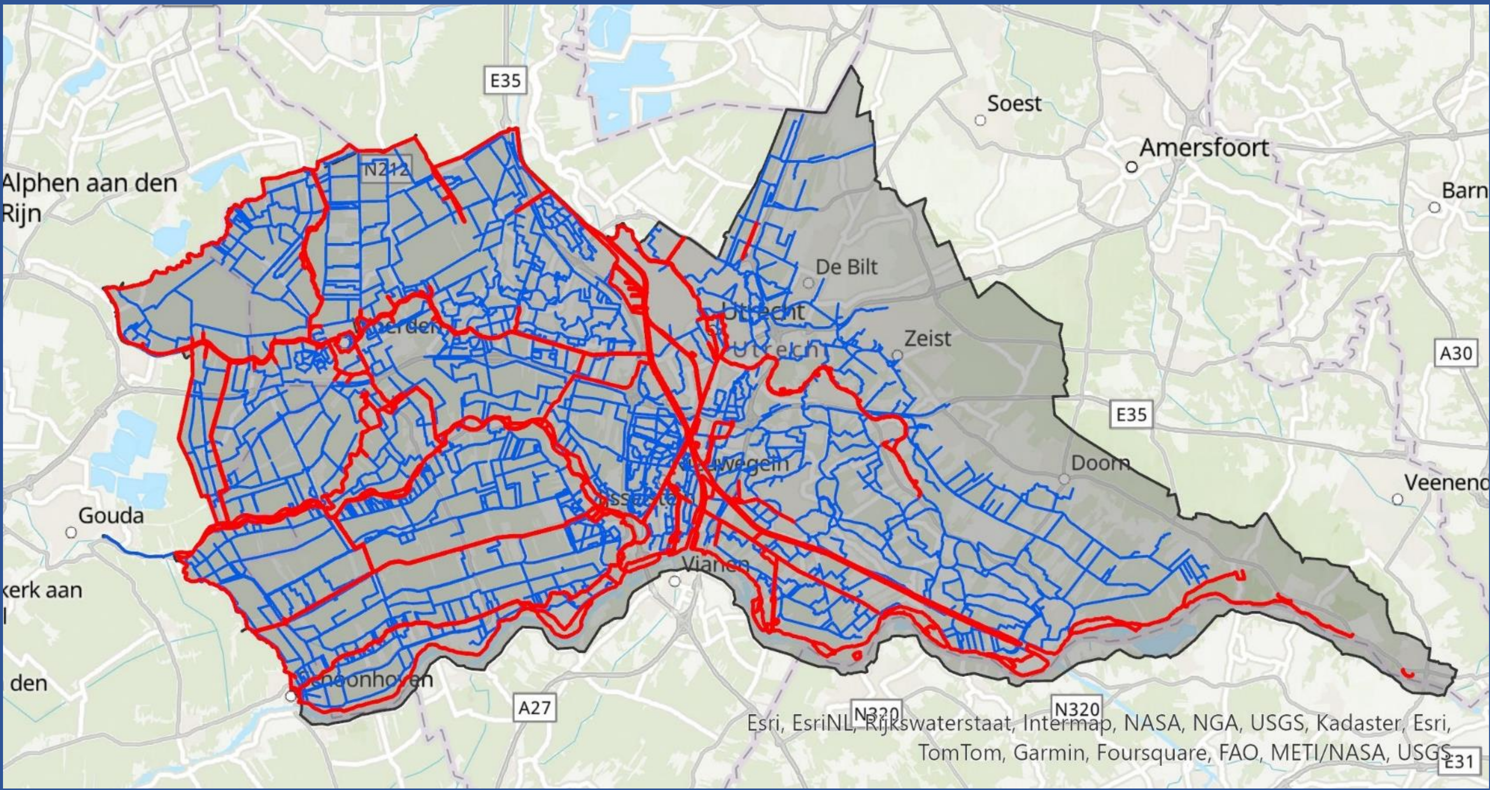

Accelerating a detailed 1D2D hydrodynamic model

Optimising the calculation time of a detailed 1D2D hydrodynamic model by resolving numerical instabilities and applying numerical simplifications

Bachelor Thesis Civil Engineering

Final version

June 19, 2024

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HydroLogic

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HydroLogic

Preface

In front of you lies the final version of my Bachelor thesis Accelerating a detailed 1D2D hydrodynamic model. This thesis completes my Bachelor's degree in Civil Engineering at the University of Twente.

First of all, I would like to thank HydroLogic for providing an interesting case study on which I could work. I enjoyed working in the office with my colleagues. I still cannot believe that I can stay at HydroLogic during my Master's studies to work for one day a week on similar projects. I am looking forward to this period and am pleased that this arose from this Bachelor thesis assignment.

While writing my thesis, I have had support from many people, for which I am very grateful. To start, I would like to thank Laura Janssen, my supervisor at HydroLogic, for her support and feedback. The possibility to always ask questions about everything, especially D-HyDAMO and D-HYDRO, facilitated my research and allowed me to do what I wanted much faster. Your feedback was also incredibly helpful in ameliorating the quality of my thesis. In addition, I would like to thank my supervisor Anouk Bomers from the University of Twente for the valuable feedback which I received every time unprecedentedly quickly after sending the document. This feedback certainly helped take my thesis to the next level. Hopefully, it did not get annoying that I asked for feedback so often.

Lastly, I want to thank family and friends for their support and encouragement.

Feel free to contact me about this document via pepijnvandiggelen@gmail.com.

I hope you enjoy reading my thesis.

Pepijn van Diggelen
Hall, June 19, 2024

Summary

Large parts of the Netherlands are located below sea level and face the threat of fluvial flooding. On the other hand, the intensity of extreme precipitation is expected to increase leading to a larger probability of pluvial flooding. To successfully incorporate potential future threats in the current water management strategy, it is important to be able to generate accurate predictions of the effects of these events. Hydrodynamic simulation models provide a solution to see the effects of extreme events on the water system but have long computation times, making them unsuitable for real-time use and evaluation of a large set of events.

The objective of this study is to get insight into methods that can accelerate these hydrodynamic simulation models to make them better suitable for application in water management practices. The research is executed with the help of a case study from HydroLogic. This case study includes a highly detailed D-HYDRO model of the study area of Hoogheemraadschap de Stichtse Rijnlanden (HDSR), which is a Dutch water board. HDSR will use the model in the future to identify bottlenecks in the water system. In this procedure, several thousands of different rainfall events must be simulated. At the start of the research, the model computed 15 times slower than reality, making it unsuitable for simulating all these events.

Two approaches are researched that can accelerate the model. First, the effect of numerical instabilities on the computation time is examined. Numerical instabilities arise if numerical errors grow and the output starts to diverge. This limits the time step that can be taken, resulting in long computation times. Indicators for numerical instabilities are high flow velocities, water depths of zero meters and waterways that fill or empty quickly. Several of these locations are present in the original model of the case study and are examined. This resulted in a list of seventeen causes of numerical instabilities. Most of these numerical instabilities are related to poor data quality and incorrect model construction. By obtaining missing data and improving the model construction, most numerical instabilities are solved. As a consequence, the computation time is reduced by 96%. Furthermore, by correcting the data and enhancing the model construction, the optimised model is also more accurate. Since resolving numerical instabilities can drastically advance the computation time while improving the accuracy, it is advised to always check for numerical instabilities in detailed 1D2D hydrodynamic models and solve potential issues.

Secondly, numerical simplifications are implemented in the optimised model to obtain surrogate models. Surrogate models approximate the detailed hydrodynamic model and are therefore faster, but come with a loss in accuracy. The tested numerical simplifications constitute a reduced 2D grid resolution, a reduction in the number of 1D calculation points and an increased maximum Courant number. Reducing the 2D grid resolution and number of 1D calculation points reduces the computation time by 2% to 57% but also introduces significant errors in the simulated water level of more than 5 cm. Increasing the maximum Courant number is the most efficient and can reduce the computation time of the optimised model by more than 90% while having an error in the water level that is less than 0.5 cm. A maximum Courant number of 5, the default is often 0.7 in detailed hydrodynamic models, seems to be the best in the trade-off between computation time and accuracy.

Overall, it can be concluded that resolving numerical instabilities is an effective first step in reducing the computation time of complex 1D2D hydrodynamic models since the accuracy will often improve as well during this step. If a larger reduction in computation time is required, the maximum Courant number can be increased between 1 and 5 depending on the required accuracy and computation time savings for the specific project.

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1 Introduction

Large parts of the Netherlands are located below sea level and several large rivers cross the country on their way to the sea. In combination with the sea level rise caused by climate change, the Netherlands faces an increasing threat of flooding from these rivers (Rijkswaterstaat, 2019). On the other hand, the intensity of extreme precipitation is expected to increase (KNMI, 2021), which can cause more pluvial flooding (Hofmann & Schüttrumpf, 2020). To successfully incorporate potential future threats in the current water management strategy, it is important to be able to generate accurate predictions of the effects of these events. Hydrodynamic simulation models provide a solution to see the effects of extreme events on the water system.

The basis of most of these models is a 2D raster grid, that allows the simulation of water flows in both horizontal directions (Zhao et al., 2021). The flow is averaged for the depth. For each time step, equations are solved for each raster cell. These detailed hydrodynamic models provide accurate results but have a significant computation time, especially for large areas (Wang et al., 2019). This makes the models unserviceable for real-time flood predictions and analysis of a large set of events (Zhao et al., 2021). A new trend is currently seen in hydrodynamic modelling practices, which make use of surrogate models. These surrogate models approximate the original detailed hydrodynamic model such that the computational load is reduced (Razavi et al., 2012). Subsequently, these simplified models are used in real-time predictions and sensitivity analysis.

Several methods exist to construct surrogate models. Most methods simplify the original model, which is also called the "high-fidelity" model. For instance, the time step can be enlarged to speed up the original model. Other options are to reduce the grid resolution, decrease the number of calculation points or scale down the dimensions that are modelled (Bomers et al., 2019a; Razavi et al., 2012). A major question evolves from these simplifications and that is to which extent these surrogates are accurate when compared to the original model.

Various software packages exist that can simulate hydrodynamics, for example, SOBEK, HEC-RAS, and TYGRON (Teng et al., 2017). A relatively new addition to the field is the D-HYDRO software that is developed by Deltares. It can be seen as the successor of SOBEK (Deltares, n.d.). Because D-HYDRO is a relatively new modelling software, limited research is executed on accelerating models in this software by means of surrogates. The topic of this research explores therefore the practice of surrogate modelling with the D-HYDRO software, although the results might also apply to other modelling software.

This thesis is structured as follows. Chapter 2 will provide a theoretical framework for this research. The basics of hydrodynamic modelling will shortly be touched upon. The main body of this chapter consists of a literature study on available methods to advance hydrodynamic models. Section 3 will clearly show the knowledge gap that follows from the literature study. Then a research objective is formulated that helps to bridge the knowledge gap. Based on this research aim, the research questions are drafted. Subsequently, the case study, including the involved parties, study area and D-HYDRO model, will be introduced in Chapter 4. Section 5 discusses the methods that will be applied to answer the research questions. The results will be presented in Chapter 6 and discussed in Section 7. Chapter 8 will answer the research questions and recommendations will be made for further research.

2

Theoretical framework

This section examines the existing literature on the topic of detailed 1D2D hydrodynamic models and covers background information on the software D-HYDRO. Moreover, it explores what type of model simplifications can be used to reduce the computation time of a detailed 1D2D hydrodynamic model. This will lead to the formulation of the research objective and questions in the next chapter.

2.1 Dimensions in hydraulic modelling

This section will briefly touch upon the basic classification of hydraulic models since this is used in almost all literature on the topic covered by this thesis. This categorization focuses on the degree of complexity of the model, see *Figure 1* for an overview of hydraulic models. Zero-dimensional (0D) models represent a certain enclosed area as a container. The water level in the container is based on the total volume that flows into the compartment and is assumed to be equal in the whole area. This method is extremely fast and easy to implement but is only suitable for small, enclosed areas without a slope (De Bruijn, 2018).

One-dimensional models (1D) calculate the hydraulic parameters along the river axis for certain cross-sections. Water depths and flow velocities can be calculated with these models (Teng et al., 2017). The number of cross-sections is often restricted such that the computation time remains low (De Bruijn, 2018). 1D models only calculate the data along the cross-sections, which introduces subjectivity about which cross-sections are used. Moreover, this method is less accurate in sharp bends (Hunter et al., 2007; Teng et al., 2017). Lastly, flooding is difficult to model with 1D as flood water often spreads in multiple directions, which cannot be represented well with 1D (Zhao et al., 2021). A 1D model that has side branches is often called Quasi-2D (De Bruijn, 2018).

Two-dimensional models (2D) work with a raster of cells that partially solve the problems of 1D models. The water is allowed to flow in both horizontal directions. One large constraint on the use of 2D models is the required computation time, which can be high for models with a high resolution (De Bruijn, 2018; Razavi et al., 2012; Teng et al., 2017). This can especially be difficult for modelling channels as these require a high resolution to yield accurate flow patterns with a 2D simulation (Zhao et al., 2021).

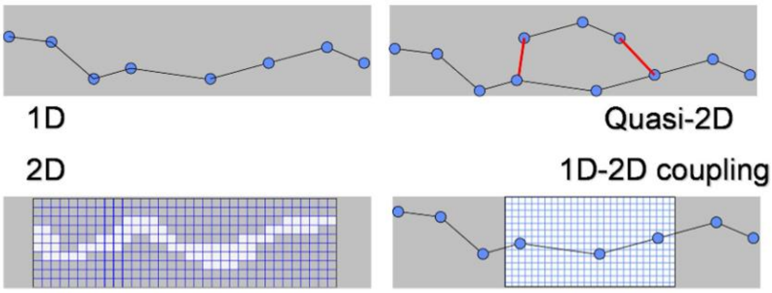

Figure 1 - Overview of model dimensions (from De Bruijn, 2018).

Three-dimensional models (3D) add equations to the 2D model that take into account variable flow characteristics in the depth dimension. These models can be more accurate than 2D models but result in more computational load and can be difficult to set up (Teng et al., 2017; Dahm et al., 2014).

Lastly, 1D2D models combine 1D flow in rivers and a 2D flow raster in the surrounding area. This limits the computation time related to the river characteristics while being able to accurately model flood propagation in the surrounding areas (De Bruijn, 2018). The same holds for a 1D3D model.

2.2 D-HYDRO

The D-HYDRO software package is used in this research to construct and simulate a detailed 1D2D hydrodynamic model. It is developed by Deltares and can simulate a variety of different water-related processes such as tsunamis, waves, water levels, river morphology, sediment transport and storm surges. The package contains different modules that are each developed for modelling one specific water-related process although all modules can perfectly be integrated (Deltares, n.d.).

This research will focus on D-FLOW Flexible Mesh (D-FLOW FM) which is the main module of D-HYDRO. This module allows the use of a coupled 1D2D or 1D3D grid for hydrodynamic simulations. Furthermore, the 2D grid cells can be flexible in size and shape (triangular or square), based on the characteristics of the area. Important areas, around structures for example, can be modelled in high resolution, while large areas with similar characteristics can be modelled in a low resolution. The D-FLOW FM module solves the 2D Shallow Water equations for each grid cell at each time step. These equations are given by *Eq. 1 - Eq. 3* (Deltares, 2023):

Depth-average continuity eq.

$$\frac{\partial z}{\partial t} + \frac{\partial (h+z)u}{\partial x} + \frac{\partial (h+z)v}{\partial y} = 0 \qquad \text{Eq. 1}$$

Momentum eq. x direction

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} = -g\frac{\partial z}{\partial x} - \frac{gu\sqrt{u^2+v^2}}{C_z^2(h+z)} \qquad \text{Eq. 2}$$

Momentum eq. y direction

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} = -g\frac{\partial z}{\partial y} - \frac{gv\sqrt{u^2+v^2}}{C_z^2(h+z)} \qquad \text{Eq. 3}$$

Where u and v represent the depth-averaged flow components in x- and y-directions respectively (m/s), h is the water depth (m), z is the water surface elevation (m), g represents the constant gravitational acceleration (m/s^2), and C_z is the Chézy friction coefficient (m$^{0.5}$/s).

D-HYDRO uses a combination of an explicit and implicit calculation scheme. An explicit scheme calculates the solution of the time step solely with the solution of the previous time step. When using larger time steps, numerical errors can grow over time with these explicit schemes (Akbari & Firoozi, 2010; Deltares, 2024). The duration of the time step in D-HYDRO is therefore dynamic and limited by the grid size. The duration of the time step is calculated with *Eq. 4* and becomes smaller if the flow velocities are higher or if the grid size is smaller. Furthermore, the time step depends on the maximum Courant number C_{max}. The default value for this parameter in D-FLOW FM is 0.7, which means that a particle of water cannot flow further than 0.7 cells during 1 time step. A maximum Courant number larger than 1 means that a particle of water can flow further than 1 cell. Values larger than 1 should be used with care because it can result in instability of the explicit scheme (Deltares, 2023). For each time step, the Δt parameter is calculated in all cells. The smallest value for Δt is then used as the duration of the time step. This means that areas with higher flow velocities are often the limiting factor for the length of the time step (Sanders, 2008).

$$\Delta t = \frac{C_{max}}{\left(\frac{u_x}{\Delta x} + \frac{u_y}{\Delta y}\right)} \qquad \text{Eq. 4}$$

Where u_x represents the flow velocity in the x-direction (m/s), u_y represents the flow velocity in the y-direction (m/s), C_{max} the maximum allowed Courant number, Δt the duration of the time step (s), Δx the length of the grid cell in the x-direction (m), and Δy the length of the grid cell in the y-direction (m).

2.3 Overview methods for advancing hydrodynamic models

Long computation times are a commonly encountered problem with 1D2D hydrodynamic models, especially when using high resolutions (see e.g. Fraehr et al., 2024; Jamali et al., 2021; Razavi et al., 2012; Zhao et al., 2021). Several methods have been proposed to reduce the computation time of detailed 1D2D hydrodynamic models. This section will elaborate on these attempts in literature.

Zhao et al. (2021) and Neal et al. (2010) suggest using parallelization of the high-fidelity model. With this method, the computational burden is spread over multiple calculation cores that work at the same time (often in multiple computers). However, the model considered in this study will be run on one single computer such that parallelization is not a viable option at this moment. It is therefore not considered in this research.

Besides parallelization, surrogate models can be used. Surrogate models approximate the original high-fidelity model to decrease the computational load (Razavi et al., 2012). Two main classes of surrogate models exist: Response Surface Surrogates (RSS) and Low-Fidelity Physically based Surrogates (LFPS).

Response Surface Surrogates (RSS) are models that use statistical and empirical regression and machine learning techniques to approximate the relations of the relevant parameters in the model. This type of surrogate model no longer uses the physical equations that underlie the original model (Razavi et al., 2012). Burrichter et al. (2023) show that the results of an RSS that estimates pluvial flooding in small-scale areas can show a high agreement with the original 2D model while significantly reducing computation times. On the other hand, the RSS developed by Jamali et al. (2021) did not perform well in a real case study that covered a larger area. Compared to LFPS models, RSS models can become complex with an increasing number of parameters and are also less suited for extrapolation of cases that were not included in the training data (Razavi et al., 2012). This research focuses on detailed hydrodynamic models that have a large range of parameters. As a result, RSS models are less suited for this purpose. Furthermore, the training data for the RSS model must be generated with the original model. For a large model considered in this research, this would take too much time and resources. It is therefore chosen to neglect RSS models in this research.

LFPS models keep the physical bases of the original model but are simplified to reduce the computation time. A large benefit is that LFPS models can extrapolate for data that is not used in the model construction (Razavi et al., 2012). Several methods to create a LFPS model exist.

The first method of creating an LFPS model simplifies the hydrodynamic equations by omitting the momentum conservation (Zhao et al., 2021). The resulting simplified equations can be solved significantly quicker (Hunter et al., 2007). This method is for example used by Yu & Lane (2006a) to be able to calculate a fine spatial grid that would take too long if the full hydrodynamic equations were used. Another example is the study of Bomers et al. (2019a) which uses simplified equations to be able to reconstruct a historic flood event. Nevertheless, Hunter et al. (2007) also show that these types of models are often only suitable for a specific purpose. This research aims to assess methods for advancing a high-fidelity model in general. Furthermore, according to Janssen (2023), it is difficult to implement this type of simplification into D-HYDRO as the full momentum equations are embedded in the software. This type of LFPS model is therefore not further explored in this research.

Secondly, LFPS models can be created by numerical simplifications while keeping the full hydrodynamic equations (Razavi et al., 2012). Using a longer time step or coarser grid size are examples of this type of LFPS model. These simplifications are easier to implement in D-HYDRO (Janssen, 2023) and are therefore considered in this research as viable options. The potential measures for accelerating high-fidelity models that fall under this category will be discussed in the next section.

2.4 Numerical simplifications in low-fidelity physically based surrogate models

This section will specifically elaborate on the construction of LFPS models that apply numerical simplifications. The three methods that are most mentioned in literature are coarsening the 2D grid, reducing the number of 1D calculation points and increasing the maximum Courant number. These solutions are often easy to implement while generating large computational benefits (Razavi et al., 2012).

2.4.1 2D grid resolution

Probably the most studied measure in this type of low-fidelity model is the reduction of the mesh resolution. It means that the size of the grid cells is enlarged, decreasing the number of grid cells and reducing the computational load. The studies of Bomers et al. (2019b), Judi et al. (2014), Yu & Lane (2006a) and Janssen (2023) show that a significant reduction in computational time of up to 90% is possible to yield with this method. However, a larger grid size can significantly impact the model results.

The studies of Hardy et al. (1999), Yu & Lane (2006a), Horritt & Bates (2001) and Mooijaart (2023) indicate that the flood extent is overestimated for a coarse 2D grid compared to a fine 2D grid. On the other hand, Judi et al. (2014) also experience underestimations of the flood extent for coarser meshes. Yu & Lane (2006b), McMillan & Brasington (2007) and Zhao et al. (2021) show that a smaller sub-grid, a locally refined mesh for important areas, can significantly improve the model accuracy while the computation time remains low.

Janssen (2023), Bomers et al. (2019b), Judi et al. (2014), Horritt et al. (2006) and Caviedes-Voullième et al. (2012) all indicate that besides flood extent, the modelled peak discharge in the rivers is often overestimated with a coarser grid. The timing of the peak discharge is also changing. Judi et al. (2014) found that the peak comes earlier, while Caviedes-Voullième et al. (2012) saw a delayed peak water flow.

In conclusion, coarser 2D grids are often effective in reducing the run time of a hydrodynamic model. However, all studies show that accuracy is almost always compromised although the extent and direction of the effect seem to depend on the model and characteristics of the modelled area.

2.4.2 Maximum allowed Courant number

As discussed in section 2.2, the maximum Courant number determines the distance a particle of water can travel during the time step. In general, the maximum Courant number is chosen to be smaller than 1 which means that a particle can flow only one grid cell during each time step. The model is more stable in this case (Deltares, 2023). However, the research of Janssen (2023) shows that increasing the maximum Courant number to 10 can reduce the computation times of a 1D2D D-HYDRO model by 80%, while the Mean Absolute Error in the water depth remains almost zero. The study of Mooijaart (2023) indicates that increasing the Courant number from 0.7 to 2.0 can reduce the computation time of a 2D D-HYDRO model by 47%, but no quantification is given of the impact on the accuracy. Hop (2021) increased the maximum Courant number of a D-HYDRO model from 0.7 to 50 which resulted in a four times smaller computation time. The accuracy remained high in this study, probably because one small area of the model limited the Courant number, while the majority of the model still had a low Courant number. Increasing the time step is equivalent to increasing the maximum allowed Courant number. Bomers et al. (2019a) use this former method and found that the surrogate model is significantly faster while having an acceptable accuracy. However, the low-fidelity model that is constructed by Bomers et al. (2019a) also simplifies the Shallow Water equations. Therefore, the found decrease in computation time might not be caused solely by an increase in time step. Nevertheless, increasing the maximum Courant number is considered a viable option for advancing detailed hydrodynamic models based on the results of Janssen (2023), Mooijaart (2023) and Hop (2021).

2.4.3 1D calculation points

Similar to a lower 2D grid resolution, the number of 1D calculation points in the representation of the channels can be reduced. Janssen (2023) could yield a reduction in computation time of 22% by increasing the distance between the 1D calculation points from 20 to 50 meters. The study by Davidsen et al. (2017) showed that reducing the number of 1D calculation points by 66% can result in a 35% decrease in computation time. However, similar to 2D grids, simplification of the 1D elements can lead to less accurate results (Davidsen et al., 2017).

The number of 1D calculation points can be reduced in D-HYDRO by either one or a combination of the following:

1. Increase the distance between the 1D calculation points (Davidsen et al., 2017)
2. Remove the bridges from the model because two additional calculation points are added for each bridge.
3. Merge culverts that are consecutively located in a channel to one culvert because two additional calculation points are added for each culvert.

For all of the three methods, the impact on the accuracy is not known and should be studied when applying these methods.

2.5 Numerical instability

Besides numerical simplifications, resolving numerical instabilities can potentially reduce the computation time of a hydrodynamic model. A model can become unstable if numerical errors grow to the extent at which the solution begins to oscillate or diverge. A result can be that the flow velocities are increased to unrealistic values. This reduces the allowed time step via the Courant condition (*Eq. 4*) which enlarges the computation time (De Almeida et al., 2012). The following factors affect numerical instability: computation time step, cross-section spacing, calculation tolerances, lateral structures, steep streams, downstream boundary conditions, cross-section geometry, bridges, culverts, (wrong) initial conditions, drops in bed profile, Manning's n and missing parameter values (Brunner, 2023). Because of the large number of potential causes of numerical instability, this topic is not treated in a literature study.

This chapter explored the current knowledge on the topic of hydrodynamic modelling and LFPS models. Based on this overview, a research gap and objective will be formulated in the next chapter.

3 Research dimensions

This section will present the research objective, questions and the terminology that is used.

3.1 Research gap and objective

The previous chapter introduced the concepts of surrogate modelling. These models are required because conventional 1D2D hydrodynamic models have a long computation time which makes them unsuitable for real-time flood predictions and analysis of a large set of runs for a sensitivity or uncertainty analysis (Zhao et al., 2021). Although the papers discussed in the literature study were all related to this topic of surrogate modelling, no study has been executed on the effect of numerical simplifications in a detailed 1D2D hydrodynamic model for pluvial flooding. The results of the other studies might therefore not apply to this case study. The study of Bomers et al. (2019b) and Mooijaart (2023) both implement numerical simplifications in a detailed hydrodynamic model but focus on fluvial flooding. This research and case study will only assess pluvial flooding. Janssen (2023) also applies the principles of surrogate modelling to a D-HYDRO model but to a single polder. This research and case study, in contrast, involves a significantly larger area that is more diverse. The effect of numerical simplifications in a detailed 1D2D hydrodynamic model for pluvial flooding on the run-time and accuracy has thus not been studied yet. Furthermore, other studies do not fully agree on the effects of 2D grid resolution on the accuracy of similar 1D2D hydrodynamic models. Lastly, the presence and effects of numerical instabilities in detailed hydrodynamic models are largely unknown. The research objective is therefore formulated as follows:

Research objective

The research objective is to optimise the calculation time of a detailed 1D2D hydrodynamic model by
(1) resolving potential numerical instabilities in the model,
(2) implementing numerical simplifications
and quantifying the effect of these measures on the accuracy of the model output.

A case study on the management area of HDSR provided by HydroLogic will be used to test the proposed principles and will be introduced in the next chapter.

3.2 Research questions

To fulfil the research objective, the next research question is drawn up:

Research question

Which methods are effective in reducing the computation time of a detailed 1D2D hydrodynamic model while keeping sufficient accuracy of the output?

Where "sufficient accuracy" will be defined in the methodology later (section 5.6). To answer this main research question, the next sub-questions are defined:

Sub-questions (SQ)

1. How do numerical instabilities affect the computation time of a detailed 1D2D hydrodynamic model?
2. How does a larger maximum Courant number affect the computation time and model accuracy of a detailed 1D2D hydrodynamic model?
3. How does a coarser 2D grid affect the computation time and model accuracy of a detailed 1D2D hydrodynamic model?
4. How does a reduced number of 1D calculation points affect the computation time and model accuracy of a detailed 1D2D hydrodynamic?

3.3 Terminology

Within this research, certain concepts and abbreviations will be used frequently. In *Table 1*, the definitions as used in this report, are stated. Furthermore, some words that are frequently used in Dutch water management are difficult to translate into English. An overview of the translations used in this report is given in *Table 2*.

Table 1 – Terminology and abbreviations used in the research.

Term	Definition
HDSR	Hoogheemraadschap De Stichtse Rijnlanden (the water board issuing the model construction of the case study).
LFPS	Low-fidelity physically based surrogate. In this research: numerical simplifications of the model and not the simplification of the governing Shallow Water equations.
GUI	Graphical User Interface.
Numerical simplification	Measure that removes detail from the model without simplifying the Shallow Water equations to reduce the computation time.
Numerical instability	A state of the model at a certain location where the grid cells are Courant limiting or have exceptionally high flow velocities.
1D calculation point	A cross-section of the channel which is used in the calculation of the flow properties.
Benchmark model or high-fidelity model	The initial model that has long computation times.
Optimised benchmark model	Model from which most numerical instabilities are removed.
NumLimdt parameter	"number of times a flow element was Courant limiting" parameter. This parameter stores how often a calculation point (1D or 2D) has been the limiting factor according to the Courant condition (*Eq. 4*).

Table 2 - Translations of typical Dutch words related to water management that are used in this thesis.

Dutch	English	Definition
Duiker	Culvert	A tube carrying a water stream underground.
Stuw	Weir	A structure across a stream with adjustable crest height to regulate the water level in an area.
Gemaal	Pumping station	Pump unit with the function of supplying water to higher areas or draining water from lower areas.
Stuurpeil	Control water level	The water level that is aimed for at a certain location.
Peilgebied	Control water level area	An area for which the same control water level is aimed.
Peilscheidend kunstwerk	Water level separating structure	A structure (weir, culvert, sluice) that is on the boundary of two control water level areas and ensures that the control water levels are maintained.

4 Case study

This chapter will introduce the case study. This includes a description of the involved parties, the study area and the underlying assumptions and working of the D-HYDRO model. The case study is used to see the impact of numerical instabilities on the computation time and to test the numerical model simplifications that were defined in the theoretical framework.

4.1 Involved parties

HydroLogic is the commissioning party for this project and case study. HydroLogic is a consultancy and research agency that specialises in future-proof water management. The company focuses on operational and regional water management, water safety, water nuisance and salinization. HydroLogic has years of experience with hydrological and hydraulic models and often builds the models themselves in existing software packages such as D-HYDRO.

HydroLogic is tasked with the development of a simulation model for the study area of the Dutch water board Hoogheemraadschap De Stichtse Rijnlanden (HDSR) in D-HYDRO. After completion, the purpose of the model is to perform a “watersysteem analyse” (water test). This is a legally required test that the water board must carry out every six years to find potentially dangerous bottlenecks in their water system (Informatiepunt Leefomgeving, n.d.). For this test, more than 10,000 different rainfall events will be evaluated with the model to see whether this will result in flooding. To be able to evaluate all these events, the run-time of the model must be limited. HydroLogic can potentially use the results of this research to optimise the run-time of the model without losing accuracy. The results might also apply to other models from HydroLogic.

The water board Hoogheemraadschap De Stichtse Rijnlanden (HDSR) is not directly involved in the research but is charged with the responsibility of performing a water test. HDSR benefits from a faster model that keeps its accuracy. The output of the water test can point to areas that are prone to pluvial flooding. HDSR can use this information to make policy decisions and take measures to solve the problem since the water boards are legally required to do so.

Lastly, unrelated organizations and practitioners that work with detailed 1D2D hydrodynamic models could potentially use the results of this research to accelerate their hydrodynamic models.

4.2 Study Area

The study area concurs with the management area of HDSR, which is in the centre of the Netherlands (*Figure 2*). The total surface area is approximately 860 km^2. The city of Utrecht is located in the heart of the study area.

The eastern boundary of the project area is formed by the Utrechtse Heuvelrug. This is a forested area with a slightly higher elevation. The soil consists mainly of sand which causes the water to infiltrate quickly. As a result, almost no waterways exist in this area.

The areas east and south of Utrecht consist of polders. These are located under sea level. The water in the polders is discharged by drainage ditches (see *Figure 2*). The water from these ditches is pumped into the “boezem” canals that have a higher elevation and transport the water to the main rivers. The main rivers in the study area are the Lek, the Amsterdam-Rijnkanaal, the canalized Hollandse IJssel and the Oude Rijn. These main waterways transport the water from the study area to the sea.

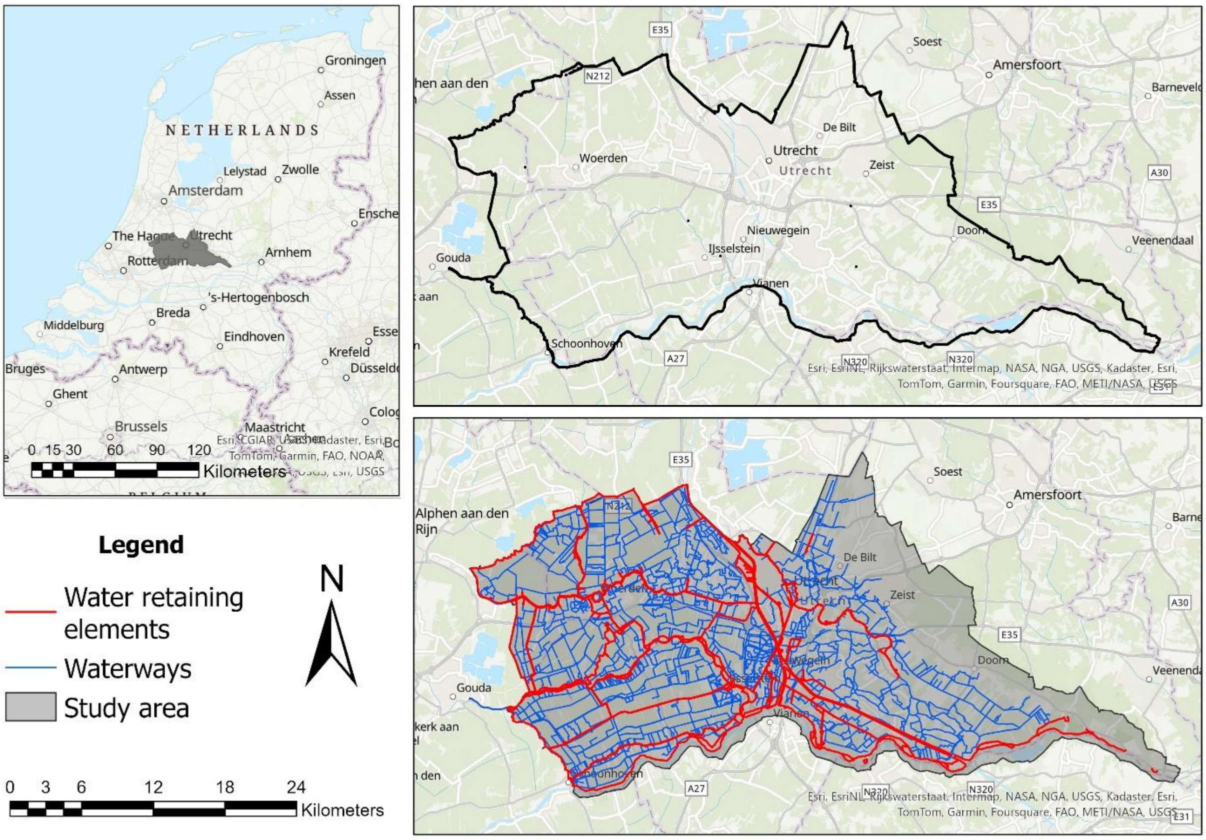

Figure 2 - The study area.

4.3 HDSR model

HydroLogic has constructed a detailed model of the management area of HDSR in D-HYDRO. The model consists of four different components (see *Figure 3*). First, the separate model ModFlow/MetaSwap (MFMS) uses the precipitation data, soil characteristics and groundwater flows to simulate the infiltration, runoff, and groundwater fluxes in rural areas. The relevant output is the runoff to the waterways which is used as input for the hydrodynamic model that simulates the water flow in the 1D channels and 2D grid. This second component is built in the D-FLOW FM module of D-HYDRO and will be the focus of this research. Besides the D-FLOW FM module, the D-HYDRO model consists of two other modules as well: D-Rainfall Runoff (RR), which simulates the runoff in urban areas, and D-Real Time Control (RTC) which simulates the control of the structures, such as the pumping stations. The model structure can be seen in *Figure 3*. The numerical simplifications that were discussed in the literature study apply to the D-FLOW FM module and this study will look only into this module. From here, this will be referred to as "the model". The D-HYDRO model is constructed with the help of D-HyDAMO. This is a Python package which allows the automatic creation of a D-HYDRO model by running a Python script (Deltares, 2022). Models with numerical simplifications can be constructed by running the script after adjusting some of the input parameters.

The runoff from the MFMS and RR components are added in the D-FLOW FM model as laterals, which are nodes in the 1D network where input water is added over time. RR models the storage of water on the streets and the sewer in the urban area. The water that falls in the urban area is either stored on the surface, accumulated in the sewer, transported to the wastewater treatment plant, or discharged onto the surface water, depending on the characteristics of the sewer. The discharge onto the surface water is transferred to the D-FLOW FM model with the laterals.

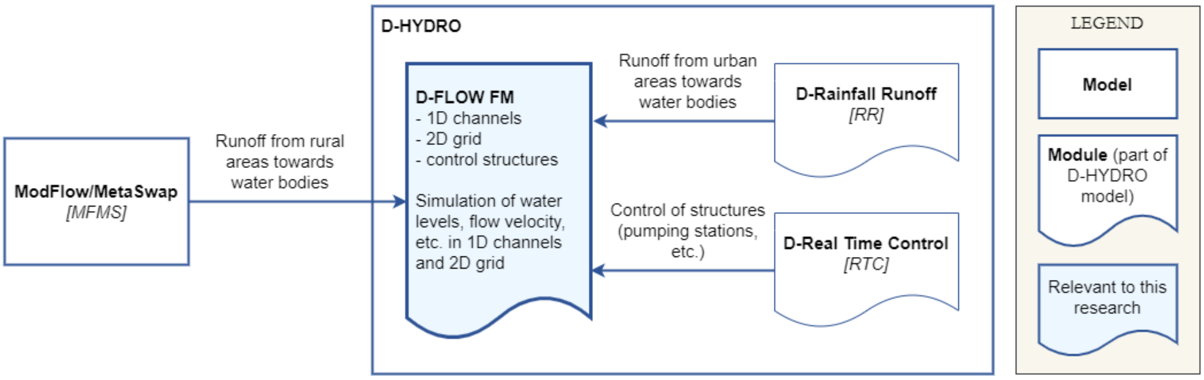

Figure 3 – Structure of the complete HDSR model.

The D-FLOW FM module contains the waterways which are constructed as 1D elements that connect with a 2D grid which covers the surrounding land. The model includes almost all drainage ditches and boezem canals that are present in the study area. The Lek and Amsterdam-Rijnkanaal that are managed by Rijkswaterstaat are not modelled but are only included as boundary conditions. The model will only evaluate pluvial flooding (due to rain) and not fluvial flooding of these large rivers. Water retaining elements such as dikes and elevated roads are also implemented in the model to be able to accurately model potential flooding (see *Figure 2*). Culverts, bridges and other structures are incorporated into the model as well. The behaviour of the structures such as the pumping stations is controlled by the RTC module.

The original version of the model uses a square mesh with grid cells of 100x100 meters. The maximum Courant number in the original model is set to the default of 0.7 to prevent any instabilities caused by a large time step.

The model has not been calibrated yet by HydroLogic at the time of writing. It is planned to be executed in July 2024. As a consequence, the results of this study can only be used to relatively compare the surrogate models with the original model. Nevertheless, the effect of different numerical simplifications on the computation time and their relative effect on the accuracy can be determined with this uncalibrated benchmark model.

5 Research methodology

This section will elaborate on the methods that are applied to answer the research questions. The methods will build on the theoretical framework and the research dimensions as discussed in the previous chapters. First, the general methodology will be introduced, after which the specific components will be discussed in more detail.

5.1 Overview of research methodology

The general methodology can be seen in *Figure 4*. The steps mentioned in the figure correspond with the numbering of the steps discussed in the upcoming sections. The first and second steps are related to sub-question 1 (SQ1). Steps 3 and 4 are relevant for answering SQ2, SQ3 and SQ4 which focus on numerical simplifications. Lastly, step 5 will answer the main research question.

The first step will address the numerical instabilities in the model (see section 5.3 for more details). Then, the optimised model will be analysed on the computation time in step 2 (see section 5.5). After this step, SQ1 can be answered. The output of these first two steps is an optimised benchmark model that can be used to assess the numerical model simplifications.

Step 3 implements the different numerical model simplifications into the optimised benchmark model which yields different surrogate models (see section 5.4). These are tested on their computation time and accuracy for two precipitation events during step 4 (see section 5.5). After finishing step 4, SQ2, SQ3 and SQ4 can be answered. Lastly, all information on the sub-questions is used in step 5 to make a recommendation about effective model acceleration methods.

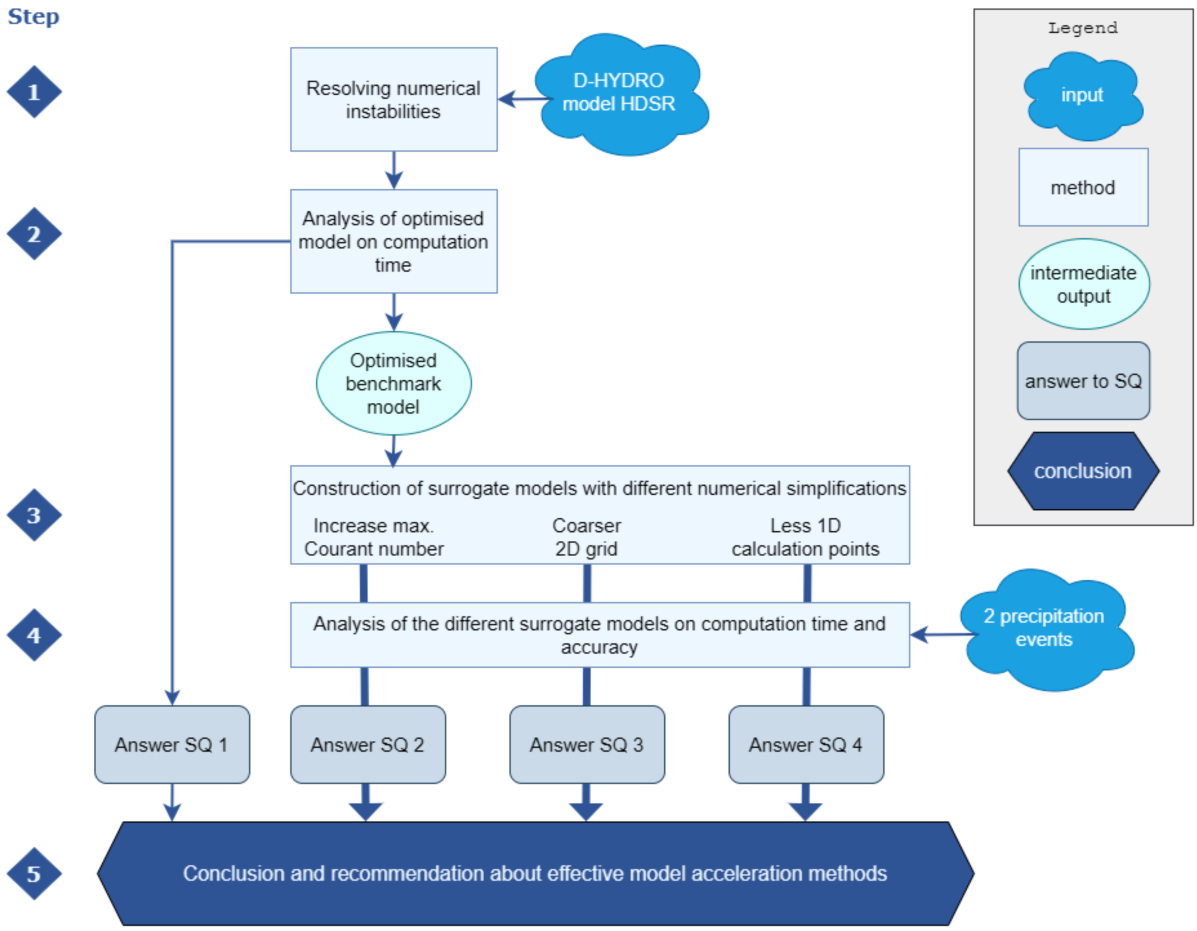

Figure 4 - Overview of the research methodology.

5.2 Input data and boundary conditions

The D-HYDRO model of the HDSR study area (as described in section 4.3) is provided. All required input data to run the model, such as bed roughness and level, are already included in this model. The water levels on the boundary nodes are constant.

The analysis of the numerical instabilities is performed by running the model for January 1, 2014, which is a day without precipitation. The model is initialized on the equilibrium conditions. In other words, all waterways start at their control water level. By using a day without precipitation, the model is expected to show stable water levels. With this approach, any changes in the water level over time can indicate numerical instability or at least point to locations with unexpected behaviour (see section 5.3).

The historical precipitation event of 27 and 28 July 2014 is chosen for simulating the surrogate models. During this event, locally up to 131.6 mm of rain fell in the study area in one day (KNMI, 2014). *Figure 5* shows the average intensity and cumulative rainfall over the study area for July 27 18:00 until July 28 18:00. A large peak in the intensity of three hours is present for this event. Parts of the study area experienced flooding which is an important boundary condition to be able to assess the numerical simplifications of the 2D grid (Lenderink & Van Oldenborgh, 2014; Kennisportaal Klimaatadaptatie, n.d.).

Considering the computation time of the original model, only one precipitation event is selected for the evaluation of all surrogate models. However, to see whether the same results can be obtained for a different rainfall event, four surrogate models are run for a second event as well. This event runs from September 4 at noon until September 7 at midnight, in 2018 (*Figure 6*).

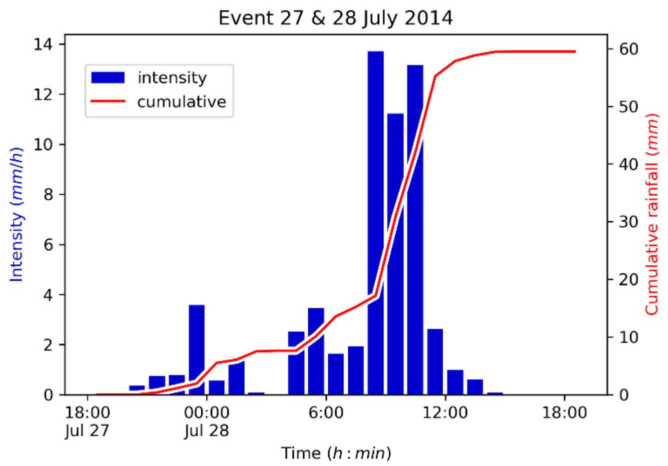

Figure 5 – Primary event for evaluating all the surrogate models: July 27 & 28, 2014.

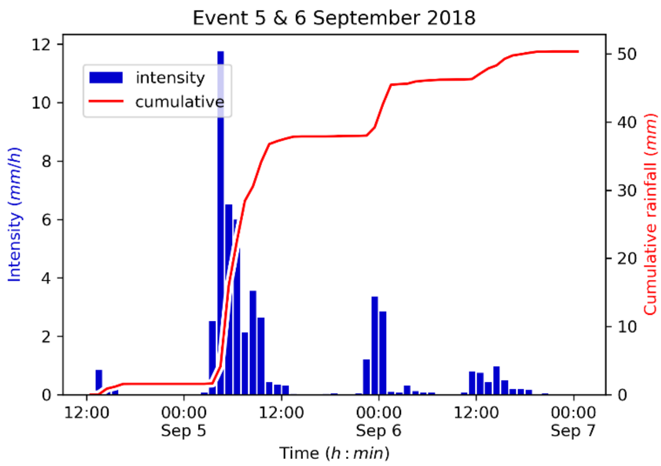

Figure 6 – Secondary event for evaluating the surrogate models to check if similar results are found: September 5 & 6, 2018.

The simulations are started seven days before the event to allow the model to find an equilibrium state and minimize the errors due to incorrect initialization. Seven days were chosen because the water levels were seen to be fairly stable after this period.

5.3 Step 1: resolving numerical instabilities

This section will focus on the first step in the general research methodology (see *Figure 4*), which addresses the numerical instabilities. As shown in section 2.5, a large range of possible causes for numerical instabilities exist. As a result, setting up a fixed methodology is difficult. However, the next steps are executed:

1. Run the model for one day (January 1, 2014).
2. To pinpoint locations with potential numerical instabilities, the following four indications are used:
    a. A "number of times a flow element was Courant limiting" (NumLimdt) parameter larger than 1000. In the original model, most calculation points are 0 to 1000 times Courant limiting for simulating 1 hour of rainfall. Locations where the NumLimdt parameter is larger than 1000 potentially experience numerical instabilities and are examined in more detail.
    b. A flow velocity larger than 2 m/s. A velocity that is larger than 2 m/s is considered high for the type of waterways in the area and might result in numerical instability.
    c. A water depth of 0 meters. A water depth of 0 meters can result in numerical instability and at least indicates an unexpected situation in the model as it is expected that all waterways contain water.
    d. A difference in water depth of more than 0.1 m between the end and start of the simulation. The model is initialized with the control water levels. The water level should therefore be stable since there is no rainfall. Large differences in water depth during the run indicate that water is flowing towards or leaking from the location, which can indicate a flaw or numerical instability.
3. Identify a possible cause for the numerical instability at this location by
    a. Checking whether the initial conditions are logical for the situation.
    b. Checking which control water level is aimed for at the location and how it relates to the surrounding control water levels. Sometimes, the control water level is defined for half of the 1D flow element, resulting in a large jump within the flow element. During the run, the model tries to maintain the jump in the water level for the flow element which results in unrealistic flow velocities.
    c. Checking the operation of the control structures (e.g. pumping stations, weirs, sluices) that are close to the location of numerical instability. Sometimes pumps are pumping in the wrong direction, or the pumps are switched on and off at the incorrect water level. Weirs and sluices can also show incorrect behaviour.
    d. Checking the characteristics of culverts and bridges that are close to the location.
    e. Checking the input parameters at the location (bed level, roughness)

    Any anomalous patterns, behaviour or values can point to the cause of the numerical instability.
4. Implement a solution to the cause. Depending on the cause, this can be adjusting the 1D flow net, the channel characteristics, the input parameters (bed level, roughness, time series of moveable weirs), the initial (boundary) conditions or the control water level. Furthermore, the operation of the control structures can be changed.
5. Run the model again and observe the NumLimdt parameter and flow velocity parameters. If the parameters are not improved, i.e. the parameter values are not decreased, implement a different solution (go back to [4]).
6. If more areas with numerical instability exist, repeat from [1]
7. Evaluate the new model on computation time (as described in section 5.5)

With this stepwise approach, most numerical instabilities are detected and solved, resulting in an optimised benchmark model. Numerical simplifications are implemented in this new model to construct surrogate models. Subsequently, these surrogates have been tested on their computation time and accuracy against this optimised benchmark model.

5.4 Step 3: construction surrogates with numerical simplifications

Section 2.4 discussed possible numerical simplifications to accelerate a 1D2D hydrodynamic model. An overview of the simplifications is given in *Table 3*, which also includes the range of variations for each simplification that will be tested. Horritt & Bates (2001) show that their 2D surrogate model has acceptable results with a grid size of 500 meters or smaller for a relatively flat catchment. Although the model of the case study does not encompass a whole catchment, it is also largely a flat area. It is therefore chosen to set the upper boundary for the resolution of the 2D grid and the distance between the 1D nodes to 500 m. The largest Courant number is selected to be 50 because Hop (2021) could still yield acceptable results in terms of accuracy with this maximum Courant number for a similar D-HYDRO model. Surrogate models will be constructed that implement one or a combination of the simplifications in the optimised benchmark model.

Each bridge and culvert add two additional 1D calculation points to the model. For the case study, around 1900 bridges are present in the model. Removing all these bridges reduces the number of 1D calculation points by 3800. By merging subsequent culverts that are on the same branch, the number of 1D calculation points can be reduced. The D-HyDAMO script is extended to be able to merge culverts because this was not yet possible in the script. The next assumptions are made in this process:

1. Culverts that are in series on the same branch are merged into one culvert with the length of all original culverts combined.
2. Culverts that function as water-level separating structures are not merged.
3. The new culvert is located in the middle of the branch.
4. The largest roughness of the original culverts is used for the new culvert.
5. The smallest dimensions of the original culverts are used for the new culvert.
6. The highest inflow height and lowest outflow height of the original culverts are used for the new culvert.

Three additional simplifications are tested that were not discussed in the theory section. First, the 2D grid cells that are directly located below the 1D channels can be removed. This clipping of the 2D cells can reduce the computation time because less water flows from the 1D network to the 2D grid during the initialization. Secondly, the user time step can be adjusted. This time step indicates how often the RR and MFMS components of the model are forced to the laterals. Lastly, a coarser 2D grid resolution can be used in combination with a local refinement of the grid around the waterways. The underlying idea is to have more detail and accuracy of the 2D grid in areas that are more prone to flooding.

Table 3 - Overview of simplifications to create surrogate models and the range of the variations.

Numerical simplification	Range	Values
Resolution squared 2D grid [m]	100-500	100, 250, 500
Distance between 1D calculation points [m]	50-500	50, 100, 250, 500
Removing bridges? (reducing 1D calculation points)	Yes/No	Yes, No
Merging culverts? (reducing 1D calculation points)	Yes/No	Yes, No
Maximum Courant number [-]	0.7 - 50	0.7, 1, 5, 10, 50
2D grid clipping	Yes/No	Yes, No
User time step [s]	3600-7200	3600, 7200
2D grid refinement resolution [m]	0-100	0, 100

In total 18 surrogates were constructed by implementing the numerical simplifications as discussed above. An overview of all surrogates is given in Appendix A.1 (*Table 11*).

5.5 Steps 2 & 4: evaluation of the surrogate model performance

The computation time of the optimised benchmark model in relation to the original model is evaluated in step 2. Step 4 assessed the accuracy and computation time of the surrogate models with numerical simplifications with respect to the optimised benchmark model.

The computation time is recorded for each simulation. The same laptop is used for each run. The specifications are given in *Table 4*.

Table 4 - Specifications of the computer used to run the model.

CPU	Intel(R) Core(TM) i7-8850H CPU @ 2.60GHz 2.59 GHz
RAM	16 GB

To assess the accuracy of the surrogate models in relation to the optimised benchmark model, several metrics are chosen for comparing the maximum and average water level in the 1D calculation points and the maximum and average water depth in the 2D calculation points. It is chosen to use water depth for the 2D grid because cells that are not flooded get a value of 0 which makes them easier to filter out later. For the 2D water level, D-HYDRO exports the bed elevation if the cell is not flooded, which makes it difficult to determine if water is present. Lee & Choi (2022) and Waseem et al. (2017) state that a good accuracy analysis should at least have one absolute error metric and one dimensionless metric.

Absolute error metrics are for example the Mean Absolute Error (MAE), Root Mean Square Error (RMSE) and the Sum of Squared Errors (SSE). The MAE (*Eq. 5*) is chosen because this metric is independent of grid size and only weakly related to dimensionless metrics, which is preferable to get a less biased output (Wöhling et al., 2013). The MAE is best if 0. It can be calculated for the average water level over time in a calculation point or the maximum water level in the calculation point.

$$MAE = \frac{1}{n}\sum_{i=1}^{n} |y_{s,i} - y_{b,i}| \qquad \textit{Eq. 5}$$

Where n is the total number of grid cells/nodes, $y_{b,i}$ is the output value in grid cell/node i according to the benchmark model, and $y_{s,i}$ is the output value in grid cell/node i according to the surrogate model.

Dimensionless error metrics compare the variability of the water level in the benchmark and surrogate over time. Examples are the Nash-Sutcliffe Efficiency (NSE), Index of Agreement (d) and Kling-Gupta Efficiency (KGE). The KGE is used in this study since it is less biased towards peak values than the NSE and d metrics (Waseem et al., 2017; Krause et al., 2005). Furthermore, the KGE assesses the accuracy based on correlation, bias and variability which is more comprehensive than the NSE and d which only evaluate variability and bias respectively (Gupta et al., 2009; Kling et al., 2012; Towner et al., 2019). The KGE can be calculated with *Eq. 6*. The score is best if 1 and can vary from minus infinity to 1.

$$KGE = 1 - \sqrt{(r-1)^2 + (\alpha - 1)^2 + (\beta - 1)^2} \qquad \textit{Eq. 6}$$

Where r is the Pearson's correlation coefficient, α is the variability ratio, and β is the bias ratio. See Appendix A.2 for more details on these.

The KGE and MAE metrics for the 2D grid will be calculated excluding the true negative calculation points. These true negatives are the calculation points that are not flooded in both the benchmark and the surrogate model. A relatively large part of the calculation points is true negative, and all these cells have no error in the water depth which will let the metrics appear to be extremely good when included in the calculation (Yu and Lane, 2006a). To determine if a cell is flooded, a threshold for the water depth must be set. Below this threshold, the grid cell is considered dry. For this study, a threshold of 0.05 m will be used, similar to Yu & Lane (2006a). A cell is true negative if the water depth in the cell is below 0.05 m in both the benchmark and surrogate model.

Besides the error in the water level, the accuracy of the flood pattern, which cells are flooded and which are not, is determined. Available metrics are the Accuracy (A), Critical Success Index (CSI) or F1-score, with the corresponding *precision* and *recall* statistics. The F1-score (*Eq. 7 - Eq. 9*) is chosen because the precision and recall statistics say something about the over- and underprediction of the flood extent respectively which is relevant to policymakers (Bermúdez et al., 2018). The F1-score is best if 1.

F1-Score	*Precision*	$$p = \frac{\#TP}{\#TP + \#FP}$$	*Eq. 7*
	Recall	$$r = \frac{\#TP}{\#TP + \#FN}$$	*Eq. 8*
	F1-score	$$F1 = 2\frac{p \cdot r}{p + r} = \frac{2 \cdot \#TP}{2 \cdot \#TP + \#FN + \#FP}$$	*Eq. 9*

Where #TP is the number of True Positives: 2D cells that are flooded in both surrogate and benchmark, #TN is the number of True Negatives: 2D cells that are not flooded in both models, #FP is the number of False Positives: 2D cells that are predicted as flooded by the surrogate but are not flooded in the benchmark model and #FN is the number of False Negatives: 2D cells that are predicted as dry by the surrogate but are flooded in the benchmark model (underprediction flood extent).

Changing the distance between the 1D and/or resolution of the 2D grid leads to a different number and location of calculation points. To compare the calculation points of the surrogate model with the calculation points in the benchmark model, the nearest calculation point in the same control water level area is taken. Other additional calculation points that are present in the benchmark model are ignored.

5.6 Step 5: recommendation on effective model acceleration methods

In this last step, the main research question will be answered, and a recommendation will be given about which methods are effective for reducing the computation time without significantly decreasing the accuracy. To make this recommendation, a threshold value for the accuracy must be set to exclude methods that compromise the accuracy significantly.

According to Gupta et al. (2009) and Kling et al. (2012), the KGE is sufficient if larger than 0.5 and good if larger than 0.75. A threshold of 0.75 will be used here. The acceptable MAE depends on the quality of the benchmark model in relation to the observations used in the validation of this benchmark. However, the benchmark model is not validated yet by HydroLogic and the error in the benchmark model is therefore not known. An initial estimation of the MAE in the benchmark model with respect to the observations is maximal 5 centimetres at the time of writing. The acceptable MAE in the surrogates with respect to the optimised benchmark model is chosen to equal this estimate. Lastly, an F1-score larger than 0.7 is generally seen as a sufficient performance (Logunova, 2023) and will be used in this study as a threshold. A surrogate must score sufficiently on all metrics to be recommended as a suitable method for accelerating a detailed hydrodynamic 1D2D model. Based on the performance of the different surrogates on the given metrics, the main research question is answered. *Table 5* summarizes the characteristics of all metrics.

Table 5 - Overview metrics for assessment model accuracy.

Metric	Acceptable range	Parameters to apply
Mean Absolute error (MAE)	0 – 0.05 m	Average and maximum water level 1D Average and maximum water depth 2D (excluding true negatives)
Kling Gupta Efficiency (KGE)	0.75 – 1	Water level over time in 1D calculation points Water depth over time in 2D calculation points (excluding true negatives)
F1-score (precision and recall included)	0.7 – 1	Flood extent (flood presence)

6 Results

This section will discuss the results of this thesis. First, the results of the investigation of numerical instabilities are discussed, which is related to the first sub-question. Secondly, the effect of numerical simplifications on the computation time and accuracy of the model is elaborated on. This relates to sub-questions 2 to 4.

6.1 Numerical instabilities

The first sub-question of this research focuses on numerical instabilities in the model and their impact on the computation time. Seventeen causes of numerical instabilities were found in the model of the case study and are largely solved. The instabilities were sometimes rooted in incorrect data, while others were related to the automatic model construction with the D-HyDAMO package. For example, some channels had no control water level defined in the data which is crucial for the operation of control structures. Besides problems with the control water levels, incomplete data for culverts, incorrect bed levels and missing elevation data caused numerical instabilities. For the model set-up, the control parameters of the weirs and sluices were incorrectly set, resulting in excessive drainage of water or unintentional water retention in some areas for instance. The data problems were largely solved by obtaining the missing data or making assumptions. Most model construction flaws were solved by adjusting the D-HyDAMO script or the input data.

Incorrect data and model construction flaws led to locations in the model with a large "number of times the flow element was Courant limiting" value, high flow velocity, a sharp decrease or increase in water depth, or a water depth of almost zero meters. *Table 6* shows the root causes of the numerical instabilities that were found. A more detailed overview and explanation of all numerical instabilities, their root cause and the implemented solutions are shown in Appendix B.

Table 6 – Root causes of numerical instability.

#	Root causes of numerical instabilities	Type
1	Bed level of some waterways was equal to the control water level	Data
2	Either the summer or winter control water level was missing for seasonally controlled waterways	Data
3	Control water levels were missing for some waterways with a fixed control water level	Data
4	2D input elevation map contained -10 mNAP values for cells without data in the original map	Data
5	Inflow height of some culverts was not present or incorrect in the data	Data
6	Control water level for some weirs was not present in the data	Data
7	No initial water depth defined on the 2D grid between the retaining elements	Model set-up
8	Control parameters of the weirs set incorrectly	Model set-up
9	Control water level of sluices incorrectly derived from control water level map	Model set-up
10	Incorrect snapping of pumps to branches	Model set-up
11	Incorrect snapping of fish passages to branches	Model set-up
12	Incorrectly merging branches that flow above each other (with the help of culverts)	Model set-up
13	Incorrect location of observation points used in the determination of the control water level of pumps	Model set-up
14	Unintended merging of branches that flow parallel to each other	Model set-up
15	Incorrect snapping of weirs to branches	Model set-up
16	No possibility of defining additional sluice/orifice parameters	D-HYDRO GUI limitation
17	Unexpected behaviour of D-HYDRO	D-HYDRO

By improving the data and model set-up, it is expected that the model has become more accurate in representing reality. The optimised benchmark model is therefore not quantitatively assessed on the accuracy in relation to the original model but only on the computation time. Both models are run with and without the 2D grid. The results are shown in *Table 7*.

Table 7 - Computational benefit of resolving numerical instabilities.

Dimensions	Simulation period [h]	Computation time [h]		Reduction computation time [%]
		Original model	Optimised benchmark	
1D only	24	28.8	1.08	96%
1D and 2D	3	45	0.99	98%

Without the 2D grid, the computation time is reduced by 96% when implementing all the solutions to the numerical instabilities. This is probably largely caused by the 96% decrease in the maximum "number of times a flow element was Courant limiting" parameter during 1 simulation hour from 82,187 to 3,115, see *Table 8*. Furthermore, the number of locations that have a flow velocity higher than 2 m/s is reduced from 21 in the original model to 2 in the optimised benchmark model (see *Table 9*). Lastly, the water levels are significantly more stable in the optimised benchmark model which can be concluded from *Figure 7*. This figure shows the difference in the water level on the 1D grid between the start and end of the simulation period. Because all branches start with their control water level, and no precipitation occurs, water levels are expected to be fairly stable during the simulation. The optimised benchmark performs significantly better.

The same patterns are seen when running the complete model including the 2D grid. The original model with the 2D grid computes 15 times slower than in real-time. This would mean that a simulation of 1 day results in a computation time of 15 days. Because the execution period is limited, it was chosen to cease the run after 3 simulation hours and compare this with the computation time of the optimised model on a simulation period of 3 hours. The difference in the maximum "number of times a flow element was Courant limiting" parameter is not as large as for the runs without the 2D grid. The number of locations with a flow velocity of 2 m/s or larger is reduced from 15 to 4. Furthermore, the optimised benchmark has no location that has a flow velocity higher than 2 m/s on the first time step. This indicates that the initial conditions are close to the equilibrium situation.

Table 8 - Number of times a flow element was Courant limiting (NumLimdt).

Dimensions	max. NumLimdt in 1 h		#locations NumLimdt > 1000 in 1 h	
	Original model	Optimised benchmark	Original model	Optimised benchmark
1D only	82,187	3,115	4	1
1D and 2D	64,414	31,110	3	3

Table 9 – Number of locations with a flow velocity larger than 2 m/s.

Dimensions	#locations flow velocity > 2 m/s		#locations flow velocity > 2 m/s on first time step	
	Original model	Optimised benchmark	Original model	Optimised benchmark
1D only	21	2	14	0
1D and 2D	15	4	15	0

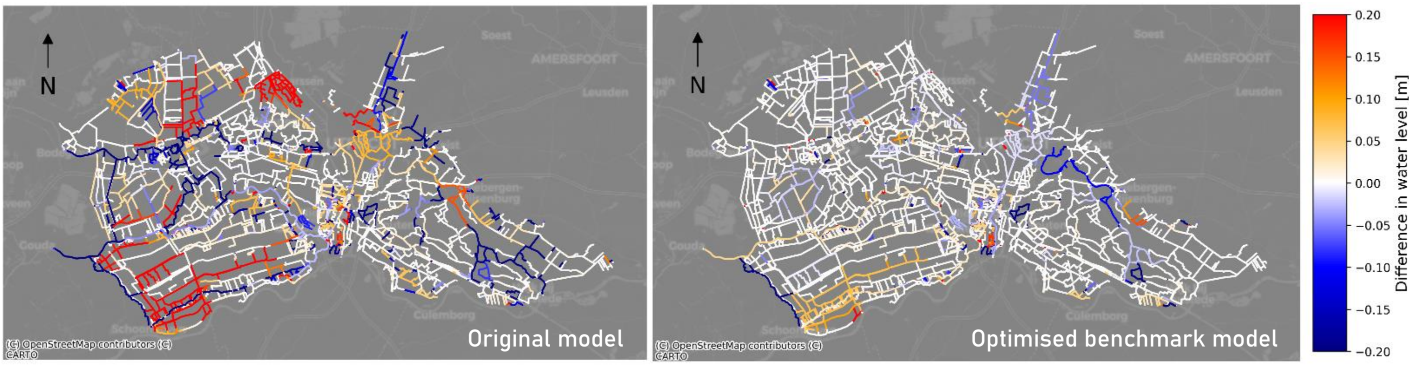

Figure 7 - Difference in water level on the 1D network between the start and end of the simulation period.

Although most numerical instabilities are found and solved, some problems with the 2D grid are still present in the optimised benchmark model. *Figure 8* shows the maximum water depth on the 2D grid for the event of July 2014. The inundated areas in the west and southwest (highlighted in red) are not flooded by precipitation but by leakage of water from the 1D nodes to the 2D grid since this area starts to inundate on the first time step when no rain has fallen yet. The water can sometimes leak outside of the retaining elements as dikes and banks, which then unintentionally floods large areas. This problem is probably caused by incorrect profiles and bed levels of the 2D grid which are derived from the Actueel Hoogtebestand Nederland (AHN). Solving this problem is difficult. Furthermore, the benchmark model and surrogate models all leak, because the same AHN data is used for both. This means that the models can still be compared and it was therefore chosen to leave this problem in the optimised benchmark. Nevertheless, Chapter 7 will discuss the potential implications of this leak on the results.

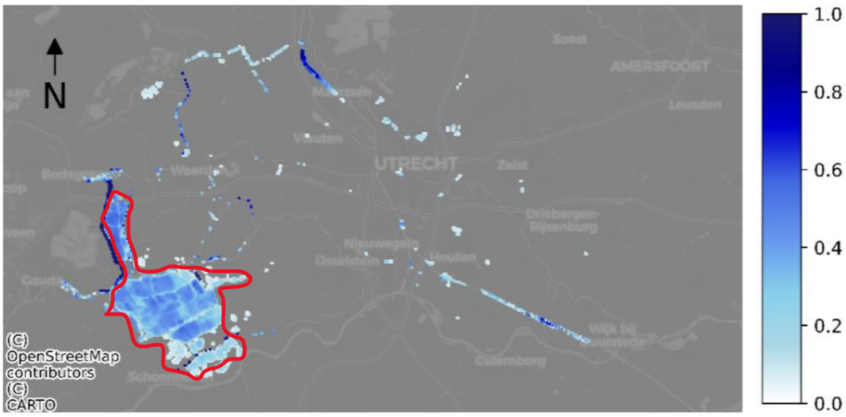

Figure 8 - Maximum water depth on the 2D grid for the optimised benchmark model. The areas that start to inundate at the start of the simulation are highlighted in red.

6.2 Numerical simplifications

This section will elaborate on the results of the surrogate modelling, for which numerical simplifications are applied to the optimised benchmark model to see their effect on the computation time and accuracy. The effect of the maximum Courant number, 2D grid resolution, number of 1D calculation points, combinations of simplifications and the results for the second event will be discussed in order. The results for all surrogates are shown in *Table 10* and visualized in *Figure 9* to *Figure 12*. Enlargements of these figures and more visualizations can be found in Appendix D. The numbering of the surrogates follows the next pattern: SUGs #dX adjust the 1D calculation points, SUGs #DX adjust the 2D resolution, SUGs #CX adapt the Courant number, SUGs #OX implement other numerical simplifications and SUGs #MX are mixes of simplifications.

Table 10a - Results of the surrogate modelling by implementing numerical simplifications. Yellow cells indicate changes with respect to the optimised benchmark. Green cells indicate if the indicator is within the acceptable limits, see Table 5. The table is continued on the next page.

Event July 2014	**Optimised benchmark**	**SUG #d1**	**SUG #d2**	**SUG #d3**	**SUG #d4**	**SUG #d5**	**SUG #D1**	**SUG #D2**	**SUG #C1**	**SUG #C2**
Distance 1D nodes [m]	50	100	250	500	50	50	50	50	50	50
Bridges removed	No	No	No	No	Yes	No	No	No	No	No
Culverts merged	No	No	No	No	No	Yes	No	No	No	No
Spatial resolution 2D grid [m]	100	100	100	100	100	100	250	500	100	100
Maximum Courant number	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	1	5
2D grid clipped	No	No	No	No	No	No	No	No	No	No
User timestep [s]	3600	3600	3600	3600	3600	3600	3600	3600	3600	3600
Resolution refinement 2D grid [m]	-	-	-	-	-	-	-	-	-	-
Results computation time										
Computation time [min]	1501	1358	1198	1158	1367	1291	1793	642	1086	126
Reduction computation time w.r.t. optimised benchmark [%]	-	9.5	20.2	22.9	8.9	14.0	-19.5	57.2	27.6	91.6
Average time step [s]	1.4	1.4	1.4	1.5	1.4	1.4	0.9	2.6	2.0	26.7
Results accuracy	**Accepted if**									
MAE avg. water depth 1D [m]	< 0.05	0.050	0.082	0.084	0.003	0.014	0.084	0.131	0.001	0.004
KGE water depth 1D	> 0.75	0.66	0.55	0.55	0.96	0.80	0.50	0.34	0.98	0.93
MAE max. water depth 1D [m]	< 0.05	0.053	0.083	0.086	0.003	0.023	0.092	0.144	0.001	0.004
MAE avg. water depth 2D [m]	< 0.05	0.147	0.225	0.223	0.006	0.008	0.144	0.300	0.00	0.001
KGE water depth 2D	> 0.75	-0.08	-0.22	-0.22	0.96	0.92	0.46	-0.81	1.00	0.99
MAE max. water depth 2D [m]	< 0.05	0.177	0.271	0.267	0.006	0.009	0.163	0.351	0.00	0.002
Precision flood extent	> 0.7	0.75	0.89	0.78	1.00	0.99	0.73	0.46	1.00	1.00
Recall flood extent	> 0.7	0.91	0.32	0.31	0.98	0.97	0.69	0.05	1.00	1.00
F1-score flood extent	> 0.7	0.82	0.47	0.44	0.99	0.98	0.71	0.09	1.00	1.00

Table 10b - Results of the surrogate modelling by implementing numerical simplifications. Yellow cells indicate changes with respect to the optimised benchmark. Green cells indicate if the indicator is within the acceptable limits, see Table 5.

Event July 2014	**Optimised benchmark**	**SUG #C3**	**SUG #C4**	**SUG #O1**	**SUG #O2**	**SUG #O3**	**SUG #M1**	**SUG #M2**	**SUG #M3**	**SUG #M4**
Distance 1D nodes [m]	50	50	50	50	50	50	50	75	75	75
Bridges removed	No	No	No	No	No	No	Yes	No	Yes	Yes
Culverts merged	No	No	No	No	No	No	Yes	No	Yes	Yes
Spatial resolution 2D grid [m]	100	100	100	100	100	200	100	150	100	100
Maximum Courant number	0.7	10	50	0.7	0.7	0.7	5	0.7	5	0.7
2D grid clipped	No	No	No	Yes	No	No	No	No	No	No
User timestep [s]	3600	3600	3600	3600	7200	3600	3600	3600	7200	3600
Resolution refinement 2D grid [m]	-	-	-	-	-	100	-	-	-	-
Results computation time										
Computation time [min]	1501	91	76	1095	1487	1481	99	1471	76	640
Reduction computation time w.r.t. optimised benchmark [%]	-	93.9	94.9	27.0	0.9	1.3	93.4	2.0	94.9	57.4
Average timestep [s]	1.4	46.0	59.6	1.9	1.4	1.2	29.7	1.1	40.6	1.6
Results accuracy	**Accepted if**									
MAE avg. water level 1D [m]	< 0.05	0.014	0.016	0.075	0.00	0.125	0.020	0.067	0.072	0.069
KGE water level 1D	> 0.75	0.86	0.82	0.40	1.00	0.30	0.72	0.63	0.40	0.48
MAE max. water level 1D [m]	< 0.05	0.015	0.017	0.076	0.00	0.141	0.028	0.072	0.088	0.084
MAE avg. water depth 2D [m]	< 0.05	0.043	0.041	0.150	0.00	0.243	0.016	0.131	0.198	0.198
KGE water depth 2D	> 0.75	0.72	0.72	0.73	1.00	-0.39	0.87	0.36	-0.62	-0.62
MAE max. water depth 2D [m]	< 0.05	0.053	0.050	0.169	0.00	0.288	0.016	0.149	0.240	0.240
Precision flood extent	> 0.7	1.00	0.99	0.80	1.00	0.53	0.99	0.58	0.68	0.68
Recall flood extent	> 0.7	0.90	0.92	0.72	1.00	0.16	0.94	0.78	0.91	0.91
F1-score flood extent	> 0.7	0.95	0.95	0.76	1.00	0.24	0.97	0.67	0.78	0.78

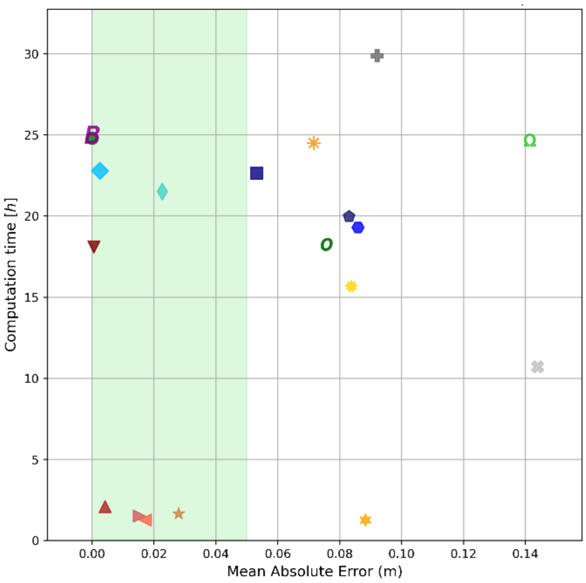

Figure 9 – MAE of the max water level in 1D calculation points.

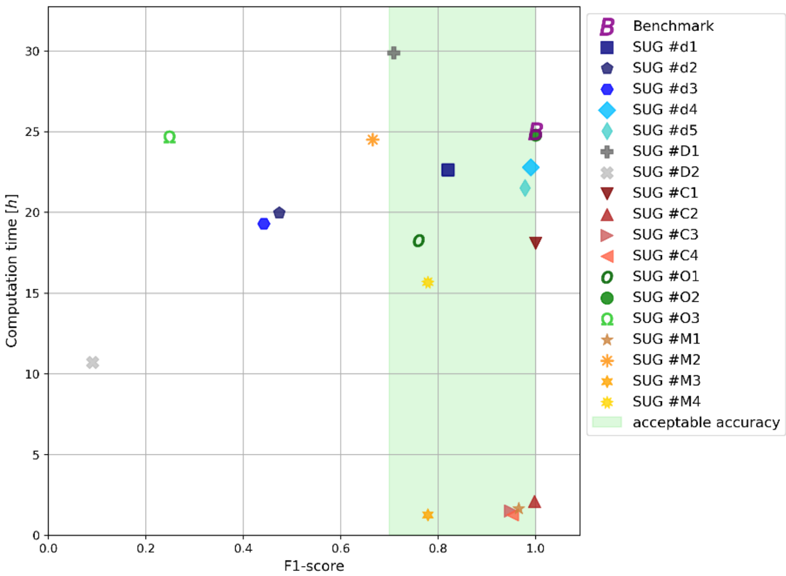

Figure 10 – F1-score of the 2D flood extent.

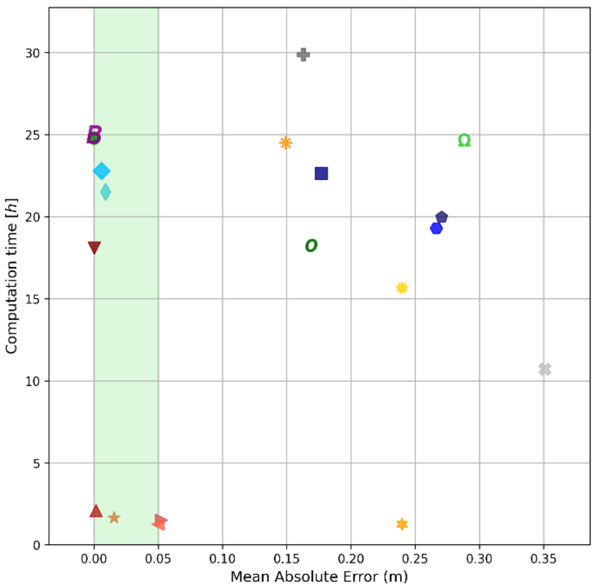

Figure 11 – MAE of the max water depth in 2D calculation points excluding TN cells.

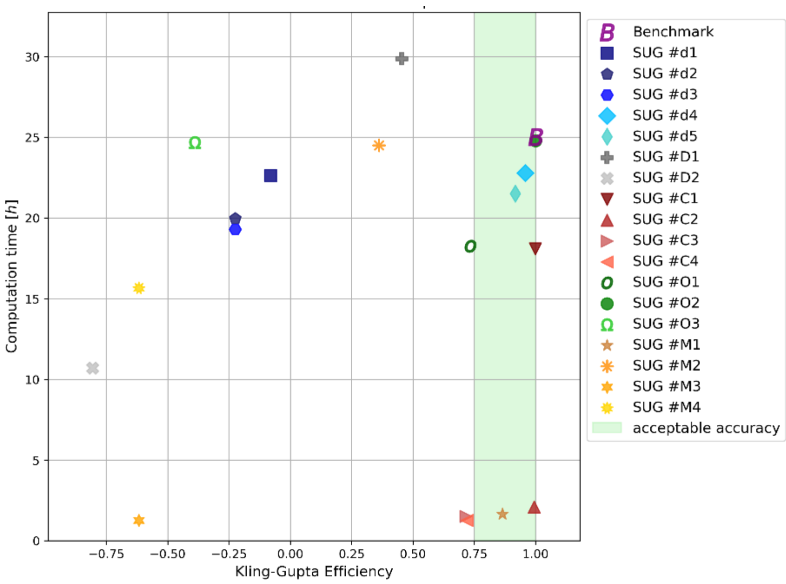

Figure 12 – KGE of water level over time in 2D calculation points excluding TN cells

6.2.1 Maximum Courant number

Increasing the maximum Courant number seems to effectively decrease the computation time by more than 90% while keeping a high accuracy, as shown by SUG #C1 to SUG #C4 (see red triangles in *Figure 9* to *Figure 12*). A maximum Courant number between 1 and 5 performs sufficiently on all indicators.

SUG #C1, which has a maximum Courant number of 1, has almost an equal output to the benchmark, with an MAE of only 1 mm for the 1D network and 0 mm for the 2D grid. Furthermore, the variability of the water level over time and the flood extent are modelled almost perfectly. This is as expected because for both the benchmark and the surrogate, particles of water can flow at most one cell in one time step since the maximum Courant number is constrained to 1 or smaller.

While SUG #C1 yields a computation time reduction of only 27.6%, SUG #C2 reduces the computation time by 91.6%. With the increase to a maximum Courant number of 5, only 2 mm of additional error in the water level is introduced. Nevertheless, some spatial differences occur in the MAE (*Figure 13*). However, 98.3% of the 1D calculation points still have an error smaller than the threshold of 0.05 m.

Figure 13 - MAE of the maximum water level in the 1D network for SUG #C2.

Increasing the maximum Courant number above 5 with SUG #C3 and #C4 barely yields an additional reduction in computation time compared with SUG #C2, while the KGE indicators start to fall below the threshold of 0.75. This means that the variability in the water level over time in the surrogate is not equal to the variability in the benchmark. By taking a larger maximum Courant number, the time step increases, which translates to a less accurate timing of the peak flow. Secondly, the KGE also includes bias. Because the error in the water level increases for these surrogates, the bias becomes larger. Both processes deteriorate the KGE score of these surrogates.

6.2.2 2D grid resolution

The 2D grid resolution has a large impact on the model performance, which is illustrated in *Figure 9* to *Figure 12* by the grey crosses that are located outside the green-shaded areas. For SUG #D1, the 2D grid resolution was decreased from 100 m to 250 m but the computation time increased by 19.5%. The maximum number of times a flow element was Courant limiting increased from 31,870 in the benchmark model to 84,745 in this surrogate model. As a result, the average time step is 0.9 seconds for this surrogate instead of 1.4 seconds for the benchmark model. This leads to a larger computation time for SUG #D1. In addition, the performance on accuracy is outside the acceptable ranges. Increasing the grid size further to 500 meters also enlarges the error in the water level of the 1D network and 2D grid (SUG #D2).

SUG #O3 uses a grid of 200 m resolution with a refinement of 100 m around the main waterways. This approach reduced the computation time by only 1.3%. At the edge of the refinements, small triangular cells are added by D-HyDAMO as a transition between the two grid resolutions. The small size of these cells limits the time step according to the Courant condition (*Eq. 4*). Besides that, none of the metrics is within the acceptable accuracy range for this surrogate.

SUG #O1 removes the 2D grid cells that are located directly under the 1D boezem branches. These cells contain the profile of the waterway in the elevation of the cells. Incorrect elevation of these cells may lead to leaking. Although the flood extent is still the same, the water depth in the west of the area is significantly smaller (*Figure 14*), which means that less water is leaking on the 2D grid with this method. On the other hand, part of the 1D-2D flow links is removed for the cells that are clipped. As a result, water that flows to the 2D grid is concentrated around the sparse number of 1D-2D flow links that are left in the model, leading to larger maximum water depths in some small areas (*Figure 14*). This problem can be solved by adding laterals, which are a special type of 1D-2D links. However, this function is not yet supported in D-HyDAMO and is therefore not applied in this thesis. The computation time of SUG #O1 is significantly faster than the benchmark model with a reduction of 27.0%. This is probably caused by a combination of a smaller amount of water that needs to flow over the 1D-2D links and 9.4% fewer 2D grid cells compared to the optimised benchmark model. This surrogate is in theory more accurate than the benchmark model since the leaking problem is partly solved.

Figure 14 - Difference in water depth on the 2D grid between the benchmark model and SUG #01. Blue indicates that the water depth in the surrogate is smaller than in the benchmark and red indicates that the water depth in the surrogate is larger than in the benchmark model.

6.2.3 Number of 1D calculation points

Reducing the number of 1D calculation points yields a relatively small reduction in computation time when compared to the other measures discussed above. Furthermore, increasing the distance between the 1D nodes (SUG #d1 to SUG #d3) scores outside the acceptable performance range for most metrics. This might be related to the number of 1D-2D links, which decreases with the number of 1D calculation points. With fewer 1D-2D links, the flow of water to the 2D grid is impeded which can lead to backwater and thus higher water levels in the 1D network. This probably also means that a larger amount of water is flowing over each of the 1D-2D links. As a consequence, the maximum water levels of the 2D grid cells in which the flow links end are higher. The precision is the only metric that scores in the acceptable range for all these three surrogates which indicates that these surrogates are not overpredicting the flood extent.

The number of 1D calculation points was reduced by 8.3% by removing all bridges from the model (SUG #d4) as each bridge normally adds two additional calculation points. This surrogate is 8.9% faster than the benchmark model. The accuracy is well within the limits, indicating that the maximum water levels, variability and flood extent can all be modelled sufficiently without the 1D points that come with the bridges in the model.

Lastly, culverts on the same branch were merged in SUG #d5 which reduced the number of 1D calculation points by 13.3%, resulting in a computation time reduction of 14%. All metrics are within the thresholds. However, local errors arise just before the entrance of long culverts with small dimensions since the water accumulates at these locations. This can be seen in *Figure 15* as dark red spots on the map. When merging the culverts, the most limiting values of the original culverts are taken and used in the merged culvert. This leads to higher backwater.

Figure 15 - MAE of the max water level in the 1D calculation points for SUG#d5.

6.2.4 Other simplifications

Adjusting the user timestep (SUG #O2) changes how often the external forcings are read and processed by the model and does not influence the maximal Courant number. It has, therefore, no influence on the average calculation timestep during the run. With an increased user time step, the model spends less time reading and processing the forcing files, but the effect on the computation time is minor with a reduction of 0.9%. The output is extremely similar to the benchmark.

6.2.5 Combinations of simplifications

Several combinations of the above simplifications were tested as well. SUG #M1 combines the removal of bridges, merging of culverts, and a maximum Courant number of 5 because the accuracy of these measures individually stayed within the thresholds while generating significant reductions (>5%) in the computation time each with 8.9%, 14.0% and 91.6% respectively. These measures combined give a reduction of 93.4%. The maximum water levels were simulated highly accurately (MAE in *Table 10*), as well as the inundation extents. However, the water level time series in the 1D network performs just below the threshold.

The surrogates changing the 2D resolution and distance between the 1D network separately did not perform well on accuracy. If the difference in the 1D and 2D resolution is large, the 1D-2D links can be unbalanced. SUG #M2 solves this by adjusting the 1D and 2D resolution with the same factor of 1.5, such that the ratio between 1D nodes and 2D cells that are connected with 1D-2D links remains the same. This surrogate is only 2% faster than the optimised benchmark and all metrics except the recall perform outside the thresholds. This combination seems therefore not suitable for accelerating detailed 1D2D hydrodynamic models.

SUG #M3 increases the 1D node distance to 75 m, merges culverts, removes bridges, increases the Courant number to 5 and sets the user timestep to 2 hours since these measures seem to have sufficient accuracy when applied individually. However, when combined, all error metrics except the F1-score are outside of the acceptable ranges. The computation time saving is 94.9% which is equal to the reduction yielded with a Courant number of 50 (SUG #C4). SUG #C4 scores significantly better on accuracy and it is therefore preferred to use a Courant number of 50 instead of this combination of measures.

SUG #M4 adopts all methods to reduce the number of 1D calculation points to see how this impacts the computation time and accuracy: increasing the 1D node distance to 75 m, merging culverts and removing bridges. This surrogate only accurately predicts the flood extent but not the water level and water level time series. Because a combination of removing bridges and merging culverts (SUG #M1) scores within the limits, it is plausible that increasing the 1D node distance in a combination of measures results in a significant error.

6.2.6 Second precipitation event

SUG #C2, SUG #O1, SUG #M1 and SUG #M2, representing almost all of the tested numerical simplifications, are run for the event of September 2018 to see if similar results can be obtained for a different event. A detailed overview of the performance of these surrogates for this event is included in Appendix C.

The same metrics fall within the acceptable ranges on both events for all of the four evaluated surrogates. The MAE values in both the 1D and 2D calculation points are of the same order of magnitude, see *Figure 16* for an illustration. The KGE metric for the event of July 2014 performs better (*Figure 17*). The event of July 2014 has only one peak of rainfall while the event of September 2018 contains three rainfall peaks (*Figure 5* and *Figure 6*). More rainfall peaks make the simulation more prone to errors in the water level time series, resulting in a lower KGE value.

All the surrogates show that the reduction in computation time is relatively similar for both events.

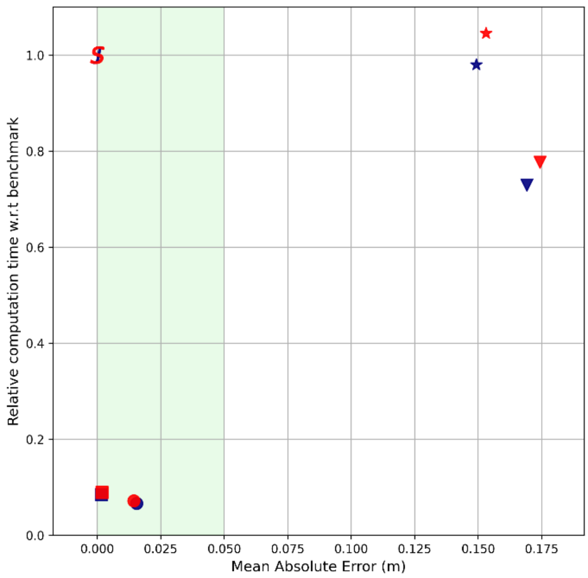

Figure 16 - MAE of the max water level in 2D calculation points for both precipitation events.

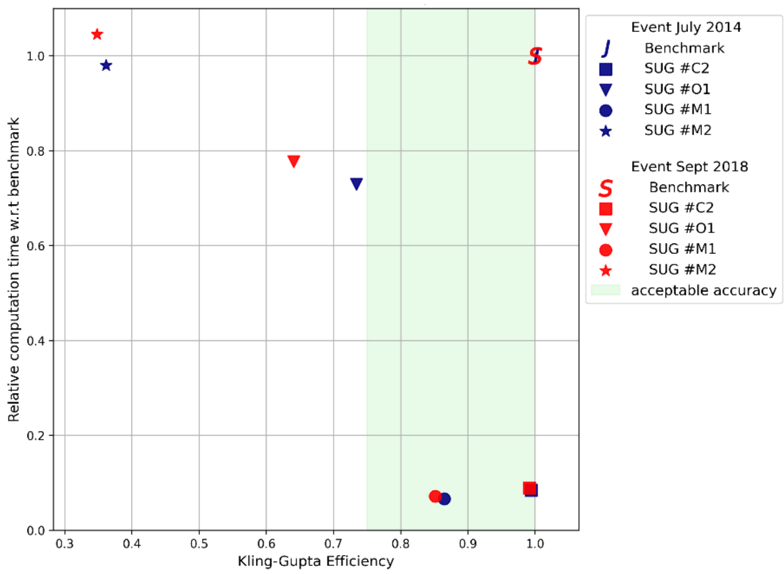

Figure 17 - KGE of the water level time series on the 2D grid for both precipitation events.

7 Discussion

This section will discuss the methods and results of this study. First by expatiating some limitations caused by the model and the choices in the methodology, then by comparing the results found in this study to other studies and seeing the differences and agreements, and lastly by elaborating on the applicability and generalisation of the results.

7.1 Limitations

Several limitations are introduced by the model characteristics and the methodological choices. These and their effects on the results will be discussed in this chapter.

7.1.1 Model restrictions

The model of the HDSR study area used in this study has some limitations. First of all, the model was not calibrated when starting with this thesis because HydroLogic further improved the model in the subsequent period. This means that the benchmark model used in this study is not calibrated which might impact the results as the water levels can be constantly too low or high. A persistent over- or underestimation of the water level can interfere with the automatic operation of the control structures that are dependent on the water level for their operation. Furthermore, if the water level in a channel is overpredicted at the start of a rainfall event, the available storage in the waterway is smaller, leading to flooding of the 2D grid faster. This is also true for an underprediction of the water level, but the flooding of the 2D grid will come later than in reality. Since this study compared the performance of the surrogates against the benchmark model, this is not a major problem as the problem occurs in both. However, when applying the HDSR model to decision-making in the water management strategy, the model must first be calibrated and validated to achieve more reliable results.

Secondly, the rainfall data is only available as an average over the whole area. However, the extreme events which are relevant for pluvial flooding are often local and it can be that some parts of the study area faced significantly more precipitation than the average for the event of July 2014. As spatial rainfall variability is not included in both the benchmark and surrogate models, still a fair comparison of model performance between the two could be made.

Thirdly, the optimised benchmark model still leaked water onto the 2D grid cells, leading to flooding without rainfall. This has likely influenced the F1-score, which depends on the number of true positive cells. The leak increases the number of true positive cells significantly, while the number of false positive and false negative cells remains almost equal because the leaking occurs at the same location for the benchmark model and surrogates. According to *Eq. 9*, this increase in TP cells improves the F1-score. As a consequence, the F1-score might seem to perform sufficiently while it might not be acceptable if no leaking had occurred. However, it is difficult to determine whether a cell is flooded by leaking, pluvial flooding or a combination of both. For SUG #D1 and SUG #O1, the F1-score is just above the threshold. If no leaking occurred, it might be that these surrogates would have scored insufficient on this metric.

Lastly, the surrogates with clipped and refined 2D grids contain some errors. No 1D-2D links are added between 1D waterways for which the 2D grid is clipped. This can be solved by adding the special ‘lateral’ type of 1D-2D links for these branches, although this is not yet possible with D-HyDAMO. As a result of the missing 1D-2D links, water from the clipped branches cannot flow to the 2D grid or only in a limited number of locations via secondary branches. This results in higher water levels on the 1D branches and in the 2D cells that are at the end of the remaining links. Furthermore, because the water from these branches flows at another location on the grid, the accuracy can be negatively impacted. In addition, the leaking is not fully solved with this measure. Also combining a local grid refinement with a clip of the 2D grid still generates errors when running the model.

7.1.2 Methodological limitations
The choices made in the methodology can influence the results too. First of all, only one rainfall event is chosen to simulate all surrogates. Similar results are found for a second rainfall event for only a selection of surrogate models. Furthermore, this second event has no completely different characteristics than the first. For other types of events, for example, events that have a longer duration but a lower average intensity, the results might not apply. Although some surrogates are run for two events, a long wet period of around a month is not covered in the analysis.

Secondly, it was chosen to terminate the run of the original model after simulating 3 hours since the simulation was 15 times slower than reality. Although the comparison between the original model and the optimised benchmark model is based only on the first three hours of the simulation, potential behaviour later in the model is not included in this comparison. All the models started to accelerate after 8 to 12 hours. By terminating the model before this moment, the behaviour of the original model is not known for this period. It can be that the original does not start to accelerate in this period which would mean that the computation time reduction of the optimised benchmark is even larger than reported.

The assumptions for merging the culverts influence the results (see *Figure 15*) since the water accumulates at the entrance of the new culverts. The worst-case parameter values were chosen to use in the merged culverts. These are, for example, the largest roughness and smallest dimensions. However, not all these parameters are necessarily originating from the one culvert which is the most limiting. These assumptions might therefore be too strict and results with lower error values could potentially be found when assuming less strict parameter values for the merged culvert.

Lastly, the choice of certain metrics and their thresholds can largely influence the conclusions. Initially, the Nash-Sutcliffe Efficiency (NSE) was calculated instead of the Kling-Gupta Efficiency (KGE) to evaluate the variability in water level over time of the surrogates. However, the NSE often approached minus infinity for some calculation nodes that had no or extremely small variability in the water level. As a result, the NSE appeared almost always unacceptable. The KGE is less sensitive to nodes with a small variability in the water level, resulting in a more balanced scoring. This shows that the choice for a certain metric can influence the results. Moreover, the recommendations on which methods are suitable for accelerating a detailed 1D2D hydrodynamic model are largely dependent on the thresholds set for each metric. For example, the KGE is good if larger than 0.75, which was here taken as a reference, and sufficient between 0.5 and 0.75 (Gupta et al., 2009; Kling et al., 2012). If the threshold was set to 0.7, still close to a “good” performance, SUG #C4 and SUG #M1 would have scored acceptable on all metrics and would be recommended as a suitable method, while these two surrogates are not recommended if a threshold of 0.75 is used.

7.2 Results in relation to existing literature
This section will shortly compare the results found in this study with the literature to see differences and agreements.

The results related to increasing the maximum allowed Courant number are largely in line with other studies although the reduction in computation time is in general slightly larger for this study. Janssen (2023) found that increasing the maximum Courant number from 0.7 to 10 could reduce the computation time by 80% while only introducing a 0.2 mm error in the water level. The results of this thesis indicate that a maximum Courant number of 10 can lead to a reduction in computation time of 93.9% while introducing around 5 cm error in the water level on the 2D grid. Compared to Janssen (2023), the reduction in computation time and the error are higher. Similar to Janssen (2023), increasing the maximum Courant number above 10 does not result in significant differences in both computation time and accuracy. Hop (2021) yielded a computation time reduction of 75% with a maximum Courant number of 50, while this study shows a reduction of 94.9%. However, both Janssen (2023) and Hop (2021) indicate that an increased maximum Courant number is the most effective numerical simplification for reducing the computation time, which is also found in this study.

Nevertheless, a maximum Courant number above 1 is seldom seen in literature (e.g. Bomers et al., 2019a; Bomers et al., 2019b; Hardy et al., 1999 all use a maximum Courant number below 1). This is because values greater than 1 can lead to numerical errors growing over time in explicit calculation schemes (Akbari & Firoozi, 2010; Deltares, 2024). Some areas in the simulation of SUG #C2, #C3 and #C4 experience significantly larger errors than the average error (see *Figure 13*). This is probably caused by this phenomenon of an unstable solution when using an explicit calculation scheme in combination with a maximum Courant number that is larger than 1. The affected areas have presumably a relatively large error at the first time steps which then grows to significantly larger errors later in the simulation. For longer simulations, the error can grow larger which can be a problem for applying a maximum Courant number larger than 1 in a simulation that is longer than the simulations discussed in this thesis.

McMillan & Brasington (2007), could yield a reduction in computation time of over 90% by increasing the grid size by a factor of 5. In this study, a grid size of 500 m instead of 100 m resulted in a 57.2% shorter computation time. With the coarsened grid, the error in the water level becomes 20 cm in the study of McMillan & Brasington (2007), which is in line with the results of SUG #D2 which has an error of around 15 cm in the 1D nodes and 30 cm in the 2D nodes. The maximum water depth on the 2D grid in this thesis is generally lower for coarser resolutions (blue areas in *Figure 18*). This is contrary to what Bomers et al. (2019b), Judi et al. (2014), Horritt et al. (2006) and Caviedes-Voullième et al. (2012) found. A coarser grid averages out the profiles of the waterways on the 2D grid. As a result, the elevation of the 2D cells beneath the 1D network will be higher, limiting the flow from the 1D network to the 2D cells. In other words, a coarser grid reduces the amount of leaking which results in lower maximum water depths on the 2D grid in this case.

Figure 18 - Difference in water depth on the 2D grid between the benchmark and SUG #D2 (resolution of 500 m). Blue indicates that the water depth in the surrogate is smaller than in the benchmark model.

The study by Davidsen et al. (2017) showed that reducing the number of 1D calculation points by 66% can result in a 35% decrease in computation time. For SUG #d2, the number of 1D nodes is reduced by 40% resulting in a reduction of the computation time by 20%, which is approximately the same ratio. However, Davidsen et al. (2017) could yield acceptable results when decreasing the calculation nodes by 66%. The surrogates affecting the number of 1D calculation points in this study are only acceptable when removing the bridges or merging the culverts, decreasing the number of 1D nodes by only 8.3% and 13.3% respectively.

7.3 Applicability and generalisation

The results are partly generalisable to other models and studies. The list of numerical instabilities presented in section 6.1 (*Table 6*) can be used as a guideline for checking potential problems in all other hydrodynamic models featuring a 1D-2D grid and/or control structures such as pumps and weirs. A large part of potential numerical instabilities can easily be found when checking all elements in this list.

Furthermore, the methods of finding numerical instabilities can be applied to other models as well to detect additional numerical instabilities that were not encountered in this study. Nevertheless, the solutions to solve most numerical instabilities are often case-specific and might need to be tailored to each project individually. For example, assuming a bed level 0.8 m below the control water level for waterways without a specified bed level is for most branches in this case study a realistic assumption. However, this might not apply to other studies. Although improving the data is a good starting point for resolving numerical instabilities, assumptions to cover missing data are more case-specific.

Concerning numerical instabilities related to the model set-up, some solutions are case-specific. For instance, pumps that snapped to the incorrect branch were manually relocated in the raw data to be as close to the correct branch as possible. For this case study, around 7 pumps were relocated in this manner. For other studies, it is neater to adjust the D-HyDAMO script to cope with this problem. On the other hand, the solution to ensure that culverts crossing above each other are not merged can be applied to other studies as well for example. In conclusion, the list of numerical instabilities and the methods for finding additional numerical instabilities can form a starting point for other studies.

The results of the surrogate modelling with numerical simplifications will probably apply to most other detailed 1D2D hydrodynamic models that describe a similar type of area and water system. The simplifications can at least be implemented in other 1D2D hydrodynamic models. The results may vary slightly in other studies depending on the set-up of the model. For example, the model of the case study contains a relatively dense 1D network. Adjusting the number of 1D nodes has for this study probably a larger impact than for a model with a less dense 1D network. Another example is the merging of culverts and removal of bridges, for which the impact on the computation time is purely determined by the number of culverts or bridges in the model. Lastly, numerical errors keep growing as long as the simulation continues when using a maximum Courant number above 1 (Akbari & Firoozi, 2010). The results might therefore not apply to simulations with a maximum Courant number larger than one that have a completely different duration than seen in this thesis. Yet it is expected that similar trends in the computation time and accuracy apply to relatively equal models.

8 Conclusion & recommendations

The main objective of this study is to optimise the calculation time of a detailed 1D2D hydrodynamic model by resolving numerical instabilities and implementing numerical simplifications. Four research questions were formulated to answer the main research question. This chapter will answer these research questions. Furthermore, some recommendations will be made for further research.

8.1 Conclusions

8.1.1 Numerical instabilities and computation time

First of all, research was done into the influence of numerical instabilities on the computation time of a detailed 1D2D hydrodynamic model to answer the following question:

> *How do numerical instabilities affect the computation time of a detailed 1D2D hydrodynamic model?*

For the case study, seventeen causes of numerical instabilities were found. After solving most numerical instabilities in the HDSR model, the model was run and compared to the original model. The optimised benchmark model appears to be 96% faster than the original model when running with the 1D network only and 98% faster when running with both the 1D and 2D grid, which shows that resolving numerical instabilities in the model is effective in reducing the computation time. Nevertheless, if no or small numerical instabilities are present in a model, the reduction in computation time will be lower. In addition to the large computation time reduction, the optimised benchmark model is more accurate by improving the data and model construction.

8.1.2 Numerical simplifications

Several numerical simplifications were implemented in the optimised benchmark model. Multiple surrogates were constructed that adjusted the maximum Courant number to answer the following research question:

> *How does a larger maximum Courant number affect the computation time and model accuracy of a detailed 1D2D hydrodynamic model?*

Adjusting the maximum Courant number appears to be the most effective numerical simplification to reduce the computation time of a detailed 1D2D hydrodynamic model. A decrease in computation time of almost 95% can be achieved while introducing less than 5 centimetres error in the water level for a maximum Courant number of 50. Using a maximum Courant number of up to 5 can still reduce the computation time by 91% and introduce less than 0.5 cm error in the water level. A Courant number of 5 seems therefore the best trade-off between accuracy and computation time.

> *How does a coarser 2D grid affect the computation time and model accuracy of a detailed 1D2D hydrodynamic model?*

First of all, a coarser 2D grid introduces significant errors in the water level of more than 8 cm in the HDSR model. In this study, a grid size of 250 meters instead of 100 meters increased the computation time by 19.5% due to new instabilities in the model. A grid size of 500 meters reduced the computation time by 57%. If no new instabilities arise, reducing the resolution of the 2D grid seems to be effective in reducing the computation time, but due to a relatively large error, this method is not recommended to apply in detailed 1D2D hydrodynamic models.

How does a reduced number of 1D calculation points affect the computation time and model accuracy of a detailed 1D2D hydrodynamic?

The distance between the 1D nodes was multiplied with a factor between 1.5 and 10, resulting in a reduction in computation time ranging from 9.5% to 22.9% and an error in the water level of 5 to 27 cm. Merging the culverts and especially removing the bridges show acceptable results in terms of accuracy. The computation time could be reduced by 14% and 8.9% respectively although this will depend on the number of bridges and culverts in the model.

8.1.3 Main research question

Which methods are effective in reducing the computation time of a detailed 1D2D hydrodynamic model while keeping sufficient accuracy in the output?

Effective methods for reducing the computation time of a detailed 1D2D hydrodynamic model are mainly resolving numerical instabilities if these exist in the model. This can reduce the computation time by 96% but this depends on the presence of numerical instabilities in the model. By resolving numerical instabilities, the accuracy of a model can improve as well. If numerical instabilities are not present or more reduction in computation time is required, increasing the maximum Courant number is the most effective numerical simplification with a reduction in computation time of over 90%. This simplification also introduces the least error of all examined simplifications, ranging between 0 and 5 cm in the maximum water level.

8.2 Recommendations on the application of the results

Resolving numerical instabilities can advance the computation time of a detailed 1D2D model significantly and can improve the accuracy. It is therefore recommended to always analyse if numerical instabilities exist in a hydrodynamic model and solve them if possible. The methodology proposed in this thesis in combination with the checklist of numerical instabilities found in this study (*Table 6*) can be used to efficiently pinpoint and identify locations with numerical instabilities in future studies and model applications.

If it is desired to reduce the computation time further after resolving numerical instabilities, increasing the maximum Courant number between 1 to 5 is highly effective and recommended as a subsequent step. Other numerical simplifications are not recommended as these introduce significantly more error in the output while resulting in a lower computation time reduction. For studies in which accuracy is extremely important, it is advised to use a maximum Courant number of 1. This will almost certainly guarantee that no new instabilities arise because water can still flow through 1 grid cell per time step while the computation time can be reduced by around 27%, depending on the model characteristics. If reduction in the computation time is favoured over accuracy, or a slight error is acceptable, a maximum Courant number of 5 can be used. For the case study in this thesis, a maximum Courant number of 5 resulted in an error of around 0.5 cm in the water level. However, the error introduced by a maximum Courant number of 5 will depend on the model characteristics and simulation time. In addition, the error in the water level can be significantly higher than the average error for some local areas (see for example *Figure 13*). A maximum Courant number larger than 5 can yield acceptable results, but the reduction in the computation time compared to a simulation with a maximum Courant number of 5 is minimal. It is therefore not recommended to use a maximum Courant number that is larger than 5. In addition, it is always recommended to plot the accuracy spatially after applying a numerical simplification to see if specific areas have larger errors than others.

The model of the case study from HydroLogic must be run for around 10,000 events, where high accuracy is required to correctly map bottlenecks in the water system. The suggested approach is to run the optimised benchmark model with a Courant number of 5 for all these events and determine which of the events pose a problem to the water system. It is recommended to simulate these events again with the optimised benchmark model with a maximum Courant number of 1 to yield more accuracy for the relevant events. For studies and projects that need to evaluate large sets of events, this approach can be helpful.

Furthermore, it is advised to HydroLogic to examine the leakage in the HDSR model in more detail. Solving this problem could potentially speed up the model significantly. Moreover, if the leaking is resolved, the evaluation metrics will better describe the actual error in the flooded areas. Starting points for investigation can be the files that contain the initial water level for the 2D grid between the retaining elements and the bed level of the 2D grid. If the leaking is solved, some or even all surrogates can be constructed again to see if the results on the computation time and accuracy are significantly different.

Lastly, it is recommended to do more research on the areas that have significantly higher MAE values when using a maximum Courant number of 5 (*Figure 13*). Understanding what causes the higher error values at these locations can help to improve the model of HDSR even further and to interpret results for the evaluation of the 10,000 events.

8.3 Recommendations for further research

Multiple recommendations can be made regarding future research on the topic of accelerating detailed 1D2D hydrodynamic models. This section will shortly touch upon the steps that can be taken in the future to deepen the knowledge on this topic.

8.3.1 Other acceleration methods

As discussed in section 2.3, other methods exist to accelerate models besides numerical simplifications. This thesis focussed on low-fidelity physically based surrogates, but data-driven methods such as machine learning, neural networks and response surface surrogates can potentially provide alternative methods for accelerating detailed 1D2D hydrodynamic models. Hop et al. (2024) were able to construct a neural network that generates probabilistic inundation forecasts within seconds for a polder system. Although the practicability of these methods is often limited for larger models (Razavi et al., 2012), a combination of LFPS and data-driven methods might potentially be successful for accelerating a detailed 1D2D hydrodynamic model.

8.3.2 Expanding research LFPS

More research can be done on LFPS models to cover more situations and obtain more detailed results. As mentioned in the discussion, the results might not apply to a rainfall event that has different characteristics than the events used in this thesis. It might therefore be useful to test if similar results can be found with completely different rainfall events, for example, an event describing an extremely wet month. The same holds for a different topography, such as a mountainous area.

Secondly, just a limited number of surrogates are calculated and it might be useful to run more surrogates to get a better understanding of where the ranges are located for parameters that will result in sufficient accuracy. For instance, having surrogates with a 2D grid resolution of 125 m, 150 m, 200 m, 250 m, 350 m and 500 m instead of having two surrogates with a 2D grid resolution of 250 m and 500 m.

Furthermore, this thesis has treated the refinement and clipping of the 2D grid rather superficial, with only two surrogates. As discussed in Chapter 7, still several problems are encountered when clipping and refining the 2D grid with D-HyDAMO. It is advised to solve these problems by creating 1D-2D links between the clipped grid and branches and then reassessing the applicability of clipping and refining the 2D grid. Furthermore, a combination of clipping and refining might also be suitable to accelerate a detailed 1D2D hydrodynamic model without compromising accuracy. This can be tested in the future.

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A Elaboration on research methods

This appendix contains more detailed information on the research methods. A.1 shows the characteristics of the surrogates that are run. A.2 shows the components of the Kling-Gupta Efficiency.

A.1 Simulated surrogate models

Table 11 shows the characteristics of all surrogate models that are run. All surrogate models are run for the event of July 2014. SUG #C2, #O1, #M1 and #M2 were also run for the event of September 2018 (*Figure 6*) to see if similar results were found.

Table 11 - Overview of all surrogates. In yellow are the changes with respect to the benchmark model.

Name	Distance 1D nodes [m]	Bridges removed	Culverts merged	2D resolution [m]	Maximum Courant number	User time step [s]	2D grid clipped	2D grid refinement [m]
Optimised benchmark	50	No	No	100	0.7	3600	No	-
SUG #d1	100	No	No	100	0.7	3600	No	-
SUG #d2	250	No	No	100	0.7	3600	No	-
SUG #d3	500	No	No	100	0.7	3600	No	-
SUG #d4	50	Yes	No	100	0.7	3600	No	-
SUG #d5	50	No	Yes	100	0.7	3600	No	-
SUG #D1	50	No	No	250	0.7	3600	No	-
SUG #D2	50	No	No	500	0.7	3600	No	-
SUG #C1	50	No	No	100	1	3600	No	-
SUG #C2	50	No	No	100	5	3600	No	-
SUG #C3	50	No	No	100	10	3600	No	-
SUG #C4	50	No	No	100	50	3600	No	-
SUG #O1	50	No	No	100	0.7	3600	Yes	-
SUG #O2	50	No	No	100	0.7	7200	No	-
SUG #O3	50	No	No	200	0.7	3600	No	100
SUG #M1	50	Yes	Yes	100	5	3600	No	-
SUG #M2	75	No	No	150	0.7	3600	No	-
SUG #M3	75	Yes	Yes	100	5	7200	No	-
SUG #M4	75	Yes	Yes	100	0.7	3600	No	-

A.2 Components Kling-Gupta Efficiency

The Kling-Gupta Efficiency (KGE) indicates the model performance in relation to observations or a benchmark model. The KGE is composed of a term scoring the correlation, a term indicating the bias and a term that compares the variability in the surrogate and benchmark model. The score can vary from minus infinity to 1 and is best if 1 (Gupta et al., 2009; Kling et al., 2012). The KGE can be calculated in each calculation point with *Eq. 10 - Eq. 13.* The overall KGE can be calculated by taking the average of the KGE values in each calculation point.

KGE	Kling-Gupta Efficiency	$1-\sqrt{(r-1)^2+(\alpha-1)^2+(\beta-1)^2}$	*Eq. 10*
r	Pearson correlation coefficient	$\frac{\sum_{i=1}^{n}(y_{s,i}-\overline{y_s})(y_{b,i}-\overline{y_b})}{\sqrt{\sum_{i=1}^{n}(y_{s,i}-\overline{y_s})^2}\sqrt{\sum_{i=1}^{n}(y_{b,i}-\overline{y_b})^2}}$	*Eq. 11*
α	Variability ratio	$\frac{\sigma_s}{\sigma_b}$	*Eq. 12*
β	Bias ratio	$\frac{\overline{y_s}}{\overline{y_b}}$	*Eq. 13*

Where n is the total number of time steps with a water level measurement for the given calculation point, $y_{s,i}$ is the water level predicted by the surrogate at time step i, $y_{b,i}$ is the water level predicted by the benchmark at time step i, $\overline{y_b}$ is the average water level in the benchmark, $\overline{y_s}$ is the average water level in the surrogate, σ_s is the standard deviation in the water level of the surrogate, and σ_b is the standard deviation in the water level of the benchmark.

B Detailed overview of numerical instabilities

Table 12 shows the numerical instabilities that were encountered, their root cause, the dimensions of the model they are related to, and the implemented solutions.

Table 12 - Detailed overview of numerical instabilities found in the original model of the case study.

#	Numerical instability	Model dimensions	Direct cause high computation time	Root cause	Implemented solution
1	The bed level of some waterways was equal to the control water level. This resulted in water depths of 0 meters at the first time step.	1D	Water depth of 0 meter	Incorrect data	The bed level for waterways is estimated 0.8 meters below the control water level if the bed level and control water level are equal in the input data.
2	Some areas with a seasonal (summer and winter) control water level had only one of the two control water levels specified. As a result, some waterways started empty.	1D/2D	Water depth of 0 meter	Incomplete data	If only a summer or winter control level was specified in the data, the missing control water level was set to equal the known control water level.
3	Some areas did not have a fixed control water level specified, resulting in empty waterways at the start of the simulation	1D/2D	Water depth of 0 meter	Incomplete data	HDSR has provided the missing data after a request.
4	The elevation map of the 2D grid cells contained values of -10 mNAP for cells with invalid elevation measurements. This created deep pools that filled with water from the 1D network.	2D	Large NumLimdt	Incorrect data	The elevation on the 2D grid cells was set to -5 mNAP for invalid values, and the grid resolution of the 2D grid was set to 100 m to average out these invalid values. All to reduce the water that will flow from the 1D network to these cells
5	For all culverts without data on the elevation, it is assumed in D-HyDAMO that the bottom elevation is 0.4 m under the control water level. However, some culverts act as water level-separating structures. In this case, the elevation of the culvert must be equal to the highest of the two control water levels between which it is the separating structure. With the default assumption, the areas with the highest control water level can drain via the culvert.	1D	High flow velocities, Large NumLimdt, Unrealistic water level drops	Incomplete data and incorrect model set-up	For culverts that act as a water level-separating structure, the elevation is set equal to the control water level if no data is available. For normal culverts without data, the assumption that the inflow elevation is 40 cm under the control water level is kept.

#	Numerical instability	Model dimensions	Direct cause high computation time	Root cause	Implemented solution
6	The control water level for some weirs was not present in the data. As a result, the weirs were always open.	1D	High flow velocities, Large NumLimdt, Unrealistic water level drops	Incomplete data	The control water level for weirs without data was derived from the map containing all control water levels.
7	The water depth on the 2D grid between the retaining structures was initially set to 0 m, while water is present here. As a result, the 1D network was filling the 2D grid between the retaining structures during the first time steps.	2D	Large NumLimdt, Water depths of 0 m	Incorrect model set-up	A new input map for the water depth on the 2D grid was generated where the cells between the retaining elements have an initial water level equal to the control water level.
8	Automatic weirs steered in the wrong direction. When the water had to be retained, the automatic weirs lowered their crest level, increasing the discharge.	1D	Water depth of 0 meter	Incorrect model set-up	Change of the automatic weir parameters (the K_p parameter was set from -0.03 to 1)
9	The control water level for the sluices was determined incorrectly if the sluice point objects in the raw data were not exactly on the border of the two control water level areas.	1D	High flow velocities	Incorrect model set-up	The raw data of the sluices was changed from point objects to line objects that all intersect the two relevant control water level areas, which is required to set the correct control water level for the sluices.
10	Pump stations snapped to the wrong branch when constructing the model, causing open channels between areas with different control water levels. This drained the areas with a higher control water level.	1D	High flow velocities, Unrealistic water level changes	Incorrect model set-up	Pump stations were relocated in the input data via QGIS to snap on the correct branch when running the D-HyDAMO script.
11	Some fish passages are several meters long but are represented in the model as a point object on a branch. It is assumed that the flow through the fish passages is zero. When the fish passage intersects with another branch, only one side of the passage is closed as the point object is placed on one side of the intersection only. The other side remains open and water can incorrectly flow through the fish passage.	1D	Water depth of 0 meters	Incorrect model set-up	When fish passages intersect other branches, the fish passage is split in two in the raw data. When constructing the model, one point element on both sides of the intersection is added to close both sides.

#	Numerical instability	Model dimensions	Direct cause high computation time	Root cause	Implemented solution
12	Waterways and culverts that crossed above each other (without an open connection) were sometimes constructed in the model as open connections, allowing the water to flow from one to the other.	1D	Large NumLimdt, Unrealistic water level changes	Incorrect model set-up	The D-HyDAMO script that constructs the branches from the raw data was adjusted such that it does not connect waterways if a culvert stretches along the intersection.
13	Some pumps were steering on the wrong control water level. The water level on which a pump should steer is determined with the help of a downstream and upstream observation point. However, some of the observation points were located in the wrong control water level area, which translates to an incorrect pump operation.	1D	Large NumLimdt, Water depths of 0 m	Incorrect model set-up	A file with new locations of the observation points was created in which the observation points are located in the correct control water level areas.
14	Some parallel branches were incorrectly snapped to each other resulting in flow around pump stations, draining areas with a higher control water level.	1D	Unrealistic water level drops	Incorrect model set-up	The script that snaps branches to each other was adjusted by a colleague of HydroLogic.
15	Some weirs were snapped on the wrong branch when constructing the model or were steering on the wrong control water level.	1D	Large NumLimdt, Water depths of 0 m, High flow velocities	Incorrect model set-up	A csv file is created for the incorrect weirs. The csv file describes either to which branch the weir should snap or on which control water level it should steer. The csv file is read by the D-HyDAMO script to ensure the correct model construction.
16	Sluices could not discharge water because the bed elevation at the start and end of the sluice was set to the default of 0 m NAP in D-HYDRO since it is not possible in the GUI to define these values. However, the sluices are located below 0 m NAP, such that a bed level of 0 m NAP blocks the opening of the sluices completely, resulting in high water levels.	1D	Large NumLimdt, Unrealistic water level increases	D-HYDRO GUI limitation	The correct bed level parameters are manually added to the structures.ini file which describes all parameters for the structures.
17	The water level of one branch was seen dropping while both sides of the branch were closed off by two switched-off pumps. After a couple of hours in the simulation, the water level dropped, while the discharge through the pumps remained 0. This is unexpected and the cause of this behaviour could not be found.	1D	Water depths of 0 m	Unexpected behaviour of D-HYDRO	-

C Results second precipitation event

This appendix contains the results of the second precipitation event in *Table 13*.

Table 13 - Results of the surrogate modelling for the event of September 2018. Yellow cells indicate changes with respect to the optimised benchmark. Green cells indicate if the indicator is within the acceptable limits, see Table 5.

Event September 2018	Optimised benchmark	SUG #C2	SUG #O1	SUG #M1	SUG #M2
Distance 1D nodes [m]	50	50	50	50	75
Bridges removed	No	No	No	Yes	No
Culverts merged	No	No	No	Yes	No
Spatial resolution 2D grid [m]	100	100	100	100	150
Maximum Courant number	0.7	5	0.7	5	0.7
2D grid clipped	No	No	Yes	No	No
User timestep [s]	3600	3600	3600	3600	3600
Resolution refinement 2D grid [m]	-	-	-	-	-
Results computation time					
Computation time [min]	1559	133	1166	107	1569
Reduction computation time w.r.t. optimised benchmark [%]	-	91.5	25.2	93.1	-0.6
Average timestep [s]	1.5	27.3	1.9	29.9	1.1
Results accuracy	**Accepted if**				
MAE avg. water depth 1D [m]	< 0.05	0.005	0.075	0.017	0.065
KGE water depth 1D	> 0.75	0.94	0.54	0.74	0.64
MAE max. water depth 1D [m]	< 0.05	0.004	0.076	0.019	0.069
MAE avg. water depth 2D [m]	< 0.05	0.001	0.150	0.015	0.130
KGE water depth 2D	> 0.75	0.99	0.64	0.85	0.35
MAE max. water depth 2D [m]	< 0.05	0.002	0.174	0.014	0.153
Precision flood extent	> 0.7	1.00	0.80	0.99	0.58
Recall flood extent	> 0.7	1.00	0.72	0.94	0.79
F1-score flood extent	> 0.7	1.00	0.76	0.97	0.67

D Additional visualizations

This appendix contains additional figures that compare the performance of the surrogates for the primary event of July 2024. These figures are partly enlargements of *Figure 9* to *Figure 12*.

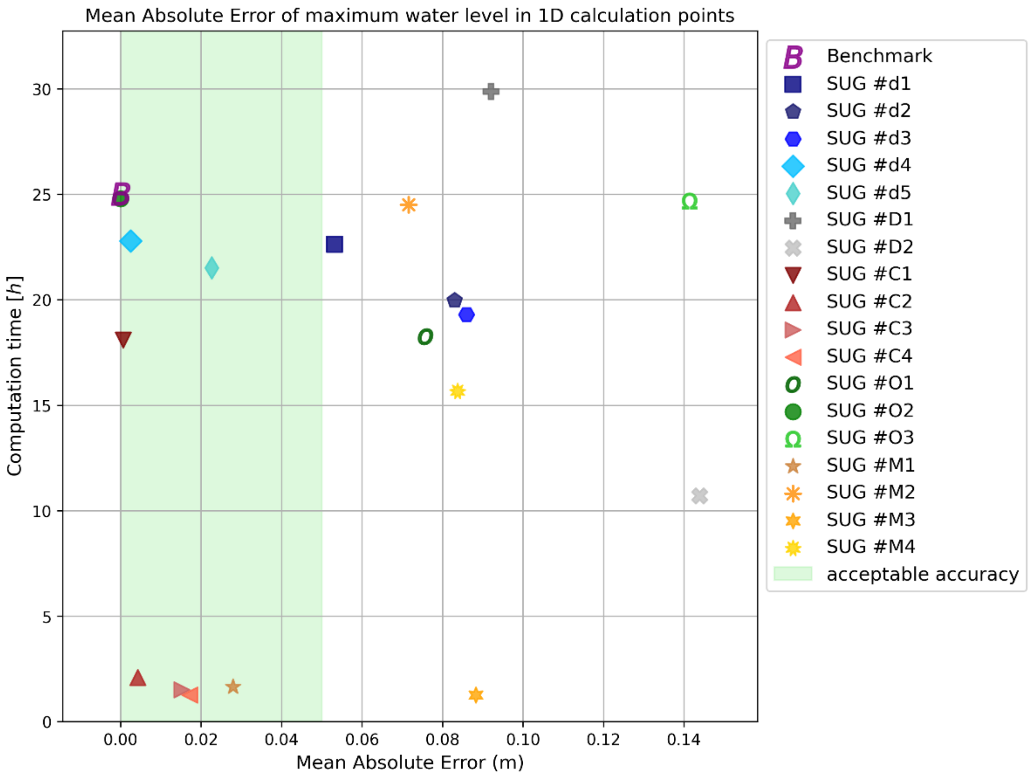

Figure 19 – MAE of maximum water level in 1D calculation points.

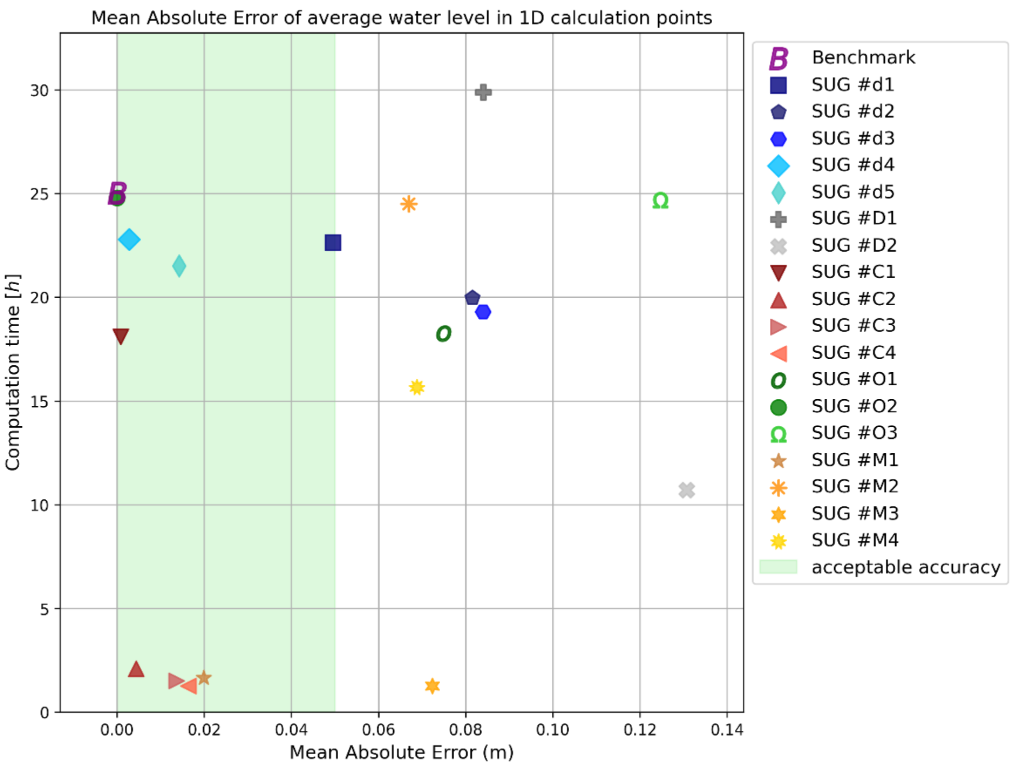

Figure 20 – MAE of average water level in 1D calculation points.

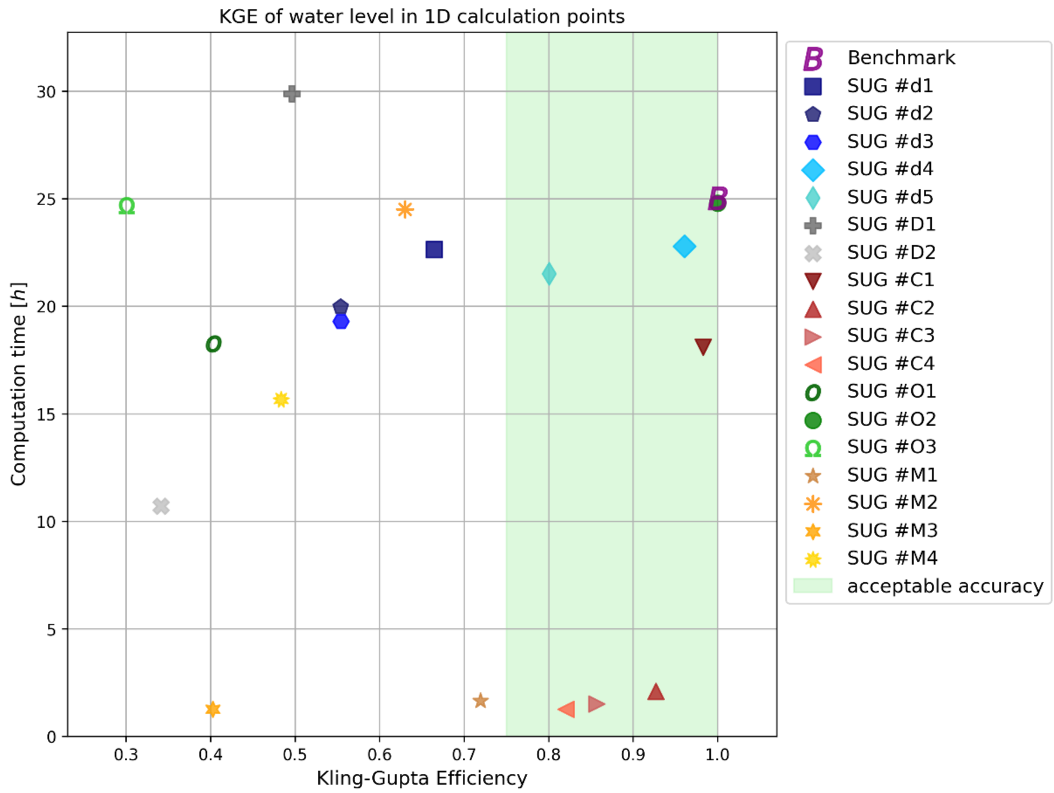

Figure 21 – KGE of water level time series in the 1D calculation points.

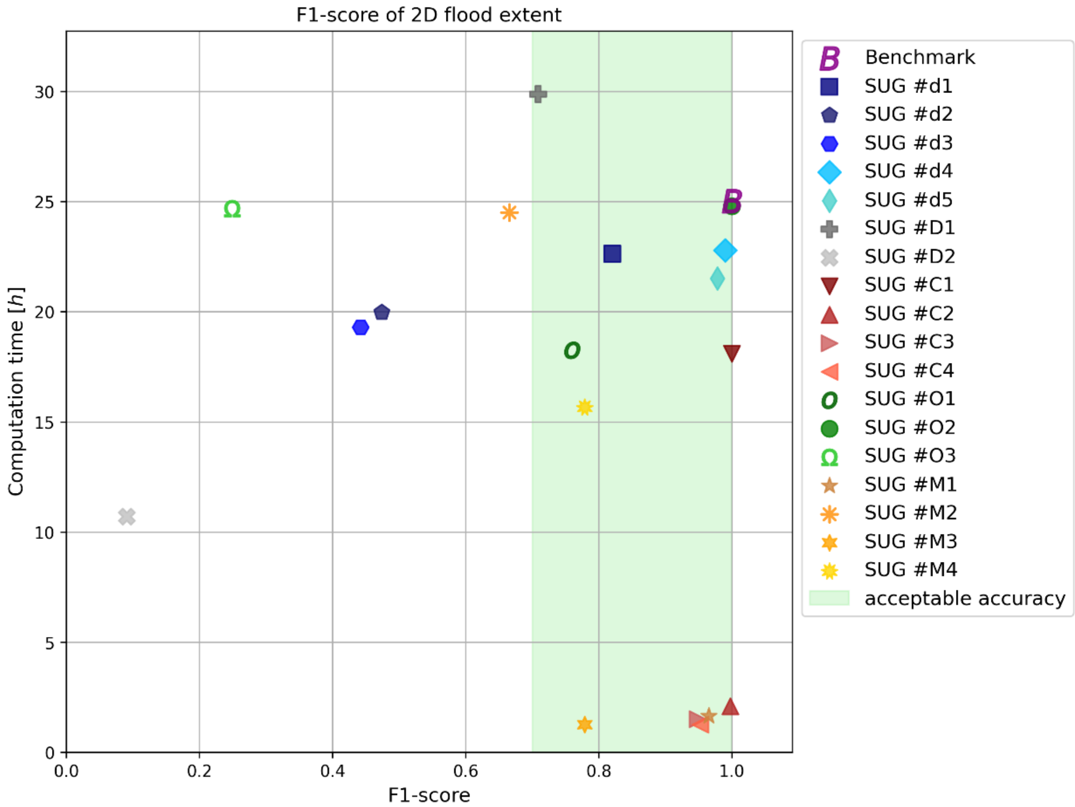

Figure 22 – F1-score of the flood extent.

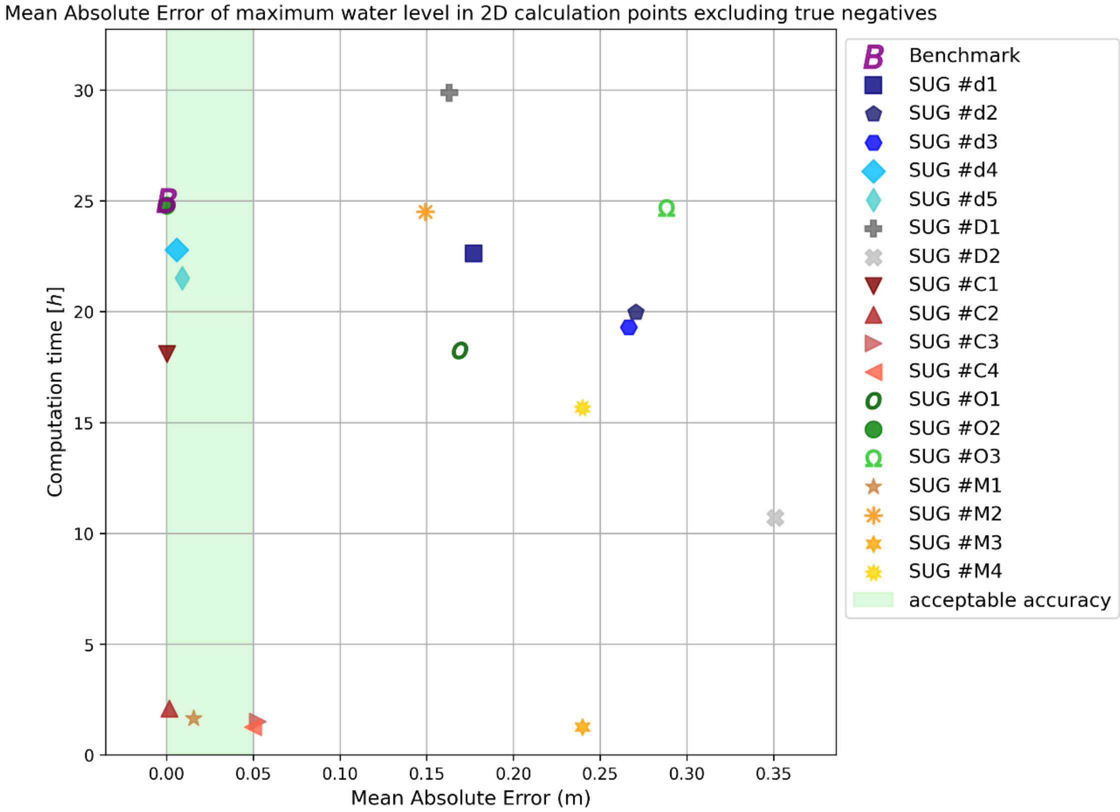

Figure 23 – MAE of maximum water level in the 2D calculation points without the true negative cells.

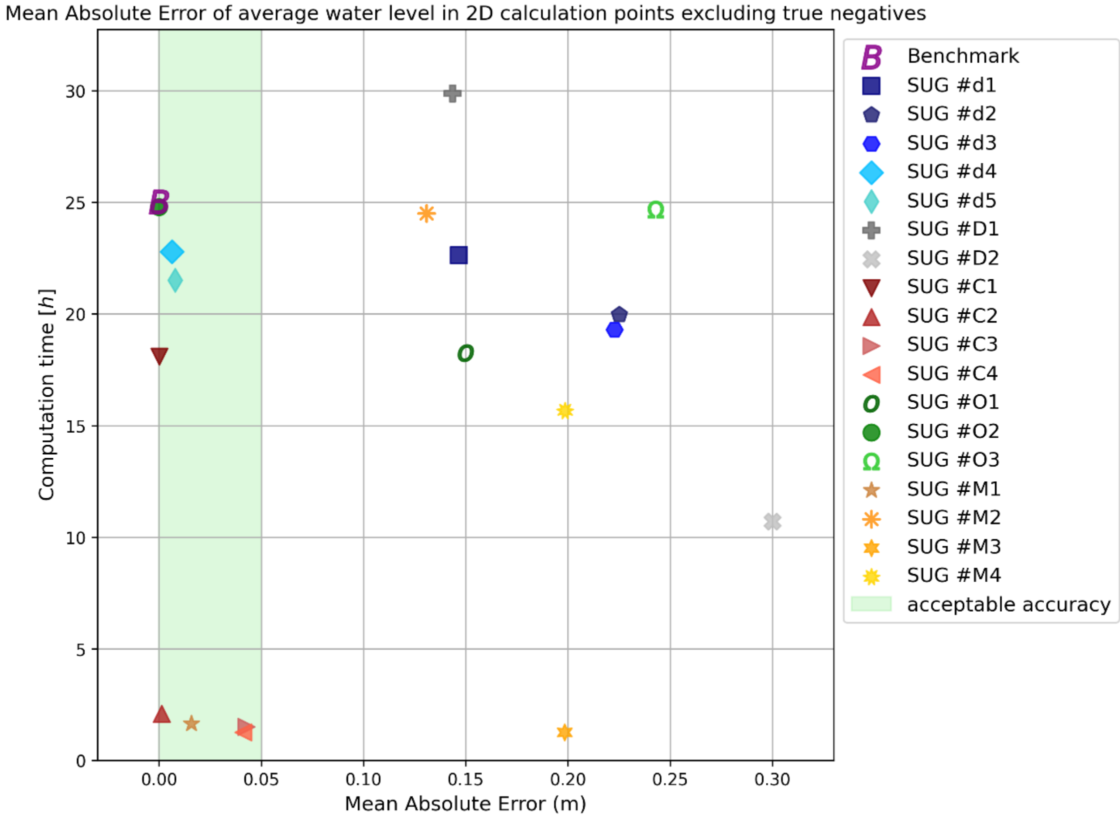

Figure 24 – MAE of average water level in the 2D calculation points without the true negative cells.

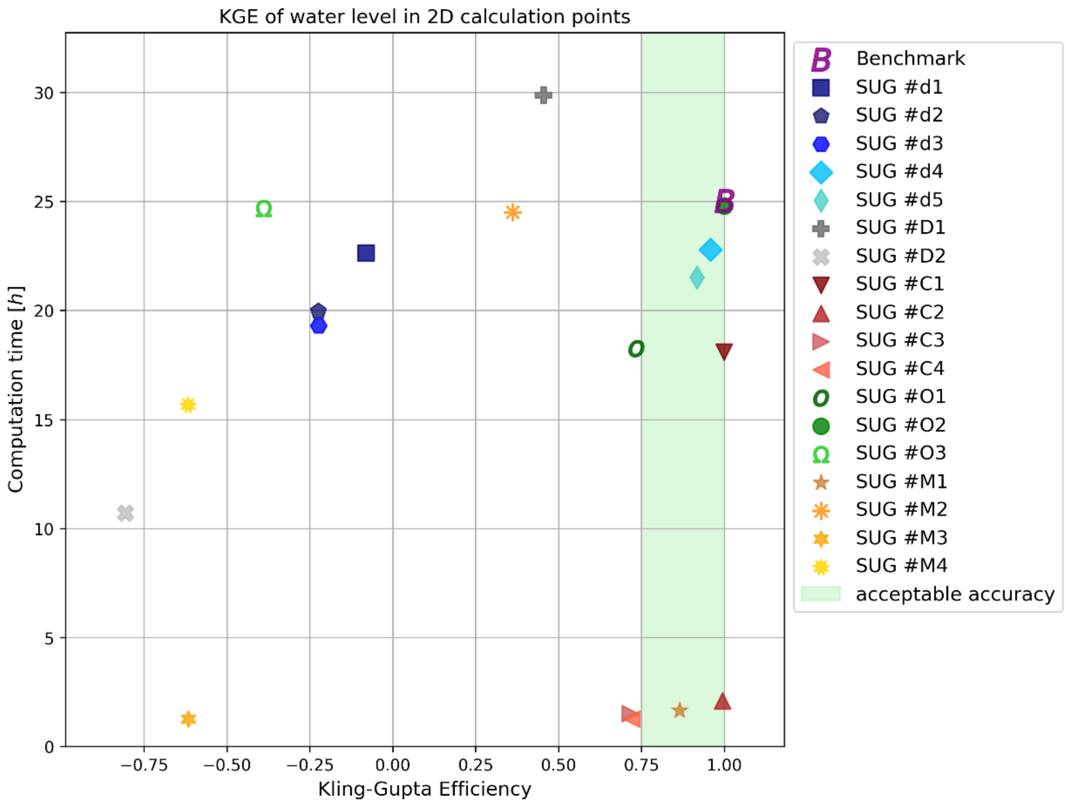

Figure 25 – KGE of water level time series in the 2D calculation points without the true negative cells.