Eindevaluatie pilot Langsdammen in de Waal

Delft3D simulations

Client

Rijkswaterstaat – WVL

Rijkswaterstaat
Ministerie van Infrastructuur en Waterstaat

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Eindrapport

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Summary

Het riviersysteem van de Rijn, met daarin alle Nederlandse Rijntakken, kent problemen met onder meer hoogwaterveiligheid, insnijding van de zomerbedbodem, daling van laagwaterstanden en grondwaterstanden, de kwaliteit van het rivierecosysteem, en het gebruik van de rivier als vaarweg. Om deze problemen integraal aan te pakken, heeft Rijkswaterstaat Oost-Nederland een idee gelanceerd onder de werknaam WaalSamen. Dit is een plan voor herinrichting van het zomerbed in de gehele Waal. De herinrichting wijzigt het principe van het bestaande normalisatiesysteem door het zomerbed te verdelen in twee parallelle stroomgeulen, gescheiden door een langsdam. Om de eigenschappen van deze systeemwijziging in de praktijk te beproeven is over een lengte van tien kilometer de pilot Langsdammen uitgevoerd: het Waaltraject Wamel-Ophemert (km 911.5-921.5) bij Tiel.

Voor, tijdens en na de aanleg van de langsdammen is een uitgebreid monitorings- en onderzoeksprogramma uitgevoerd door de partners van de samenwerkingsovereenkomst 'WaalSamen'. Dit programma is afgesloten met een integrale eindevaluatie, onderverdeeld in 12 inhoudelijke deelprojecten (werkpakketten). Voor u ligt het deelrapport van WP1 dat gericht is op numerieke simulaties met Delft3D; de resultaten van de numerieke simulaties zijn gebruikt in andere werkpakketten.

This report is part of the project "Final Evaluation Pilot Longitudinal Training Walls" (Work Package 1, WP1). It concerns numerical simulations with a state-of-the-art Delft3D-model covering the Waal between Nijmegen and Zaltbommel. The impact of the Longitudinal Training Walls (LTW's) on the river (water levels, flow velocities, morphology, etc.) depends on the geometry and properties of the LTW's, the bank channels behind them and geometry of the openings. The discharge distribution between the main channel and the bank channel is mainly controlled by the geometry of intake sills at the upstream side of each LTW. Therefore, numerical simulations are performed with a broad range of hydrodynamic conditions and design variants of the LTW's, openings and bank channel. The results of the simulations are used within the other work packages of the overall project to carry out various analyses.

At first, we updated the hydrodynamic model and evaluated its performance in a verification step. The model was found to be sufficiently accurate in capturing the observed water levels, which deems it more accurate in the intended comparative analysis, in which direct comparisons between variants are made to evaluate their effectiveness or their impact on the river.

Based on an inventory of the analysis requirements of the other WP's, we made a setup for a wide range of simulations, which included a number of design variations. The boundary conditions ranged from the very low flow

(800 m^3/s at Lobith) to extreme discharge (18.000 m^3/s at Lobith). The design variations distinguish between base variants and optimisation variants. The base variants are the situation without LTW's (V0) and the situation with LTW's and fully opened intake sills (V1) and fully closed intake sills (V2). For the optimisation variants (Vopt_01 to Vopt_08), the geometry and properties of the LTW's, bank channel and openings are varied with respect to base variant V1.

The hydraulic and morphodynamic simulations of the base variants are used by WP2, WP3, WP6, WP7, W9 and WP10 to study the effect of LTW's and intake sill configuration on various river functions. For the optimisation variants, only hydraulic simulations are performed. These are used to analyse the effects on high water safety (WP6) and to evaluate whether the local design can be optimised (WP2) and LTW's can be implemented elsewhere (WP3).

Morphodynamic simulations are performed for 20 years for a selection of base variants (V0, V1 and V2). Two sets of simulations were carried out, with and without maintenance dredging, in order to generate a broad range of results, for better understanding of the morphological response to LTWs. The accuracy of the model was checked to be sufficient to conduct the intended comparative analysis; and improved by executing a model initialisation step.

After finishing the WP1-Delft3D-simulations in Summer 2020, "Work Package 0" (WP0) performed extensive data-analysis based on field measurements of water level, velocity and bed level. WP1 and WP0 provide separate input of results and data to other work-packages to perform their analysis. Nevertheless, we performed a global consistency check between WP0 and WP1. There is sufficient agreement between model results and measurements in terms of the effect of LTW's (V1) with respect to the situation with groynes still present (V0); both in velocity/water level change as well as in bed level change.

Both the hydrodynamic and the morphodynamic model results give insight into the function, performance and impact of the different design variants of the LTW's. In this report we only carried out the most basic analysis and provided the results and model limitations and assumptions for the other work packages for their intended detailed assessment. Together with the detailed data-analysis of WP0, WP1 model results provide sufficient material for the other work packages of the project to carry out the intended detailed analysis.

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1 Introduction

1.1 Background

Het riviersysteem van de Rijn, met daarin alle Nederlandse Rijntakken, kent problemen met onder meer hoogwaterveiligheid, insnijding van de zomerbedbodem, daling van laagwaterstanden en grondwaterstanden, de kwaliteit van het rivierecosysteem, en het gebruik van de rivier als vaarweg. De laatste decennia wordt onderkend dat de sectorale aanpak niet efficiënt is. De beleidsdirecties van het ministerie van Infrastructuur en Waterstaat hebben de wens uitgesproken voor een meer innovatieve systeem- en gebiedsgerichte aanpak, met integrale aandacht voor alle probleemvelden tegelijk. Deze integrale aanpak beoogt de som van alle problemen te reduceren in plaats van slechts de problemen van een beperkt aantal sectoren.

Voor deze integrale aanpak heeft Rijkswaterstaat Oost-Nederland een idee gelanceerd onder de werknaam WaalSamen. Dit is een plan voor herinrichting van het zomerbed in de gehele Waal. De herinrichting wijzigt het principe van het bestaande normalisatiesysteem door het zomerbed te verdelen in twee parallelle stroomgeulen, gescheiden door een langsdam. Om de eigenschappen van deze systeemwijziging in de praktijk te beproeven is over een lengte van tien kilometer de pilot Langsdammen uitgevoerd. Het doel daarvan is een proof of concept, om meer zekerheid te verkrijgen over de integrale werking en de potenties van een dergelijke systeemwijziging.

Voor de pilot werd het Waaltraject Wamel-Ophemert (km 911.5-921.5) bij Tiel gekozen. Om redenen van efficiëntie werd de pilot tegelijk uitgevoerd met Fase III van het project Kribverlaging Waal van het programma Ruimte voor de Rivier. Hiervoor leverde Rijkswaterstaat Oost-Nederland op 30 juni 2011 de producten van een SNIP-3-besluit op aan de Programmadirectie Ruimte voor de Rivier van Rijkswaterstaat, inclusief een omwisselbesluit om geplande kribverlaging te vervangen door langsdammen. De Staatssecretaris van Verkeer en Waterstaat bekrachtigde dit eind 2011. De langsdammen tussen Wamel en Ophemert werden vervolgens in de periode van augustus 2014 tot maart 2016 gerealiseerd.

Voor, tijdens en na de aanleg van de langsdammen is een uitgebreid monitorings- en onderzoeksprogramma uitgevoerd door de partners van de samenwerkingsovereenkomst 'WaalSamen'. Dit programma is afgesloten met een integrale eindevaluatie, onderverdeeld in 12 inhoudelijke deelprojecten die worden aangeduid met "WP" (werkpakket). Voor u ligt het deelrapport van WP1 over het onderdeel van de evaluatie van het tweegeulensysteem met langsdammen dat gericht is op numerieke simulaties met Delft3D; de resultaten van de numerieke simulaties zijn gebruikt in andere werkpakketten. De deelrapporten vormen de ondergrond van het *hoofdrapport, maar de inzichten en conclusies zijn bij het opstellen van dat hoofdrapport integraler beschouwd, verder geëvolueerd en verduidelijkt. Waar dat mogelijk tot verschillen heeft geleid, zijn de conclusies van het hoofdrapport leidend.*

The remainder of this report is in English.

1.2 Structure of this report

The specifications of the longitudinal training walls (LTW's), including their openings, are crucial for a good understanding of the set-up of the model (variants) and simulation settings. Therefore, we discuss some details of the LTW design in paragraph 1.3. Paragraph 1.4 gives a brief description of the Delft3D (4)-model that we use in this study, which is based on the model developed by Omer et al (2019b). The scope of this Work package WP1 is explained in paragraph 1.5.

The Delft3D-simulations (WP1) are input for WP2, WP3, WP6, WP7, WP8, W9, WP10. Based on consultation with these WP's we made a simulation plan with a list of required simulations to answer relevant questions from the WP's (Paarlberg et al, 2020). Tailoring the set of simulations to the needs of the various WP's was a continuous activity and resulted in the set of simulations presented in this report.

In WP1, there is a clear distinction between hydrodynamic simulations and morphodynamic simulations. The reason for that distinction is twofold: (1) not every question requires a morphodynamic simulation, and (2) morphodynamic simulations take a lot of computational effort, which puts pressure on fulfilling the project's tight schedule.

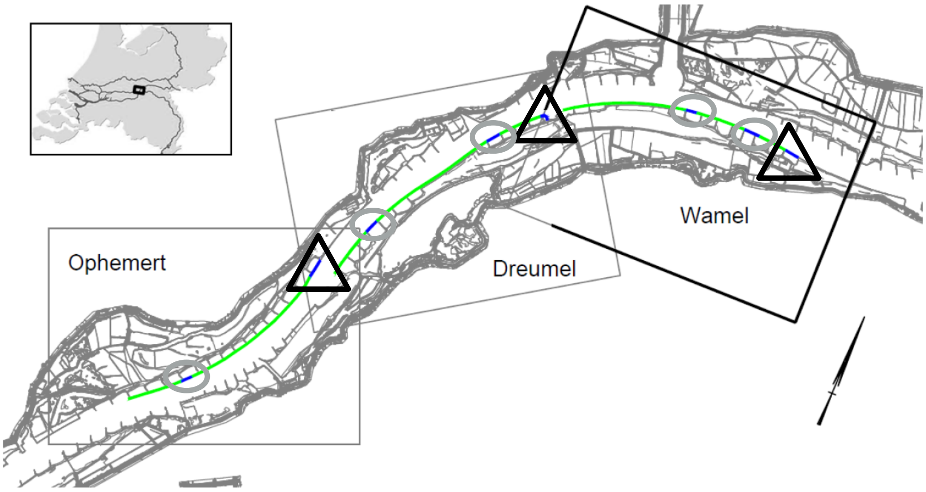
In our analysis, we distinguish between the so-called base variants ('hoekpunten') and optimisation variants. The base variants represent the situation without LTW's and basic LTW designs with either fully opened or fully closed intakes at the upstream side of each LTW. For these base variants both hydrodynamic and morphodynamic simulations are performed, which are used by all WP's. The WP's dealing with the optimisation of the LTW design (WP2), possible application elsewhere (WP3), and functions flood safety (WP6), nature (WP8) and fresh water (WP9) mainly use the results of hydrodynamic simulations. The WP's navigation (WP7) and maintenance (WP10) mainly use results of morphodynamic simulations supported with some hydrodynamic simulations.

The hydrodynamic simulations are treated in chapter 2 and the morphodynamic simulations in chapter 3. In these chapters we discuss how the variants are implemented and which model settings are used. Some concluding remarks are given in chapter 4.

1.3 Some specifications of the LTW’s

The layout of the LTW’s in the Waal near Tiel is shown in Figure 1. There are three dams: Wamel, Dreumel and Ophemert. The height of the LTW’s with respect to OLR (= water level for 1020 m^3/s at Lobith) slowly decreases in downstream direction; Wamel 2,78 – 2,66 m; Dreumel 2,65 – 2,44 m and Ophemert 2,52 – 2,35 m (note that these numbers need to be added to the OLR (2012)-level to get the height of the dam).

Figure 1
Layout of the LTW’s.



Each dam has a primary intake opening at the upstream side (black triangles in Figure 1) and either 1 (Ophemert) or 2 intermediate openings (Wamel and Dreumel) (grey ovals in Figure 1). The impact of the dams on hydraulics and morphology is mainly controlled by the distribution of water and sediment between main channel and bank channel. Therefore, the upstream intake sill of each LTW is *flexible*. Basically, this means that the height and width of the intake sills can be altered (within certain limits) to influence the distribution of water and sediment between the channels.

The crest elevation of the dams, intermediate openings and the intake sills is illustrated in Figure 2. The figure also includes a typical bed level in the main channel of 2018 (pmap) and the water level at various (Lobith) discharges. Each dam has a sill at the intake which is below OLR (= water level for 1020 m^3/s at Lobith) and intermediate openings having a crest level which is 1,15 m lower than the crest of the LTW’s (OLR+1,25 m).

The intake sills were constructed in 2015/2016 with fully opened intakes, i.e., the crest level of the sills is at OLR-1,75 m (Sieben, 2020). Note that this doesn’t mean that the sill is at the same level as the main channel, since it has rock foundation on top of which the sill can be raised.

The topography of the dams and intake sills constructed in the field is illustrated in Figure 3. The topography of the main channel, flood plain and bank channel represents the situation in 2018 (Baseline-j18). Within this Baseline-database, the intake sills are fully opened, i.e., only the rock foundation is present.

Figure 2
Elevation of dams and sills in Baseline rijn-j18_5-v1 in black. Black triangles are intake sills (below OLR), grey ovals are intermediate openings” which are at approx. Qlobith=1800 m^3/s. The crest level of the LTW’s is at approx. Qlobith=3000 m^3/s. The water levels indicated in the figure are taken from “betrekkingslijnen 2018”. The numbers in the legend are Lobith discharges.

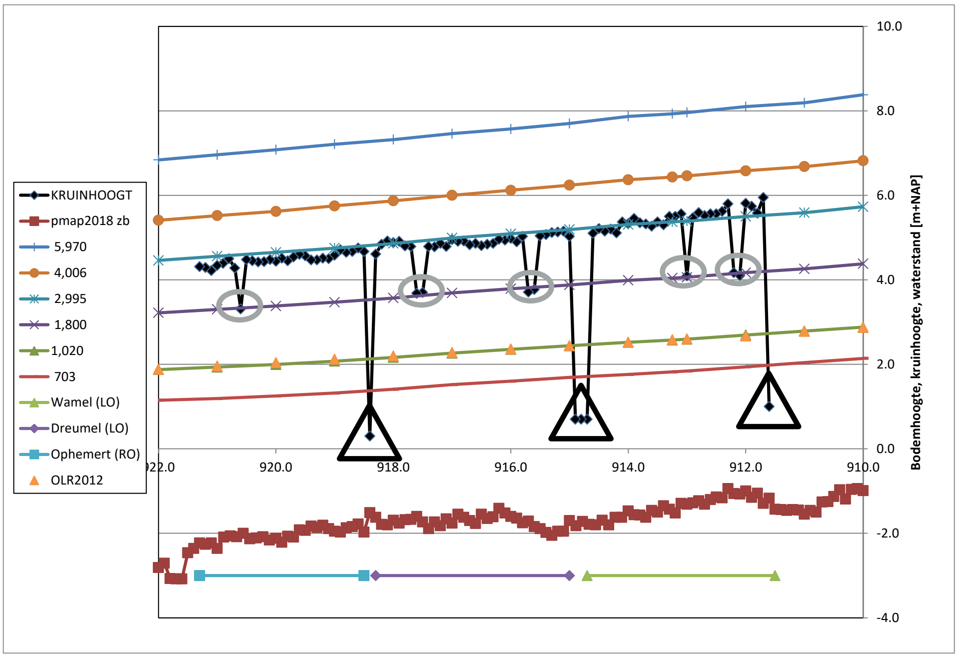
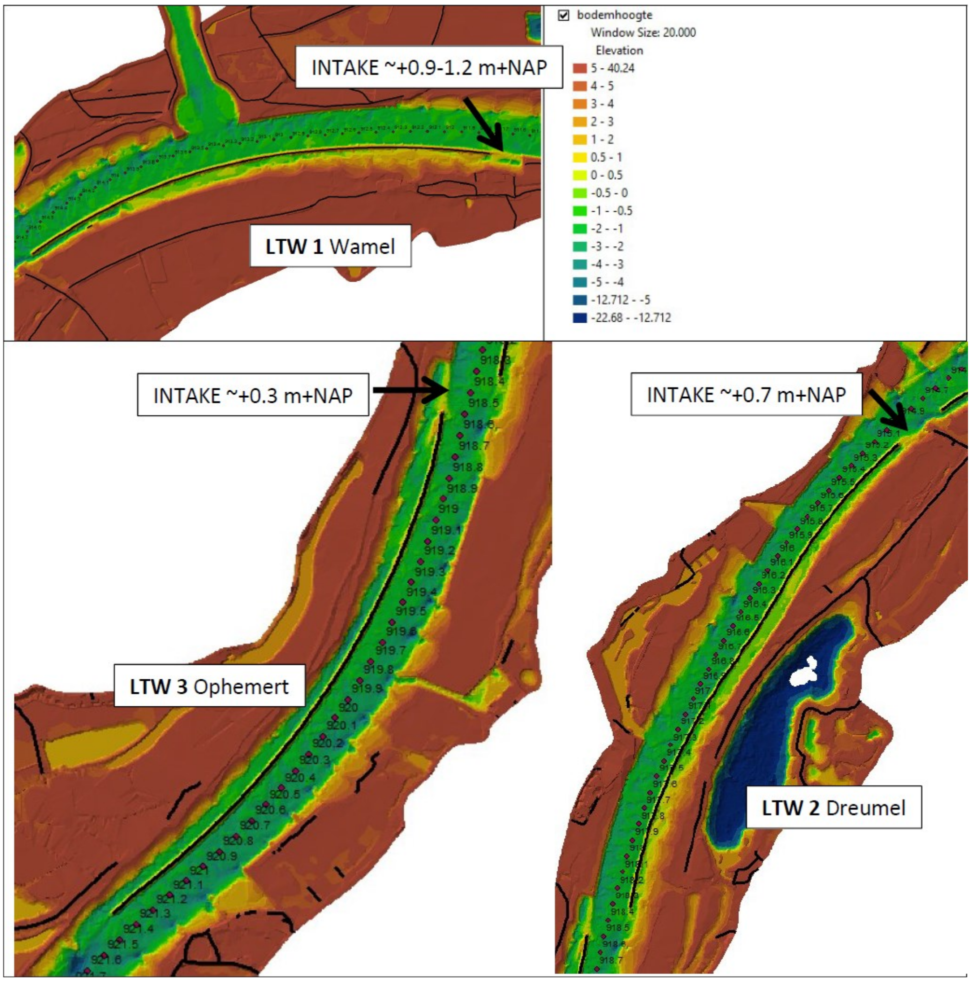


Figure 3
Maps of LTW’s with bed level (m+NAP) on background (Baseline-j18).



1.4 Delft3D-model Nijmegen-Zaltbommel with LTW's

Omer et al (2019a, b) developed a 2D-dimensional depth-averaged Delft3D-4-model covering the area of the Longitudinal Training Walls (LTW's) in the Waal. This Delft3D (4)-model is used as a starting point in this project. The model domain length is 47 km, starting downstream of Nijmegen (~km 888) until Zaltbommel (~km 934,7), see Figure 4.

The model is based on the widely-used 'DVR-model' (DVR = "Duurzame Vaardiepte Rijndelta") which is considered the state-of-the-art model to determine the morphological impacts of river interventions, and is set-up with the greatest care, building on about 15 years of application to the Dutch Rhine. The model contains the LTW's and bank channels, based on Baseline-j18, i.e., the configuration as described in paragraph 1.3.

To correctly schematise the cross-sectional shape of the LTW's, the configuration of sills and bank channel geometry, the grid is locally refined (see Omer et al., 2019b) in the area of the LTW's (in both streamwise and crosswise direction). This refinement resulted in a resolution of approximately 20 m (streamwise) x 5 m (cross-wise) in the vicinity of the LTW's. Using *local* refinement where needed, instead of *global* refinement as done by Huthoff et al (2011), the computational times for morphodynamic simulations are kept acceptable.

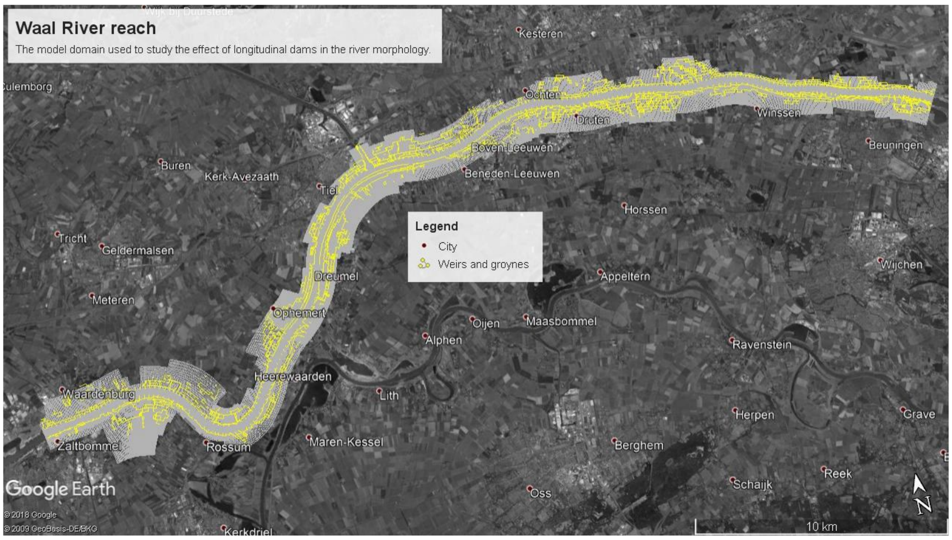


Figure 4
Overview of model domain.

1.5 Scope of report and analysis

Scope of model application: relative effects

The Delft3D-model used in this study (paragraph 1.4) is set-up with the greatest care, building on about 15 years of application to the Dutch Rhine branches. Local grid refinement has been applied to correctly capture the LTW's, sills and bank channel in the schematisation (paragraph 1.4). However, as with all models, simplifications or assumptions have to be made because certain processes are not (correctly) captured by the model (equations or resolution). Think of applying non-alluvial floodplains, not including bank erosion of bank channels, uncertainty in sediment transport over sills and assuming bedload and suspended load transport of bed material only (see also paragraph 3.3.2). This introduces uncertainties, which increase when the model is used in an absolute manner. Therefore, it is common to use the model for comparative analysis to isolate the relative impact of interventions by comparing hydraulic (water levels, discharge) and morphological (bed levels) development with that of a reference simulation. As such, the model results of WP1 are meant to help understanding the relative effect of various configurations of LTW/channel-designs.

It is known that large-scale sedimentation and erosion patterns such as bars and large-scale bed changes are captured well by the DVR-models. This does not hold for small-scale features like bed forms and their associated roughness since the grid resolution does not capture this scale. Also dredging and dumping volumes are known to be rather sensitive to specific dredging criteria (OLR reference plane, dune height, tolerance) and should be treated with care. Specifically, for the LTW's it should be noted that the process of sediment transport over the intake sills into the bank channels is not captured properly by the model. Even in the field this is a very uncertain process. Therefore, in this project we focus on differences between variants, rather than absolute results. Additionally, and for a better understanding of the morphological impacts, simulations are performed both with and without the dredging and dumping module activated.

Scope of model application: considered variants

The base model (Omer et al., 2019a, b) is built from Baseline-j18, and more or less represents the as-built situation (see paragraph 2.3) with fully opened intake sills. We call this variant 'V1'. For the hydrodynamic analysis this is used as a reference case. Variant 'V2' is a fictive situation with the crest of the intake sills at the same level as the LTW's. This sill level is even higher than the maximum design height of the sills to investigate the possible band width of effects. Variant 'V3' is a situation with partly closed intake sill and only simulated hydrodynamically.

For some WP's, the effect relative to the situation without LTW's has to be considered. Therefore, we constructed a variant 'V0' which is the situation prior to the construction of LTW's with groynes still present. Furthermore, for WP2, a range of 'optimisation variants' is developed which are run hydrodynamically.

Scope of analysis in WP1

The model results are intended to support the analysis that is carried out by other WP's. The results are used and intensively analysed by other work-packages, see e.g., the following reports:

- WP0: De Jong, J., V. Chavarrías & W. Ottevanger (2021), Eindevaluatie pilot Langsdammen in de Waal; Hydromorphological data and observations. Rapport Deltares, 11204644, Delft, september 2021.

- WP2: Zuijderwijk, W.M. & J. de Jong (2021), Eindevaluatie pilot Langsdammen in de Waal; Optimalisatie. Rapport Witteveen+Bos & Deltares, 117743/20-006.749 (Wi+Bo), 11204644 (Deltares), Delft, september 2021.
- WP6: Asselman, N. & P. de Grave (2021), Eindevaluatie pilot Langsdammen in de Waal; Functie Hoogwaterveiligheid. Rapport Deltares, 11204644, Delft, september 2021.
- WP7: Van der Mark, R. & R. van der Wijk (2021), Eindevaluatie pilot Langsdammen in de Waal; Functie Vaarweg. Rapport Deltares, 11204644, Delft, september 2021.
- WP10: Chavarrías, V., C.J. Sloff & E. Mosselman (2021), Eindevaluatie pilot Langsdammen in de Waal; Morphology and maintenance. Rapport Deltares, 11204644, Delft, september 2021.

This WP1-report is intended to show the used Delft3D-model, considered variants, simulation set-up (chapters 2 and 3) and relevant model limitations and/or assumptions (chapter 4). The report includes some basic analysis to support modelling choices and assumptions and to check whether results are plausible. As such, this report does *not* include detailed analysis and/or interpretation of results.

The calculation results help to understand the effect of several design changes of the LTW's on hydraulics and morphology. We looked at relative effects as requested by other work packages. Hydromorphological data from WP0 and computational results from WP1 form separate sources of information for application in other work packages. Results from both sources are integrated and interpreted within those work packages.

Use of (field) data

Table 1 gives a brief summary of used (field) data in this project. This concerns both specific data to set-up the model (variants), as well as data to perform a global verification of the model of Omer et al. (2019a, b). Model geometry (bed level, roughness, weirs) was taken directly from various Baseline-databases. Bed level measurements of the main channel from the pilot were not used, since we adopted a morphological "spinned-up" bed level (paragraph 3.4).

The aim of WP1 is to determine relative effects using scenario-analysis as requested by other WP's, rather than predicting absolute effects of certain field situations. This is because (1) absolute values of modelled bed levels are known to be uncertain and (2) WP0 performs an extensive analysis of field data. This project (WP1) started with a global verification of the Delft3D-model of Omer et al (2019a, b) for the area of interest based on V1, by comparing simulated water levels and discharge distribution between main channel and bank channel (paragraph 2.3.2). Detailed comparison with field data (e.g., bed level and flow velocity measurements) is outside the scope of WP1 and is done in WP0. The data from Table 1 is sufficient to set-up our model (variants) and to verify that our model is fit-for-purpose considering relative effects.

Table 1
Brief overview of data used in this project to set-up Delft3D models and perform a global verification of the model results.

Data source:	**Used to:**
Various Baseline databases	Used to construct geometry of various variants considered
Baseline measures of RWS-ON with raised/changed sills (weirs) at intakes of bank channels	
Delft3D model including earlier model runs (Omer et al, 2019a, b)	Since this model was extensively tested and reviewed this model is used as basis for the Delft3D model settings in this study
Delft3D model schematisation ‘delft3d_4_dvr-rijn-2015-v1’	Used for various model settings (settings morphology, including dredging/dumping)
Various data sources for Lobith / Waal discharge distribution: • Spruyt & Asselman (2017) – Discharge distribution for Qlobith>= 6000 m^3/s • Discharge distributions from calibrated WAQUA and SOBEK model for j19 (RWS-ON) • Stuwprogramma 2016 (pers. comm. Max Schropp, RWS-WVL).	Used to determine the hydraulic boundary conditions of the model (translate Lobith discharge to Waal discharge)
As-built bank channel topography	Used to schematise the bed level of the bank channel in morphodynamic simulations
Webviewer langsdammen (RWS-ON)	Underlying data such as bank channel contour and bank line used to schematise the bank channel in the morphological simulations
Betrekkingslijnen 2018	Used for global model verification on water levels and determination of hydraulic boundary conditions
“Overzicht afvoermetingen 2016-2019.pdf” (Sieben, 2020)	Used to (1) translate Lobith discharge to Waal discharge and (2) do a global verification on modelled discharge through bank channels
OLR2012	Schematisation of optimisation variants (hydrodynamics only). Note that for the field design OLR/OLW-2002 has been used.
Data-analysis from WP0	Global consistency check on model results

The work packages WP6, WP7, WP8, WP9 and WP10 draw conclusions on flood levels, navigability, nature and freshwater supply by combined evaluation, analysis and interpretation of outcomes from WP0 (detailed analysis of hydro- and morphological measurements) and scenario results from WP1 (Delft3D-simulations). Hence this WP1 report does not give a detailed comparison using field data. So, within WP1, no detailed comparisons were made to e.g., measured 2D flow patterns and/or measured bed levels (8 weekly multi-beam measurements). Also, a comparison with WAQUA is outside the scope of this study. Absolute water levels are known to have several decimetres of inaccuracy in both WAQUA and Delft3D. The significance of model outcomes lies in capturing the difference in water level or bed level between different variants of the Delft3D-model. From other studies we know that water level differences show good agreement between WAQUA and Delft3D.

We did perform a global consistency check by performing a rough comparison between WP0 and WP1 (see chapter 4). If needed, a more detailed comparison can be made/reported within the WP’s where combined information from WP0 and WP1 is used in the analysis.

2 Hydrodynamic simulations

2.1 Introduction

This chapter gives an overview of the set-up of the hydrodynamic simulations performed with the Delft3D model. The boundary conditions are treated in paragraph 2.2. Paragraph 2.3 and 2.4 describe the base variants and optimisation variants, respectively.

2.2 Boundary conditions

2.2.1 Discharge at Nijmegen

To cover all questions for the WP's, we consider a wide range of discharge conditions (Table 2). Specific aspects, such as flood safety or cross-currents, are usually based on certain predefined Lobith discharges (RWS, 2018). Therefore, the set of discharges is specified in terms of Lobith discharges.

Table 2
Considered steady discharges for hydrodynamic analysis.

STEP	Qlobith [m^3/s]	Qwaal [m^3/s]	Qlobith -> Qwaal based on
1	800	536	Sieben (2020)
2	1020	796	
3	1500	1242	
4	2500	1825	
5	4000	2920	
6	6000	4112	Spruyt & Asselman (2016)
7	8000	5420	
8	10000	6497	
9	12000	7742	
10	16000	10166	
11	18000	11729	

The low discharge is needed for (at least) WP9 (water supply), the extreme discharges for (mainly) WP6 (flood safety). The Lobith discharge 1020 m^3/s is the so-called "Overeengekomen Lage Rivierafvoer". The water level (OLR) belonging to this discharge is important for navigation and is important for the dredging operations within the morphodynamic simulations. Therefore, this discharge is also included in the set of discharges. Discharge step 3 (1500 m^3/s) is below the level of the intermediate openings, step 4 (2500 m^3/s) below the crest and step 5 (4000 m^3/s) well above the crest of the current crest level of the LTW's. Steps 5-8 are mainly used in WP2 (optimisation of LTW design) and WP7 (navigability) e.g., for analysis of cross-currents ("dwarsstromingen").

Since the model starts at Nijmegen, we need to translate the Lobith discharges to the Waal discharges, for which numerous relationships exist. Figure 5 and Figure 6 show some of these relationships:

- Omer et al (2019b), which originates from the DVR-model that is used in morphological studies and is defined for Qlobith 1020-8592 m^3/s.
- Spruyt & Asselman (2017) – Qlobith >= 6000 m^3/s (used in flood risk studies).
- Sieben (2020) – Qlobith < 5500 m^3/s (based on measurements in pilot area). This relationship is shown in Figure 7.
- Data RWS-ON: discharge distributions from calibrated WAQUA and SOBEK model for j19 (pers. comm. Daniël van Putten, RWS). SOBEK is specifically calibrated on low water.
- Stuwprogramma 2016 (pers. comm. Max Schropp, RWS-WVL).

For Boven-Rijn discharges at Lobith larger than 6.000 m^3/s the relationships generally agree. Therefore, we used the relationship of Spruyt & Asselman (2017) to translate Lobith discharges to Waal discharges (Table 2).

For medium discharge, the relationships differ a lot. Sieben (2020) reports that direct measurement in the project area deviates from the LMW (= "Landelijk Meetnet Water") values on corresponding days. The directly measured Waal discharge is higher than the calculated discharge, from the Qf-relations, which is based on LMW-data, see Figure 7. Since the relationship of Sieben (2020) is based on *direct* measurements, we use his relationship to derive Waal discharge values in the low to medium discharge regime. The resulting Waal discharges are given in Table 2.

Figure 5
Various discharge distributions between discharge of Boven-Rijn at Lobith and Waal.

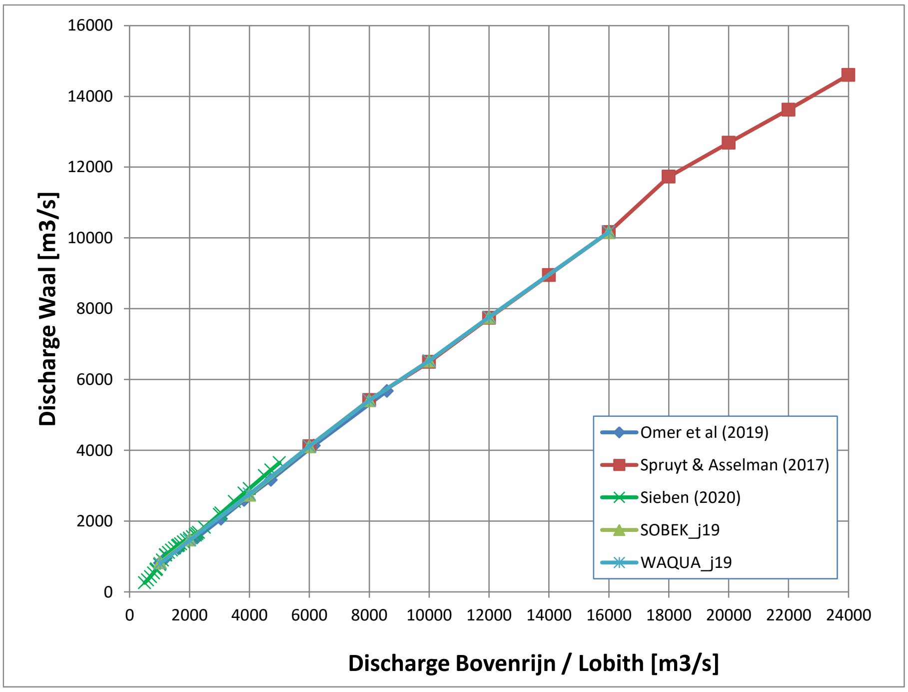


Figure 6
Various discharge distributions between discharge of Boven-Rijn at Lobith and Waal in the range of low to medium discharges.

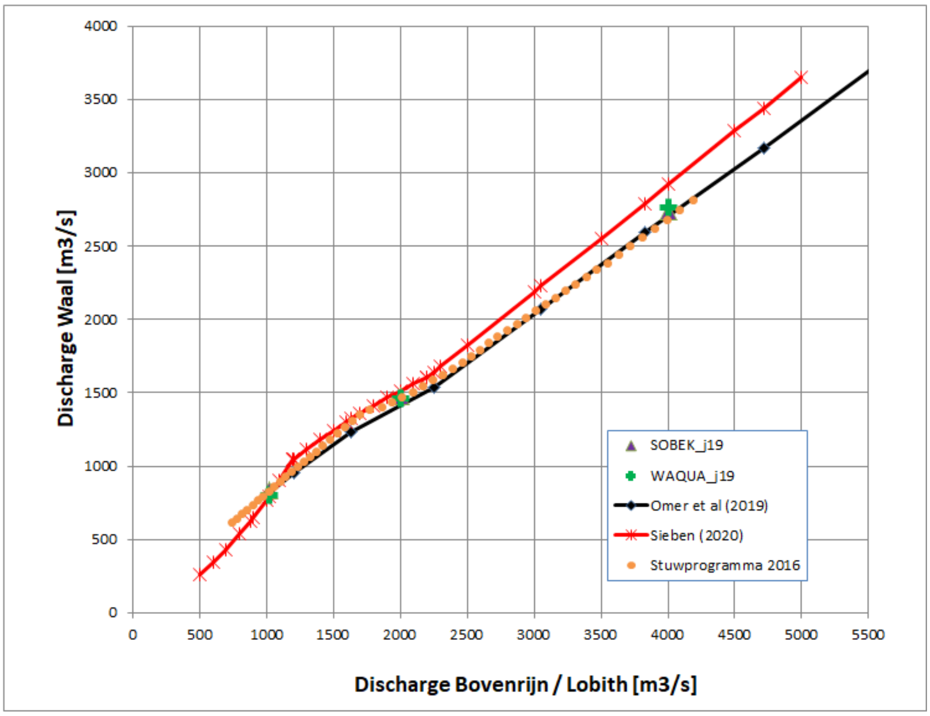
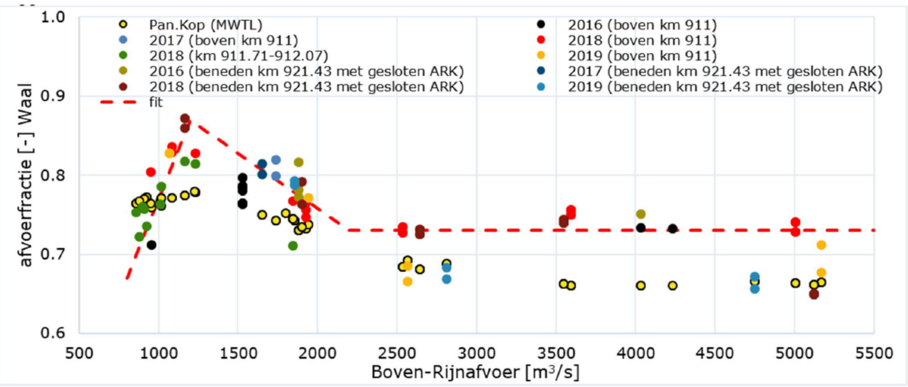


Figure 7
Discharge fraction of Lobith discharge to Waal, from Sieben (2020). Black outlined markers are MWTL-data. Others are direct measurements.

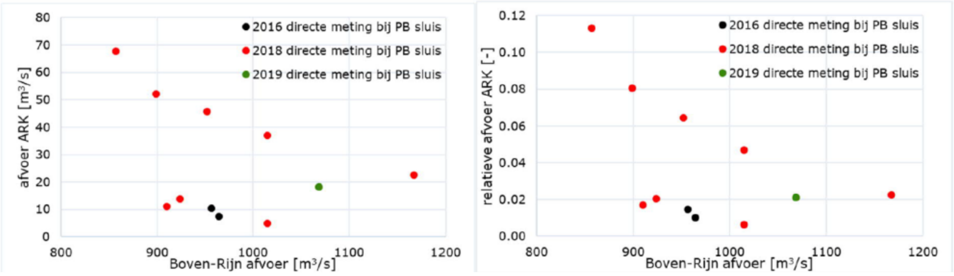


2.2.2 Discharge Amsterdam-Rijnkanaal (ARK)

At low river discharge, water can be extracted from the Waal through the Amsterdam-Rijnkanaal (ARK) for fresh water supply to other water systems (Van der Vat, 2021). Note that at low discharge, the water level in the Waal and ARK are equal, meaning that the water level in the Waal is no controlling parameter for the discharge to the ARK. Sieben (2020) found that in the period of 2016-2019 the discharge to the ARK was significant (Figure 8) for Lobith discharge up to 1200 m^3/s. At present, when the water level at Tiel (km 913) is lower than approximately +3,20 m+NAP (Q Lobith below ~1300 m^3/s), a significant amount of water is extracted through the Bernard sluice towards the river Lek (fresh water supply).

The main focus of WP9 is to know whether LTW's mitigate negative effects of water extraction through the ARK by setting up water levels to ensure safe navigation. This question can be answered based on the simulations with low discharge without discharge extraction to the ARK. In the morphodynamic simulations, discharge extraction to the ARK is not included since the lowest modelled discharge in the hydrograph is a Lobith-discharge of 1203 m^3/s (apart from the OLR-computation), see paragraph 3.3.1.

Figure 8
Overview absolute (left) and relative (right) discharge toward ARK (measurements), from Sieben (2020).

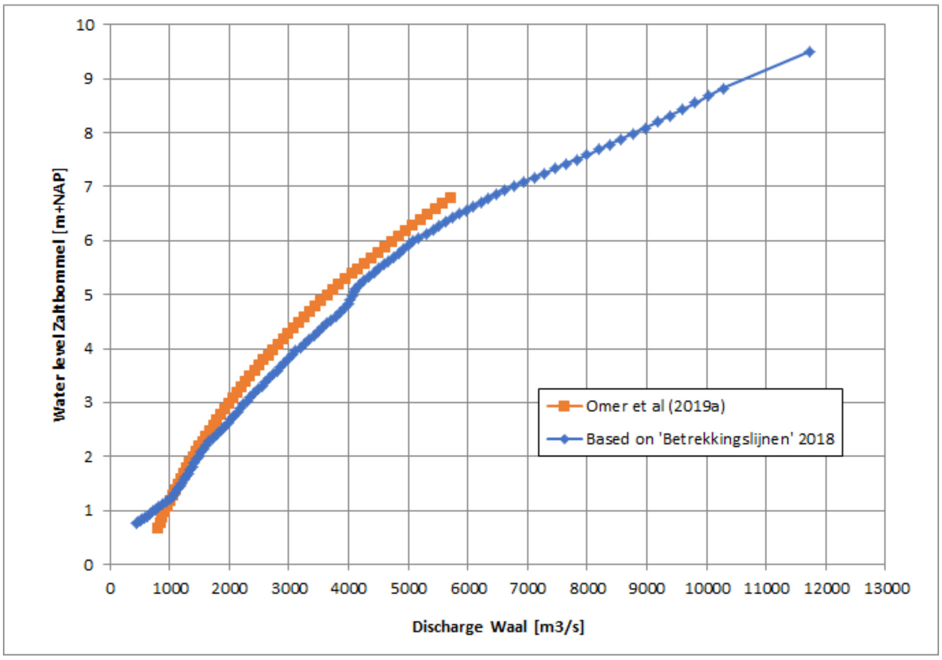


2.2.3 Q-h relations at Zaltbommel

So-called “Betrekkingslijnen” give the relation between Lobith water levels and water levels along the Rhine branches. The Betrekkingslijnen are updated every two years, based on discharge and water level measurements as much as possible and model simulations if needed. Zaltbommel is one of the LMW-stations used to construct the Betrekkingslijnen, which means that the water level at our downstream boundary is directly useable. However, the Betrekkingslijnen relate a local water level to the discharge at Lobith. Using the relationships from paragraph 2.2.1 (Qlobith->Qwaal), the water level at Zaltbommel is specified as a function of the Waal discharge. The resulting Qh-relationship at Zaltbommel is presented in Figure 9. Our Qh-relationship differs from the one used by Omer et al. (2019a). That is because the latter used LMW discharge measurements to construct the relationship, resulting in lower Waal discharges for the same water level. For low discharges, the deviations in Figure 9 could be related to discharge extraction to the ARK (paragraph 2.2.2).

Note that there is quite some tidal variation at the downstream boundary at Zaltbommel. However, in the area of interest (at Tiel) the (tidal) water level fluctuations are only marginal, even at low discharge (see Figure 36 in paragraph 3.3.1). Therefore, we use a fixed water level for a certain discharge as downstream boundary in both hydrodynamic and morphodynamic simulations.

Figure 9
Q-h relationship at Zaltbommel.



2.3 Base variants

2.3.1 Schematisation of variants

The following base variants (‘hoekpunten’) are considered for the hydrodynamic simulations:

- V1 = situation 2018.
    - LTW’s from Baseline rijn-j18_5-v1 (BL-j18) (bed level + sill)
    - Bed level at upstream intake sills taken from BL-j18 (which is representative for the as-built situation), i.e., fully opened intakes.
    - No 2D weirs at upstream intake sills, since bed level at sills is ‘smooth’ (see paragraph 2.3.1 and paragraph 3.3.4).
    - Outside the area of LTW’s we have lowered groynes as present in BL-j18.
    - The two lowered and lengthened groynes just downstream of the Ophemert-LTW (between kmr 921.6 en 922.0) are included.
    - For the bank channel geometry, we use the topography as stored in Baseline-j18 (*so, not the as-built situation*). With respect to the as-built situation there has been quite some sedimentation (see also paragraph 3.3.4).
    - The side channel ‘Passewaaij’ is included in the model geometry, but the culvert at the upstream side is not (see chapter 4).

- V0 = V1 with 2014 geometry in the area of LTW's, meaning:
  - No LTW's present.
  - Original (non-lowered) groynes in area of LTW's (BL-j14) (note that this is different from Huthoff et al (2011), where planned lowered groynes were considered the reference case).
  - Lowered groynes outside this area (BL-j18).
- V2 = V1 with fully closed openings by using a 2D weir.
  - The height of the weir is taken the same as the connecting LTW
  - Note that this is a fictive case, considering a higher sill crest elevation than designed (the maximum design height is OLR/OLW-2002 +1,25 m).
  - The *bed level is not changed* at the location of the weir (for morphodynamic simulations this is done, see paragraph 3.2).
  - Intermediate openings in the LTW's are unchanged.

Note that both V0 and V2 are 'fictive' variants, and only intended for the intended scenario analysis in the relevant WP's.

In addition to these base variants for hydrodynamic simulations, we also considered the situation with partially closed intakes as constructed in May 2018: V3. This variant is considered to estimate the effect of partially closing intakes on hydraulics (WP2). Note that no morphodynamic simulation is performed for variant V3, and that also in V3 the intermediate openings are unchanged with respect to V1.

V1 schematisation

V1 (2018) is the reference case for optimisation of the design. Therefore, a global verification of the model results has been performed for V1 (see next paragraph). The bed level, weirs, roughness and thin dams for V1 are obtained from Baseline rijn-j18_5-v1 using the grid of Omer et al (2019b). Bed levels are projected from cell corner to cell centres by taking the mean of four surrounding grid points. This enables to easily implement optimisations/changes for the bed level. The resulting bed level and weirs for the pilot area are shown in Figure 10.

Figure 10
Bed level and location of 2D-weirs (white lines) for V1.

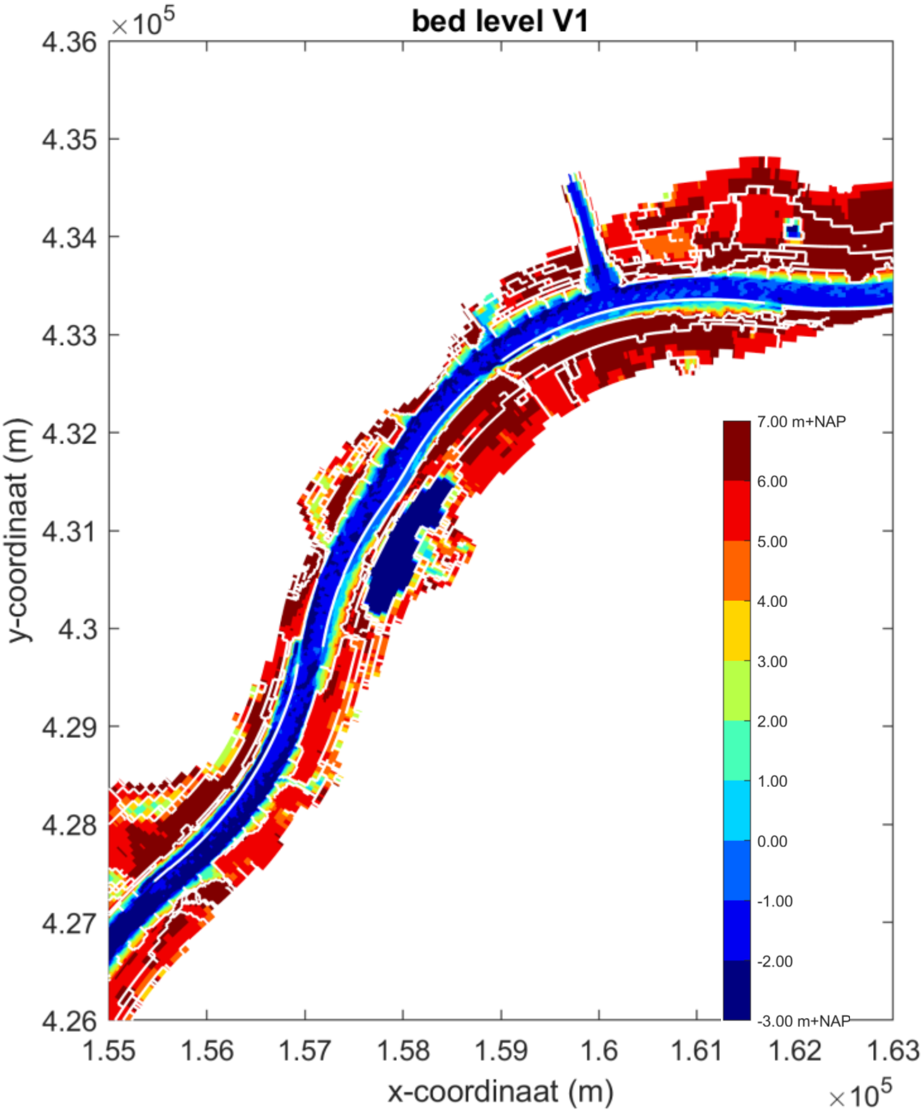


Figure 11 shows the bed level and location of 2D-weirs at the intake of the Wamel LTW for V1 (i.e., BL-j18). A cross-section of the bed level at the LTW is given in Figure 12. From these figures we observe for V1 that:

- At the intake sill no weirs are schematised;
- The LTW is schematised as a combination of a raised bed level and a weir on top of that.

For the intake sills and LTW’s at Dreumel and Ophemert the same schematisation method is used. The reason for not including weirs at the intake sills is that the bed level gradients are ‘smooth’, i.e., the slopes do not exceed the general rule of thumb of 1:7 in horizontal direction (see also paragraph 3.3.4).

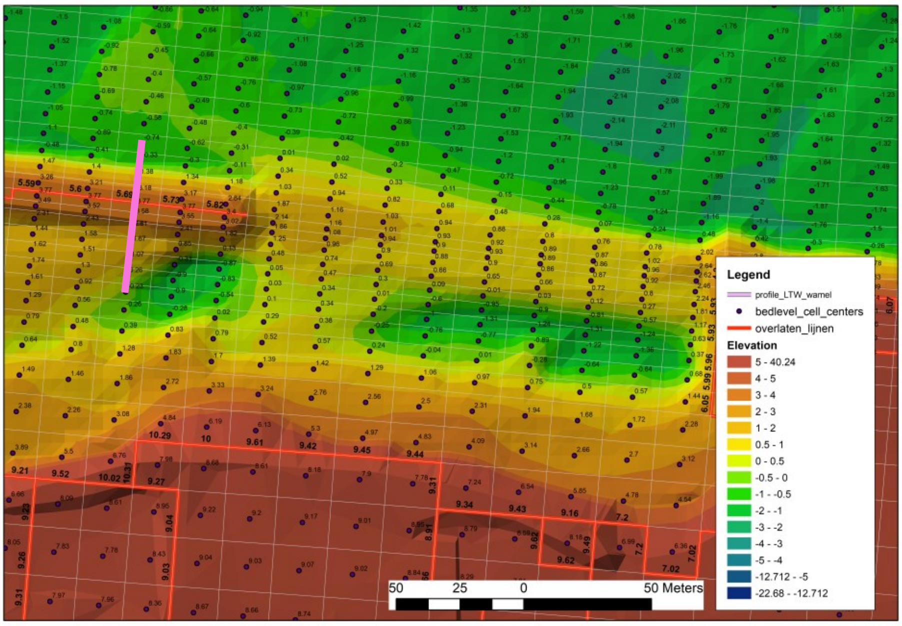


Figure 11
Detail of V1 intake schematisation of Wamel bank channel. The bed elevation at the pink line over the dam is given in Figure 12. Red lines are 2D-weirs.

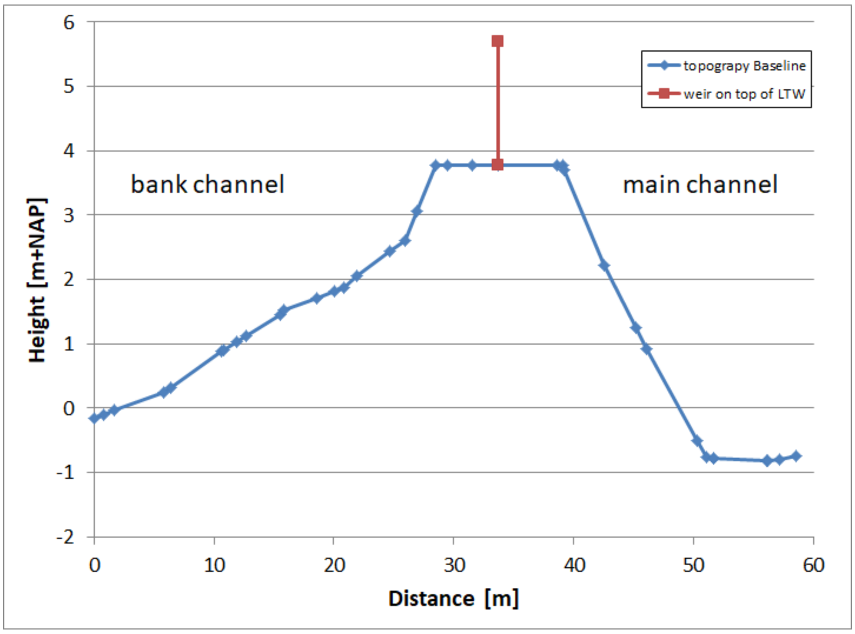


Figure 12
Schematisation LTW in Baseline (for location of profile, see Figure 11).

V0 schematisation

For some work packages (WP6, WP7, WP9), the difference of V1 (LTW's with fully opened upstream intakes) with the situation without LTW's (V0) needs to be considered. To this end, the model schematisation near the longitudinal dams is 'reset' to the situation prior to dam construction. This is done by replacing Delft3D-input within a certain 'polygon' by using input coming from Baseline rijn-j14_5-v1.

The areas where the data are replaced are shown in Figure 13 and the resulting topography in Figure 14. Also, roughness codes within these polygons are replaced by j14-data in the aru/arv Delft3D input files.

Note that only the first groyne directly downstream of the Ophemert-LTW is set back to the original (shorter) length and (greater) height; the second groyne indicated with an arrow in Figure 13 is (accidentally) not replaced. Since the first groyne is most important in steering the flow towards the main channel, no effect on the morphological results is expected.

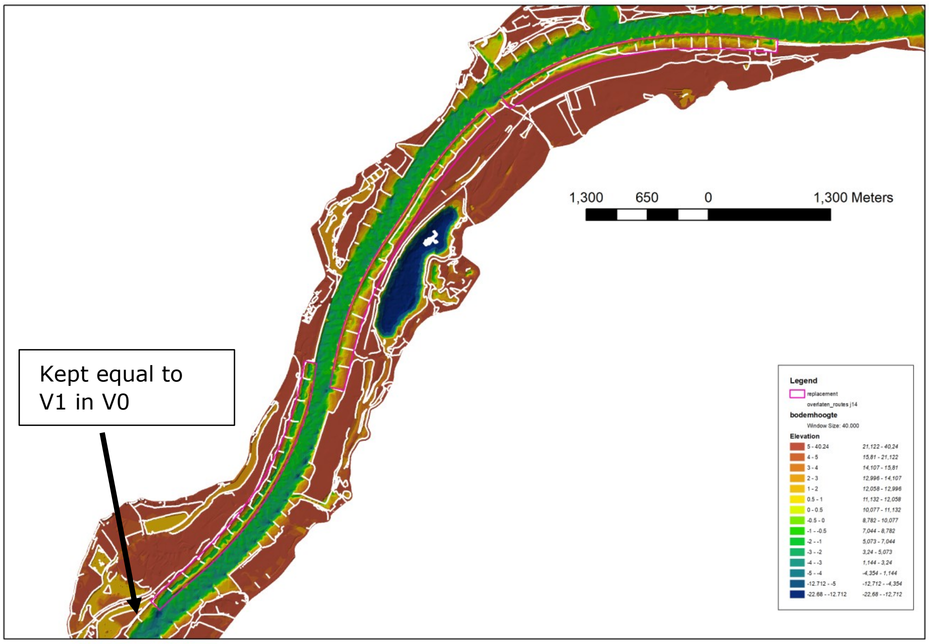


Figure 13
Area of replacement V1 data to give V0.

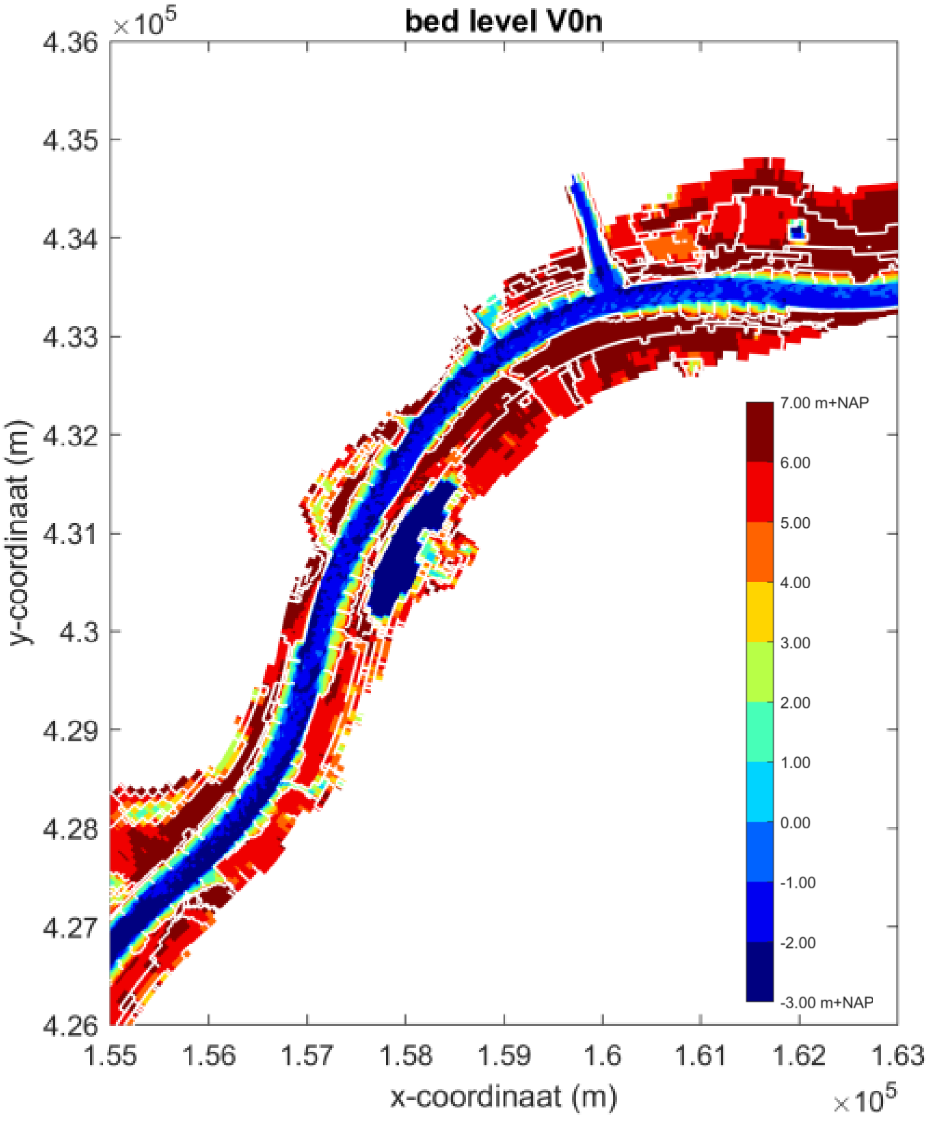


Figure 14
Bed level and weirs for V0.

V2/V3: (partially) closed intakes

For hydrodynamic simulations, it is common to include sills, which are typically smaller than the grid size, only as 2D weirs. V2 considers fully closed intakes by adding weirs at the intake sills with the same height as the connecting LTW.

In 2018 the intakes of Wamel and Dreumel were adjusted. The sill level of the Wamel intake was raised to OLR + 1,25 m (NAP+3,9 m). The part of the sill of the bank channel of Dreumel outside the navigation signs was raised to OLW + 1,25 m (NAP+3,7 m). These adjustments are included in a Baseline measure by RWS-ON which is available for this project. In that measure, also the intake sill of Ophemert is partially closed, which is also included in the model.

The bank channel topography and bed level at the intake sills are not changed, so:

- V3 = V1 + sills partially raised/closed using 2D weirs, bank channel geometry from V1 (j18).
- V3 represents the intakes as they were changed in the field in 2018.
- For Ophemert, 1 additional weir was added to have a proper connection to the LTW (at 4,64 m+NAP).
- Figure 15 to Figure 17 show the weirs and bed level at the intakes for V3.

Figure 15
Wamel: sill/weir for V3 at 3,9 m+NAP, bed level at ~ 1 m+NAP.

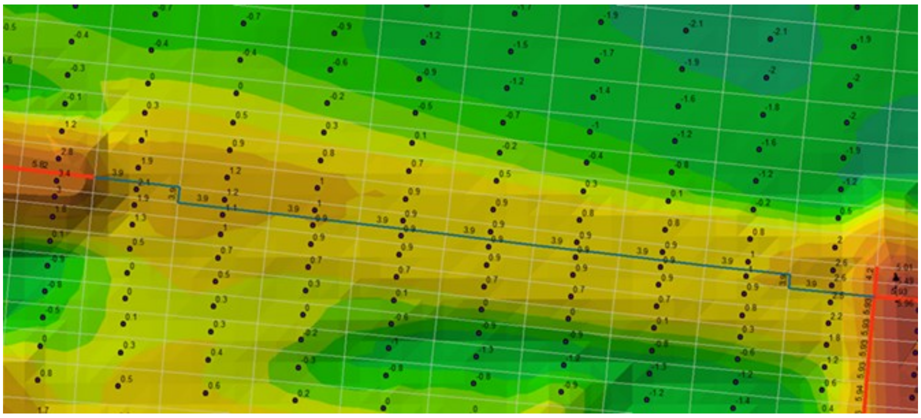


Figure 16
Dreumel: sill/weir for V3 at 3,7 m+NAP, bed level at ~ 0,7 m+NAP; opening in centre at 0,7 m+NAP, which is more or less the bed level in V1.

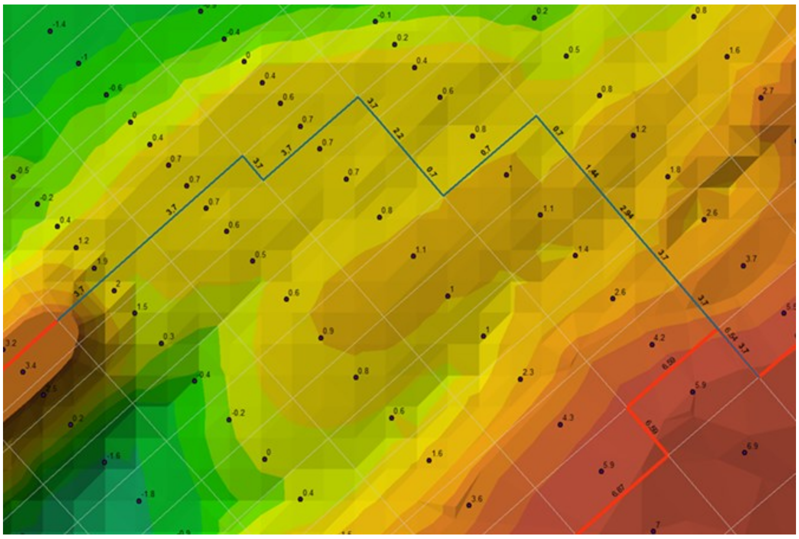


Figure 17
Ophemert: partially closed sill/weir for V3 at 3,15 m+NAP, bed level at ~ 0,3 m+NAP.

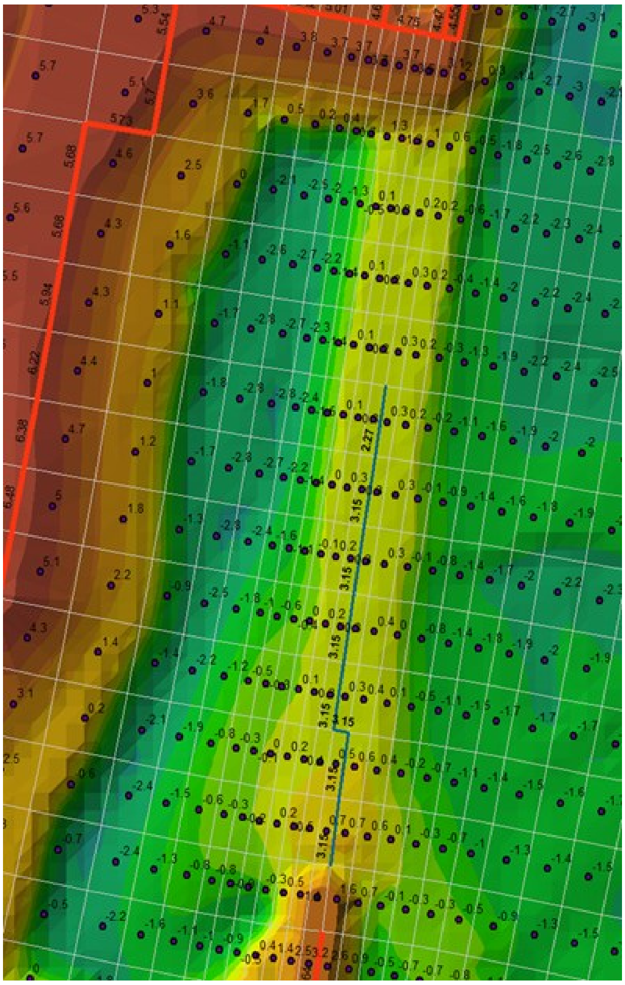


Table 3 gives an overview the schematisation of intake sills in the hydrodynamic simulations. The LTW's are schematised identically in V1, V2 and V3. Only for V2 and V3 (resp. fully and partly closed intakes), weirs are used to schematise energy losses at the intake sills.

Note that the energy losses over a 2D-weir are not specified by the user in Delft3D (such as in WAQUA where the height difference that is used in energy loss calculation can specified by the user). Instead, the energy loss is based on the difference between the crest height of the 2D-weir and the local bed level in the computational core of Delft3D.

Table 3 Schematisation of intake sills in hydrodynamic simulations.

	Schematisation of intake sills
V1	Ground body in bed level, no local 2D weirs.
V2	Fully closed intake sills, height of sills schematised using a 2D weir at the crest level of the LTW, connecting to the weir that represents the crest level of the LTW's. The bed level is not raised. Note that this is a fictive case, considering a higher sill crest elevation than designed (the maximum design height is OLR/OLW-2002 +1,25 m).
V3	Wamel: partially closed intake by raising the sill over the entire intake width. Dreumel/Ophemert: partially closed intake by raising the sill over part of the intake width. In all cases, the highest level of the intake sills is still well under the LTW crest level. In the model, bed levels are identical to V1; changed sill configurations are implemented by changing the heights of 2D weirs. Note that for this variant no morphodynamic simulations are performed within WP1.

2.3.2 Global verification of V1

The simulations presented herein are used to carry out a comparative analysis to identify the hydrodynamic and morphodynamic differences between variants. Nevertheless, in order to gain trust in the model results in general, we performed a global verification of absolute water levels and discharge to have a feeling of model performance. This 'global verification' is done by:

- Comparing simulated water levels with so-called "betrekkingslijnen" (2018) in the river axis;
- Comparing modelled and measured discharge distributions between bank channel and main channel.

Water levels

Omer et al. (2019a) tuned simulated water levels by changing the roughness coefficients of the model, resulting in a global roughness calibration factor of 1,7. Since we changed the upstream boundary conditions in this project (other Waal discharge for same Lobith discharge, see paragraph 2.2.1), we compared model results with 'betrekkingslijnen' and found that using a calibration factor of 1,0 gives better agreement with measurements. Therefore, we set the calibration factor of the roughness coefficients to 1,0.

The model results using a calibration factor of 1,0 are given in Figure 18 and Figure 19; the water level difference is defined as 'model' minus 'betrekkingslijnen' (BL's). The difference in water level is roughly between +20 and -20 cm. Since we are mainly interested in relative numbers, this is considered to be acceptable for WP1 (see paragraph 1.5).

Note that:
- The 'betrekkingslijnen' are interpolations between so called LMW-stations;
- For 18000 m^3/s no information is available in the BL's.

Figure 18 Comparison modelled water level with BL's for Q1020 (left) and Q16000 (right).

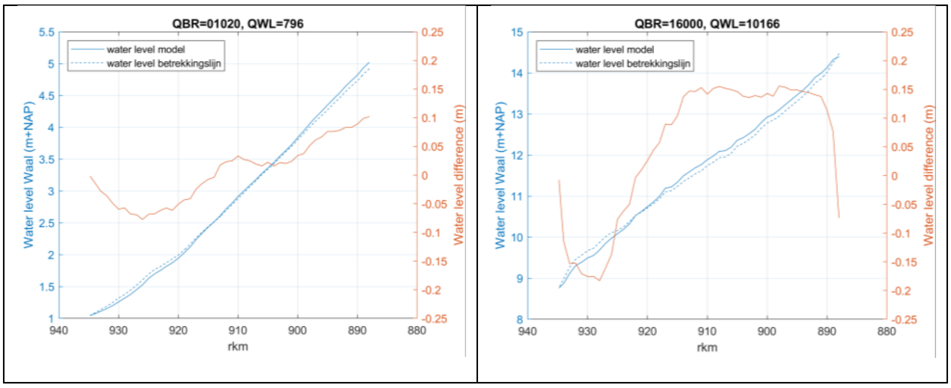
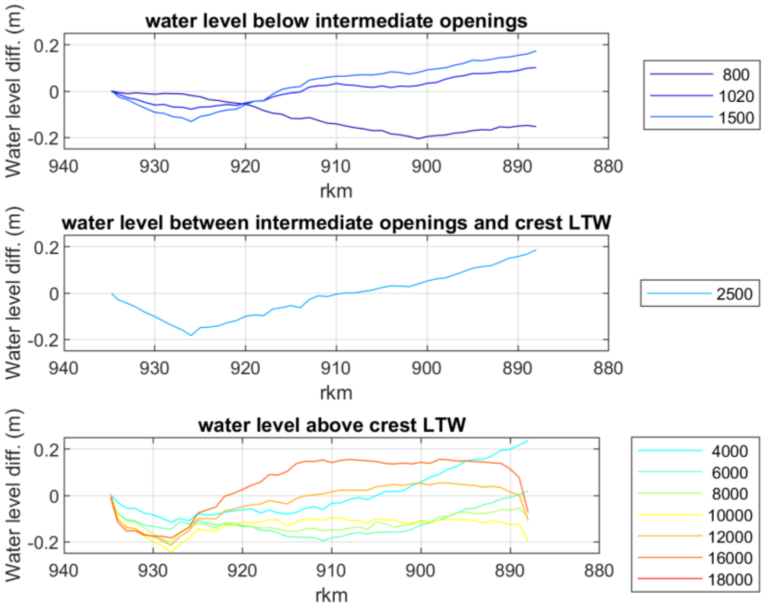


Figure 19 Comparison modelled water level with BL's for all considered (Lobith) discharges.



Discharge distribution

In the model various observation cross-sections are included, which are used to analyse the discharge distribution between bank channel and total discharge through a cross-section.

Figure 20 shows the modelled discharge distribution (solid lines) and measured distributions taken from Sieben (2020). The black solid line is the "threshold value" defined by Sieben (2020). This threshold value is important for obtaining a proper balance between hydraulic and morphodynamic effects in the field situation. Below this threshold, at low river discharge, the combination of main channel narrowing, and bank channel capacity leads to erosion in the main channel. Above this threshold, at higher river discharges, a large(r) discharge fraction through the bank channel results in a reduction of flow velocity in the main channel (w.r.t. the situation without LTW's) and thus sedimentation in the main channel. Since the discharge fraction through the bank channel is adjustable through the inlet sill configuration, also the morphology (and hydraulics) in the main channel can be influenced.

In general, the agreement between model and measurements is quite good. For Wamel, the model captures that at low discharge, the discharge fraction to the bank channel is below the threshold value, and for higher discharge above the threshold. For Dreumel, the model gives a similar pattern while measurements are on or just above the threshold, indicating that the model underestimates the discharge fraction to the bank channel, especially for low-medium discharges. For Ophemert, the model captures that the discharge fraction is well above the threshold for all discharges, but also here the discharge fraction is underestimated with approximately 25%. We reflect on this in chapter 4.

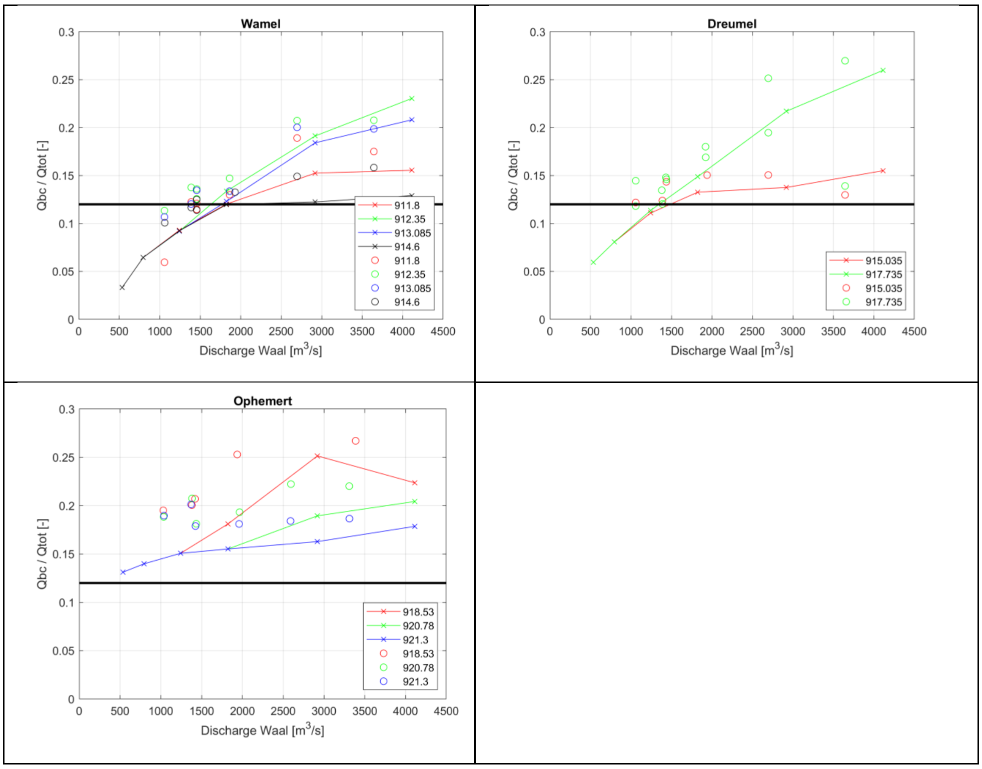


Figure 20
Comparison of modelled (lines) and measured (symbols) discharge distribution between bank channel and total discharge through cross-section.

Concluding remarks on model performance

The comparison between calculated and measured water levels and discharges indicates that the model sufficiently captures, in absolute terms, the observations from the field. Accordingly, we have confidence that the model can be used for the comparative analysis and would sufficiently accurately capture the hydraulic effect of the different variants of the LTW's.

2.3.3 Effect LTW's (V1 versus V0)

The effect of the LTW's can be identified by comparing V1 with V0. The water levels in the river axis are compared between V1 and V0 in Figure 21. The 'MHW'-discharge (16.000 m^3/s at Lobith) is shown in black. At the lowest discharges (800 m^3/s & 1020 m^3/s) the LTW's lead to ~4 cm increase in water levels. At all other discharges the water level is lower in V1 compared to V0.

The discharge through the bank channels for V1 is given in Figure 22.

Figure 21
Water level V1 minus V0.

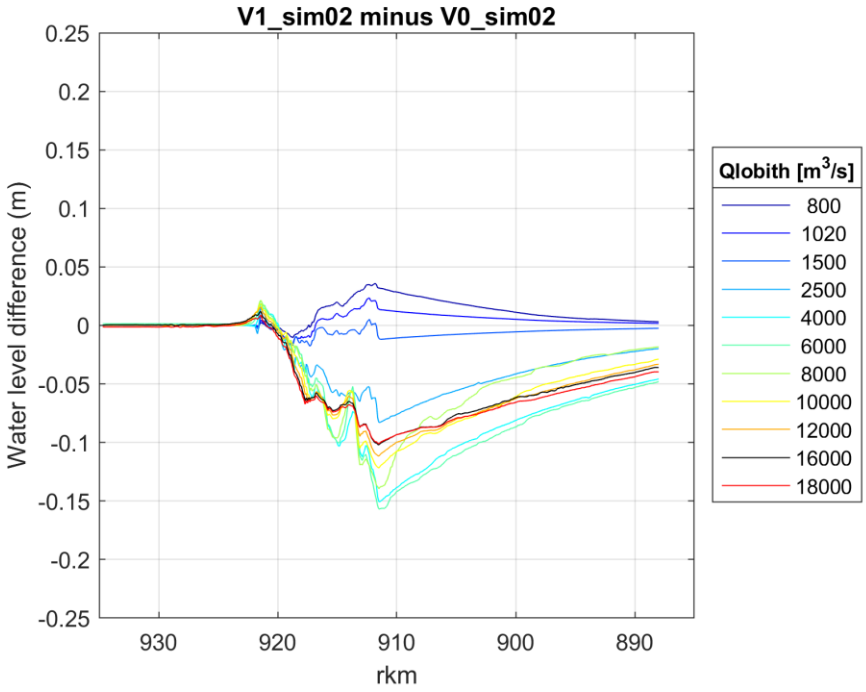
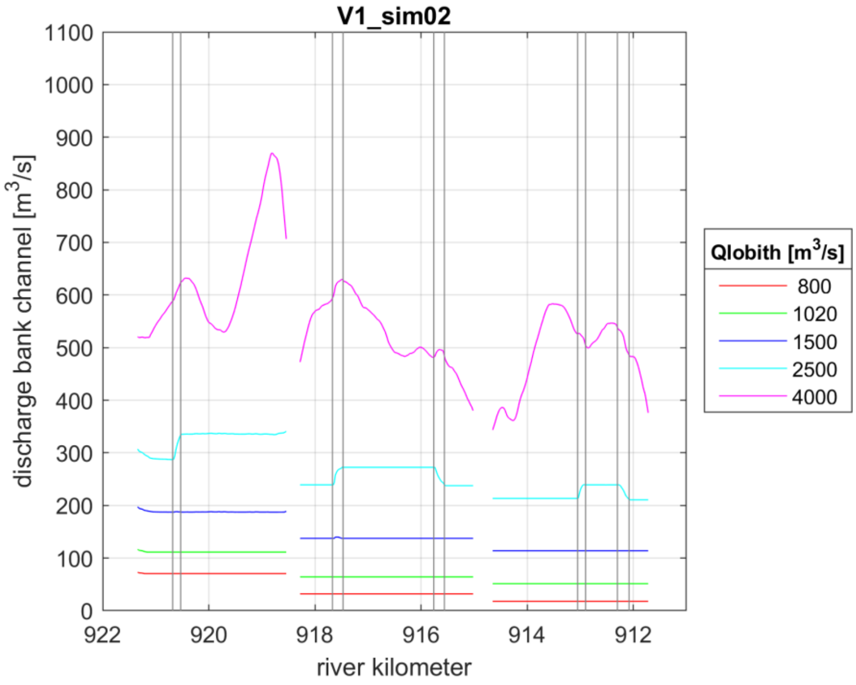


Figure 22
Discharge through bank channels V1.



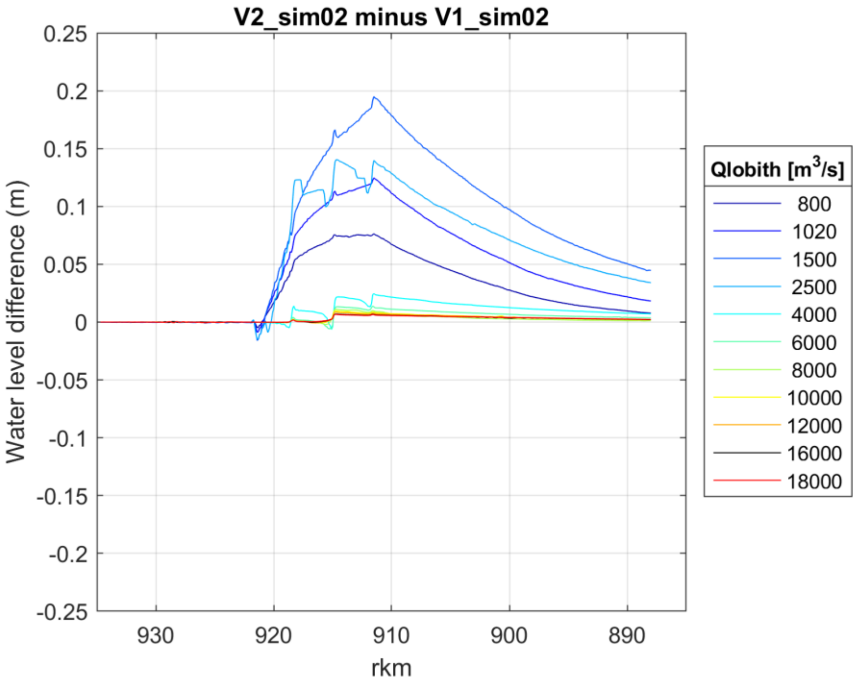
2.3.4 Effect of (partially) closing intakes (V2 and V3)

In order to evaluate the isolated effect of (partially) closing the intakes, we compare the LTW's variants with (partially) closed intakes (V2 & V3) with the base variant of LTW's (V1). The optimisation variants (variants based on V1) are treated in paragraph 2.4.

Fully closing intakes (V2 versus V1)

V2 considers fully closed openings (see Table 3). The results shown in Figure 23 indicate that full closure of the inlets can result in up to 20 cm higher water levels at rkm 911. The largest effect is expected for Q~1500 m^3/s (Lobith discharge).

Figure 23
Effect V2 vs V1 on water level.



Partially closing intakes (V3 versus V1)

Figure 24 shows the effect of partially closing the intakes with 2D-weirs on the water level. The effect of partial closure on the water levels is only visible for a narrow range of discharge classes (800 to 1500 m^3/s). The effect on the discharge through the bank channels is shown in Figure 25. The largest effect on discharge is for the Wamel bank channel reducing it to zero for the lowest discharge levels.

Figure 24
Effect V3 vs V1 on water level.

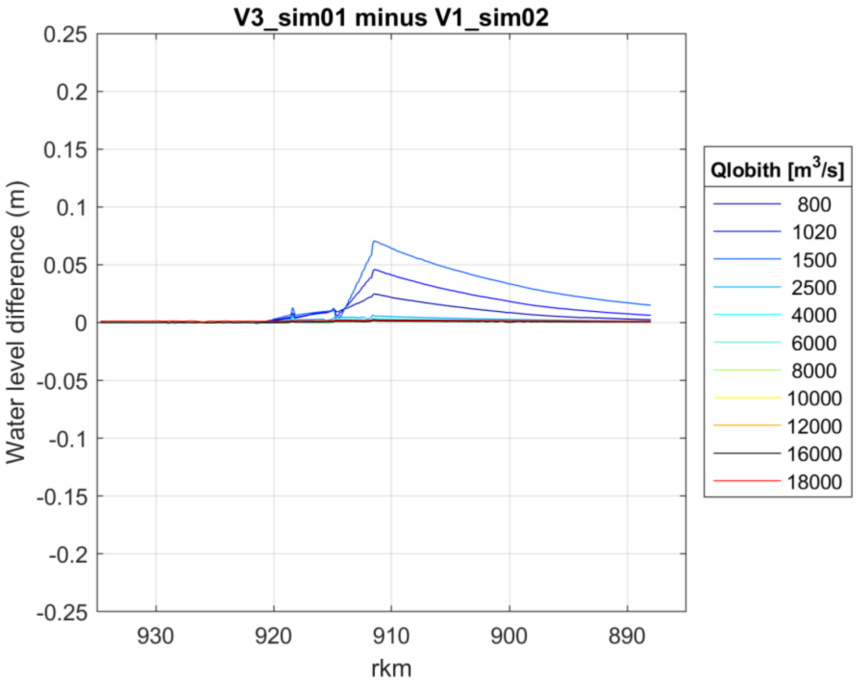
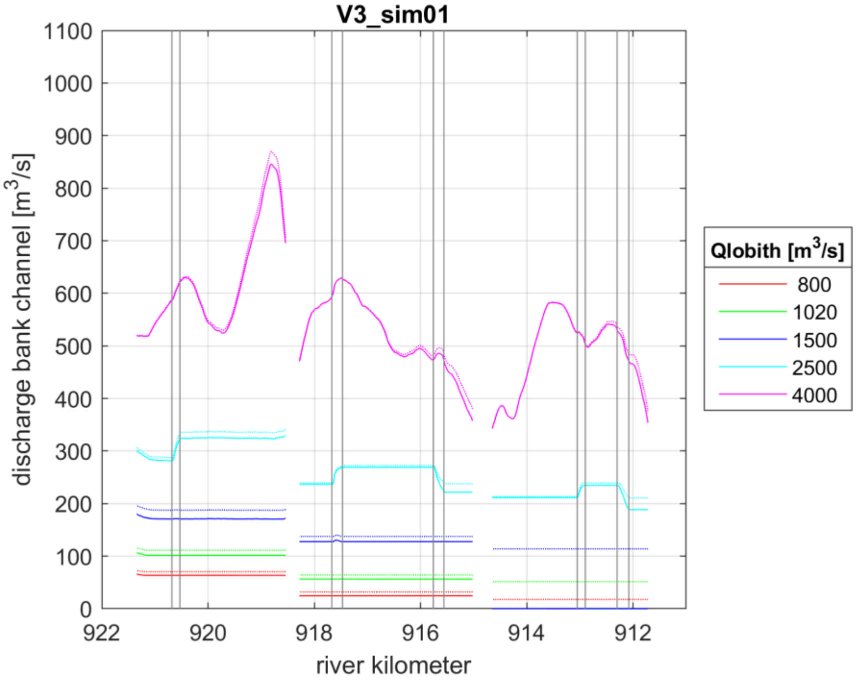


Figure 25
Effect V3 vs V1 on discharge bank channel (dashed = V1, solid = V3).



2.4 Optimisation variants (only hydrodynamics)

For WP2 various “optimisation variants” are considered. The purpose is to investigate the relative hydrodynamic effects of various LTW-designs, compared to V1, needed for various WP’s (e.g., WP2/WP6). Using model results from these simulations, WP2 analyses whether the local design can be optimised for various river functions. WP6 uses the results to analyse effects on flood safety. The results are also useful for analysing effects on nature functions (WP8). In paragraph 2.4.1 we show the variants considered. Some basic (example) results are shown in paragraph 2.4.2 together with an overview of the data.

In the next sections, we present the approach of modelling these variants (the schematisations) and the raw results of the model. The detailed analysis of the performance and effects of the optimisation variants is executed within WP2 (see Zuijderwijk and De Jong, 2021).

2.4.1 Schematisations

Table 4 gives an overview and short description of the considered optimisation variants, including the simulation code. All variants are based on V1, and OLR = OLR2012. The optimisation variants shown in Table 5 are taken from WP2 (Zuijderwijk and de Jong, 2021), which gives an overview how the various considered optimisation variants relate to each other.

Table 4
Considered cases for optimisations WP2.

Vopt	Code	Description
Vopt_01	LD_h_min_0p5	Crest height of LTW lowered by lowering 2D-weirs by 0,5 m
Vopt_02	LD_h_min_1p0	Crest height of LTW lowered by lowering 2D-weirs by 1,0 m
Vopt_03	TO_h_dicht_050p	Intermediate openings, height 50% closed
Vopt_04	TO_h_dicht_100p	Intermediate openings, height 100% closed
Vopt_05a	OG_h_OLR_min_0p00	Minimum bed level bank channel at OLR min 0,00 m
Vopt_05b	OG_h_OLR_min_2p75	Minimum bed level bank channel at OLR min 2,75 m
Vopt_06a	OG_w_100p_d_OLR_min_2p75	Depth bank channel at OLR min 2,75 m, 100% width
Vopt_06b	OG_w_075p_d_OLR_min_2p75	Depth bank channel at OLR min 2,75 m, 75% width
Vopt_06c	OG_w_050p_d_OLR_min_2p75	Depth bank channel at OLR min 2,75 m, 50% width
Vopt_07	V1_no_dam_in_bed_weir	Dam removed from topography, weir kept
Vopt_08	V1_no_dam_in_bed_ppl	Based on Vopt_07, but weir replaced by porous plate

Table 5 Overview optimisation scenarios.*

Onderdeel	**Para-meter**	**Pilot (situatie in V1)**	**Simulaties voor ontwerpvarianten**	**Vergelijk met**
Langsdam-lichaam	hoogte	OLR + 2,75 m	Vopt_01: OLR +2,25 m (0.5 m verlaagd) Vopt_02: OLR +1,75 m (1.0 m verlaagd)	V1 V1
	talud	1:2.5	Vopt_07: Verticale damwand	V1
	materiaal	breuksteen	Vopt_08: Poreuze verticale damwand	Vopt_07
Instroom-opening	breedte			
	hoogte	OLR -1,75 m	*V2: Volledig gesloten* *V3: Deels gesloten (zoals nu in het veld)*	V1 V1
Tussen-openingen	breedte			
	hoogte	OLR +1,25 m	Vopt_03: OLR +2.0 m (50% gesloten) Vopt_04: OLR +2.75 m (100% gesloten)	V1 V1
oevergeul	breedte	varieert (ca. 50 tot 120 m)	Vopt_06a**: Breedte constant op ong. 110/120 m, en bodemhoogte constant op OLR -2.75 m. Vopt_06b: 75% breedte van Vopt_06a Vopt_06c: 50% breedte van Vopt_06a	V1 Vopt_06a Vopt_06a
	diepte	Varieert (ong. OLR -4,75 m)	Vopt_05a**: Minimaal bodemhoogte OLR -0.00 m Vopt_05b: Minimaal bodemhoogte OLR -2.75 m	V1 Vopt_05a

* Table text only available in Dutch from Zuijderwijk and De Jong (2021).

** These variants (Vopt06a/Vopt05a) are no design variants but implement uniform dimensions in the model as a reference case. These variants are used as reference for other design variants (see last column).

Vopt_01/02: crest level of LTW

The LTW's are schematised in Baseline (and thus in Delft3D) in the topography with a low weir on top of it (Figure 12). The highest bed level points are approximately 1,5 m below the crest level of the 2D-weirs. The lowering of the LTW crest level is implemented by lowering the crest level of the 2D-weirs. The geometry (bed level) of the LTW's including the intermediate openings is not changed.

Vopt_03/04: crest level intermediate openings

In the Delft3D-model of Omer (2019a, b), the height of the weirs, representing the crest levels of the LTW's and intermediate openings, does not decrease monotonic in downstream direction (there are slight 'fluctuations' in height, see Figure 2). The height of the dam (upper bound of opening closure) is taken as the height of the 2D-weir two grid cells away from the opening. The height of the intermediate openings is the average height of the opening, see Table 6. Remark: from the table it can be seen that the downstream height is not always lower than upstream (as a result from the Baseline schematisation).

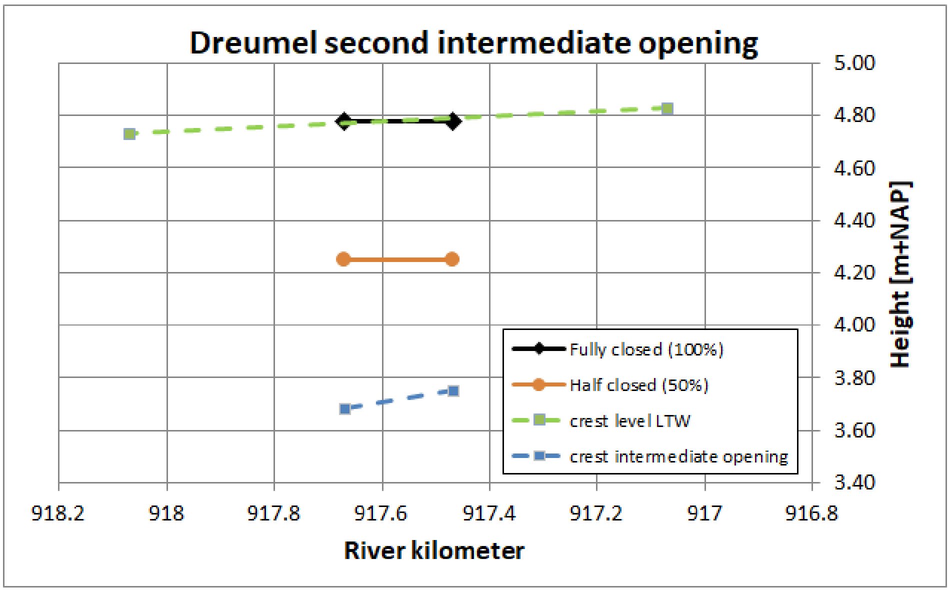
Table 6
Crest levels intermediate openings.

Opening	Location	hD_U	hD_D	hT_U	hT_D	hT_N _100%	hT_N _50%
Wam1	912.08-912.30	5.66	5.77	4.11	4.18	5.72	4.93
Wam2	912.90-913.05	5.50	5.53	4.10	4.14	5.52	4.82
Dreu1	915.56-915.76	5.17	4.99	3.78	3.77	5.08	4.43
Dreu2	917.47-917.67	4.83	4.73	3.75	3.68	4.78	4.25
Oph1	920.53-920.68	4.49	4.32	3.29	3.34	4.41	3.86

Location = location of opening (river kilometer)
hD_U/D = height dam upstream (U) or downstream (D) of opening [m+NAP]
hT_U/D = height intermediate opening upstream (U) or downstream (D) [m+NAP]
hT_N = new height intermediate opening, 100% and 50% closed [m+NAP]

As an example, the resulting heights of the intermediate openings for the second downstream opening of the Dreumel LTW intermediate opening is shown in Figure 26 for the two considered cases: half closed, and fully closed (also the base level is shown). These heights are the crest heights of the 2D-weirs in the hydraulic simulations. Note that, just as for variant V2, in this fictive scenario the openings are closed even further than the maximum design height, which is OLR+1,25 m.

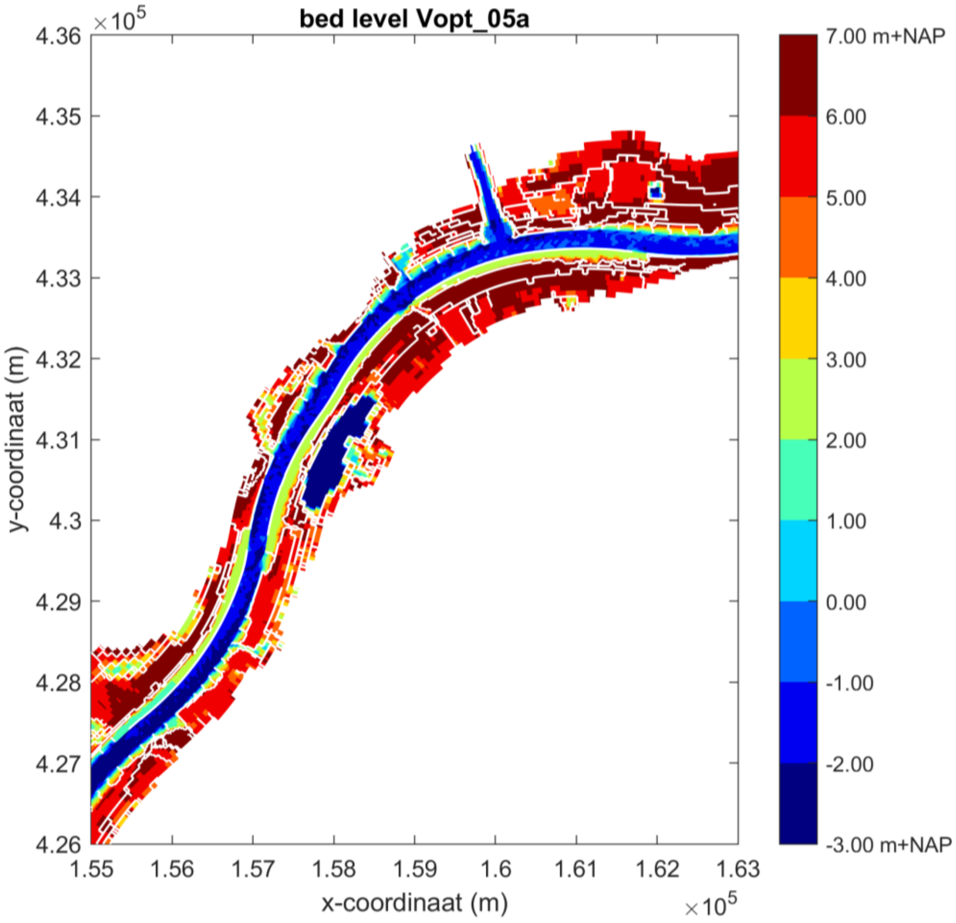
Figure 26
Schematisation intermediate openings: example for Dreumel LTW, second opening.



Vopt_05: depth bank channel

To mimic siltation in the bank channel, the local bed elevation is increased to the level of (a) OLR, and (b) OLR-2,75m. The bed level of the former is shown in Figure 27. The locations of 2D-weirs are not changed with respect to V1 (see paragraph 2.3.1). Filling up for case (b) results in higher bed levels especially in the Ophemert channel; the other channels are quite shallow in V1 already.

Figure 27
Bed level and location of 2D-weirs (white lines) for Vopt_05a, filling up to OLR.



Vopt_06: width bank channel

In the base variant V1 the cross-sections of the bank channels vary significantly. This makes an analysis of effects of varying width difficult. Accordingly, and for the purpose of evaluating the effects of width variations, we created a schematisation with a channel with a base (100%) width and a bed level of OLR-2,75 m. Next, we use this as a reference to reduce the width of the bank channel. Figure 28 illustrates the maximum contour of the area (between the black lines) where the cross-section of the bank channel is changed to obtain the base variant with 100% width, resulting in a maximum bank channel width of about 120 m for Wamel and Dreumel and about 110 m for Ophemert.

We considered two reduced widths: 75% and 50% w.r.t. the case with 100% width (so reduction in width of by approximately 25 and 50% respectively), where the deep part is adjacent to the LTW. Changes in model geometry are implemented based on Delft3D grid lines. Because grid lines do not entirely align with the bank channels, the resulting width is not constant but approximately as intended. For the case with 100% width, the bed level change extends beyond the bank of the V1 bank channel at some locations; at these locations also the 2D-weirs representing these banks are removed. The roughness code is set to 105 in the parts with changed bed level, representing "nevengeul" with Nikuradse roughness height of *k*=0,20 m. For the widths 75% and 50%, part of the original bank channels remains in the model, since the new reduced width is calculated from the LTW crest. The resulting 100% and 50% case are shown in Figure 29.

Figure 28
Area where 100% is implemented for Vopt_06.

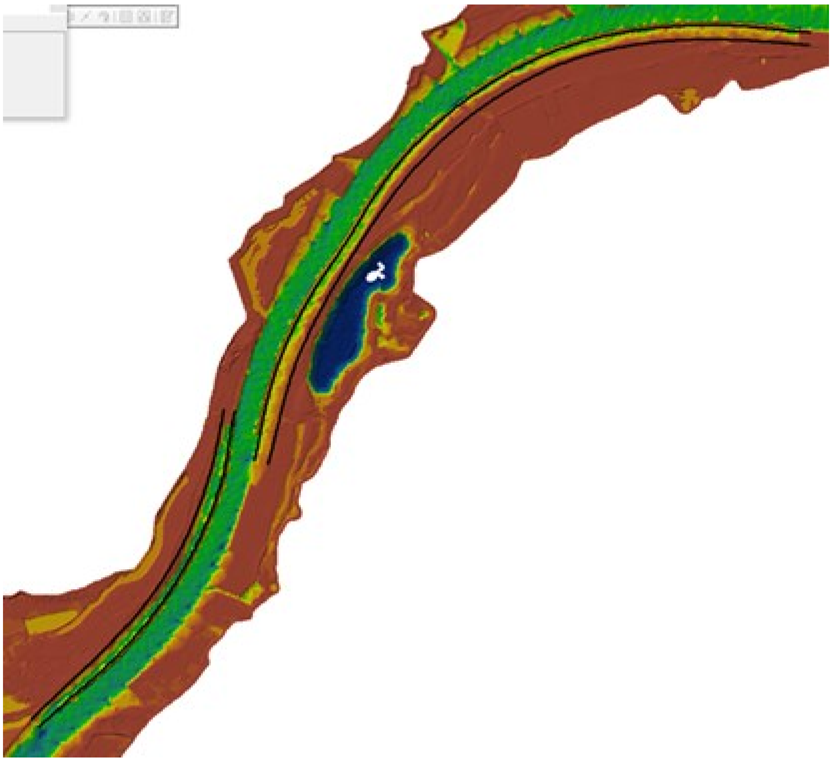
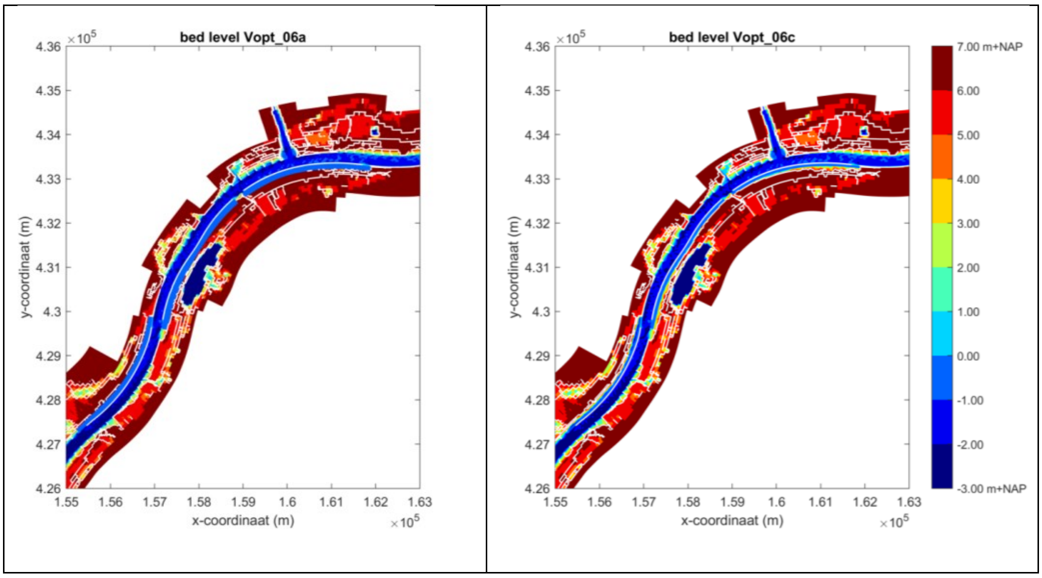


Figure 29
Bed level and weirs for Vopt_06, 100% width (left), 50% width (right).



Vopt_07: LTW as weir only (removed from topography)

This variant is made to evaluate the performance of LTW's with minimum volume blocking the flow, e.g., LTW's made out of sheet piles. For this case, the dams are schematised by being removed from the bed topography both in the main channel and bank channel. This is done by removing the bed level points of the dam (including the slopes) and fill missing bed levels using linear interpolation (see result in Figure 30). The roughness code 112 representing the dam crest (rough bank / "ruwe oever", Nikuradse roughness height *k*=0,40 m") is replaced by 105 representing the roughness of the bank channel (side channel / "nevengeul", Nikuradse roughness height *k*=0,20 m"). The weirs that represent the LTW crests are kept in the model.

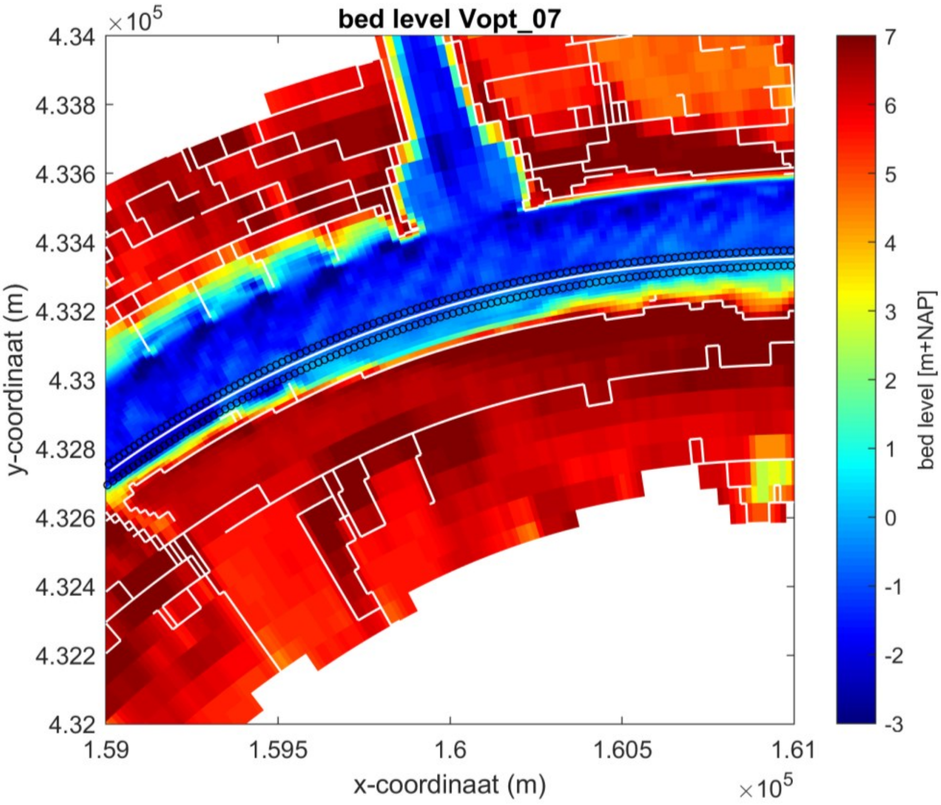
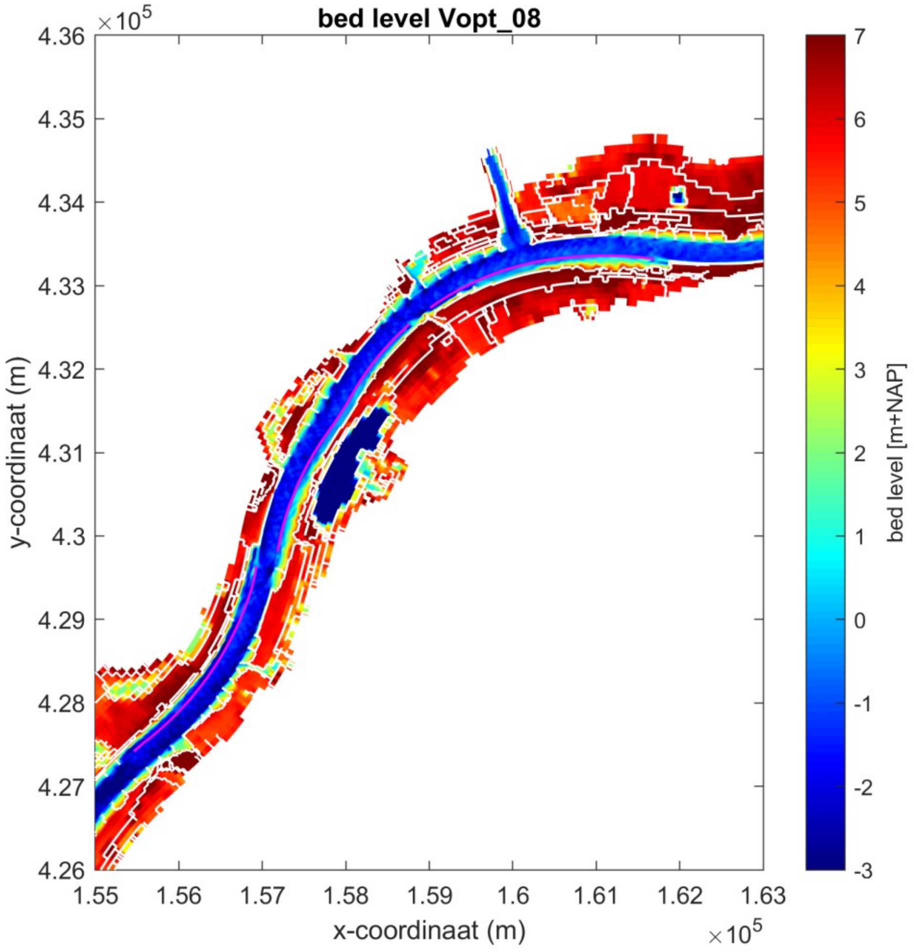


Figure 30
Bed level and weirs for V1 with dam removed from bed level.

Vopt_08: dam as porous plate

This variant is based on Vopt_07, but the weir is replaced by a porous plate (Figure 31). The porosity of a porous plate is controlled by a quadratic friction coefficient (see paragraph 10.9.2 in Deltares, 2021). Thus, the porosity is not an input parameter as such, and proper values of the friction coefficient can only be found by trial and error (pers. comm. Erik de Goede, Deltares). We considered the following friction coefficients: (a) 1, (b) 1000, (c) 1000000, (d) 0.1, (e) 10, and (f) 100.

Figure 31
Bed level and location of 2D-weirs (white lines) and porous plates (magenta lines) for Vopt_08.



2.4.2 Results and data overview (base + optimisations)

The model simulations discussed in the previous paragraph are analysed by WP2 (Zuijderwijk and De Jong, 2021) in detail. In this paragraph we give some exemplary results to verify model settings. Note that for the optimisations a subset of the discharge classes is relevant; therefore, not all discharges are simulated for each variant.

All produced figures and data (Delft3D output and CSV-files) are provided to WP2 and other relevant WP's to perform their analysis. The cross-section discharges are available every river kilometre and for some cross-sections in the bank channel and at the channel intakes. At observation stations water levels and flow velocity magnitudes are tabulated.

The stations are subdivided in:
- MWTL ("programma Monitoring Waterstaatkundige Toestand des Lands") and other stations;
- River kilometer points (888.00_WA - 934.00_WA);
- River hectometre points (hm_888.00_WA - hm_934.70.00_WA);
- Stations in bank channels Wamel, Dreumel, Ophemert (<channel>_<m>_<n>).

Some example figures are given below:

- Figure 32 gives the water level difference for a higher bed level in the bank channel (Vopt_05a) w.r.t. V1.
- Figure 33 gives the water level difference for a situation with a wider and deeper bank channel (Vopt_06a) w.r.t. V1.
- Figure 34 gives the effect on the discharge through the bank channels if some sedimentation would have occurred w.r.t. V1 (Vopt_5b).

For detailed analysis and interpretation of the results of hydrodynamic simulations, we refer to WP2 (Zuijderwijk and De Jong, 2021).

Figure 32
Effect on water level for Vopt_05a, bed level in bank channels to OLR.

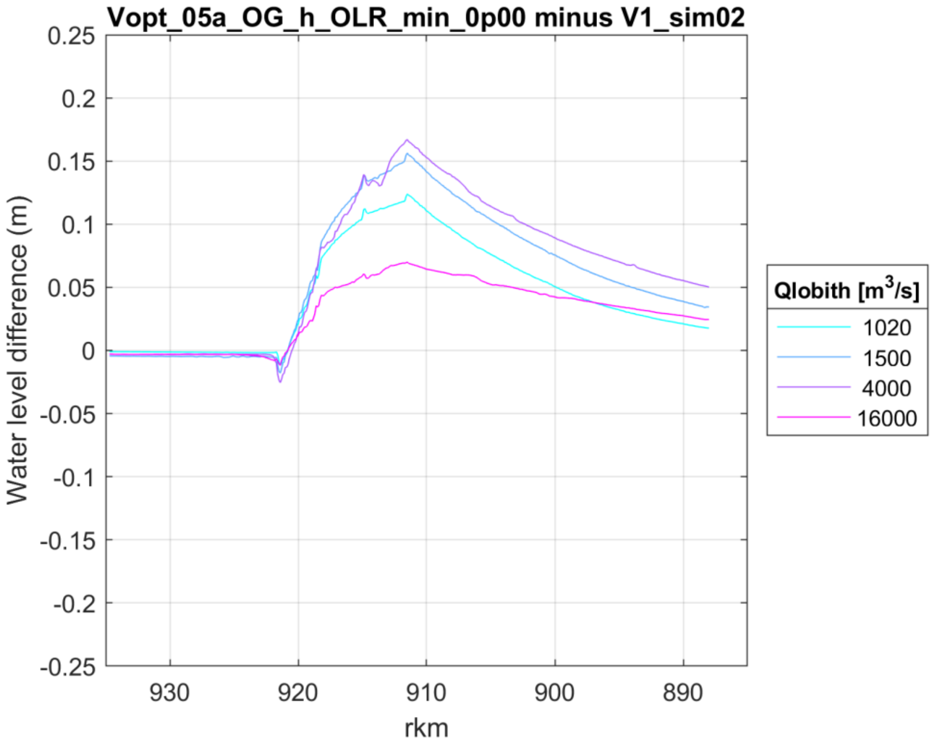


Figure 33
Effect on water level for Vopt_06a, widening the side channel, bed level at OLR-2,75 m.

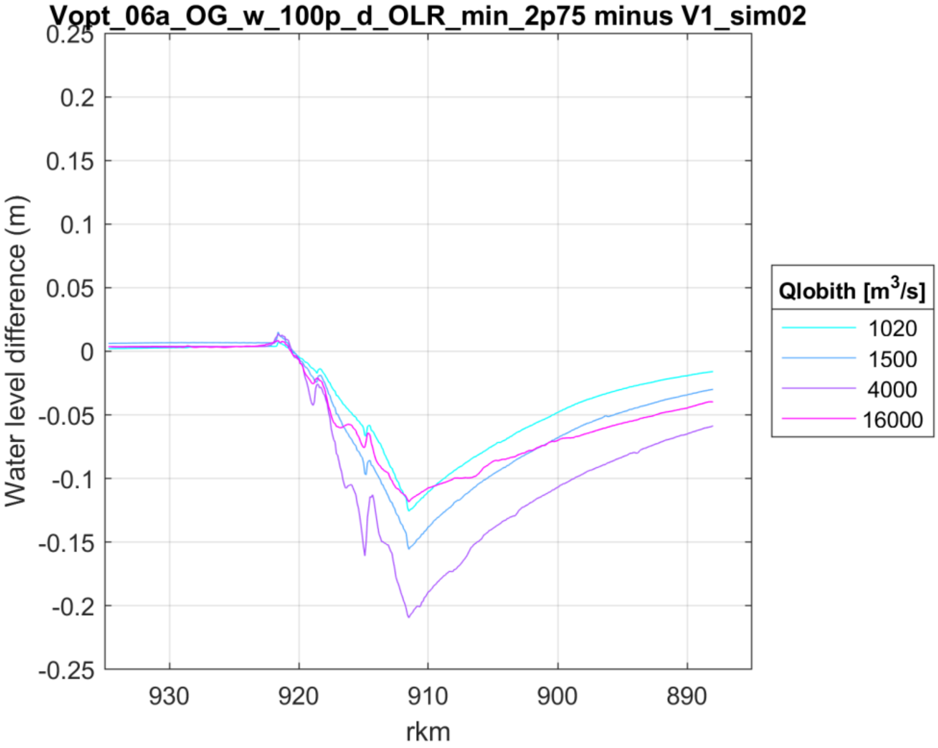
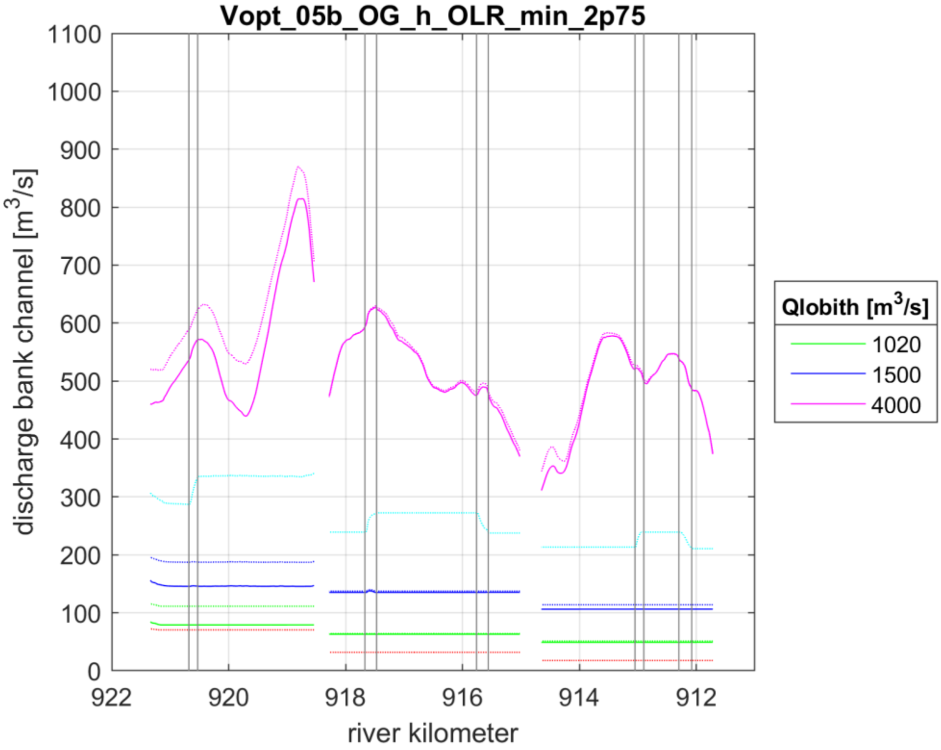


Figure 34
Effect on discharge through bank channel with some sedimentation in bank channels. Dashed lines are V1, solid lines are Vopt_05b (the latter only computed for Qlobith 1020, 1500, 4000). Grey vertical lines are the locations of the openings.



3 Morphodynamic simulations

3.1 Introduction

This chapter gives an overview of the set-up and results of the morphodynamic simulations performed with the Delft3D model. For the morphological analysis, a selected number of variants is modelled. Each simulation was run for 20 years and was made with and without activating the dredging and dumping module.

The variants considered are described in paragraph 3.2. Model settings, specific for the morphological simulations, are given in paragraph 3.3. Before starting the morphological simulations with a representative hydrograph, a so-called spin-up simulation is performed which is discussed in paragraph 3.4. Model results are presented in paragraph 3.5 (with dredging and dumping active) and paragraph 3.6 (no dredging and dumping). In both cases, it is basic analysis –see WP7 and WP10 reports for detailed analysis. Paragraph 3.7 compares the effects of V1 and V2 relative to the reference case V0, giving an indication of the range in effects (bed level, low water level, dredging quantities).

3.2 Variants for morphological analysis and simulations performed

For the morphodynamic simulations, we focus on the base variant (‘hoekpunten’), since WP7 and WP10 are mostly interested in the effects of these variants. The schematisation (geometry, roughness) of the base variants V0, V1, V2 is largely identical to the one of the hydrodynamic simulations (see paragraph 2.3.1). In the overview below, only the differences w.r.t. the hydrodynamic variants are mentioned:

- V1 = situation 2018. For the bank channel geometry, we use the measured as-built situation (see paragraph 3.3.4), instead of the geometry as stored in BL-j18 that is used in hydrodynamic simulations (chapter 2). This will probably lead to a larger discharge fraction to the bank channel (see chapter 4).
- V0 = V1 with 2014 geometry in area of LTW’s
- V2 = V1 with fully closed openings (by using a 2D weir and locally increased bed level, see paragraph 3.3.4). The height of the weir and bed level at sills is taken the same as the connecting LTW (Table 7). This is needed to (i) block sediment (bed level) and (ii) have correct energy loss (weir).
- V3 = partially closed intake sills: not used in morphodynamic analysis.

Table 7 Schematisation of intake sills in morphodynamic simulations.

Variant	Description and schematisation of intake sills
V0	Reference case: no LTW's present with original (non-lowered) groynes in area of LTW's (BL-j14). Lowered groynes outside this area (BL-j18).
V1	LTW's and intake geometry from BL-j18. Ground body in bed level, no local 2D weirs. Geometry of bank channel is the as-built situation.
V2	Identical to V1, but fully closed intake sills; sills schematised in the same way as LTW's in V1, thus raised bed level with low weir on top of that (conform Figure 12). Note that this is different from the approach in the hydrodynamic simulations, where the closure was implemented by only using a 2D-weir.
V3	Not used in morphodynamic analysis.

General remarks:

- The schematisation of the intake sills in V1 and V2 is explained in more detail in paragraph 3.3.4.
- The LTW's are included in the bed level, with a weir on the crest to represent local energy losses.
- During the construction of the LTW's sediment was extracted from the main channel. Since it is not known exactly where and how much sediment was extracted, this is not taken into account in WP1 (refer to WP10 for further information on this topic). To prevent that the sediment extraction affects the model results we (i) use a spinned-up bed level in the main channel as initial condition in the actual morphodynamic simulations (see paragraph 3.4) and (ii) we only consider differences between variants.
- In May 2018 the intake sills were changed/raised/partly closed. This is not included in V1. This situation is considered in the hydraulic simulations as V3. However, since WP7 and WP10 are mainly interested in the effects with either fully opened or fully closed intake sill, V3 is not run morphologically.

Remarks on simulations:

- As requested by WP7 and WP10 (see Paarlberg et al., 2020), variants V0, V1 and V2 are run for a period of 20 years. Simulations are performed both with and without dredging and dumping active.
- The long-term morphological simulations are intended to judge the relative effect of design variants. The studied variants differ from situations in the field, meaning that direct comparison to the field situation cannot be made. A detailed analysis of field measurements can be found in WP0. Hydro-morphological data from WP0 and computational results from WP1 form separate sources of information for application in other work packages. Results from both sources are integrated and interpreted within those work packages (see also chapter 4).
- Each year to simulate takes about one day computation (wall) time.

- For the morphodynamic simulations with a hydrograph we use the Simulation Management Tool (SMT). This means that:
  - The hydrograph is specified in a series of discharges (Qseries)
  - The initial flow field for each discharge is stored in 'local_database'
  - The output is stored, for each simulated discharge, in an output folder, that includes sub-folders named with the hydraulic time.
  - We use a relatively small "InitialPeriod" (spin-up time before morphological changes start at a new discharge level) of 60 minutes. A larger spin-up period would result in too large simulation times.
- All simulation results are provided to and used by relevant WP's to perform their analysis. Further details can be found in the report of WP0, WP7 and WP10.

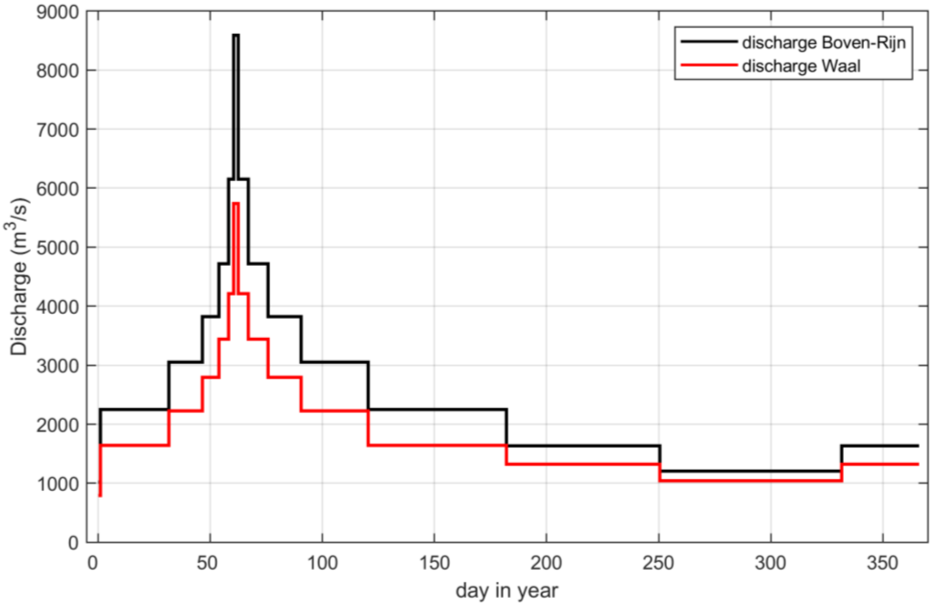
3.3 Model set-up

3.3.1 Hydrodynamic boundary conditions and calibration

We use the following hydrodynamic boundary conditions:

- The discharge hydrographs in terms of Lobith discharge (Qseries) are taken from previous work / studies (i.e., from the DVR-model). We use one representative 'common' hydrograph (repeated each year), see Figure 35.

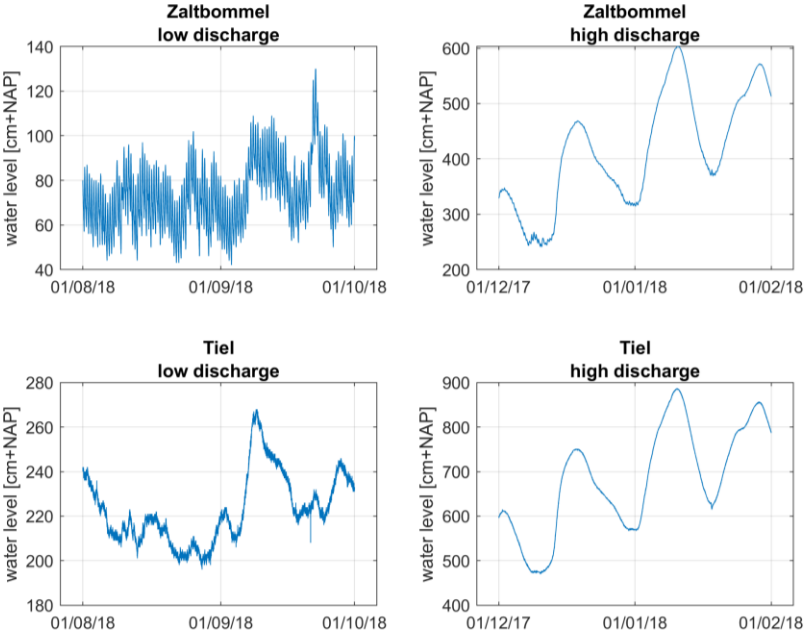
Figure 35
Yearly hydrograph used in morphodynamic simulations.



- Lobith discharges are converted to Waal discharges using the same relations as used for the hydrodynamic simulations (paragraph 2.2.1). In line with hydrodynamic simulations, we use a roughness calibration factor of 1,0 (paragraph 2.3.2).
- The downstream boundary condition (Qh-relation Zaltbommel) is taken equal to the hydrodynamic simulations (paragraph 2.2.2), meaning that it is based on the Betrekkingslijnen 2018.

- We do not include a discharge to the Amsterdam-Rijnkanaal (ARK). The lowest discharge considered in the morphodynamic simulations is 1203 m^3/s at Lobith. As was discussed in paragraph 2.2.2, discharge to the ARK is only structural and significant for discharge below 1200 m^3/s at Lobith. Note that this means that for a Lobith discharge of 1020 m^3/s, which is used to update the OLR reference plane for dredging, extraction to the ARK might be relevant, but is not included. The reason for this choice is that the model predicts the OLR (2012) well without an extraction to the ARK.
- At the downstream boundary of the model (Zaltbommel) there is a tidal influence, especially at low Waal discharge, see Figure 36. In line with previous morphodynamic Waal-models, we did not include tidal effects in the morphodynamic simulations. In the main area of interest near Tiel, there is virtually no tidal variation, even at extremely low river discharge (Figure 36). Including the effects of tidal variations in a morphodynamic simulations requires either (i) a totally different model setup since tidal and morphodynamic effects occur at different time scales, or (ii) a parametric approach to mimic the enhanced sediment transport rates due to the tidal variations. Both approaches are outside the scope of this study, since we focus on relative effects. Having said that, we are confident that the model is sufficiently fit-for-purpose without the tidal downstream boundary condition.

Figure 36
Water level at Zaltbommel and Tiel during periods of high (1600-5100 m^3/s) and low (625-850 m^3/s) Waal river discharge (measured at Tiel).



3.3.2 Sediment transport and morphology

Overview of model settings w.r.t. sediment transport and morphology:

- Most of the settings are equal to the calibrated 'DVR model', which is available through the HelpdeskWater ('`delft3d_4_dvr-rijn-2015-v1`').
- The settings used by Omer et al. (2019b) were extensively checked and obtained during rough morphological tuning in the past, and it was shown that they provide best agreement.

- Values for e.g., morphological acceleration factor, grain size (D50), secondary flow, A/BShield, etc., are kept equal to the DVR-models.
- The model uses the DVR-defined sediment transport formula of Van Rijn (1977/1984) [Delft3D code: `VRIJN84_RIV_77`]. Thus we assume bedload and suspended load transport of bed material only, which means that suspended load of bed material is taken into account by means of a total load formula for the bed material and that entrainment/deposition and transport by advection-diffusion are not explicitly calculated.
- The model uses uniform sediment (D50 varies in longitudinal direction).
- The model includes one fixed layer at St Andries. This fixed layer has the same roughness coefficients (Van Rijn formulation) as the alluvial sections up- and downstream, while the inner bend is rougher (see paragraph 3.4.2 for details).
- The fixed layer causes a reduction of the sediment transport rate as a function of the thickness of the layer of sediment on top of the fixed layer relative to a user-specified thickness (= 0,7 m).
- We use a fixed bed level (no bed degradation) at the upstream boundary of the model.
- In morphological simulations the LTW's are not permeable.
- Bank erosion processes are not included in the model.
- The initial sediment layer thickness in the bank channels is set to 0, which implies that the initially present bed is non-erodible. Sediment can be deposited and re-entrained on top of the bed, but no erosion can occur below the original bed level. The main reasons for this choice are:
  - The Delft3D computations are (in this project) not meant for simulating the morphological development of the bank channel in detail.
  - The sediment transport (processes) over the inlet sills towards the bank channel are not (correctly) captured in Delft3D. This means that the calculated sediment transport towards the bank channel and associated morphological development of the bank channel is uncertain (see also chapter 4).
  - Note that the distribution of flow between the bank and main channels does influence the morphological development of the main channel (see also chapter 4).
- The LTW's and intake sills are fixed by excluding these parts from the alluvial part of the river (initial sediment layer thickness is set to 0) such that no erosion can occur below the initial bed level (see paragraph 3.4.2).

3.3.3 Dredging and dumping

For the dredging and dumping module, we use the following settings:

- The settings for dredging/dumping are based on '`delft3d_4_dvr-rijn-2015-v1`'
- Dredging polygons for the navigation channel are not changed. Around the fixed layer of St. Andries (km 925-928) no dredging polygons are included.

- Dumping polygons are changed for V1/V2 (with wall), to prevent dumping in the bank channels.
- We use a dredge depth of 2,80 m below OLR for (Lobith) for the entire Waal, based on the settings from '`delft3d_4_dvr-rijn-2015-v1`'. This deviates from Rijkswaterstaat (2018), where the minimum depth under OLR increases downstream of Tiel. Since the morphological simulations are intended to be used in a relative way, the impact of this deviation on differences in dredging quantities is not investigated in WP1 (it is expected to be justified/interpreted in WP10).
- Dredging is executed only during the discharge levels Q1203, Q1635, Q2250.
- OLR is computed (in Delft3D) every year, as the water level for QLobith = 1020 m^3/s.
- In the lower reaches of the Waal, between km 928-935, sand mining is implemented using a maximum volume rate that is allowed to be extracted for km-blocks. The maximum volume rate is about 4400 m^3 per year, and the DredgeDepth = 5,4 m. Mined sediment is removed from the model domain.
- It is possible to include ploughing ("ploegen") in Delft3D, but it was decided not to use this since in the new dredging contracts ploughing will be forbidden and is thus not relevant when predicting morphological effects of the LTW's (instead dredgers should apply sufficient clearance). Conform the DVR-model, we use a clearance of 0,5 m which is sufficient.
- Dunes are modelled using a sub-grid model and included in triggering dredging, since dunes are important for this area of the Waal. To predict dune height, we use DVR-settings (i.e., bed form height predictor of "Fredsoe (1982) for MPM (1948)")
- Two aspects of the DVR-model are not used in this study:
  - The DVR-model uses (automatic) bed stabilisation in eroding river parts (Ottevanger et al DVRII). We do not use that in our simulations.
  - The DVR-model takes various 'WFD' measures ('KRW maatregelen') into account by adding nourishments representing bank erosion in certain areas. Since they will complicate interpretations (there is an area at Dreumel as well) we decided not to include these WFD measures; we have removed them from the dad-files.

Dredging is typically implemented as follows:

```
[Dredge]
    Name                = WA_0910.0_b
    Dump                = WA_0910.0_s
    Dump                = WA_0911.0_s
    Dump                = WA_0909.0_s
    Dump                = OS_u_s
```

In this example:

- Material is dredged at km 910 (region from km 910-911). And:
- Dumped either sideways between 'normaallijnen', downstream, or upstream (in that order).
- Using these settings, if there is not enough margin for dumping in one of the dump polygons, the sediment is removed from the model (dump area "REMOVED_FROM_MODEL"). For the simulations reported in WP1, all

sediment fits within prescribed dumping polygons, and no sediment is "removed from model".

- The dump polygons *OS_g_s* and *OS_u_s* are defined but not used in this model (this is only relevant for models combining both graded and uniform sediment domains).

3.3.4 Topography bank channel and sill V1, V2

Bank channels

For the morphological simulations, for V1 and V2 the as-built measurements of the bank channel are used as initial bed level. This is different from the hydrodynamic simulations, where the Baseline rijn-j18_5-v1 topography is used. The difference is shown in Figure 37 and Figure 38: in general the as-built bed level in the bank channel is lower than in BL-j18 (so sedimentation has occurred). Likely, this also leads to a different distribution of water between main channel and bank channel.

Figure 37 Difference as-built bank channels Wamel/Dreumel with Baseline-j18.

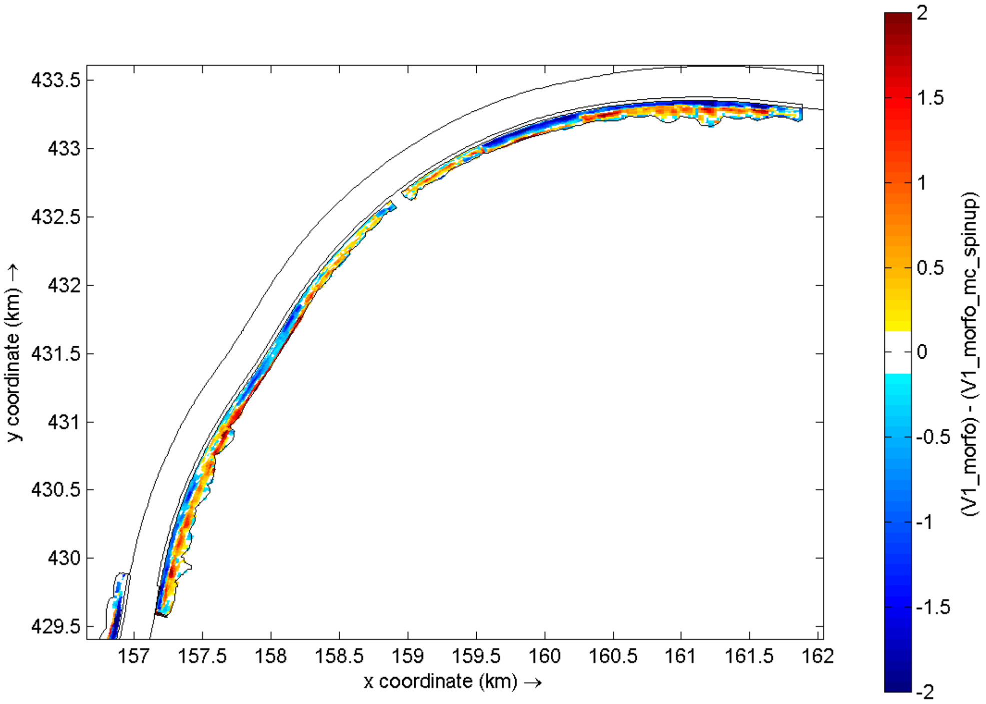
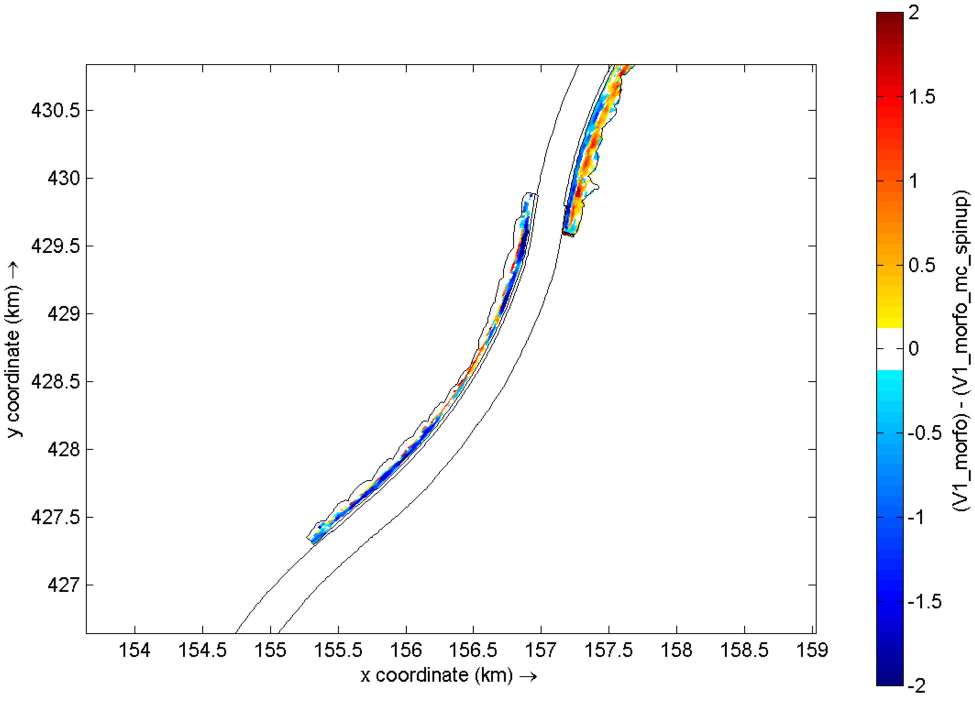


Figure 38
Difference as-built bank channel Ophemert with Baseline-j18.



Schematisation of intake sills in V1

Figure 39 and Figure 40 show the bed level for V1 at the Wamel and Dreumel intake sills, based on data in Baseline-j18. The red numbers in these figures show the as-built constructed sills, while the colours and black numbers represent the bed level in Baseline-j18.

On top of the Wamel sill the as-built bed levels and bed levels from Baseline rijn-j18_5-v1 are more or less the same. For Dreumel, there are some differences (Figure 40), but these differences are minor and the effects on the hydrodynamics / morphology will be negligible. This means that the sill-topography in Baseline rijn-j18_5-v1 represents the as-built sills. Only downstream of the sills, there appears to be some sedimentation, especially at the Dreumel sill.

For V1 the sills are not raised compared to the as built situation. Therefore, the sill geometry is taken from Baseline, which means that for V1 there is no 2D-weir at the sills. Because of the smooth bed level slopes (smaller than 1:7 slopes, see also paragraph 2.3.1 and Figure 41) at the sills in V1, no additional energy losses are modelled to account for flow separation behind the sills.

Figure 39
V1 topography at intake sill Wamel. Black numbers: Baseline rijn-j18_5-v1 (thus in simulation), red number: as-built measurement. The bed level on the background is BL-j18.

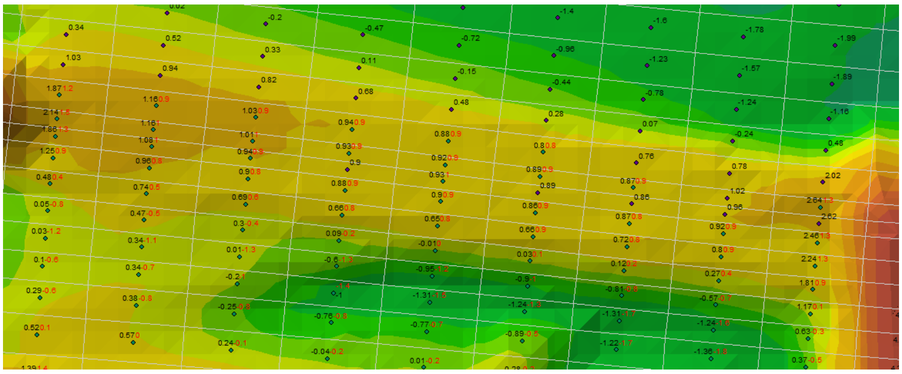


Figure 40
V1 topography at intake sill Dreumel. Black numbers: Baseline rijn-j18_5-v1 (thus in simulation), red number: as-built measurement. The bed level on the background is BL-j18.

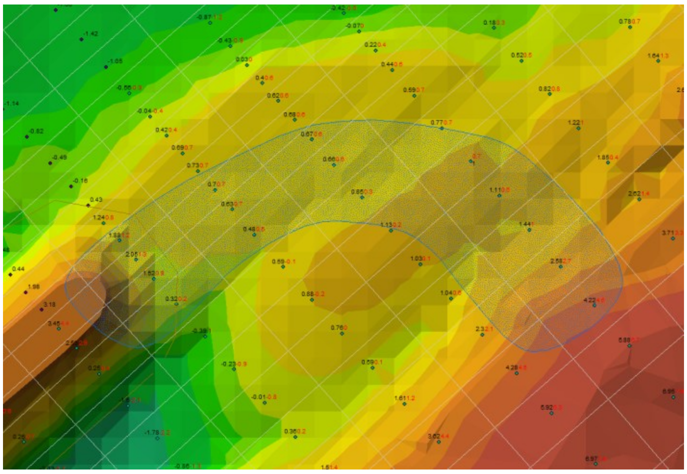
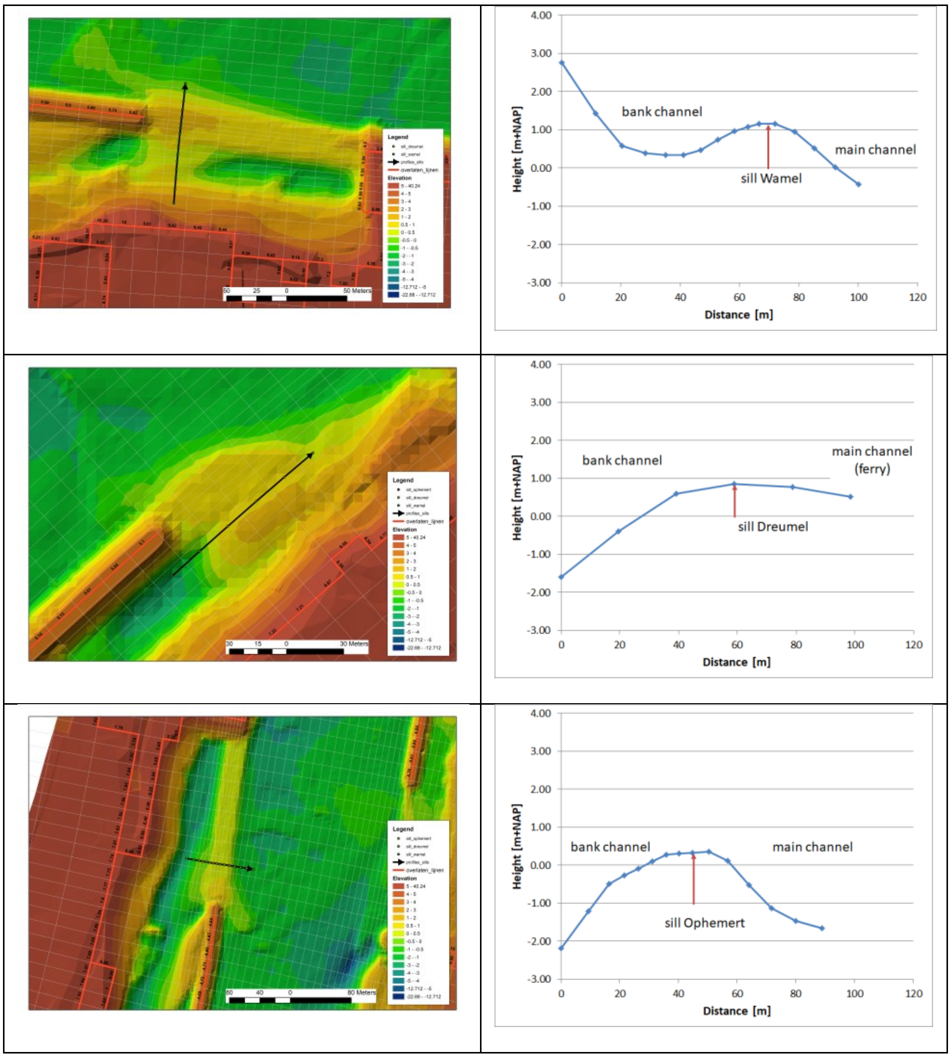


Figure 41
Bed level of sills: profiles for Wamel (top), Dreumel (middle) and Ophemert (bottom). The figures on the left give the locations of the cross-sections.



Schematisation of intake sills in V2

In variant V2 the intake sills are fully closed until the crest level of the LTW's. Apart from a weir at the crest of the sill, the sills are raised in the topography to prevent sediment from passing through the sills (2D-weirs in Delft3D do not block sediment). This is done by replicating the schematisation of the LTW's: a raised bed elevation with a low 2D weir on top of the bathymetry (cf Figure 12). The amount of bed level increase is visualised in Figure 42. Essentially this means that the schematisation of the intake sills in V2 is identical to the schematisation of the LTW's.

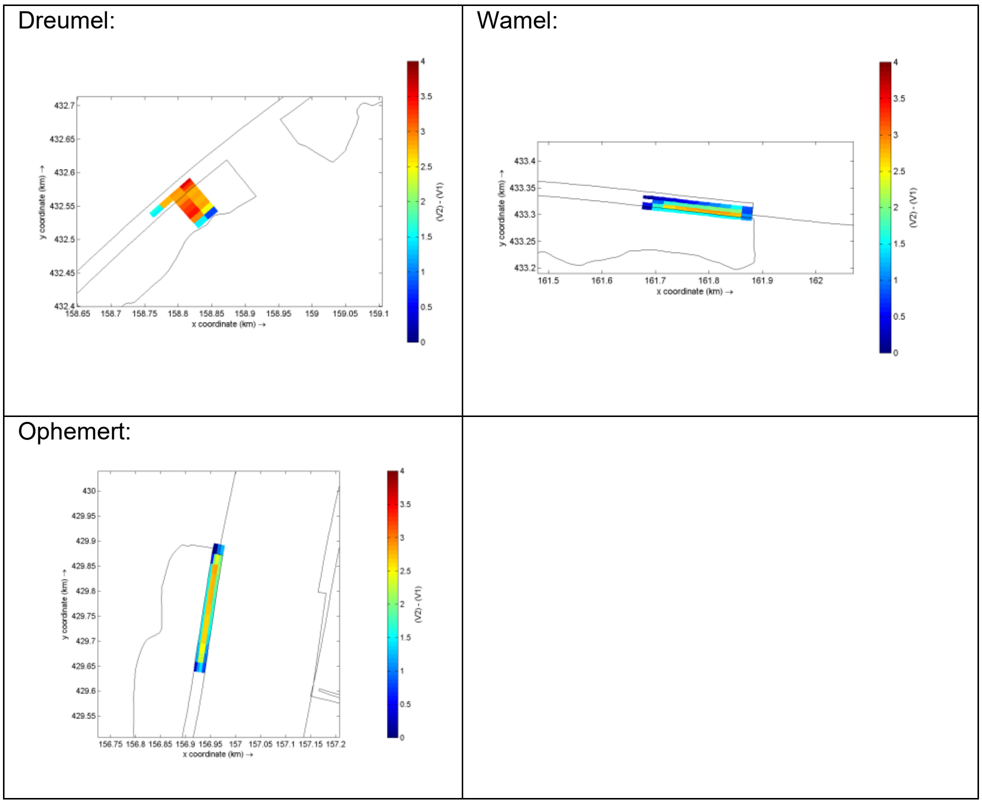


Figure 42
V2 - intakes closed in bed level by connecting them to the adjacent LTW topography. The colours represent the difference in bed level between V2 and V1. Note that there is also a weir on top of the sill (not shown).

3.4 Initialisation of the morphological simulations (Morphodynamic spin-up)

3.4.1 Approach for morphodynamic spin-up

The multibeam bed level measurements include bed elevation details relevant to a range of morphodynamic processes with different scales (e.g., groyne flames, bars, dunes, ripples). Not all these processes and scales can be captured by the model. When starting from a measured bed level, the model exhibits a rapid bed level change to reach a so-called model preferred initial state (viz. initialised bed level), which correspond to the selected sediment transport formula, the modelled discharge, model schematisation, and the model ability to capture the relevant processes. In a well-calibrated model, the deviation between measurements and the initialised bed level is not large. If the temporal scale of concern is short, this initialisation hampers the ability to carry out direct comparison between the model and prototype. However, the model ability for comparative analysis is still intact, if the initialisation step is carried out. For that, we use an approach that starts with an initialisation step (i.e., a spin-up) based on our reference case V0, which sets the initial bed conditions for all simulations, with a representative hydrograph.

The spin-up simulation helps to get rid of the initial bed changes (pulses) which may occur due to the use of 'measured bed', and it develops a 'model bed'. As the model is not perfectly matching the measurements this is essential. For the spin-up, we use a steady discharge, because it provides a smooth bed that fits very well to the model schematisation. Without a proper spin-up, a morphodynamic simulation with a dynamic discharge (hydrograph) will show pulses that travel through the system with ~1 km per year. If the start condition is polluted with such pulses, the actual simulation will give a lot of disturbances that prevent from distinguishing the impacts of the LTW's from the changes in propagation of these relicts. In the model with LTW's these waves will have a different speed than in the reference. So, after some years the difference plots will become very difficult to interpret. Instead, using a spinned-up bed (using steady discharge, so a steady state), which means that most disturbances (differences compare before and after LTW-construction) can be attributed to the schematisation differences.

For the spin-up simulation, we us a constant discharge of QLobith = 2250 m^3/s (as was done by Omer et al 2019b). This is a Waal discharge of 1643 m^3/s. Note that:

- This discharge is in good agreement with the so-called 'channel-forming discharge' for the bed slope found in the IRM (= Integraal Riviermanagement), the "Quickscan Rivierbodemligging" and research by Blom et al (2017).
- For this discharge the lowered groynes are submerged.

The spin-up simulation is done for our reference case being the situation before LTW-construction (V0), without dredging and dumping active. By following the strategy to use a spinned-up bed level in the main channel to assess the effect of the LTW's, the (probable) sediment extraction in the building period is not influencing the assessment (this aspect is briefly studied by comparing volume changes as an effect of the LTW's to measured volume changes in WP10).

From the spin-up simulation we use the main channel bed elevation (between 'normaallijnen') and sediment thickness after 5 years as initial state for the morphological simulations with a hydrograph.

3.4.2 Relevant model details

First, some remarks on the schematisation (which also hold for simulations with hydrograph presented in the next sections):

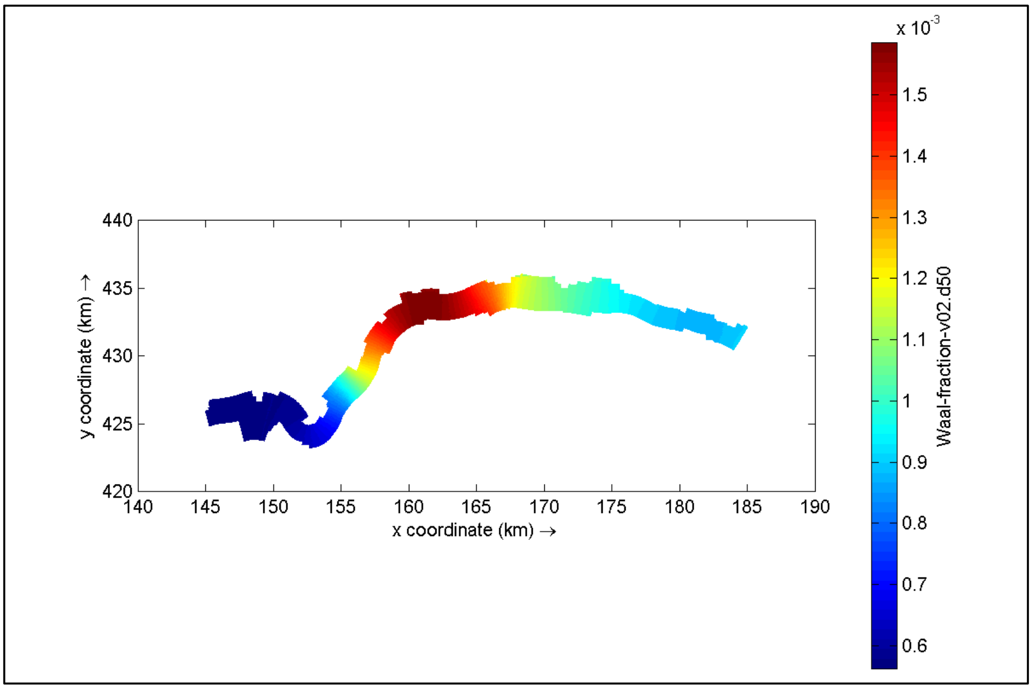
- The spatial distribution of D50 is shown in Figure 43.
- Only the main channel is alluvial, see Figure 44 as example for V0 (lines are weirs).
- In addition to V0, for both V1 and V2, the LTW's, bank channel and intake sills are fixed by excluding these parts of the schematisation from the alluvial part. Effectively this means specifying a (near) zero sediment layer thickness in these parts. This is illustrated in Figure 45.
- Roughness along the model: see Figure 46 for left and right side of main channel.

- The fixed layer at St. Andries has different roughness definitions in the inner and outer bend:

  ```
  # vaste laag St. Andries;
  132  101  0.0400  2.5;
  # binnenbocht vaste laag St. Andries;
  # alfa = 1.37 * alfa traject 624;
  672  101  0.1388  2.5;
  ```

  Thus, the inner (right side) bend is rougher (A=0,1388) than the outer (left side) bend (= fixed layer) (A=0,04), see also Figure 46. From the roughness definition file we obtain: code 101 means Nikuradse roughness height formulation according to Van Rijn (k = A h^0.7 * {1 – exp (-B h^(-0.3))}. At the outer bend (left side), the roughness of the fixed layer is reduced to the same roughness value of the alluvial upstream and downstream of the fixed layer. This is to prevent unrealistic bed changes at this outer bend.
- We use a Total Discharge at the upstream boundary (thus not per cell), which is no problem because of the applied fixed upstream bed level boundary condition (so no degradation imposed).

Figure 43
Spatial grain size distribution (D50 in m).



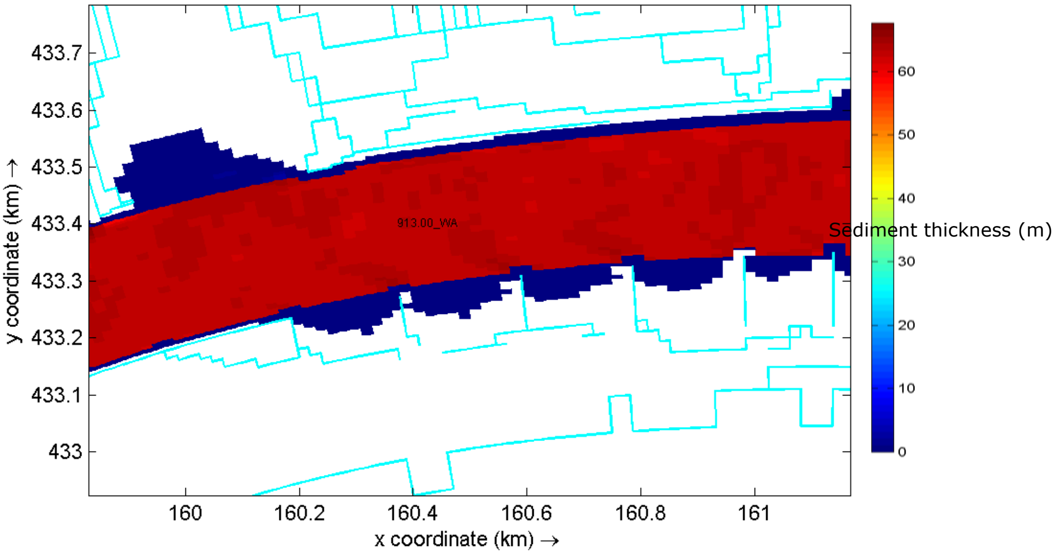


Figure 44
Initial sediment thickness, cyan lines are weirs. Sediment thickness is in m.

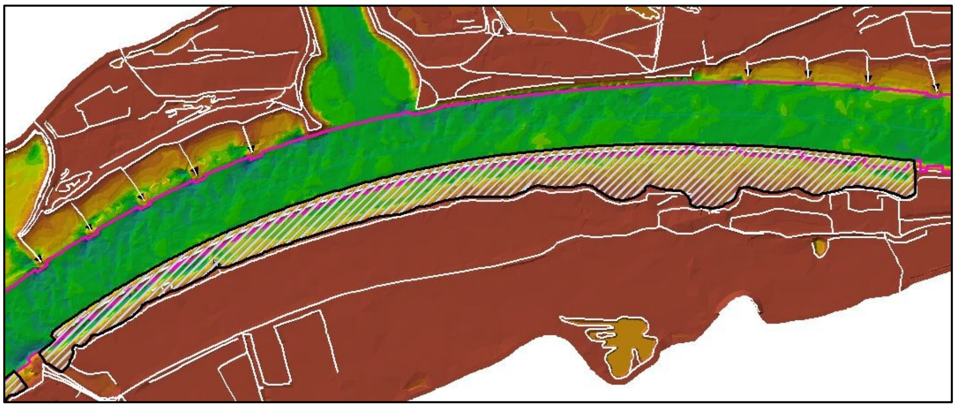


Figure 45
Adaptation of initial sediment thickness for V1 and V2 at Wamel bank channel. Background color is the bed level. White lines are weirs. Pink lines are the alluvial part in V0. The part added to the non-alluvial part is indicated as black polygon with white diagonal lines.

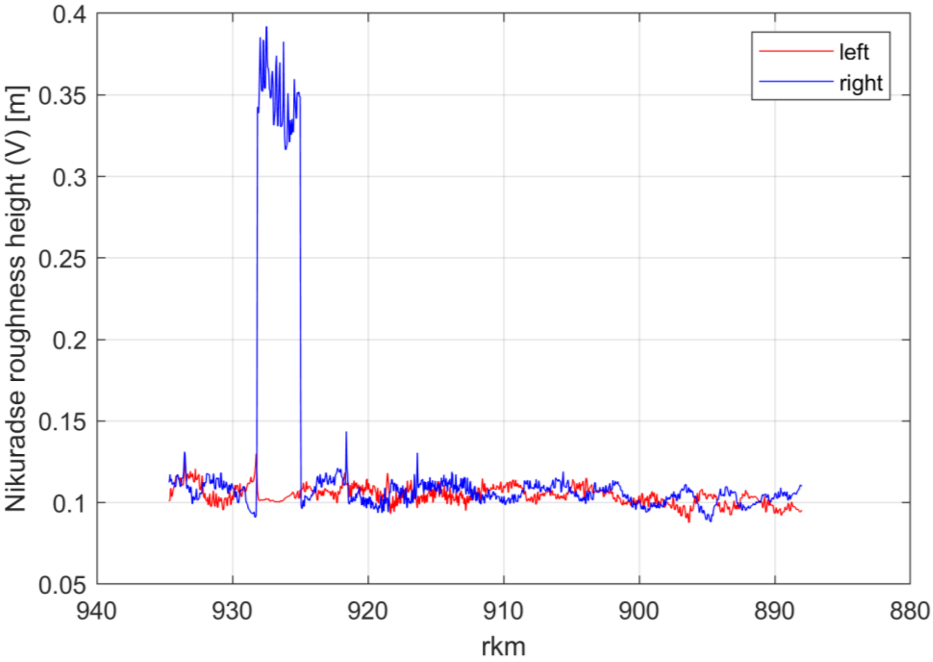


Figure 46
Nikuradse roughness height (V-direction) on left/right side of main channel. The inner bend (right) at the fixed layer (left) of St. Andries has a clearly different roughness from the fixed layer itself in the outer bend.

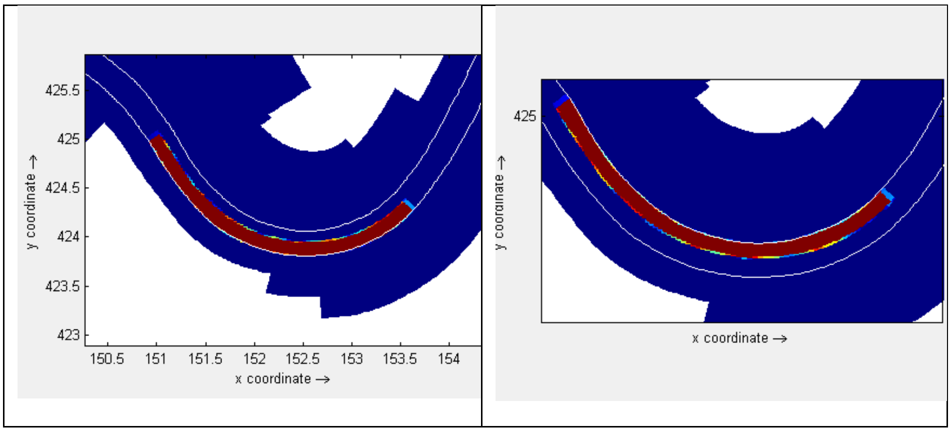


Figure 47
Fixed layer St. Andries. Left: fixed layer outer bend, code 132. Right: inner bend, code 672.

Figure 48 shows the simulated water level for the spin-up simulation at Qlobith 2250 m^3/s (Qwaal 1643 m^3/s). This water level is above the crest level of the (lowered) groynes. The groyne height is the crest height of the groynes in Baseline at the river side.

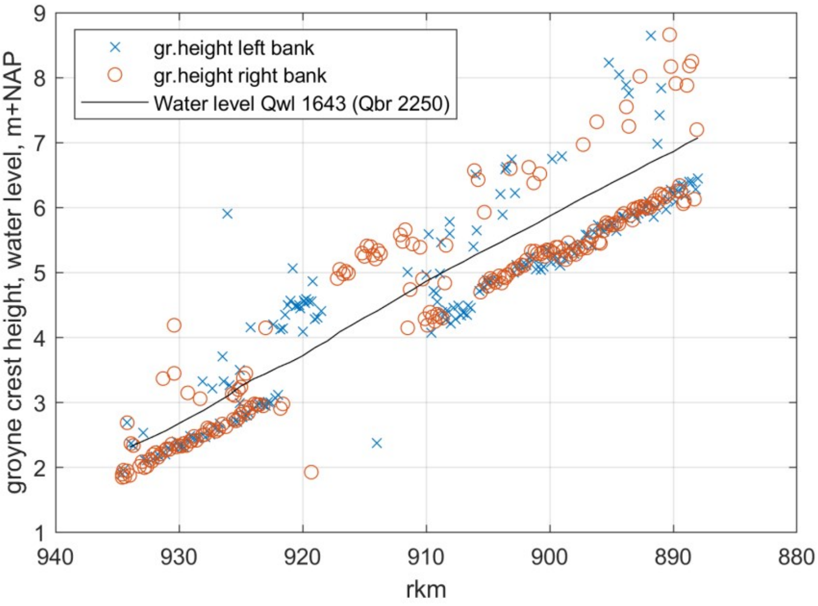


Figure 48
Groyne height in V0 (and thus also in V1/V2 outside area of LTW's) and water level for spin-up simulation. The groyne heights are the crest heights of the Baseline groynes on the river side on each bank.

3.4.3 Results morphodynamic spin-up

The result of the model initialisation step is demonstrated in the following figures, which give an overview of the morphological changes in the spin-up simulation (i.e., V0 with constant discharge).

Figure 49 and Figure 50 show the bed level time-evolution after 1, 5 and 10 years at the left bank, right bank and cross-section averaged. The three horizontal lines are the locations of the LTW's, the vertical dash-dotted lines represent the location of the fixed layer at St. Andries (km 925-928).

Initially the bed level comes from a direct (multi-beam) measurement, which contains small-scale features, especially in the area with a refined grid at the

LTW's. These features are quickly smoothened in the model (since small-scale features are not resolved by the model). After 5 years the bar/pool pattern is stabilised (after 10 years almost no difference).

Figure 49
Bed level after 1, 5, and 10 years of steady discharge for V0. Top: left bank (m=52), centre: right bank (m=61), bottom: cross-section averaged (m=47..67). The black thick horizontal lines are the LTW's (not in this sim, but for reference). The fixed layer St. Andries is located at the vertical dash-dotted lines. The grey line is the initial bed level.

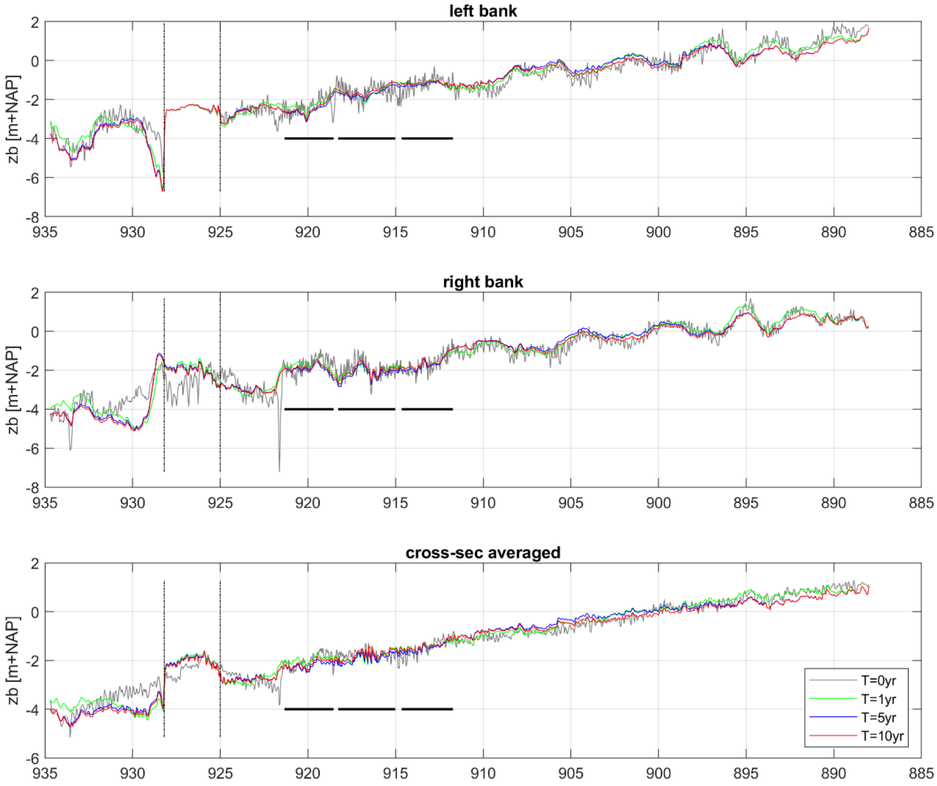


Figure 50
Bed level after 0 (initial), 1, 5, and 10 years of steady discharge for V0. In each subplot, the left and right bank are shown. The black thick horizontal lines are the LTW's (not in this sim, but for reference). The fixed layer St. Andries is located at the vertical dash-dotted lines.

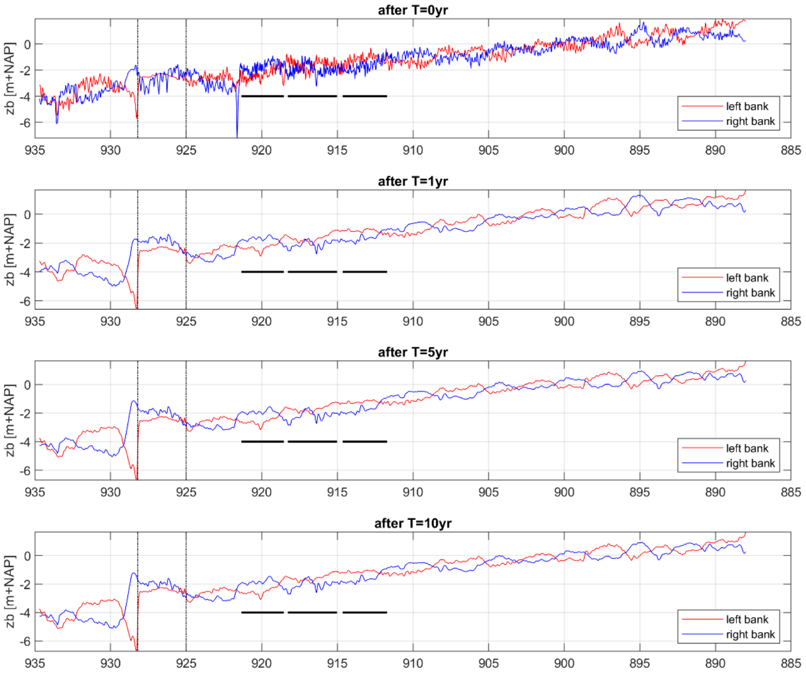


Figure 51 shows the bed level changes after 5 years w.r.t. the initial condition. Figure 52 shows the difference after 1 and 10 years w.r.t. the bed level after 5 years. From these figures we observe:

- The pattern of large-scale erosion / sedimentation resembles the spin-up simulation done by Omer et al. (2019b) to a large extent (compare Figure 51 and Figure 53);
- There is some erosion on the left bank in the upstream part of the model;
- The large peak at km 922, just downstream of the Ophemert LTW, is probably due to filling of a scour hole;
- There is sedimentation of the inner bend at the fixed layer of St. Andries and erosion directly downstream of the fixed layer (note that this simulation is without dredging and dumping);
- After 5 years there are only bed level changes at very large spatial scale with wave lengths in the order of 30 km (Figure 52).

Based on these results, we chose to use the bed level after 5 years as initialised bed level for the remainder of the analysis. This means that we use the results after 5 years as initial condition for simulations with a representative hydrograph: i.e., bed level and sediment thickness (sdb) between ‘normaallijnen’.

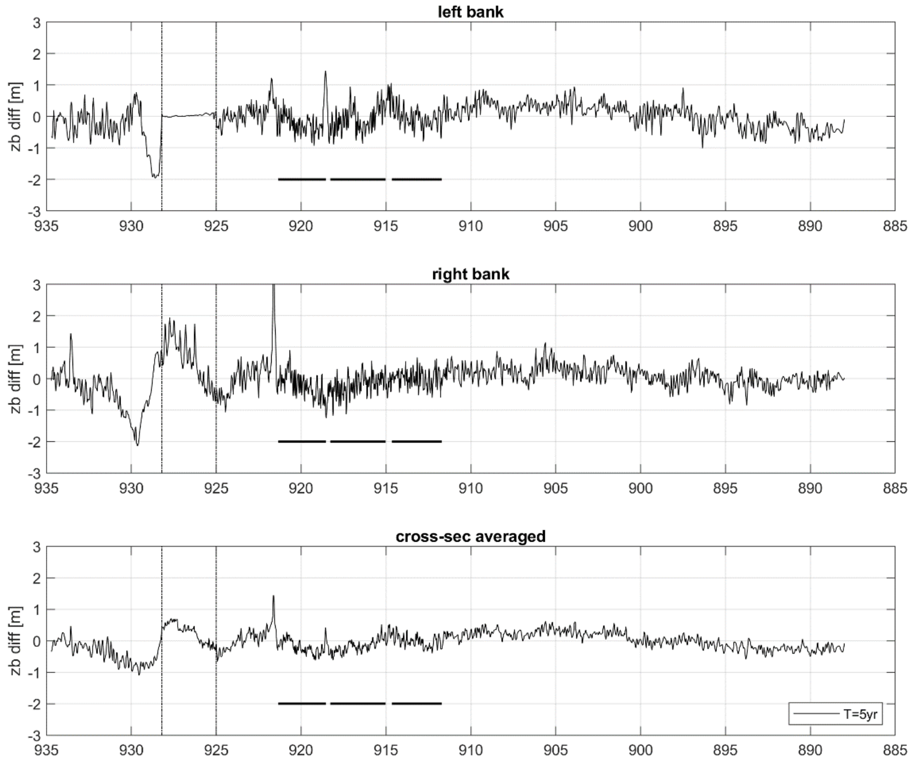


Figure 51
Bed level after 5 years of steady discharge for V0 w.r.t. initial condition. Top: left bank (m=52), centre: right bank (m=61), bottom: cross-section averaged (m=47..67). The black thick horizontal lines are the LTW’s (not in this sim, but for reference). The fixed layer St. Andries is located at the vertical dash-dotted lines.

Figure 52
Bed level difference in spin-up simulation after T=1 and T=10 years, w.r.t. bed level after 5 years.

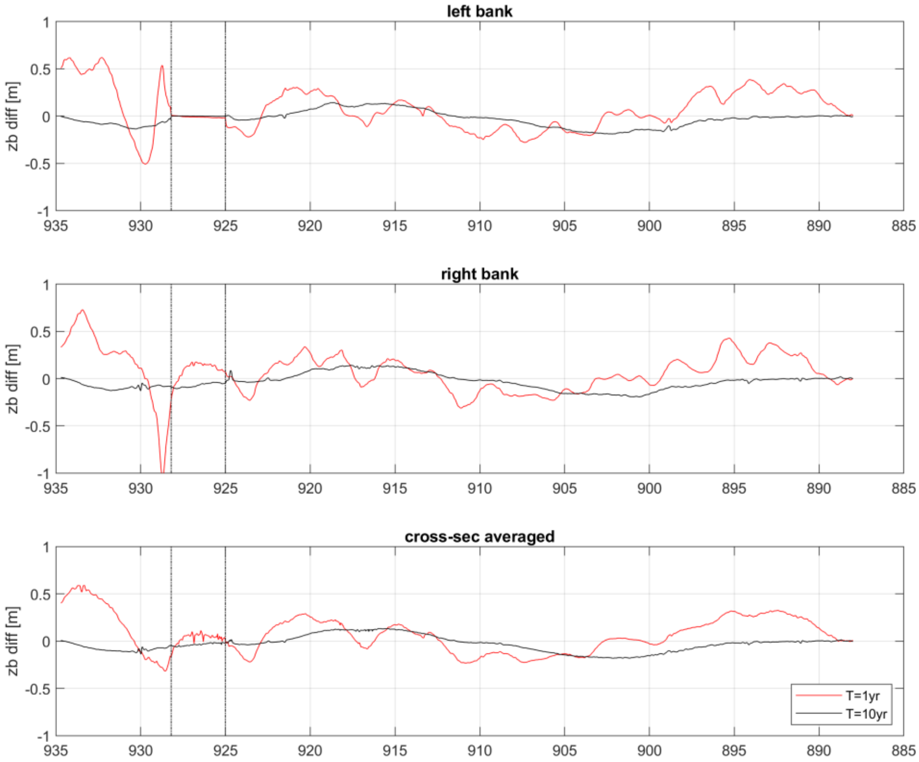
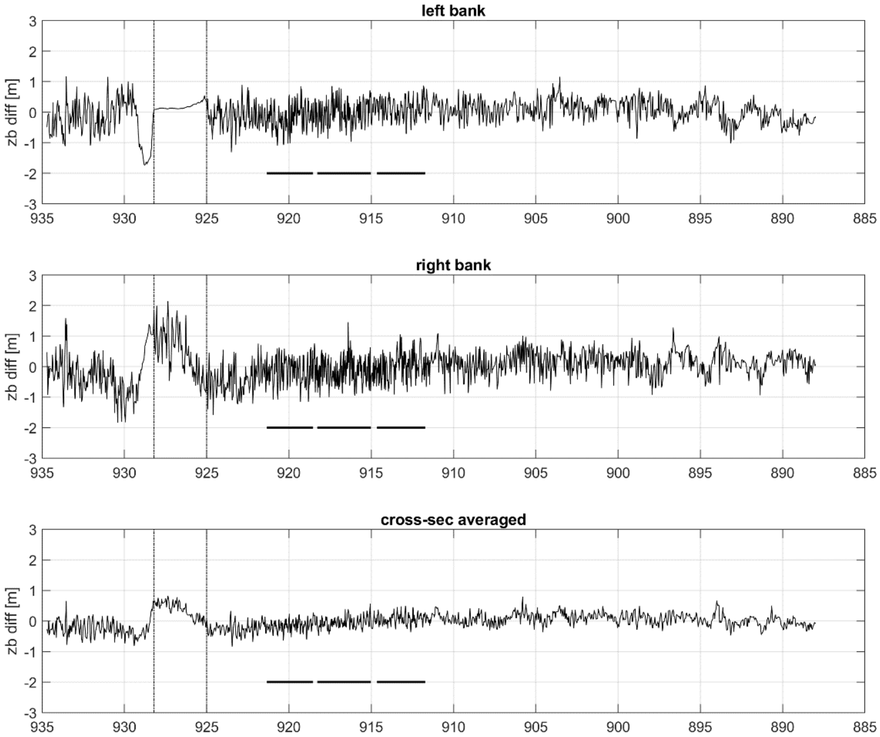


Figure 53
Same as Figure 51, but from spin-up simulation of Omer et al (2019b).



3.5 Results with hydrograph **with** active dredging and dumping

3.5.1 Hydraulic spin-up

Before morphological simulations with a hydrograph are started, a 'local database' with stable flow fields for each discharge is generated to ensure that the simulations start with a correct hydraulic initial condition. For most discharge levels, the 'convergence' is perfect, with only a few m^3/s of difference in discharge through different cross-sections in the model. Figure 54 and Figure 55 show two examples where there is some variation, but a longer simulation does not prevent this. This is considered to be no problem for the morphological simulations, because morphological changes will influence water levels anyhow.

Figure 54
V0, Q6151: time-evolution of discharge (left) and water level (right) for hydrodynamic spin-up. The star represents the water level from the "betrekkingslijnen".

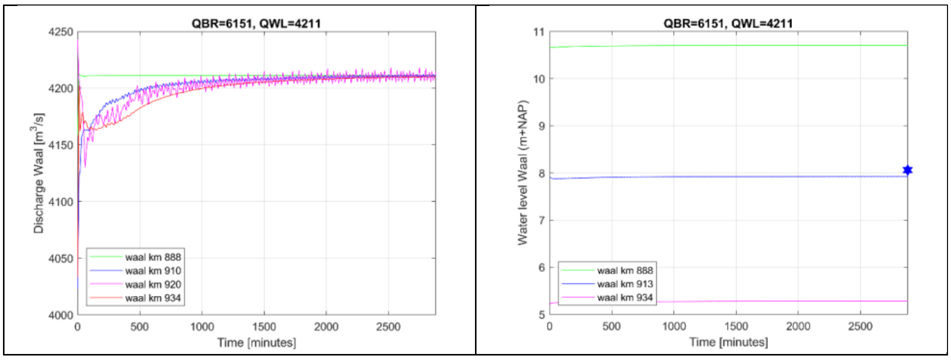
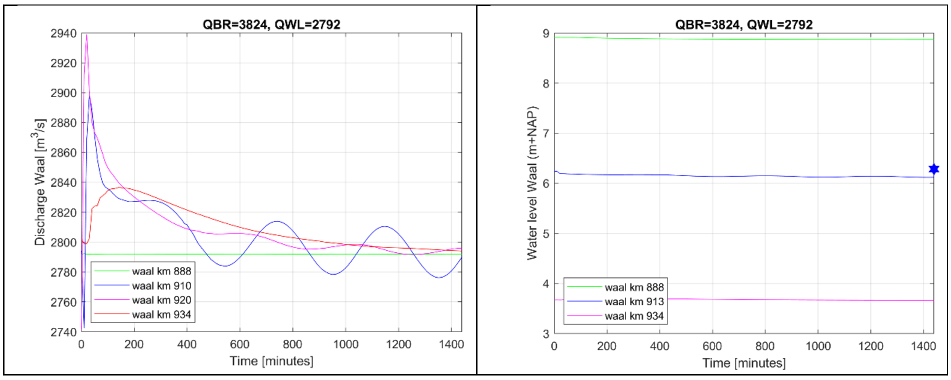


Figure 55
V1, Q3824: time-evolution of discharge (left) and water level (right) for hydrodynamic spin-up. The star represents the water level from the "betrekkingslijnen".



3.5.2 Morphological development V0, V1, V2

Water level changes in first year for V0, V1 and V2

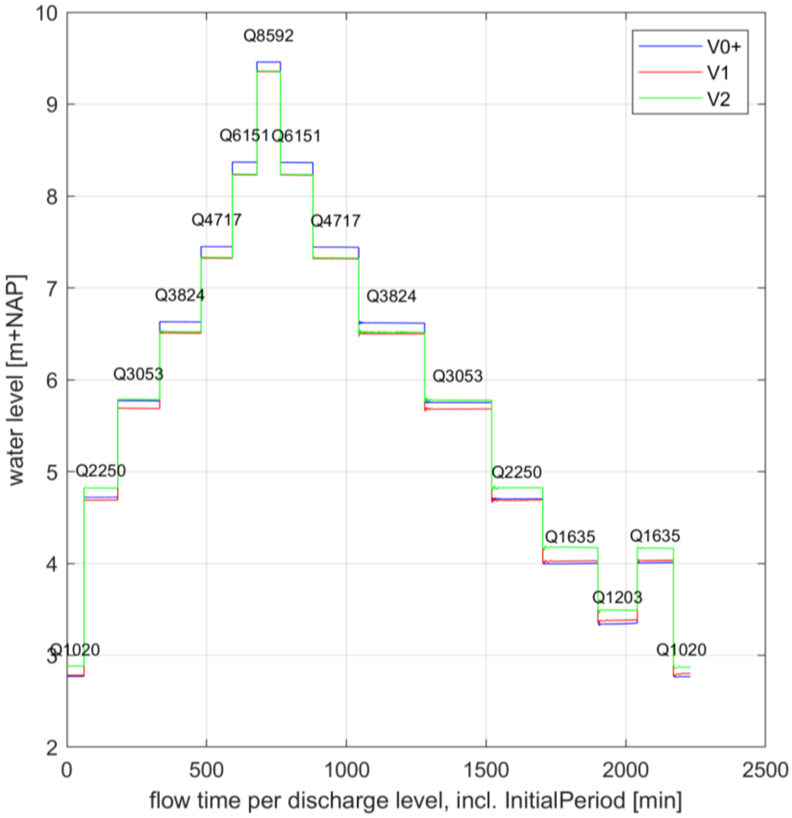
The hydrograph (Qseries) is a representative yearly hydrograph (Figure 35), which is repeated each year, such that 20 years are morphologically simulated. After each year, the OLR (reference plane for dredging and dumping) is updated by running a simulation at OLA=1020 m^3/s (Lobith) without updating the bed level / bed composition. Note that this might lead to deviations from OLR (2012), but it reflects the effect of morphological changes on dredging through the yearly updated reference plane.

Figure 56 shows the water level (on hydraulic time scale) just upstream of the area of the LTW's (km 910) for the first year of the simulation.

We can observe from this figure that:

- V1 gives slightly higher water levels at low flow and lower water levels at high flow compared to V0, which is what we would expect because of the narrowed main channel at low flow and increased discharge capacity at high flow.
- V2 gives higher water levels than V1 at low discharge, which is what we would expect since the bank channels are not conveying water at low flow.

Figure 56
Water level at km 910, just upstream of the LTW-area during the first year of hydrograph (time is on hydraulic time scale and it includes spin-up time (InitialPeriod)). The label is the Lobith discharge.



Bed level differences: V1 compared to V0

Figure 57 and Figure 58 show 2D-plots of the bed level difference between V1 and V0. Note that the bank channel is fixed and the "changes" we see here are simply because of the channel being constructed. With respect to morphological changes in the main channel we observe:

- Some minor changes outside of the LTW area (mostly upstream), which are a result of (i) the method to replace the spinned-up main channel topography in the model, (ii) different morphological behaviour because of the upstream water level effect of the LTW's (back water curve) and (iii) dredging/dumping differences related to (ii).
- After the first year a large scour near the exit of the Ophemert channel.
- A scour zone downstream of the LTW's, extending downstream of the fixed layer of St. Andries.

Figure 57
Bed level difference V1– V0) after 5 years of morphological development, complete model area.

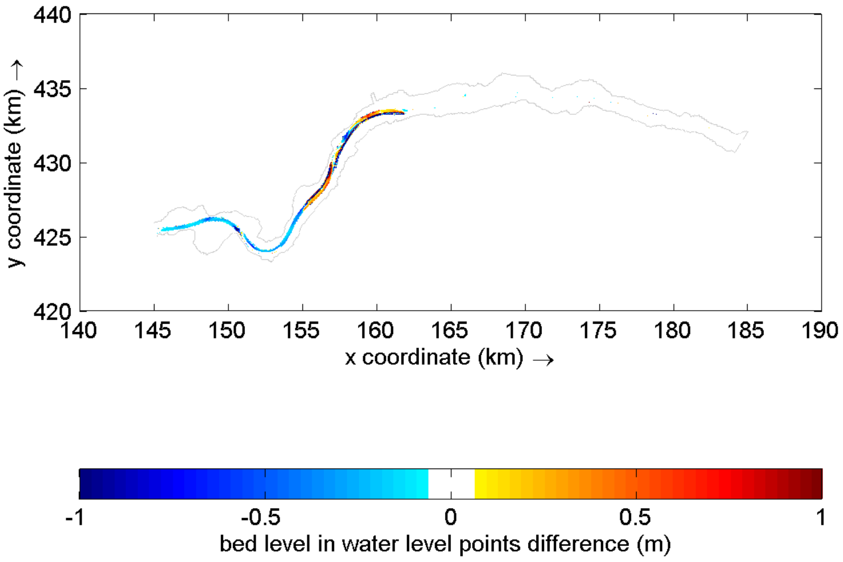


Figure 58
Bed level difference (V1 – V0) after 1 year of morphological development, focus on LTW area (for colour scale, see Figure 57). Note that the dark blue areas are not 'real' morphological changes, but represent the construction of the bank channels.

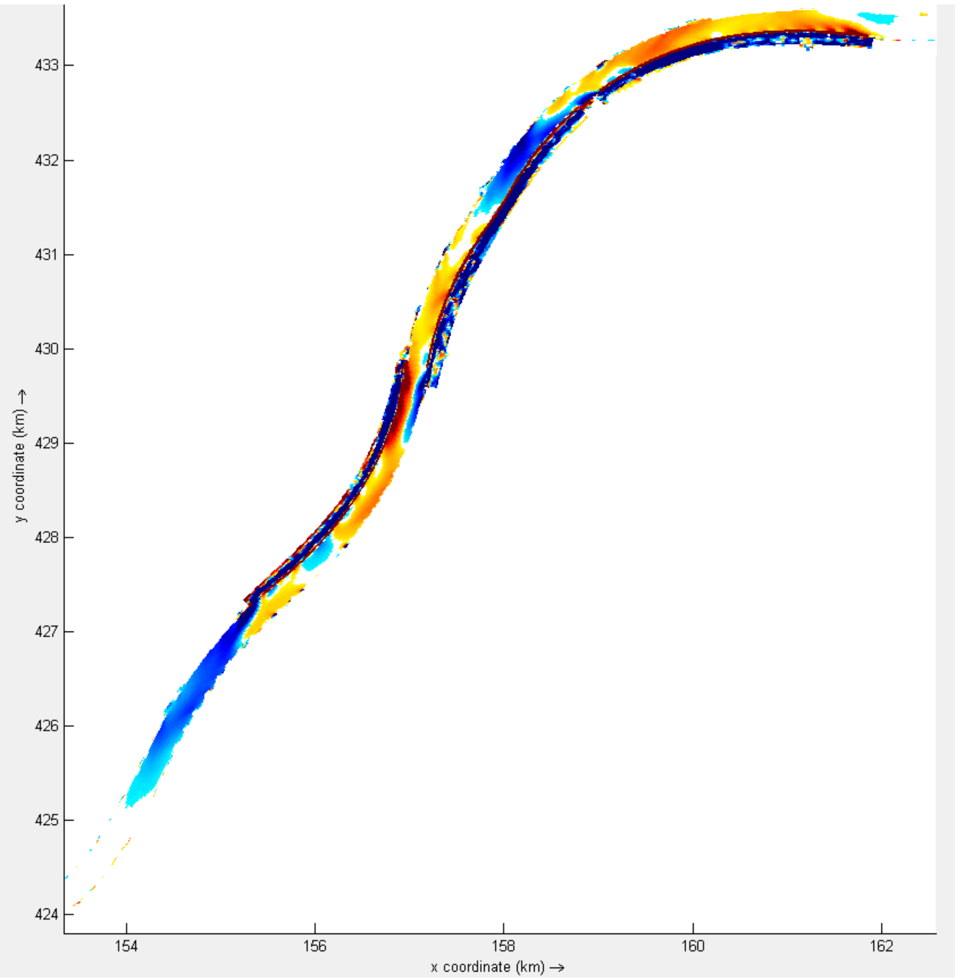


Figure 59 (whole model) and Figure 60 (LTW area and downstream) show 1D-plots of the bed level difference between V1 and V0 in the main channel. With respect to morphological changes we observe:

- Sedimentation in main channel at Wamel and Ophemert LTW.
- Erosion in main channel at Dreumel LTW.
- Scour in main channel downstream of LTW's.
- Quite some morphological changes in main channel downstream of fixed layer at St. Andries.

We note that an extended analysis of the morphological results is carried out in WP7 (Van der Mark and van der Wijk, 2021) and WP10 (Chavarrías et al, 2021).

Bed level differences: V2 compared to V0

Figure 61 (whole model) and Figure 62 (LTW area and downstream) show 1D-plots of the bed level difference between V2 and V0 (V2 – V0). From these figures we observe:

- On the left bank, the sedimentation for Wamel is concentrated more downstream compared to V1.

On the centre and right bank, there is a lot more sedimentation downstream of the Ophemert LTW, because the sediment transport capacity in the main channel at the Ophemert dam increases and decreases again downstream of the LTW's.

Bed level differences: V2 compared to V1

Figure 63 and Figure 64 show the bed level difference of V2 compared to V1. At the LTW's there is generally more erosion in V2 than in V1 because of the closed bank channel. Downstream of the LTW's this is compensated by more aggradation in V2 than in V1, since the sediment that erodes in the LTW-area is deposited downstream of it (except on the fixed layer).

Figure 59
*Bed level difference between **V1** and **V0** at left bank, centre of channel and right bank, whole model.*

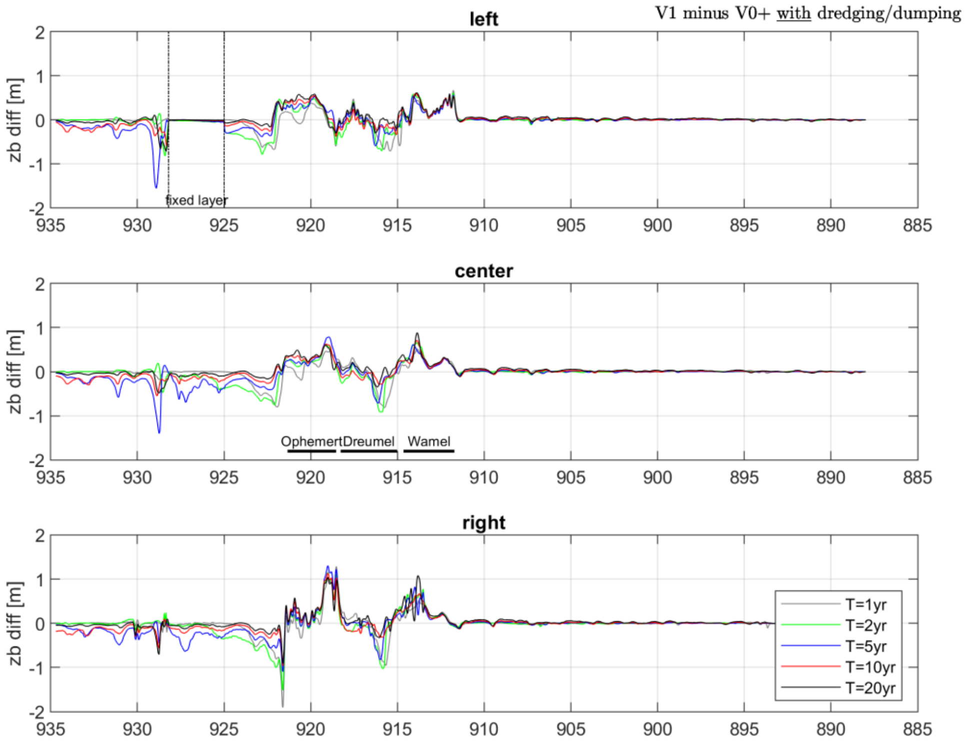


Figure 60
*Bed level difference between **V1** and **V0** at left bank, centre of channel and right bank, **LTW-area** and downstream.*

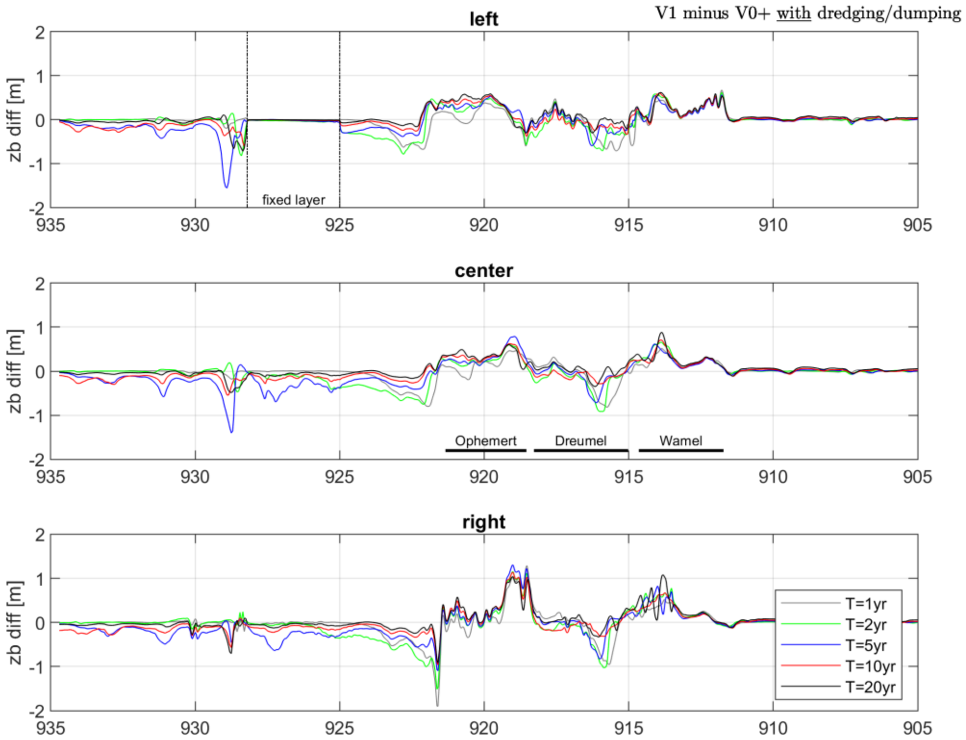


Figure 61
*Bed level difference between **V2** and **V0** at left bank, centre of channel and right bank.*

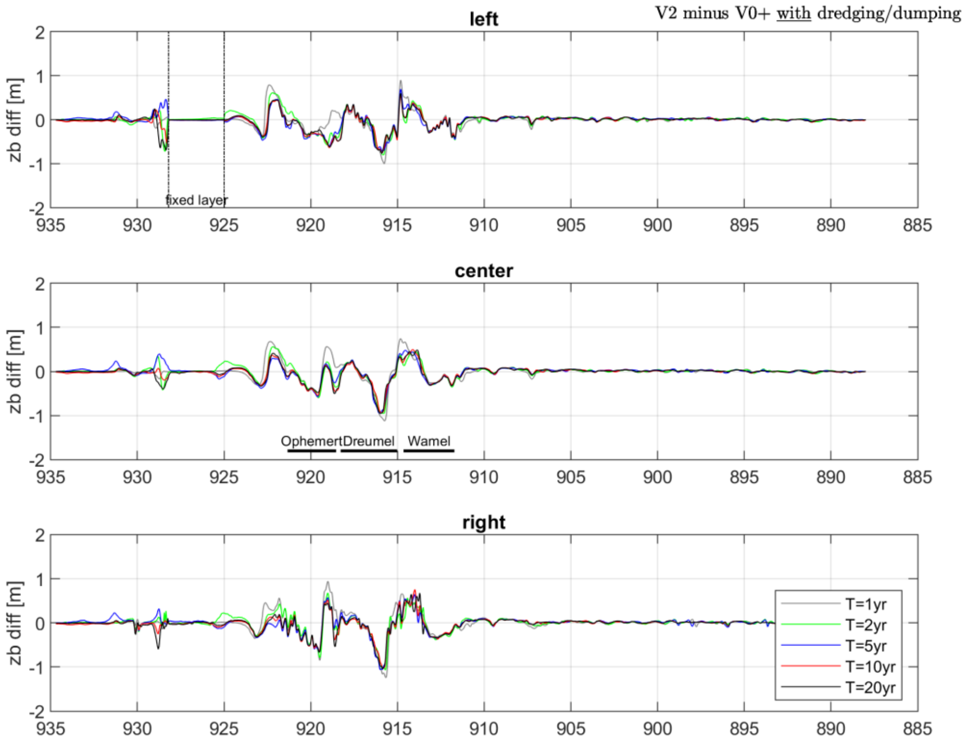


Figure 62
*Bed level difference between **V2** and **V0** at left bank, centre of channel and right bank, **LTW-area** and downstream.*

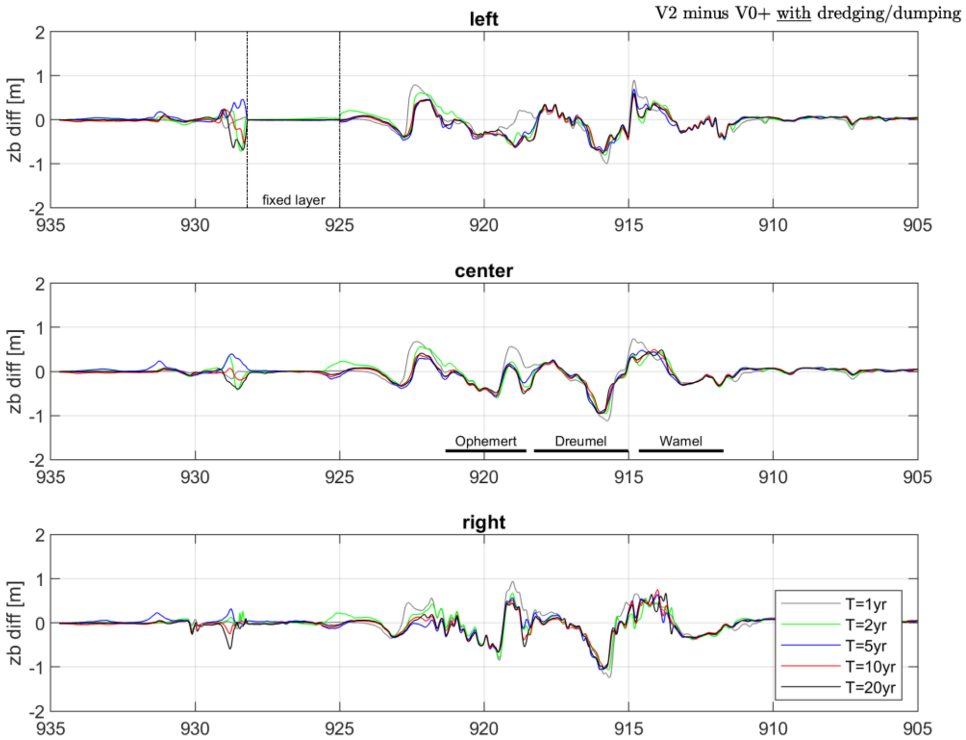


Figure 63
*Bed level difference between V**2** and **V1** at left bank, centre of channel and right bank.*

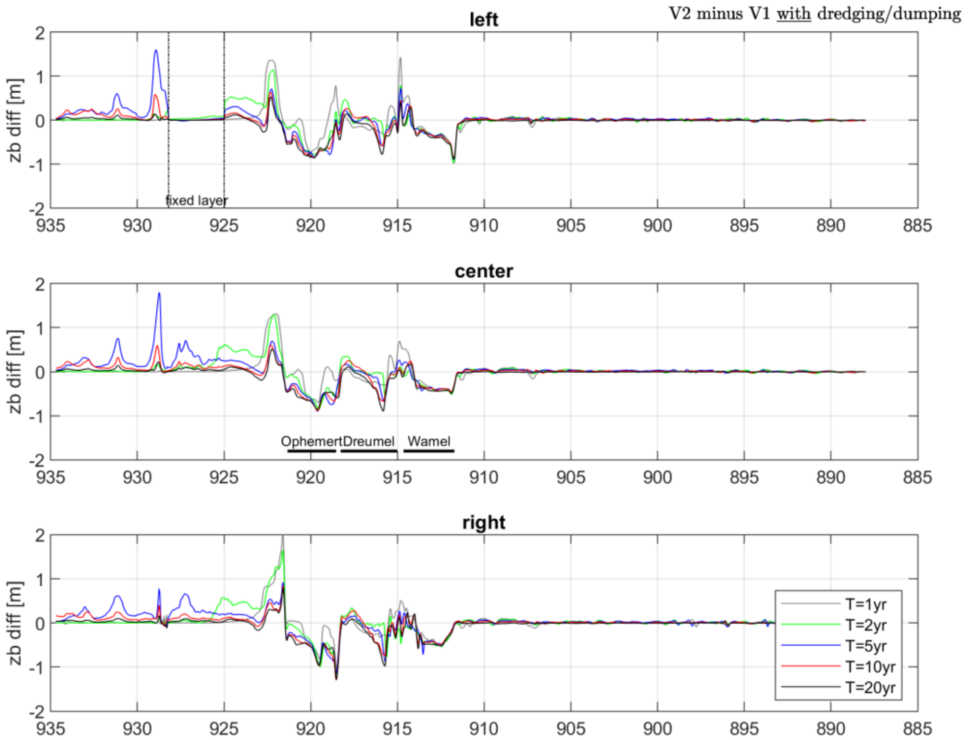
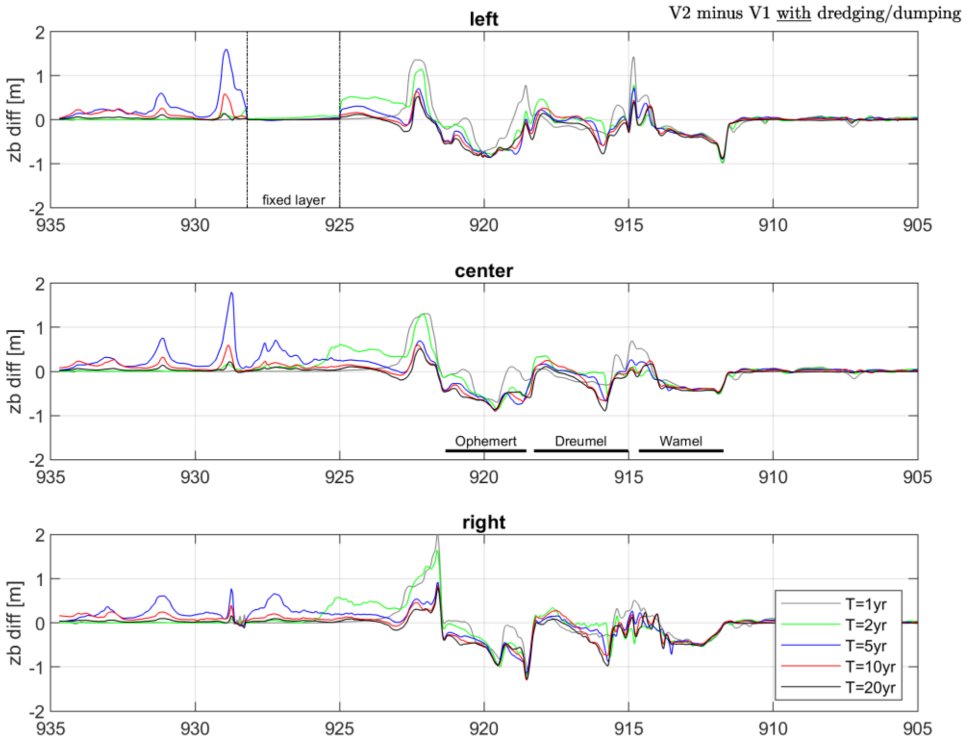


Figure 64
*Bed level difference between **V2** and **V1** at left bank, centre of channel and right bank, **LTW-area** and downstream.*



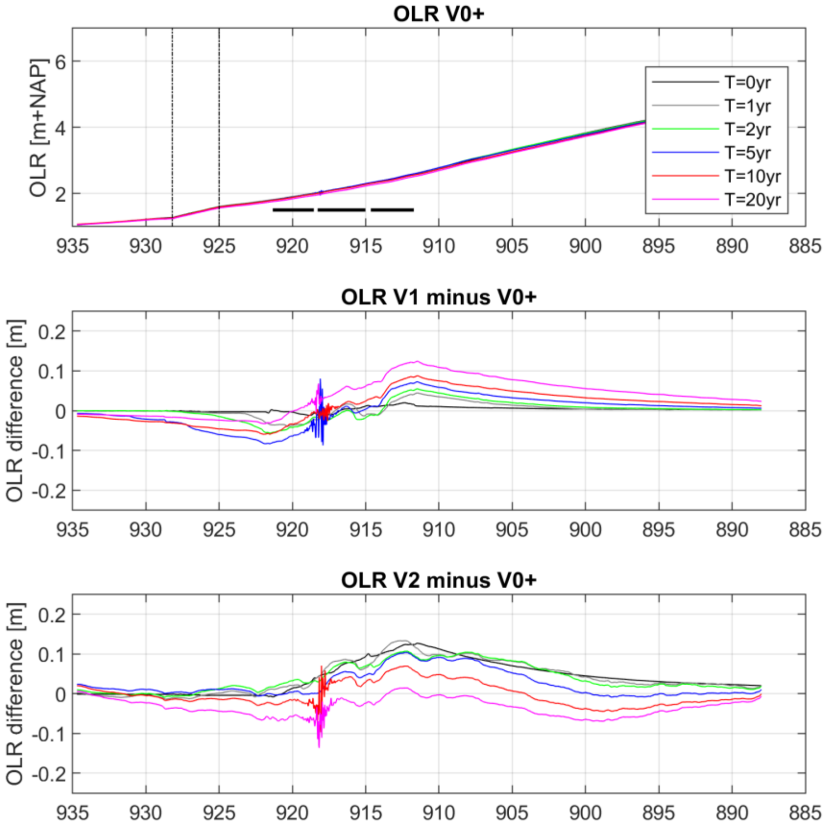
3.5.3 Dredging/dumping quantities

Details on the dredging/dumping settings can be found in paragraph 3.3.2.

Figure 65 shows the initial and time development of the OLR reference plane during the simulations.

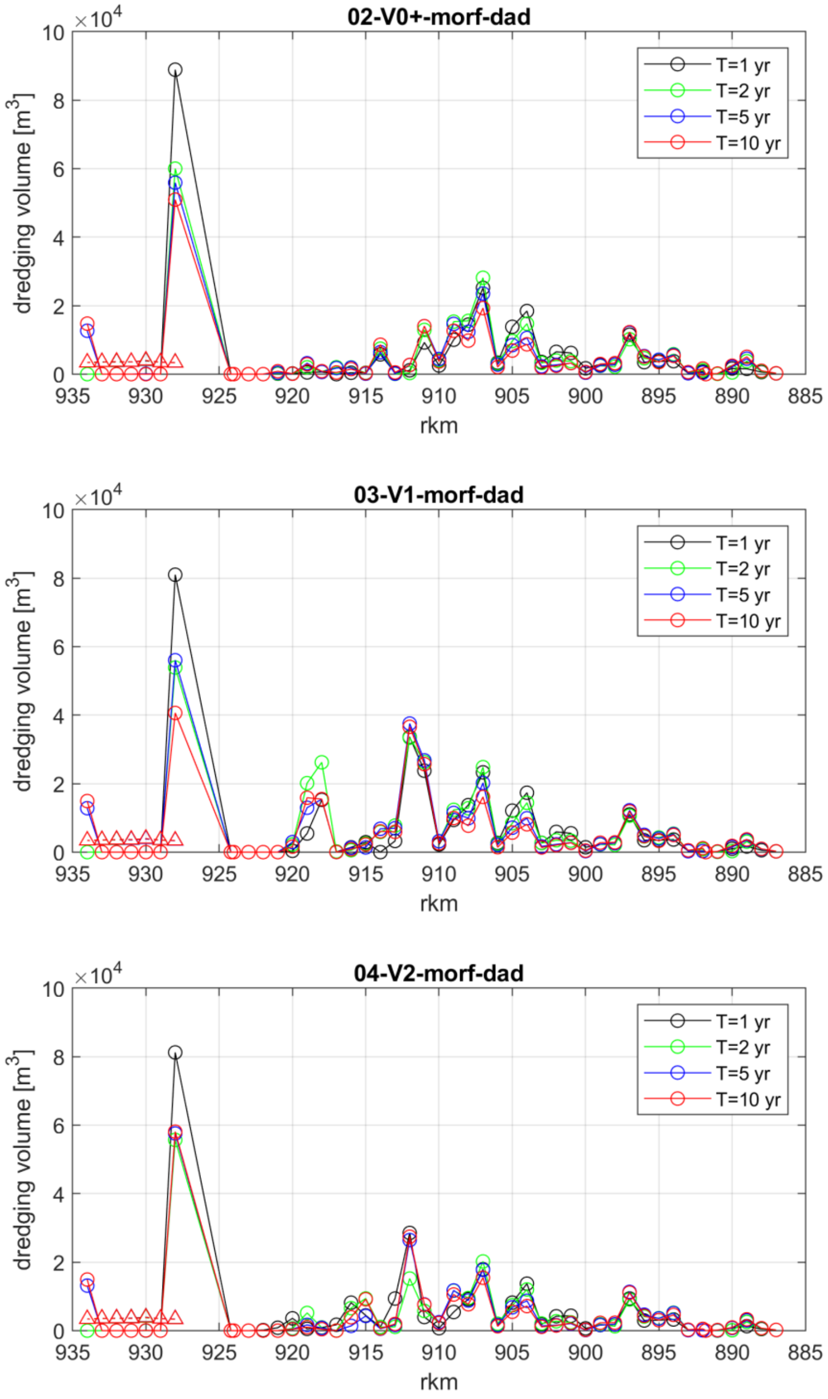
- For V2 we clearly see the effect of closure of the channels and related increase of OLR;
- For V1, initially, the OLR is almost identical compared to V0. This is slightly different from the hydrodynamic simulations (Figure 21, Q=1020 m^3/s). For those simulations, the initial water level increase is circa 3 cm. Probably this is because of the different topography in the bank channels, which are deeper in the morphological simulations (Figure 37, Figure 38);
- For V0 and V1 we see oscillations in the water level / reference plane. This might be related to the short initial period of 60 minutes, which is needed to have acceptable simulation times;
- For V1 and V2 we clearly see that the OLR is influenced by the morphological development of the main channel.

Figure 65
OLR reference plane for V0, V1 and V2 in top panel (lines on top of each other). Centre/bottom: difference in OLR for V1 and V2 compared to V0.



From Figure 66 we can observe that the dredging volumes generally increase in the area of the LTW’s for V1 but appear to decrease for V2. The latter might be related to the increased OLR reference plane, both in the area of the LTW’s, as upstream in the Waal. Dredging and dumping quantities are extensively analysed and interpreted in WP7, see Van der Mark and van der Wijk (2021).

Figure 66
Dredging quantities, summed per km for each year. Top V0, centre V1, bottom V2. Sand mining is shown with triangle symbols (between km 928 and 934).



3.6 Results with hydrograph **without** dredging and dumping activated

The simulations presented in the previous paragraph are also performed without activating the dredging and dumping module in Delft3D. Except for switching off dredging and dumping the settings of the simulations are identical.

Figure 67 and Figure 68 show the bed level differences of V1 and V2 compared to V0, respectively. Figure 69 shows the bed level differences of V2 compared to V1. The bed level differences are similar to those for the simulations with dredging and dumping activated.

For detailed analysis and interpretation of the bed level changes, the reader is referred to Chavarrías et al (2021).

Figure 67
*Bed level difference between **V1** and **V0** at left bank, centre of channel and right bank, LTW-area and downstream, simulation **without** dredging/dumping.*

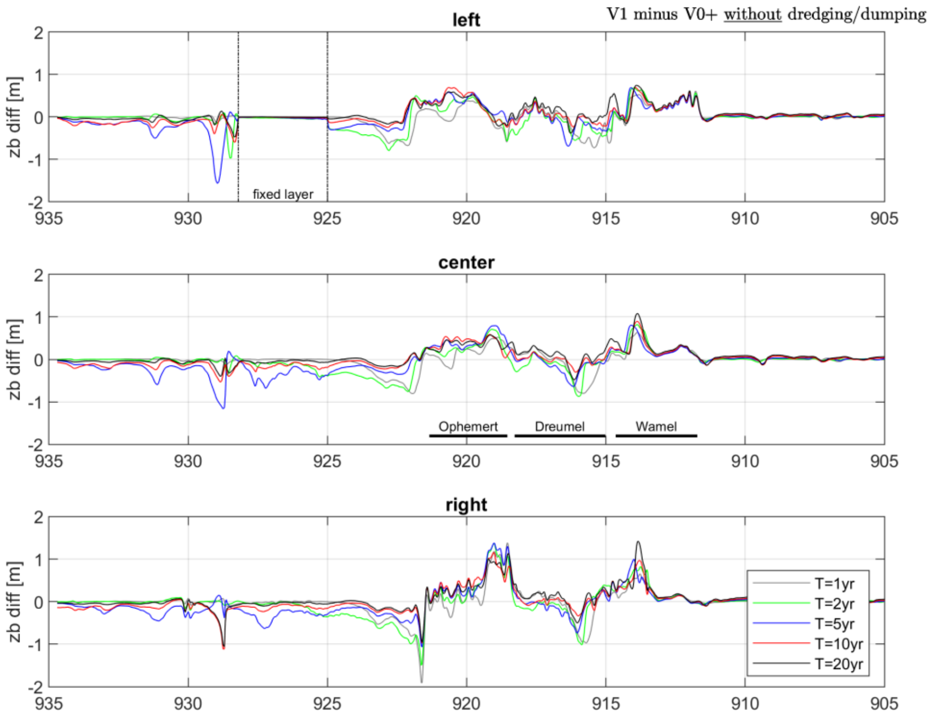


Figure 68
*Bed level difference between **V2** and **V0** at left bank, centre of channel and right bank, LTW-area and downstream, simulation **without** dredging/dumping.*

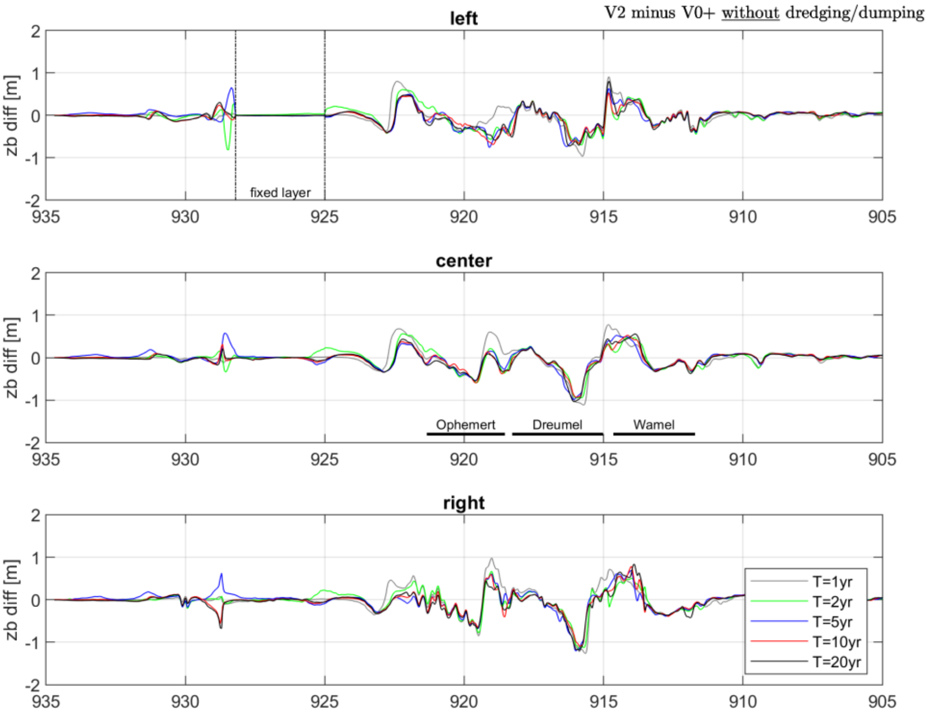
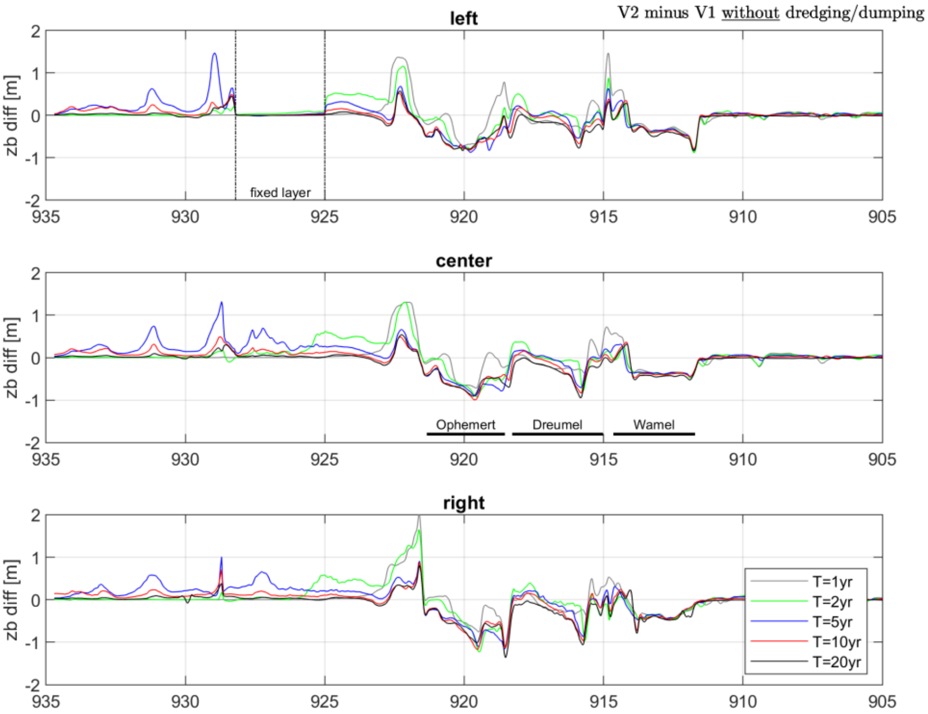


Figure 69
*Bed level difference between **V2** and **V1** at left bank, centre of channel and right bank, LTW-area and downstream, simulation **without** dredging/dumping.*



3.7 Comparison of V1/V2 with reference V0

In this paragraph we compare the effects of V1 and V2 relative to the reference case V0 (no LTW's), giving an indication of the range in effects for bed level (Figure 70), low water level (Figure 71), and dredging quantities (Figure 72).

In all cases we use the simulations with dredging and dumping activated for this analysis. For the bed level, V1 shows larger areas with (relative) sedimentation in the main channel, while V2 shows quite some erosion e.g. at the Dreumel and Ophemert LTW's. On the other hand, V2 (fully closed intake sills) can potentially lead to a larger water level set-up at low discharge. V2 appears to lead to smaller dredging quantities, as a result of both erosion and (more) increasing low water levels.

Figure 70
Effect of V1/V2 w.r.t. V0: bed level after 10 years.

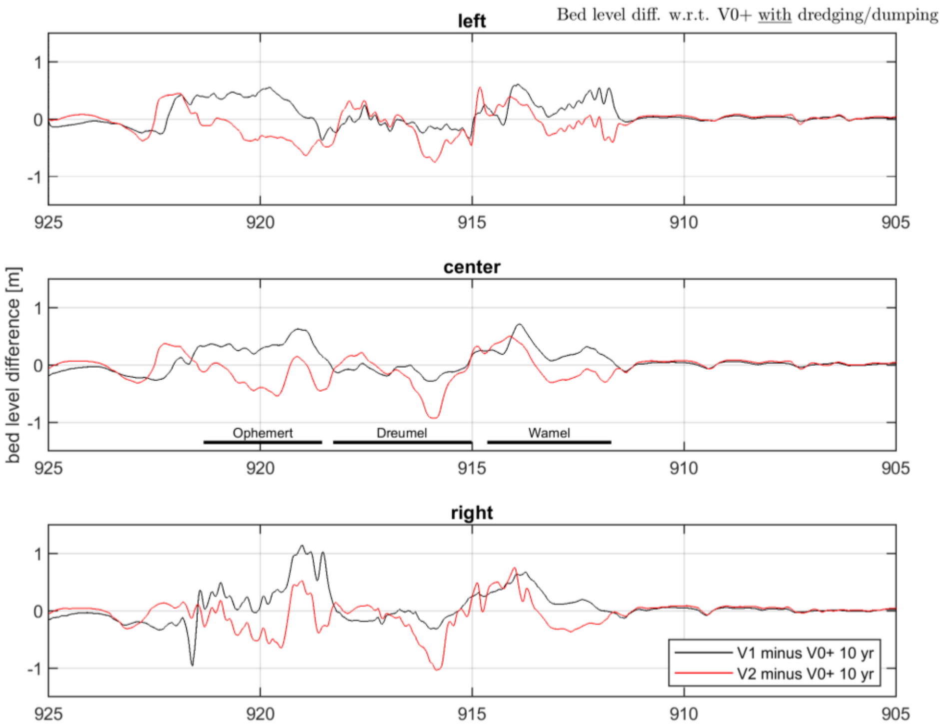


Figure 71
Effect of V1/V2 w.r.t. V0: low water level (Q1203 m^3/s) after 10 years.

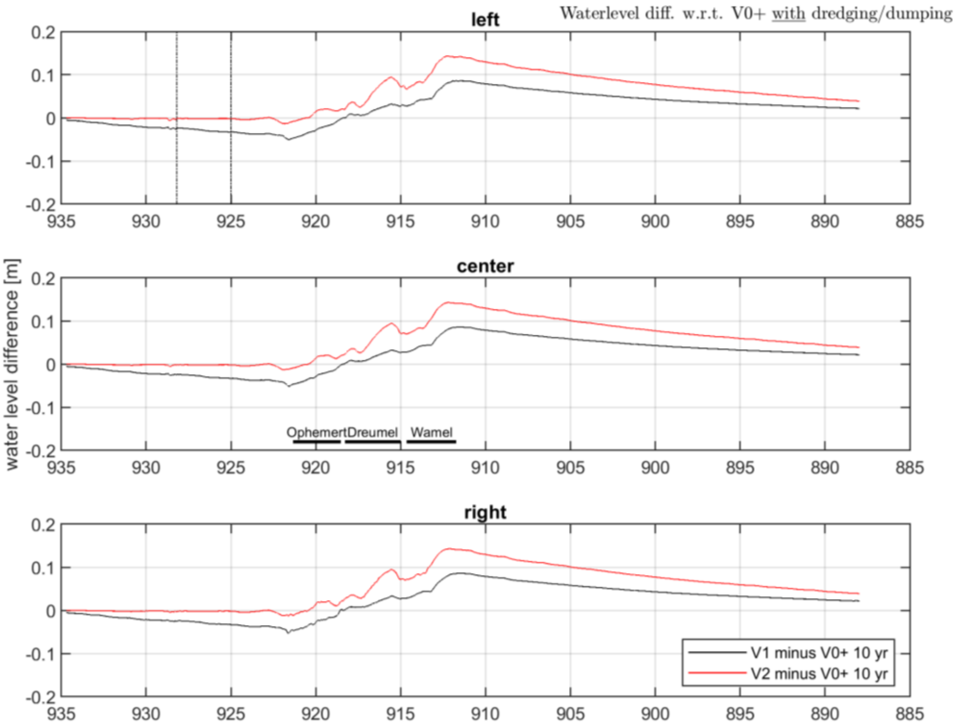
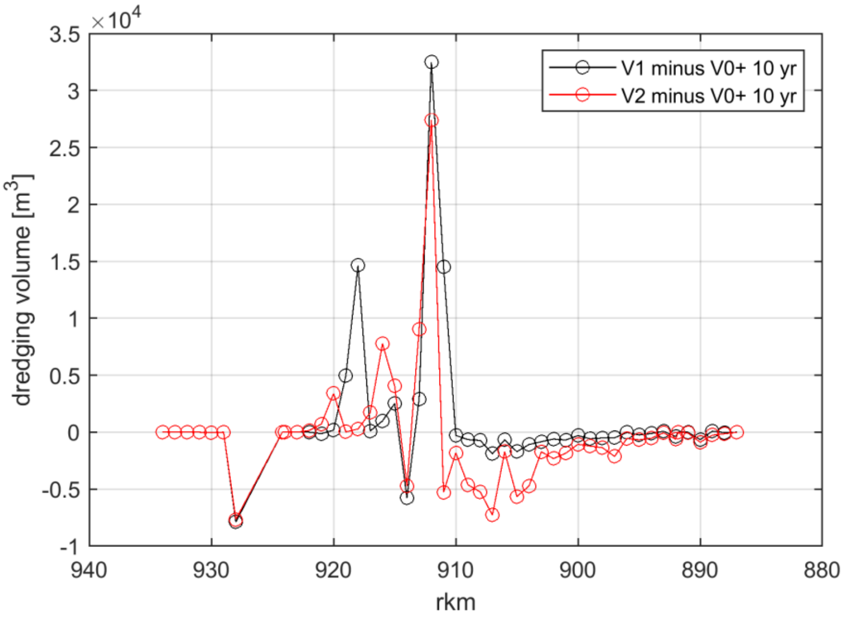


Figure 72
Effect of V1/V2 w.r.t. V0: dredging quantities after 10 years.



4 Concluding Remarks

This report concerns Work Package 1 (WP1) of the project “Final Evaluation Pilot Longitudinal Training Walls” ("Eindevaluatie pilot Langsdammen in de Waal”). We present the setup and results of numerical Delft3D-simulations, which are needed to carry out the different analysis components, which are addressed in the other work packages of the Final Evaluation.

Firstly, we made an inventory of the required hydrodynamic and morphodynamic simulations in terms of boundary conditions and design variants. The applied boundary conditions covered a wide range of discharge conditions. Apart from a situation without LTW’s (i.e., with original (non-lowered) groynes in area of LTW’s: variant V0) we considered (a) the basic variants with fully opened intake sills (V1) and fully/partly closed intake sills (V2/V3) and (b) a set of optimisation variants where the LTW’s and bank channel geometry are altered.

Secondly, we adapted the hydrodynamic model and evaluated its performance in a verification step. The model was found to be sufficiently accurate in capturing the observed water levels and is considered fit-for-purpose for the intended comparative analysis. The comparative analysis basically addressed direct comparisons between design variants and optimisation variants to evaluate their effectiveness or their impact on the river (hydraulic analysis only). In this report we only carried out the most basic analysis and provided the model results to the other work packages for their intended detailed assessment.

Finally, we carried out morphodynamic simulations for 20 years for a selection of design variants. The dynamic simulations were carried out with and without dredging in order to generate a broad range of results. The accuracy of the model was found to be sufficient to conduct the intended comparative analysis (for which using a spinned-up bed level as initial state for simulations with a hydrograph is a key element).

Both the hydrodynamic and the morphodynamic model results give insight into the function, performance and impact of the different design variants of the LTW’s. Together with the detailed data-analysis of field measurements from WP0, the computational results from WP1 deliver the necessary information for application in other work packages. Results from both sources are integrated and interpreted within those work packages to carry out their intended detailed analysis. Within WP1, a global consistency check with WP0-results have been made (see below).

Global comparison between WP1 and WP0

After finishing the WP1-Delft3D-simulations in Summer 2020, “Work Package 0” (WP0) performed extensive data-analysis based on field measurements of water level, velocity and bed level (De Jong et al, 2021). As outlined in paragraph 1.5, in WP1 we focus on differences between variants (relative effects). This is based upon consultation of requirements from the WP’s that will use the results, but also because absolute values on modelled bed levels are known to be uncertain (due to e.g., grid resolution or known model limitations). Relative effects due to different variants are considered significant. Therefore, comparison of absolute model predictions with measurements from WP0 is meaningless and outside the scope of WP1. Results from WP0 and WP1 are integrated and interpreted within other work packages to carry out their intended analysis.

Nevertheless, a global comparison between WP0 and WP1 has been performed. In general, there is good agreement between model results and measurements in terms of the effect of LTW’s (V1) with respect to the situation with groynes (V0); both in velocity/water level change as well as in bed level change.

For low discharge (i.e., no overtopping of LTW’s) it was anticipated that the water level would increase at Tiel (km 913) because of a width reduction in the LTW-trajectory. WP0 did not find this from water level measurements; this is in agreement with model results: the water level at Tiel is only marginally higher after LTW-construction (Figure 21, QLobith <= 1500 m^3/s). For higher discharges, measurements show a water level reduction after LTW-construction of around 20 cm at Tiel. Model results show a somewhat smaller water level reduction at Tiel of around 10 cm (Figure 21, QLobith >= 2500 m^3/s); this is probably related to the fact that in the hydraulic simulations the bank channel geometry of 2018 is used where, in reality, quite some sedimentation after construction has occurred, or to the highly variable width of the auxiliary channel which might not fit the (relatively coarse) grid.

WP0 performed velocity analyses for three discharge levels (Q-Lobith roughly 3207, 3983 and 4920 m^3/s), and found that for these discharges the velocity in the main channel reduces by 0-15% in the LTW-trajectory after the construction of LTW’s. This is in agreement with model results (not shown in this report). Both model and measurements show the largest reduction of the main channel velocity at LTW Ophemert. Note that there are no T0 (prior construction) ADCP measurements at discharges where water level set-up is expected according to model result, so no comparison of velocities at low discharge between model and measurements could be made.

Bed level measurements indicate less bed degradation in the area of the LTW’s after construction. The maximum cross-section-averaged sedimentation from 2016-2020 (obtained from p-map data) is in the order of 30-40 cm (section 4.4.1 in WP0-report). Model results show a slightly higher relative sedimentation of around 50-60 cm after five years (Figure 60). Note that in the simulations the sill dimensions are fixed during the entire simulation, while in the field they were changed over time (openings made smaller). Model results show that dredging volumes increase after LTW-construction at km 911/912 (downstream of inlet Wamel) and km 918/919 (downstream of inlet Ophemert) (Figure 66 and Figure 72). De Jong et al (2021) did not analyse dredging quantities. However, at these locations both the model (Figure 67) and field measurements (Figure 4.12 in De Jong et al, 2021) show sedimentation, although it appears stronger in the model at the Wamel inlet (which might be related to changes in sill configuration which are not taken into account in these morphological simulations). The increased dredging volume at km 918/919 is in line with a frequent MGD-location also reported by De Jong et al (2021). The measurements show large erosion at the downstream end of the longitudinal training walls, which is also predicted by the model. Further information and analysis about dredging and maintenance can be found in WP10 report (Chavarrías et al, 2021).

Model assumptions and discrepancies relevant for other WP’s

The Delft3D-model used in this project is based on the widely-used DVR-model of the Waal and tailored to this specific LTW-evaluation-project with greatest care. Notable are, (i) local grid refinement in the area of LTW’s, (ii) the schematisation of intake sills and bank channels and (iii) a verification of water levels and discharge distribution between main channel and bank channel. The model schematisation has been updated to fit study goals of the relevant work packages. However, as with any modelling study, there is room for improvements of the modelling tool and model schematisation. We trust that the model and the analysis approach we followed fits for the study purpose. They provide sufficient quality for confidence in the results and insight into the performance and impacts of the different LTW design variants. Below, we highlight some relevant model assumptions and limitations giving guidance to other WP’s to perform their analysis and proper interpretation based on WP1-model results.

Variants considered

The base variants V0 and V1 are based on Baseline-j14 and Baseline-j18 respectively and intend to represent the situation before LTW-construction (V0) and maximum opened (as-built) intake openings (V1) as closely as possible. Base variant V2 is a fictive variant where the intake sills are raised all the way to the LTW crest elevation, thus even higher than the maximum design height (i.e., OLR+1,25m). These base variants are intended primarily for assessing the possible range of morphodynamic effects (‘hoekpunten’) in WP7 and WP10. Variant V3 is intended to analyse the hydrodynamic effect of partly closing the intake sills in WP2. In addition, as requested by WP2, various optimisation variants are considered in hydrodynamic simulations.

Discharge distribution between main channel and bank channel

The discharge distribution between the main channel and bank channel is a key parameter for both the hydrodynamic as well as the morphodynamic effects. In the global verification of V1 (paragraph 2.3.2), a measured bank channel geometry from BL-j18 was used. Although measured discharge fractions are quite ‘scattered’ and difficult to obtain consistently (Figure 22), the general pattern (i.e., w.r.t. the threshold value) is reproduced quite well, but it appears that the model gives a (systematic) underestimation of the bank channel discharge of about 25%. Potentially this might lead to an underestimation of sedimentation with 25% as well.

In the morphodynamic simulations (deeper) bank channels are applied (as-built situation). From the hydrodynamic optimisation variants (Vopt05/06) we can clearly see that a different bed elevation in the bank channel leads to a significant change in discharge to the bank channel (e.g. Figure 34; for detailed analysis, please refer to WP2; Zuijderwijk and De Jong, 2021). This, however, does not lead to a significant change in discharge distribution w.r.t. the situation with the Baseline-j18 topography in morphodynamic simulations (not shown). Detailed analysis to the reason of the systematic underestimation of the bank channel discharge is outside the scope of WP1. We advise that other WP’s carefully consider this systematic underestimation when using and interpreting simulation results since model simulations potentially underestimate velocities in the bank channel (WP2) or underestimate sedimentation in the main channel (WP7/10). Future research should investigate the reason for this discrepancy (e.g., roughness, geometry, resolution, sills and/or comparison with WAQUA) for future modelling exercises using this or similar Delft3D-models. Note that the discharge distribution between bank channel and main channel also has an (indirect) effect on the water level at Tiel.

Sediment transport over intake sills and in the bank channels

The process of sediment transport over the intake sills is very complex, not fully understood, and not fully captured by the model. Also, bank erosion processes, which are observed in the field, are not included in our model since we lack proper (large-scale) morphological models of this area to do so. This means that (i) the sediment distribution between main channel and bank channel does not represent the field situation, and (ii) modelled morphological evolution of the bank channels is not necessarily representative for the field situation. Therefore, we follow a common strategy to focus on predicting the long-term morphological evolution of the main channel, neglecting detailed morphological effects in the bank channels, and use the model in a comparative way rather than in an absolute way. This is quite common when it comes to evaluating the morphological effects of measures in floodplains such as side channels (e.g. Paarlberg, 2013). The bank channels constructed in this area, however, are far more morphologically active than typical side channels. This means that their morphological evolution could alter the discharge distribution. Since the (required/desired) discharge distribution between bank and main channel is a key parameter for the morphological development of the main channel, we analysed the (change in) discharge distribution between bank and main channel for the 20 year morphological simulations (Figure 73). Although locally there is quite some sedimentation in the bank channels in the model, the discharge distribution is more or less constant over 20 years. This means that the behaviour of the bank channel for the sake of correctly modelling the behaviour of the main channel in the long term is correctly captured in our model.

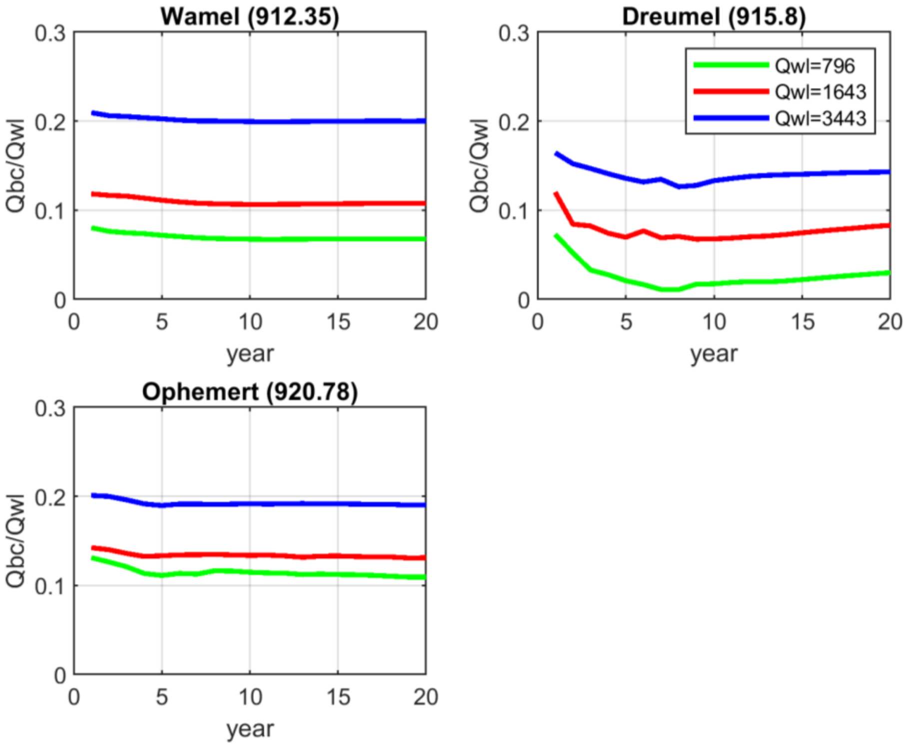


Figure 73 Development of discharge distribution between bank channel and main channel for V1 in the simulation with dredging and dumping active.

Absence of the Passewaaij secondary channel in the model
On the right bank, opposite of the Dreumel LTW (between km 916.1 and 917.4) is side channel 'Passewaaij'. This side channel is connected to the river on the upstream side by a culvert (Paarlberg, 2013). This culvert is not included in the model, meaning that at low flow the side channel is not connected to the river. This leads to an underestimation of sedimentation in the main channel at Passewaaij (in all simulations, both reference and design variants).

Small-scale bed level features and dredging/dumping
Small-scale features like bed forms are not taken into account explicitly in the model, since the grid resolution does not capture this scale; their associated roughness contributes to the (calibrated) roughness coefficients in the flow model. Also (absolute) dredging and dumping volumes are known to be rather sensitive to specific dredging criteria (OLR reference plane, dune height, tolerance) and should be treated with care.

Discharge extraction Amsterdam-Rijnkanaal
We do not include a discharge extraction to the Amsterdam-Rijnkanaal (ARK) in both hydrodynamic and morphodynamic simulations. The main focus of WP9 is to know whether LTW's mitigate negative effects of water extraction through the ARK by setting up water levels to ensure safe navigation. This question can be answered based on the simulations with low discharge without discharge extraction to the ARK. The lowest discharge considered in the morphodynamic simulations is 1203 m^3/s at Lobith. As was discussed in

paragraph 2.2.2, discharge to the ARK is only structural and significant for discharges below 1200 m^3/s at Lobith. Therefore, no discharge extraction to the ARK is considered in morphodynamic simulations. Note that this means that for a Lobith discharge of 1020 m^3/s, which is used to update the OLR reference plane for dredging, extraction to the ARK might be relevant, but is not included. The reason for this choice is that the model predicts the OLR (2012) well without an extraction to the ARK.

Tidal effects

At the downstream boundary of the model (Zaltbommel) there is a tidal influence, especially at low Waal discharge, see Figure 36. In line with previous morphodynamic Waal-models, we did not include tidal effects in the morphodynamic simulations. In the main area of interest near Tiel, there is virtually no tidal variation, even at extremely low river discharge (Figure 36). Including the effects of tidal variations in morphodynamic simulations requires either (i) a totally different model setup since tidal and morphodynamic effects occur at different time scales, or (ii) a parametric approach to mimic the enhanced sediment transport rates due to the tidal variations. Both approaches are outside the scope of this study, since we focus on relative effects. Having said that, we are confident that the model is sufficiently fit-for-purpose without the tidal downstream boundary condition.

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