsim2_mean <- eliminator[period] * ((Q_all[period] - mean(Q_all[period]))^2)
errormult_mean <- errorsim_mean * error_mean
R2 = (sum(errormult_mean)^2/((sum(errorsim2_mean))*(sum(error2_mean))))

NB = sum(error)/(length(flow[period])*mean(flow[period]))

measuredvolerr<- ((sum(flow[period]) - sum(Q_all[period]))/sum(flow[period]))*100

DV <- sum(Q_all[period])/sum(flow[period])

#daily indicators
D_Q_all <- apply.daily(Q_all, sum)
D_flow <- apply.daily(flow, sum)

D_error <- (D_flow[period] - D_Q_all[period])
```

```
  D_error_mean <- (D_flow[period] - mean(D_flow[period]))
  D_NSE = 1 - sum(D_error^2)/sum(D_error_mean^2)

  D_error_mean <- (D_flow[period] - mean(D_flow[period]))
  D_errorobs_mean <- (D_Q_all[period] - mean(D_Q_all[period]))
  D_error2_mean <- ((D_flow[period] - mean(D_flow[period]))^2)
  D_errorobs2_mean <- ((D_Q_all[period] - mean(D_Q_all[period]))^2)
  D_errormult_mean <- D_errorobs_mean * D_error_mean
  D_R2 =
(sum(D_errormult_mean)/((sum(D_errorobs2_mean)^0.5)*(sum(D_error2_mean)^0.5)))^2

  D_NB = sum(D_error)/(length(D_flow[period])*mean(D_flow[period]))

  if (NSE >= maxNSE) {

    maxNSE <- NSE

    best_tw <- tw
    best_f <- f
    best_Tr <- Tr
    best_c <- c
    best_p <- p

    best_Cd <- Cd
    best_Cf <- Cf
    best_WHC <- WHC

    best_ah1 <- ah1
    best_ah2 <- ah2
    best_ah3 <- ah3

    best_ap1 <- ap1
    best_ap2 <- ap2
    best_ap3 <- ap3

    best_ad1 <- ad1
    best_ad2 <- ad2
    best_ad3 <- ad3

    best_nh1 <- nh1
    best_nh2 <- nh2
    best_nh3 <- nh3

    best_np1 <- np1
    best_np2 <- np2
    best_np3 <- np3

    best_nd1 <- nd1
    best_nd2 <- nd2
    best_nd3 <- nd3

    best_Sch <- Sch
    best_Scp <- Scp
    best_Scd <- Scd

    best_bhinf <- bhinf
```

```
    best_mhinf <- mhinf

    best_bpinf <- bpinf
    best_mpinf <- mpinf

    best_bdinf <- bdinf
    best_mdinf <- mdinf

    best_Q_all <- Q_all

  }
}

#montecarlo-loop end

#end time of simulation
end_time <- Sys.time()

#simulation duration
sim_duration <- end_time - start_time
```

Storage model (leachate ponds)

```
#loading packages
library(xts)
library(lubridate)

#Importing observed storage from the facility
rec_lak_st <- read.csv("Recorded Storage Lakdammar.csv", header=T, sep=";")
treclak <- as.POSIXct(strptime(rec_lak_st$Index,"%Y-%m-%d %H:%M:%S",
tz="Europe/Stockholm"))
reclakst <- xts(x= rec_lak_st$Storage, order.by = treclak)

#leachate ponds (Lakvattendammar)
#bottom level of Lakdamm1 = 66.750m + 0.300m (sedimentation) = 67.050m
#bottom level of Lakdamm2 = 67.500m + 0.300m (sedimentation) = 67.800m

#ponds area
Alak1_bot <- 4566 #bottom area of Lakdamm1 (m2)
Alak1_dr <- 6475 #area of Lakdamm1 at stage 70.020m (drain level) (m2)
Alak1_top <- 6769 #catchment area of Lakdamm1 (stage 70.500m) (m2)

Alak2_bot <- 10083 #bottom area of Lakdamm2 (m2)
Alak2_dr <- 11008 #area of Lakdamm2 at stage 70.090m (drain level) (m2)
Alak2_top <- 11152 #catchment area of Lakdamm2 (stage 70.500m) (m2)

#maximum allowed volume of Lakdamm 1 at drainage level (70.020m) = 16597 m3
#maximum allowed volume of Lakdamm 2 at drainage level (70.090m) = 24269 m3
#maximum allowed volume of Lakdamm 2 at stage (70.020m) = 23499 m3
#thus maximum allowed volume of the two ponds = 16597 + 23499 = 40096 m3
allowedvol <- 40096

#initiating model
Slak <- c()
Slak_sim <- c()
Slak[1] <- 15000 #starting storage t = 0
Slak_sim[1] <- 15000 #starting storage t = 0 (simulated)

#level of stopping pumping
lowlevel <- 5054

#correction (infiltration)
corr <- 2.1

#in and out flow separation
in_damm <- c()
out_damm <- c()

in_damm_sim <- c()
out_damm_sim <- c()

#calculation period
period <- "20200101 00:00:00/20201231 23:00:00"
P <- P[period]
Q_all <- Q_all + corr
Q_all <- Q_all[period]
```

```
flow <- flow[period]
tprecip <- index(P)

#incoming flow to the pond is the simulated flow
#pumped flow from the ponds = 50 m3/hr towards the treatment plant
#treatment plant maximum capacity is 500 m3/day (10 hrs of pumping/day)
#treatment plant minimum capacity is 100 m3/day (2 hrs of pumping/day)
#during [dec-feb] pumping is 100 m3/day (2 hrs of pumping/day)
meanTemp <- apply.daily(Temp, mean)
series <- xts(replicate(length(P), 0), order.by = tprecip)
sp_meanTemp <- series + merge(meanTemp, index(series))
sp_meanTemp <- na.locf(sp_meanTemp, fromLast=TRUE)

maxTemp <- apply.daily(Temp, max)
series <- xts(replicate(length(P), 0), order.by = tprecip)
sp_maxTemp <- series + merge(maxTemp, index(series))
sp_maxTemp <- na.locf(sp_maxTemp, fromLast=TRUE)

Qren <- replicate(length(P), 0)
Qren <- xts(Qren, order.by = tprecip)
Qren["T08/T09"] <- 50
Qren[sp_meanTemp > 5]["T08/T17"] <- 50
Qren[((month(Qren) %in% c(1:2) | month(Qren) %in% 12) & hour(Qren) %in% c(10:17))] <-
0

QrenSt <- Qren
Qren <- QrenSt

#the two ponds will be treated as one pond since they are connected

#average annual evaporation from medium sized lakes 600 mm/year (SMHI)
Evap <- 600/365/24 #year to hour
Evap <- replicate(length(P), Evap)
Evap <- xts(Evap, order.by = tprecip)
Evap <- Evap[period]

#infiltration (adpopted from the facility environmental report)
Infilt = 0.9

#climate change
P <- P * precip_change_frac
Temp <- Temp + Temp_change_inc

for (i in 1:(length(P)-1)) {

  if (Slak[i] < lowlevel) {
    Qren[i] = 0
  }
    in_damm[i] = flow[i] + (P[i]*(Alak1_top + Alak2_top)/1000)
  out_damm[i] = Qren[i] + (Evap[i]*(Alak1_curr + Alak2_curr)/1000) + Infilt

  Slak[i+1] = in_damm[i] - out_damm[i] + Slak[i]
}

  Slak[length(P)] = in_damm[length(P)-1] - out_damm[length(P)-1]
                         + Slak[length(P)-1]
```

```
Slak_xts <- xts(x = Slak, order.by = tprecip)
Slak_xts[is.na(Slak_xts)] <- 0
#end of analysis

#resetting pumping periods
Qren <- QrenSt

#same calculation with the simulated flow
for (i in 1:(length(P)-1)) {
     if (Slak_sim[i] < lowlevel) {
      Qren[i] = 0
     }

  in_damm_sim[i] = Q_all[i] + (P[i]*(Alak1_top + Alak2_top)/1000)
  out_damm_sim[i] = Qren[i] + (Evap[i]*(Alak1_curr + Alak2_curr)/1000) + Infilt
  Slak_sim[i+1] = in_damm_sim[i] - out_damm_sim[i] + Slak_sim[i]
}

Slak_sim[length(P)] = in_damm_sim[length(P)-1] - out_damm_sim[length(P)-1]
+ Slak_sim[length(P)-1]

Slak_xts_sim <- xts(x = Slak_sim, order.by = tprecip)
Slak_xts_sim[is.na(Slak_xts_sim)] <- 0
#end of analysis

#summary
summary(flow)
summary(Q_all)
summary(Slak_xts)
summary(Slak_xts_sim)
summary(Q_all)
summary(in_damm)
summary(out_damm)

#NSElak
calcperiod <- "20200101 00:00:00/20201231 23:00:00"
errorlak <- eliminator[calcperiod] * (Slak_xts[calcperiod] -
Slak_xts_sim[calcperiod])
error_meanlak <- eliminator[calcperiod] * (Slak_xts[calcperiod] -
mean(Slak_xts[calcperiod]))
NSE_lak = 1 - sum(errorlak^2)/sum(error_meanlak^2)
```