The effect of flooding along the Belgian Meuse on the discharge and hydrograph shape at Eijsden

Guus Rongen
The effect of flooding along the Belgian Meuse on the discharge and hydrograph shape at Eijsden

by

Guus Rongen

in partial fulfillment of the requirements for the degree of

Master of Science

in Civil Engineering

at the Delft University of Technology,

to be defended publicly on Tuesday January 26, 2016 at 15:30.

Student number: 4017846
Supervisor: Ir. B. Maaskant HKV Lijn in Water
Thesis committee: Prof. dr. ir. M. Kok, TU Delft
Dr. ir. E. Mosselman, TU Delft
Dr. ir. O. Morales Napoles, TU Delft
Ir. H. Buiteveld, Rijkswaterstaat
Ir. M. Hegnauer, Deltares

An electronic version of this thesis is available at http://repository.tudelft.nl/.
Acknowledgments

This thesis marks the end of my master degree Civil Engineering, and with that the end of my time as student in Delft. As such, I would like to thank everyone who supported me in bringing this research to a satisfying end, but especially the people who made my life the past six years an unforgettable experience.

I would like to express my gratitude to my daily supervisor, Bob Maaskant. His clear and broad view on the subject helped me throughout the research to keep the final goal in sight. I am much obliged to my professor, Matthijs Kok, for his discussions on the subject, which were truly inspiring and motivating. I thank Mark Hegnauer for his helpfulness and advice on GRADE related subjects, Oswaldo Morales Napoles for his insights on the subjects related to statistics and Erik Mosselman and Hendrik Hegnauer for their constructive criticism, which greatly improved the quality of this thesis. Besides my committee, I would like to thank the numerous people at HKV and Deltares who helped me, but especially Joost Pol for his in-depth discussions, Stef Hummel for his guidance into SOBEK, and Ymkje Huismans for her expertise on the Meuse and effort to transfer it to me.

To all my friends, thank you for making the time in Delft the time of my life. Without them, the past years would never have been as amusing, engaging, lively, inspiring and exciting as they have been.

I want to thank my family, who supported me on all possible levels. Even though I have lived in Delft for six years now, visiting them still feels like coming home. Finally, special thanks go out to Enza, for her loving and unconditional support.

Guus Rongen
Delft, January 2016
Executive summary

The Belgian Meuse is a river with a relatively low flood risk. The floodplains are narrow, and the estimated impact is not large until the flood reaches a magnitude of a once in a 200 years event. Flooding along the Belgian Meuse could however have an effect on the flood safety in the Netherlands, since there the flood defences are dimensioned on such extreme events. In addition, the climate is changing, causing more extreme rainfall in the Meuse basin. Therefore, knowledge of what will happen in case of extreme floods is becoming more important. Accurate numerical models and better estimates of extreme rainfall and flood events give the possibility to research flooding on the river Meuse. The goal of this thesis is to analyse extreme floods on the Belgian Meuse: What happens along this river under extreme conditions, and what are the consequences in the Netherlands?

To analyse what areas along the Belgian Meuse are prone to flooding and how severe the flooding in these areas can be, a terrain analysis is carried out. This analysis shows that the river reach between Namur and the Dutch border is prone to flooding. The spatial limit that the flood could potentially have is not more than a few kilometres from the river axis, due to the valley shape of the floodplains. These flood prone areas have a potential storage capacity of 70 million m$^3$, and can thus have a significant influence on the flood wave. The valley shape of the Meuse ensures that there is no upper limit to the conveyance capacity, so the discharge at Eijsden has no cut-off level.

A numerical model of the Belgian Meuse is built to simulate extreme floods. The new SOBEK 3.4 software, which combines SOBEK 1D and Flexible Mesh 2D, is used to build the model. Although the software is still under development, it has sufficient possibilities to model the area of research. The created flood model gives good results; it can reproduce the results from a similar but more accurate 2D model of the University of Liège. The resulting impact of the flooding on the discharge is a few percent damping, increasing gradually from nothing for once in 50 years wave to 7% for a once in 50,000 years wave.

A sensitivity and uncertainty analysis are carried out to examine the uncertainties in the flood model. The first describes the selection of the parameters with the largest influence on the result. Two of these parameters, the main channel roughness and emergency measures, are further analysed in the uncertainty analysis. The main channel roughness gives the largest spread in discharge at Eijsden: a variation of 100 m$^3$/s up or down. Placing sandbags has a small effect on the discharge, since these emergency measures will likely not be sufficient to protect the floodplains. A third parameter, the timing of the inflow from the side branches has a large influence too. It is recommended to use a smaller GRADE time step for the extreme events on the Meuse, since the timing is important for the fast genesis of floods in the Meuse basin. When combined with the uncertainties in the rainfall and run-off models, the contribution of the hydraulic uncertainty is minimal.

When all GRADE uncertainties are incorporated in the design discharge curve, the impact of flooding along the Belgian Meuse leads to a maximum reduction of 100 m$^3$/s compared to the case of no flooding. Considering flooding when determining the exceedance frequency curve is recommended, it gives a more realistic estimate for the discharge extremes. The hydrograph shape plays an important role here, since the wider wave compromises the effect of retention basins. It is thus also recommended to use a different, wider, hydrograph when considering flooding. The water level reduction due to flooding is estimated to be up to 30 cm along the Dutch Meuse. The current method of determining discharge extremes, HR2006, gives lower discharges than GRADE. When switching to the GRADE frequency curve with (or without) flooding and with uncertainty, the same safety standards will give higher design conditions. By considering flooding, these higher GRADE values are mitigated.
Samenvatting

De Belgische Maas is een rivier met een relatief laag overstromingsrisico. De aangrenzende vallei is smal en de geschatte impact is niet groot, tot het moment dat de vloedgolf een grootte bereikt die eens in de tweehonderd jaar verwacht wordt. Overstromingen rond de Belgische Maas zouden echter wel een effect op de Nederlandse waterveiligheid kunnen hebben, waar de waterkeringen wel op extreme gebeurtenissen worden gedimensioneerd. Bovendien is het zo, dat door klimaatveranderingen meer extreme regenval in het stroomgebied van de Maas wordt verwacht. Om deze redenen wordt kennis over wat er gebeurt tijdens extreme omstandigheden steeds belangrijker. Precieze numerieke modellen en betere schattingen voor extreme regenval en vloedgolven bieden nu ook de mogelijkheid om overstromingen rond de Belgische Maas te onderzoeken.

Het doel van dit onderzoek is kennis vergaren over overstromingen rond de Belgische Maas: wat gebeurt er langs deze rivier in België onder extreme omstandigheden, en wat zijn de gevolgen in Nederland?

Om te analyseren welke gebieden langs de Belgische Maas kwetsbaar zijn voor overstromingen en hoe hevig deze kunnen zijn, is een omgevingsanalyse uitgevoerd. Deze analyse laat zien dat de Maas tot Namen niet, maar tussen Namen en Eijsden wel kwetsbaar is voor overstromingen. De uitgestrektheid van zo’n mogelijke overstroming is tussen Namen en Eijsden niet meer dan drie kilometer aan weerszijden van de rivieras. Dit komt door de valleivorm van de omgeving. Deze gebieden hebben samen een bergingscapaciteit van zeventig miljoen kubieke meter, en kunnen daarmee een significante invloed hebben op de vloedgolf. De valleivorm van de Maasvallei zorgt ervoor dat de afvoercapaciteit niet aan een bovenlimiet gebonden is. De afvoer bij Eijsden heeft dus geen fysieke maximum.

In het onderzoek is een numeriek stromingsmodel van de Belgische Maas opgezet om daarmee overstromingen te simuleren. Het nieuwe SOBEK 3.4, dat SOBEK 1D en Flexible Mesh 2D combineert, is gebruikt om het model te bouwen. Hoewel de software nog in ontwikkeling is, biedt SOBEK 3.4 voldoende mogelijkheden om het onderzoeksgebied te modelleren. Het gecreëerde model geeft goede resultaten, want het kan resultaten van een vergelijkbaar preciezer 2D model van de Université de Liège reproduceeren. Uit toepassing van het model komt als resultaat naar voren, dat de impact van overstromingen op het debiet een paar procent demping is, die toeneemt van niets bij een eens in de vijftig jaar golf tot zeven procent bij een eens in de vijftigduizend jaar vloedgolf.

Een gevoeligheids- en een onzekerheidsanalyse zijn uitgevoerd om de onzekerheden in het overstromingsmodel te onderzoeken. Met de gevoeligheidsanalyse worden de parameters geselecteerd die de grootste invloed hebben op het resultaat, de uitkomsten van het model. Twee van deze parameters, de hydraulische ruwheid van de rivier en eventuele noodmaatregelen, worden verder geanalyseerd in de onzekerheidsanalyse. De ruwheid geeft de grootte spreiding in de afvoer bij Eijsden: 100 kubieke meter per seconde naar boven of naar beneden. Het plaatsen van zandzakken heeft slechts een klein effect op de afvoer, omdat deze maatregelen waarschijnlijk niet effectief genoeg zijn als beschermingsmaatregel voor de achterliggende gebieden. Een derde variabele, de timing van de instroom van de zijrivieren heeft wel weer een grote invloed op de afvoer bij Eijsden. Wanneer de onzekerheden van het overstromingsmodel worden gecombineerd met de andere regen- en hydrologische onzekerheden in GRADE, is de bijdrage van het overstromingsmodel echter minimaal.

Wanneer alle onzekerheden worden verwerkt in de overschrijdingsfrequentielijn, leidt de invloed van overstromingen rond de Belgische Maas tot een maximale afvoerreductie van ruim honderd kubieke meter per
seconde ten opzichte van het hypothetische geval zonder overstromingen. Het advies is daarom overstromingen in het bepalen van de overschrijdingsfrequentielijn mee te nemen: het geeft namelijk een realistischere schatting voor afvoerextremen. De golfvorm speelt ook een belangrijke rol, omdat een bredere golf tot een geringere effectiviteit van retentiegebieden leidt. Het verdient dus ook een aanbeveling om een bredere golf te gebruiken, wanneer overstromingen worden meegenomen in de hoogwatervoorspelling. Het onderzoek leidt tot de schatting, dat in de meest extreme gevallen de afname van het waterpeil langs de Maas ten gevolge van overstromingen kan oplopen tot 30 cm. De huidige methode om extreme afvoer te bepalen, HR2006, geeft echter lagere afvoeren dan GRADE met (of zonder) overstromingen en onzekerheden. Wanneer wordt overgestapt op GRADE, zullen dezelfde veiligheidseisen tot hoger ontwerpvoorwaarden leiden. Door de aanvullende toevoeging van overstromingen in de Belgische Maasvallei, worden de hogere waarden door GRADE weer gemitigeerd.
## Contents

1. **Introduction**  

2. **Context**  
   
   2.1. Overview of the Meuse Basin  
      
   2.1.1. Topographic properties  
   2.1.2. Genesis of the flood wave  
    
   2.2. Current method of determining extreme discharges  
    
   2.3. GRADE  
    
   2.4. Uncertainty analysis carried out for GRADE  
    
   2.5. Modelling of floods around the Meuse  

3. **Terrain analysis**  
   
   3.1. Introduction  
   3.2. Methods  
      
   3.2.1. Locating flood-prone areas  
   3.2.2. Analysing area characteristics  
    
   3.3. Results  
      
   3.3.1. Significant areas  
   3.3.2. Area characteristics  
    
   3.4. Discussion  
      
   3.4.1. Conveyance capacity and mining subsidence areas  
   3.4.2. Quality of the results  
   3.4.3. Relevance of flooding for the Netherlands  
    
   3.5. Conclusions  

4. **Flood scenarios**  
   
   4.1. Introduction  
   4.2. Methods: model schematization  
      
   4.2.1. Model software  
   4.2.2. Eijsden and Borgharen  
   4.2.3. Schematization  
   4.2.4. Hydrograph selection  
    
   4.3. Results  
      
   4.3.1. Flooding per area  
   4.3.2. Exceedance frequency curve  
   4.3.3. Hydrographs  
    
   4.4. Discussion  
      
   4.4.1. Model schematization  
   4.4.2. The results  
    
   4.5. Conclusions and recommendations  

1
5. Sensitivity analysis

5.1. Introduction ................................................. 29
5.2. Methods .................................................. 30
  5.2.1. Main channel roughness ................................. 30
  5.2.2. Floodplain roughness .................................. 31
  5.2.3. Floodplain topography ................................ 31
  5.2.4. Embankment height ................................... 32
  5.2.5. Weir operation ....................................... 32
  5.2.6. Hydrograph shape .................................... 33
  5.2.7. Emergency measures .................................. 34
5.3. Results .................................................... 34
  5.3.1. Main channel roughness ................................. 34
  5.3.2. Floodplain roughness .................................. 35
  5.3.3. Floodplain topography ................................ 35
  5.3.4. Embankment height ................................... 36
  5.3.5. Weir operation ....................................... 36
  5.3.6. Hydrographs: Timing of the lateral peaks .......... 37
  5.3.7. Hydrographs: Smoothness of the wave .............. 37
  5.3.8. Emergency measures .................................. 38
  5.3.9. Upper and lower bound ............................... 39
5.4. Discussion .................................................. 39
  5.4.1. The important parameters .............................. 40
  5.4.2. The impact of the used hydrograph ................. 41
5.5. Conclusion .................................................. 41

6. Uncertainty analysis

6.1. Introduction ................................................ 42
6.2. Methods .................................................... 42
  6.2.1. Main channel roughness ................................. 42
  6.2.2. Embankments .......................................... 46
  6.2.3. Emergency measures ................................... 46
  6.2.4. Incorporating uncertainties in the exceedance frequency curve ......................... 46
6.3. Results ...................................................... 48
  6.3.1. 1D Roughness .......................................... 48
  6.3.2. Embankments .......................................... 50
  6.3.3. Emergency measures ................................... 51
  6.3.4. Combining the uncertainties ........................... 53
6.4. Discussion .................................................. 54
  6.4.1. Main channel roughness ................................. 55
  6.4.2. Influence of the hydrograph shape ................. 55
  6.4.3. Correlation between sections ......................... 56
  6.4.4. Emergency Measures .................................. 56
  6.4.5. Incorporating all uncertainties in the exceedance frequency function ............. 57
6.5. Conclusion .................................................. 57

7. Impact in the Netherlands

7.1. Introduction ................................................ 58
7.2. Methods ..................................................... 60
8. Conclusions and recommendations 65

References 66

A. Model schematization and results 69
   A.1. Schematization 69
      A.1.1. 1D main branch 69
      A.1.2. 2D floodplains 71
      A.1.3. River control structures 74
   A.2. Verification 77
      A.2.1. Input and schematization 77
      A.2.2. Results 77
   A.3. Calibration of the main channel roughness 81
      A.3.1. Topographical overview of calibration data 81
      A.3.2. Data availability 82
      A.3.3. Calibration with OpenDA 83
   A.4. Flood maps 84
   A.5. Model error 86

B. Uncertainty analysis 88
   B.1. Hydraulic analysis of the basin 88
# List of abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ahn2</td>
<td>Actueel Hoogtebestand Nederland 2</td>
</tr>
<tr>
<td>CDF</td>
<td>Cumulative distribution function</td>
</tr>
<tr>
<td>CRS</td>
<td>Coordinate Reference System - A system to reference geographical locations. For this research mainly the Belgian Lambert 1972 (EPSG:31370) is used.</td>
</tr>
<tr>
<td>DEM</td>
<td>Digital Elevation Model - A general term for raster elevation maps.</td>
</tr>
<tr>
<td>DNG</td>
<td>Deuxième Nivellement Général (French for TAW)</td>
</tr>
<tr>
<td>DSM</td>
<td>Digital Surface Model - A DEM which shows the top of the land surface, thus included objects like trees.</td>
</tr>
<tr>
<td>DTM</td>
<td>Digital Terrain Model - A DEM in which non-terrain objects like trees or houses are filtered out.</td>
</tr>
<tr>
<td>EFC</td>
<td>Exceedance Frequency Curve</td>
</tr>
<tr>
<td>ENW</td>
<td>ExpertiseNetwerk Waterveiligheid</td>
</tr>
<tr>
<td>GRADE</td>
<td>Generator of Rainfall And Discharge Extremes</td>
</tr>
<tr>
<td>NAP</td>
<td>Nieuw Amsterdams Peil - Dutch vertical coordinate reference system</td>
</tr>
<tr>
<td>OSM</td>
<td>Openstreetmap - An open source map service.</td>
</tr>
<tr>
<td>PDF</td>
<td>Probability density function</td>
</tr>
<tr>
<td>RMSE</td>
<td>Root Mean Squared Error - A variation estimator based on a sample or model.</td>
</tr>
<tr>
<td>RWS</td>
<td>Rijkswaterstaat</td>
</tr>
<tr>
<td>SPW</td>
<td>Service Public de Wallonie</td>
</tr>
<tr>
<td>TAW</td>
<td>Tweede Algemene Waterpassing (NAP + 2.33m) - Belgian vertical coordinate reference system</td>
</tr>
<tr>
<td>VNK</td>
<td>Veiligheid Nederland in Kaart</td>
</tr>
<tr>
<td>WFS</td>
<td>Web Feature Service - A method to download features, for example shape files, from the Internet.</td>
</tr>
<tr>
<td>WMS</td>
<td>Web Mapping Service - A method to download parts of maps as images from the Internet.</td>
</tr>
</tbody>
</table>
**List of symbols**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a$</td>
<td>$[\text{m}^2/\text{s}]$</td>
<td>Fitting coefficient for a rating curve</td>
</tr>
<tr>
<td>$b$</td>
<td>[-]</td>
<td>Fitting coefficient for a rating curve</td>
</tr>
<tr>
<td>$C$</td>
<td>$[\text{m}^{0.5}/\text{s}]$</td>
<td>Chézy coefficient</td>
</tr>
<tr>
<td>$C_2$</td>
<td>$[\text{s}^{-2}]$</td>
<td>Curvature of a hydrograph</td>
</tr>
<tr>
<td>$f(x)$</td>
<td>[-/-]</td>
<td>Probability density</td>
</tr>
<tr>
<td>$h$</td>
<td>[m]</td>
<td>Water level</td>
</tr>
<tr>
<td>$n$</td>
<td>[m]</td>
<td>Manning $n$</td>
</tr>
<tr>
<td>$P$</td>
<td>[-]</td>
<td>Probability</td>
</tr>
<tr>
<td>$q$</td>
<td>$[\text{m}^3/\text{s}]$</td>
<td>Threshold level of the discharge</td>
</tr>
<tr>
<td>$Q$</td>
<td>$[\text{m}^3/\text{s}]$</td>
<td>Discharge</td>
</tr>
<tr>
<td>$Q_p$</td>
<td>$[\text{m}^3/\text{s}]$</td>
<td>Peak discharge</td>
</tr>
<tr>
<td>$R$</td>
<td>[m]</td>
<td>Hydraulic radius</td>
</tr>
<tr>
<td>$T$</td>
<td>[years]</td>
<td>Return period</td>
</tr>
</tbody>
</table>
Some important terminology

A flood is the occurrence of a high discharge on the river, which mostly lasts for a week or two. We will often talk of a hydrograph, which is the description of the discharge during a flood, as a function of time. When we are talking about flooding, we mean that the river exceeds its embankments, and the floodplains or banks are flooded. Talking about these last concepts, a floodplain is a region along the river, which can be flooded in case of a large flood. The word insinuates that the area is meant to be flooded, but that is not the case for our area of research. We might also refer to these regions as the valley floor in a geographical sense. The embankment is a general term for the border between the river and the floodplains. This can be a quay, levee, or just a riverbank as will mostly be the case.
## List of Figures

2.1. Map showing the total Meuse basin ................................................. 5
2.2. Current design discharge curve (HR2006) ........................................ 6
2.3. Flow chart illustrating components of GRADE .................................... 7
2.4. Catchment map of the Meuse ............................................................. 8
2.5. GRADE exceedance frequency curve for the Meuse ............................... 9
2.6. GRADE exceedance frequency curve for the Rhine ............................... 10
2.7. Extent of flood from AMICE research ................................................ 11

3.1. Maximum flood levels for different simulations ..................................... 14
3.2. Elevation map showing the outer limit of the flood extent ..................... 15
3.3. Map showing potential flooded in case of a 5000 m$^3$/s flood ............... 16
3.4. Detailed map showing potential inundation in Liège in case of a 5000 m$^3$/s flood ............................... 16
3.5. Flood area and volume per region and per scenario ............................. 17
3.6. Map showing the comparison between the terrain analysis and AMICE ....... 18

4.1. Model schematization of the created flood model ................................. 22
4.2. Left: cross section from existing model located in Liège. Right: Top view of the cross section ......................... 23
4.3. Schematic overview of the existing model and the new models ................ 23
4.4. Check for the hydrograph selection ..................................................... 24
4.5. Flood map of Liège for the 50,000-year wave ..................................... 25
4.6. Exceedance frequency curves for all three models ............................... 26
4.7. Comparison of the peak discharge with the old model, the case of no flooding and the current practice .................. 26
4.8. Change of the hydrograph shape due to the inundations ....................... 27

5.1. Roughness ranges per river reach based on Chow .................................. 31
5.2. Variations in the DEM ................................................................. 32
5.3. Percentiles of the embankment height ................................................. 32
5.4. Variation in the weir control used for the sensitivity analysis .................. 33
5.5. Variations in the hydrograph shape .................................................... 34
5.6. Discharges at Eijsden for different values of the main channel roughness ....... 35
5.7. Hydrographs at Eijsden for different embankment levels ....................... 36
5.8. Discharges at Eijsden for a different weir operations ............................ 37
5.9. Discharges at Eijsden for a different timings of the laterals .................... 37
5.10. Discharges at Eijsden for a regular and smooth wave ......................... 38
5.11. Flood volumes for different variations on placing sandbags .................. 38
5.12. Discharge at Eijsden for the upper and lower limit ............................. 39

6.1. Roughness ranges per river reach from calibration ............................... 43
6.2. Correlations between roughness values ............................................. 44
6.3. Roughness ranges per river reach from calibration ................................ 45
6.4. Correlation in the quadrants ............................................................. 45
List of Figures

<table>
<thead>
<tr>
<th>Figure Number</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>Top: cell lengths along the river axis. Bottom: cell length ratios. Liège up and Liège down refer to up and downstream of the bifurcation.</td>
<td>70</td>
</tr>
<tr>
<td>4.2</td>
<td>Downstream boundary conditions</td>
<td>71</td>
</tr>
<tr>
<td>4.3</td>
<td>Hypothetical example explaining choice of embankment height</td>
<td>72</td>
</tr>
<tr>
<td>4.4</td>
<td>Graphical example of determining the floodplain roughness</td>
<td>73</td>
</tr>
<tr>
<td>4.5</td>
<td>A typical water control structure which consists of a weir and a navigational lock.</td>
<td>74</td>
</tr>
<tr>
<td>4.6</td>
<td>A composite structure in SOBEK, left the weir (belongs in middle), right the overflow 'structure'.</td>
<td>74</td>
</tr>
<tr>
<td>4.7</td>
<td>Location and schematization of the weirs which are validated</td>
<td>75</td>
</tr>
<tr>
<td>4.8</td>
<td>Weir calibration results for Lixhe and Anseremme</td>
<td>75</td>
</tr>
<tr>
<td>4.9</td>
<td>Comparison between the built in RTC module, and the scripted weir movement</td>
<td>76</td>
</tr>
<tr>
<td>4.10</td>
<td>Hypothetical example explaining how the input embankment for SOBEK is determined: The maximum of the 10 percentile value from the terrain analysis height and the grid level is taken.</td>
<td>77</td>
</tr>
<tr>
<td>4.11</td>
<td>Comparison of the water levels between the model of the university of Liège and the created 1D2D model.</td>
<td>78</td>
</tr>
<tr>
<td>4.12</td>
<td>Discharge AMICE and SOBEK for $Q = 3100$ m$^3$/s</td>
<td>78</td>
</tr>
<tr>
<td>4.13</td>
<td>Comparison of the hydrographs at Eijsden between SOBEK 3.4 and AMICE for the $Q = 4100$ m$^3$/s wave.</td>
<td>79</td>
</tr>
<tr>
<td>4.14</td>
<td>Validation of the flooded area for $Q = 4100$ m$^3$/s</td>
<td>79</td>
</tr>
<tr>
<td>4.15</td>
<td>Comparison of the hydrographs at Eijsden between SOBEK 3.4 and AMICE for the $Q = 5000$ m$^3$/s wave.</td>
<td>80</td>
</tr>
<tr>
<td>4.16</td>
<td>Validation of the flooded area for $Q = 5000$ m$^3$/s</td>
<td>80</td>
</tr>
<tr>
<td>4.17</td>
<td>Topographical overview of the calibration data.</td>
<td>81</td>
</tr>
<tr>
<td>4.18</td>
<td>Above: discharge series at Chooz. Below: data availability during this period for all stations.</td>
<td>82</td>
</tr>
<tr>
<td>4.19</td>
<td>Contribution of the Sambre to the discharge.</td>
<td>83</td>
</tr>
<tr>
<td>4.20</td>
<td>Example of a roughness calibration result</td>
<td>83</td>
</tr>
<tr>
<td>4.21</td>
<td>Flood map of Namur for the 50,000 year wave.</td>
<td>84</td>
</tr>
<tr>
<td>4.22</td>
<td>Flood map of Andenne for the 50,000 year wave.</td>
<td>84</td>
</tr>
<tr>
<td>4.23</td>
<td>Flood map of Amay for the 50,000 year wave.</td>
<td>85</td>
</tr>
<tr>
<td>4.24</td>
<td>Flood map of Seraing for the 50,000 year wave.</td>
<td>85</td>
</tr>
<tr>
<td>4.25</td>
<td>Flood map of Visé for the 50,000 year wave.</td>
<td>86</td>
</tr>
</tbody>
</table>
A.26. [Above] Hydrograph at Eijsden for both waves. [Below] Difference in discharge. Note that the one without embankments is almost always below the one with embankments. 86
A.27. [Above] Embankments, [Below] No embankments. The green line shows the cumulative difference of the discharge at Borgharen, with a no flooding situation. The orange line shows the stored volume. The sum of both lines should eventually be zero, meaning no water is added or lost. 87

B.1. [Discharges (left) and water levels (right) at Eijsden for different waveforms. 88
B.2. Histogram of time lag between lateral peak and peak at Borgharen 89
B.3. 10 Waves with a peak discharge around 4000 m$^3$/s disaggregated 90
## List of Tables

2.1. Overview of uncertainties in GRADE ................................................................. 10

3.1. Comparison of flood volumes between the terrain analysis and the research carried out by Dewals et al. (2012). Only the volumes of the overlapping areas are summed, to come to a fair comparison................................................................. 18
3.2. Inundated volumes and areas for the four scenarios........................................... 19

5.1. Table summarizing all sensitivity variations ...................................................... 40

6.1. Average peak discharge when protecting or not protecting a region with sandbags. 52
6.2. Average flooded area per number of protected regions. The reference discharge is the discharge without sandbags at Eijsden. ................................................................. 53

7.1. Table with discharges for HR2006 and GRADE with and without flooding at the design return periods. Note that these discharges do not translate one-on-one to design conditions. For example the wind might also contribute to levee failure..................................................... 59

A.1. Walloon and corresponding Dutch land use types with roughness. .................... 73
A.2. Comparison of flooded volume and area between SOBEK 3.4 and AMICE for $Q = 4100 \text{ m}^3/\text{s}$ 79
A.3. Comparison of flooded volume and area between SOBEK 3.4 and AMICE for $Q = 5000 \text{ m}^3/\text{s}$ 80
1. Introduction

When you drive along the Meuse at Liège, flood risk is probably not the first thing that comes to your mind. The quays along the canalised river are high, and the floodplains are not low-lying areas as often is the case in the Netherlands. Still, the Belgian Meuse has exceeded its embankments a few times in the past. In 1880 and 1926 large floods flooded the centre of Liège, causing flooding in the area around Liège (Lecouturier, 1930; Keimeul, 2007). After the second of the two floods, activities were carried out to prevent flooding in the future. Obstructions in the river were excavated and quay walls constructed to keep the river within the embankments. During the flood events of 1993 and 1995, both with a close to 3000 m$^3$/s peak discharge but slightly lower than 1926, Liège remained dry.

So is Liège safe against floods? Ogink and Barneveld (2002) investigated the effect of potential flooding on the conveyance capacity of the Belgian Meuse and discharge in the Netherlands. They carried out rough calculations showing that with the current climate and required safety for flood risk, flooding has no important influence on the discharge in the Netherlands. At a discharge of 4,600 m$^3$/s, Liège would flood severely, limiting the discharge in the Netherlands to that value. More recently, the University of Liège performed flood simulations which showed that there is no physical maximum to the discharge at Eijsden, but flooding will occur for discharges from 3000 m$^3$/s and larger. This causes the flood wave to be damped reducing the discharge at Eijsden with a few percent (Dewals et al., 2012).

With new guidelines for flood risk, the standards for the regions along the Dutch Meuse will become stricter. The required safety against flooding will be determined by more extreme events. Ministerie van Infrastructuur en Milieu (2014) shows that the return periods for the river Meuse will vary between 100 and 30,000 years. These events are linked to floods that will likely cause flooding in Belgium. Besides these stricter norms, climate change causes more extreme rainfall events leading to extremer floods. Within the GRADE framework, which is a new method of determining discharge extremes, 50,000 years of rainfall are generated and the river run-off is simulated. In these 50,000 years (to be understood as 50,000 possibilities next year, not a forecast for the period between now and 52016) flood waves with peak discharges up to 5,000 m$^3$/s occur, which will definitely flood regions along the Belgian Meuse based on the current day situation. If these events are used as design conditions for dimensioning the Dutch levees, the potential damping due to flooding has to be taken into account: considering flooding might prevent an overestimation of the design conditions.

With on one side the new safety standards in the Netherlands and a changing climate, the need for better knowledge increases. On the other side, the GRADE data and advanced numerical models create the possibilities to research this topic. The knowledge of how large the flooding and damping is for different flood events, can be used for better estimates on the flood risk in the Netherlands. That is what the goal of this thesis is: investigating what the effect of flooding along the Belgian Meuse is on the Dutch exceedance frequency curve of discharges at Eijsden, and with that, the magnitude of the loads on the Dutch flood defences along the Meuse.
1. **Introduction**

**Problem Description**

Before going to the specific research questions, the problem is elaborated more clearly. The goal is to find out what the impact of flooding is on the discharge and hydrograph shape in the Netherlands. Since there are no measurements of flooding along the Belgian Meuse, the way to investigate this is by making a flood model. Before setting up this model, the regions to include in this model need to be determined. With other words, determine which areas are sensitive to flooding, the so-called flood prone areas.

For the flood model, it is important that the flood pattern be simulated correctly. From the GRADE analysis for the Rhine one of the conclusions was that a 1D-2D or 2D model should preferably be used to achieve this goal. Since using such models increases cost and effort, the consideration has to be made whether the reduction due to flooding is worth the effort of using 2D models. When making a more complicated simulation of reality, extra uncertainties are introduced. When simulating extreme events that are much graver than anything we have ever seen, we are extrapolating on several aspects. Uncertainties in model variables can then have a significant effect on the results. These aspects will be analysed and taken into account too.

To find out what the impact in the Netherlands is, the reduction for the Dutch design conditions needs to be investigated. Important results for this translation are an exceedance frequency curve with flooding and with uncertainties, and an adjusted hydrograph shape, since that might change too in case of flooding.

**Research questions**

*Main question:*

What is the effect of flooding along the Belgium Meuse on the water levels in the Netherlands?

*Subquestions:*

1. Which areas in Belgium are likely to be flooded?
2. What is the impact of flooding on the downstream discharge and hydrograph shape?
3. What is the uncertainty in the flooding parameters and their effect on the discharge?
4. What is the impact of the modified discharge and hydrograph shape on the water levels in the Netherlands?

**Report structure and methodology**

This section gives an overview of the report structure, and with that a short overview of the methods used to answer the main research question. Each of chapters 3 to 7 also contains a section describing the methodology, but that is a more specific description whereas this chapter explains the general line.

After this introduction, the context of the research is described in chapter 2. It gives a short overview of the Meuse basin, as well as the carried out literature study.

Chapter 3 presents the answer to the question which areas are flood prone, and what their potential impact on the flood wave is. To answer this question, the a terrain model and flood estimate are used. With this information, the spatial limits of potential flooding, as well as the area characteristics can be analysed.

To answer the question what that impact of flooding is on the discharge at Eijsden a flood model of the flood prone areas is set up. Chapter 4 described the set up, as well as the results. With the SOBEK 3.4 model flooding can accurately be described. By simulating the GRADE waves, the model gives the discharge reduction due to flooding, as well as the changed hydrograph shape.
Chapter 5 and 6 present a sensitivity analysis and an uncertainty analysis. The first is carried out to see for which variables the model results are sensitive. This is done by imposing plausible variations on the model. The important variables are further analyzed in the uncertainty analysis. There, a probabilistic approach is used, which gives a probabilistic estimate for the outcome. The resulting uncertainties are combined, which gives an adjusted exceedance frequency curve for GRADE with flooding and with uncertainties.

To find out what the impact of the flooding is, the results are translated to the Netherlands in chapter 7. The relevant results are presented first, after which enhanced rating curves are used to estimate the water levels on the Dutch Meuse.

Chapter 9 summarizes the conclusions and answers to the subquestions, and subsequently answers the main research question.

A schematization of the sequence of chapters and elaborated subquestions is shown in the flowchart below.
2. Context

This chapter presents the background of the research subject. First, it presents a topographical overview of the Meuse basin. After this, it describes the current method of determining extreme exceedance frequency. Next GRADE is shortly explained, as well as the uncertainty analysis done for GRADE. The fifth section presents the research that has already been done on flood modelling for the Belgian Meuse.

2.1. Overview of the Meuse Basin

2.1.1. Topographic properties

The Meuse springs in France and flows into the Hollands Diep at Keizersveer. The basin covers part of France, Belgium, Germany, and the Netherlands. As Ogink (2006) explains, the river Meuse can be divided into three parts (illustrated in figure 2.1):

- **The upper reach** from the source to Charleville-Mézières. A long and narrow area with little height difference. The floodplains are wide and the contribution to the peak discharge at Eijsden is small.
- **The middle reach** from Charleville-Mézières to Eijsden, which roughly matches to the Belgian part of the Meuse, the area of research. In this reach, the Meuse has a relatively steep slope and the floodplains are narrow. The side branches from the Ardennes, which contribute largely to the peak discharge at Eijsden, flow into the main river on this reach.
- **The lower reach** from Eijsden to Keizersveer. The first part (border Meuse) still has a steep slope, but afterwards it becomes a typical Dutch river with summer levees, floodplains and winter levees.

2.1.2. Genesis of the flood wave

Peak discharges in the Meuse only occur in the winter season, when the rainfall is long lasting, the evaporation small and snowmelt can contribute to the discharge. During these extreme events, the bottom is often saturated, so the storage capacity of the soil does not play a large role. The timing of the laterals in comparison to the timing of the flood wave on the main branch is important. A rainfall event that follows the flood wave can more easily cause an extreme event. The travel time of the flood on the river is small, from Chooz to Borgharen takes approximately 16 hours. In case of an extreme rainfall event, the peak can reach Borgharen very quick. Interesting is that the side branches downstream of Chooz react faster to a rainfall event than the side branches upstream of Chooz. Slowing down the run-off in Belgium, by changing the land use, can therefore increase the peak discharge at Eijsden.
2.1. Overview of the Meuse Basin

Figure 2.1: Map of the Meuse basin. The upper, middle and lower reaches, which roughly coincide with the French, Belgium and Dutch Meuse, are indicated with the arrows.
2. Context

2.2. Current method of determining extreme discharges

Ogink (2012) elaborates the current method of determining extreme discharges. It is based on extrapolation of a measured discharge series. Several extreme value distributions are fitted to the data:

1. For return periods $T < 25$ y the Generalized Pareto distribution is fitted to the peaks ($Q > 1300$ m$^3$/s).
2. For $T > 25$ y the Exponential distribution is fitted to the peaks, and Lognormal, Gumbel and Pearson III are fitted to the annual maxima (figure 2.2).

![Figure 2.2: Adapted from Ogink (2012). Extreme value distributions derived by fitting to the annual maxima between 1911 and 2008.](image-url)

Figure 2.2 shows the fit for the return periods $>25$ y. The years of discharge measurements we have are fitted and extrapolated, which is done by taking the weighted average of the extreme value distributions given above. This method of estimating extremes has a few sources of inaccuracies (Ogink, 2012):

- If large flood waves cause flooding, this influences the hydrograph in such a way that it cannot be extrapolated any more from events that do not cause floods.
- Changes in the morphology of the river and land. River works have been carried out and land use has changed. The first is compensated but the second, which can influence the run-off, is not.
- Due to climate change, the amount of winter rainfall in the Meuse basin is expected to increase (Pfister et al., 2004). Due to the complexity of the climate trends are hard to discover. No correction for climate change is made in the current method.

2.3. GRADE

The GRADE project, in which GRADE stands for Generator of Rainfall and Discharge Extremes, is focused on improving the estimation of extreme events. This is done by simulating very long discharge series. In these series of 50,000 years length, the extreme events that normally have to be determined by extrapolation, are already present. Statistical analysis is still necessary, but the result does no longer depend on the choice of a particular mathematical distribution. It does however depend on the quality of the models, so statistical uncertainties are changed for model uncertainties. GRADE consists of three different models:

1. A climatological model, which generates rainfall series
2. A hydrological model (HBV), which simulates the run-off to the river
3. A hydraulic model (SOBEK), which simulates extreme flood waves

Figure 2.3 gives a schematic overview of the models and processes in GRADE. The three models in GRADE will be briefly explained below. For a complete report on GRADE, one is referred to (Hegnauer et al., 2014).

**Rainfall generator**

Very long rainfall series are generated with a stochastic weather generator. It uses a few decades of precipitation data from several stations (figure 2.4), and re-samples these data based on a nearest neighbour technique (Leander et al., 2005). The method of re-sampling does not allow wetter days to be generated than present in the measurements. However, this is no problem since a multi-day rainfall event determines the highest discharge peaks. The technique does allow generating extremer series than have been measured.

**Hydrological model: HBV**

The generated rainfall series are processed with the hydrological HBV model, a Swedish conceptual model. Each sub basin, outlined with red in figure 2.4, is calibrated individually to measured data. The model simulates hydrological processes that transport water to the river branches, and simulates flood wave development in these branches with the layered Muskingum approach. The output given by the HBV model is a discharge series of the $N$ simulated years, at the boundary of the basin. This output can serve as input for another sub-basin or a hydraulic model.

**Hydraulic model: SOBEK**

To simulate large flood waves, where the conveyance capacity is exceeded, a different approach with a broader physical basis is preferred. The 1D SOBEK model gives a good balance between a physical base and limited computation time. SOBEK solves the 1D shallow-water, or Saint-Venant, equations, to simulate the flood wave properties. Even though the computation time is minimal compared to a 2D model, it is still significant.
for 50,000 years of simulation. That is why only the large flood waves are modelled with SOBEK. The output of the SOBEK model is $N$ discharge extremes, which can be processed to an exceedance frequency curve using statistics.

2.4. Uncertainty analysis carried out for GRADE

For the hydrological model and the rainfall generator, for both the Meuse and Rhine an uncertainty analysis has been carried out, which is described in Van den Boogaard, Hegnauer & Beersma (2014). For the Rhine, also an uncertainty analysis for the hydraulic model has been carried out. This is described in Prinsen, van den Boogaard & Hegnauer (2015). The first will briefly be described, the second more extensively.

Meteorological and hydrological model

To assess the uncertainties in the rainfall generator, a jack-knife approach is used. With this method, a number of resamples are made, each with a different part of the measured series left out. In this way, the method is used to check the influence of extremes in the measured series on the generated series. To quantify the uncertainties in the hydrological model a set of different parameters that do all give representative outcome, has been simulated. The variety in the resulting discharges gives the uncertainty. Figure 2.5 shows the described uncertainties for the Meuse.

More information on the uncertainty analysis of the meteorological and hydrological model can be found in (Van den Boogaard et al., 2014).
2.4. Uncertainty analysis carried out for GRADE

Figure 2.5.: Adapted from Van den Boogaard et al. (2014). Exceedance frequency curve for extreme discharges of the Meuse at Borgharen, according to GRADE/SOBEK. The solid curve in blue represents the estimate of $Q_{RP}$ for the various return periods $RP$. The lower and upper bounds of the 95% confidence in this estimate (rain series + HBV) are denoted by the red coloured dashed curves. The curves in black represent the $Q_{RP}$ and its confidence intervals for the uncertainty in the HBV-models only.

**Hydraulic model: SOBEK**

Prinsen et al. (2015) carried out an uncertainty analysis for the hydraulic model of the Rhine. They mainly focused on the uncertainty of the parameters that affect flooding, since these contribute most to the total uncertainty. The parameters that have been varied are the addition of sandbags, terrain roughness, storage volume, inflow height and levee failure. The last three are varied between five large river-sections, whereas the first 2 are global parameters. This gives 17 parameters that are independently (no correlation) varied.

The approach used is as follows: From the reference set, a representative wave for one of the 16 discharge levels is chosen. This wave is simulated with 251 combinations of the 17 parameters, chosen with a Latin Hypercube Sampling approach. The 251 resulting discharges give a probability distribution $f(Q)$ for that return period. To combine this probability distribution with the one from the rainfall generator and hydrological model, a Monte Carlo approach with 201 samples is used, giving the uncertainty limits. Figure 2.6 gives the exceedance frequency curve generated by GRADE with its uncertainties.

This uncertainty analysis treats the schematization errors: storage volume, inflow height, terrain roughness and the two ’statistical’ unknowns: the addition of sandbags and levee failure. Two other factors might add something to the uncertainty limits. The first are simplifications made when treating the 2D (or even 3D) problem of flooding as a 1D problem. The second are numerical errors. SOBEK is a numerical model that implies errors due to the finite cell size. The impact of these errors is probably negligible in comparison with schematization errors.

A note on the uncertainty analysis by Prinsen et al. (2015) is the effect of the chosen roughness. A conservative approach is to take a low roughness, but this is not consistent with the roughness used in the WAQUA simulations for the Dutch Rhine. The issue is broached in ENW (2015) and further elaborated by Bieman (personal communication, 2015). Two possibilities are analysed: full correlation, and no correlation of roughness. The difference in water level between full and no correlation is approximately 30 cm. The actual situation will be somewhere in between: a partial correlation. For the Meuse this difficult issue will also play a role, and affect the exceedance frequency curve.
2. Context

Figure 2.6.: Adapted from Prinsen et al. (2015). Final result for the exceedance frequency curve (middle line) and the 95% uncertainty limits (with SOBEK) for the Rhine at Lobith. The red triangles show the measured yearly maxima.

Table 2.1.: Overview of uncertainties in GRADE

<table>
<thead>
<tr>
<th>Source of uncertainty</th>
<th>Statistical/Model</th>
<th>Treated by Van den Boogaard et al. (2014) or Prinsen et al. (2015)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rain series</td>
<td>Statistical</td>
<td>Yes</td>
</tr>
<tr>
<td>Rain measuring devices</td>
<td>Model</td>
<td>No</td>
</tr>
<tr>
<td>Interpolation between rain stations</td>
<td>Model</td>
<td>No</td>
</tr>
<tr>
<td>HBV model simplification</td>
<td>Model</td>
<td>No</td>
</tr>
<tr>
<td>HBV parameter uncertainty</td>
<td>Model</td>
<td>Yes</td>
</tr>
<tr>
<td>SOBEK Schematization errors</td>
<td>Model</td>
<td>Yes</td>
</tr>
<tr>
<td>Sandbags</td>
<td>Statistical</td>
<td>Yes</td>
</tr>
<tr>
<td>Levee failure</td>
<td>Model</td>
<td>Yes</td>
</tr>
<tr>
<td>SOBEK Numerical errors</td>
<td>Model</td>
<td>No</td>
</tr>
</tbody>
</table>

Overview of uncertainties in GRADE

Table 2.1 gives an overview of the uncertainties present in the different components of GRADE. A distinction is made between model uncertainties, which are the result of using a simplified or inaccurate schematization of reality, and statistical uncertainties, which are factors that we ‘cannot’ know.

Note that assessing the magnitude of uncertainties also adds an extra error, which is assumed negligible.

2.5. Modelling of floods around the Meuse

Within the AMICE (Adaptation on the Meuse to the Impact of Climate Evolutions) project, flooding has been modelled on the stretch from Ampsin to Eijsden. Dewals, Huismans, Archambeau, de Keizer, Detrembleur, Erpicum, Buiteveld & Pirotton (2012) describe the results of the project. A 2D model developed by the University of Liège, called WOLF, was used to simulate a flood wave with a roughly similar shape to the flood of 1995. Three scenarios were modelled: $Q_{100}$, $Q_{100} + 15\%$ and $Q_{100} + 30\%$. $Q_{100}$ is the peak discharge of a once in a 100 years flood. The three discharges (which match roughly with 3100, 3630 and 4100 m$^3$/s at Eijsden)
are estimates for climate scenarios for respectively present, 2021-2050 and 2071-2100. The grid size is 5×5 m², which is also the accuracy of the elevation model used for the schematization. Dewals et al. specifically treated the difference between running WOLF in steady and unsteady mode. Figure 2.7 shows the resulting flooding at Liège for the unsteady calculations. The figure shows significant flooding for $Q_{100} + 30\%$ and to a lesser extent for $Q_{100} + 15\%$. The $Q_{100}$ flood wave does not give flooding.

Paarlberg & Barneveld (2013) used the research of Dewals et al. to extend the SOBEK model of the Meuse with retention basins. Since no detailed data on the topography are available, the flooded areas given in the AMICE report are used to model the retention basin. The 100 km² mine subsidence area, which is seen as the retention area limiting the discharge to 4600 m³/s (Ogink, 2012) is not located in the area of this research, and therefore not taken into account. The question is whether the mine subsidence around Liège causes flooding. There is significant mine subsidence along the Meuse, but this mainly is along the Grensmaas, as De Wit (2004) describes. Bastings (1998) shows that after the flood of 1925-1926, several measures have been carried out around Liège to counteract the floods due to subsiding ground. These measures were dredging the river, constructing quay walls and removing islands. The elevation of the terrain around the modelled reach is shown in figure 2.7.

To calibrate the SOBEK model to the WOLF results, the breach width is used. Since Dewals et al. (2012) modelled discharges up to 4000 m³/s, the SOBEK model is calibrated up to this discharge. The 4600 m³/s discharge, which lies outside the calibrated range, is also simulated with the properties of the model calibrated to a discharge of 4000 m³/s. The level of the retention basin is assumed 4 metres below the inflow level. Whether this assumption is correct can be determined from a DEM. Making use of a detailed DEM could also improve the accuracy of the SOBEK model outside the calibrated range.

The AMICE report describes the WOLF model schematization for the Meuse, used to simulate three climate scenario waves. The boundary conditions as well as the flood results are given. The wave damping for the most extreme scenario is given to be 5%. For the simulation of the 5000 m³/s wave a final report is available (ULg, 2015). It gives a 3% peak discharge damping as result.
3. Terrain analysis

Chapter summary

Note: from now on the $Q_{100}$, $Q_{100} + 15\%$, $Q_{100} + 30\%$ will respectively be referred to as: $Q = 3100 \text{ m}^3/\text{s}$, $Q = 3630 \text{ m}^3/\text{s}$ and $Q = 4100 \text{ m}^3/\text{s}$, so the relation between the different flood waves is more clear. These numbers have been retrieved from AMICE.

This chapter describes the terrain and river analysis. The terrain around the Belgian Meuse is very much a valley. In the upper regions upstream of Namur, there are generally no floodplains. From Namur on the valley floor becomes wider. The possible extent of the flood will however not be larger than a few kilometres from the river. For four discharge levels, the potential flooding is calculated, by extending the maximum water level to the floodplains. The table below shows the results. The numbers only give an indication of the amount of flooding, with which we can get a feeling for the potential reduction (As reference, Lob van Gennep can store a volume of roughly 20 million m$^3$).

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Volume [million m$^3$]</th>
<th>Area [km$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q = 3100 \text{ m}^3/\text{s}$</td>
<td>5.8</td>
<td>5.4</td>
</tr>
<tr>
<td>$Q = 3630 \text{ m}^3/\text{s}$</td>
<td>17.4</td>
<td>14.3</td>
</tr>
<tr>
<td>$Q = 4100 \text{ m}^3/\text{s}$</td>
<td>32.4</td>
<td>25.8</td>
</tr>
<tr>
<td>$Q = 5000 \text{ m}^3/\text{s}$</td>
<td>78.2</td>
<td>43.8</td>
</tr>
</tbody>
</table>

The analysis shows no areas around the Belgian Meuse that can store such amounts of water that the discharge at the Dutch border is restricted to a physical maximum. There are however areas with significant flooding in case of an extreme flood, which are located from Andenne to the Dutch border and around Namur.

3.1. Introduction

In this chapter, the question which areas along the Belgian Meuse are likely to be flooded, is answered.

Research carried out indicates the area from Huy to the Dutch border, especially around Liège, as vulnerable to floods. Some research notes that mine subsidence in the region has caused large low-lying areas, which can physically limit the discharge of the Meuse entering the Netherlands.

Within this terrain analysis, the entire Belgian Meuse is taken into account to see if the areas from Huy to Eijsden cover all flood-prone areas. Also the existence of a physical maximum to the conveyance capacity due to mine subsidence is investigated. From that point on, the scope will be narrowed and focused on the important areas. Flood areas and volumes will be determined, as well as the embankment height.

In this chapter many analysis are done with a digital elevation model (DEM). The Service Publique de Wallonie, which is the owner the DEM, has granted permission to use it within this research.
3.2. Methods

3.2.1. Locating flood-prone areas

Before going into details and determine the area characteristics, we need to determine which areas are prone to flooding. To narrow down the regions along the Belgian Meuse, we use a hypothetical flood wave of 5000 m$^3$/s and the terrain model. How this flood level is determined is described in section 3.2.2. When we subtract the flood level from the terrain model, we can easily see which areas are located below the flood level. With the + 10 meter contour, the outer bounds of the flood prone areas are defined. All potential large storage areas are within these bounds.

3.2.2. Analysing area characteristics

To assess the importance of different areas, we calculate the potential inundation volumes and position where the embankments are overflown. The steps to get to these results are shown in the flowchart below. The steps are elaborated shortly in the following paragraphs.

Determining water levels from AMICE report and SOBEK runs

To assess the amount of flooding during a flood, the water level for that scenario is needed. The four chosen flood waves are the ones with peak discharges of $Q = 3100$ m$^3$/s, $Q = 3630$ m$^3$/s, $Q = 4100$ m$^3$/s and $Q = 5000$ m$^3$/s. They are chosen such that they match with the simulations carried out by the University of Liège. In this way, the flood levels from these simulations can be used as comparison. The water levels that result from this approach are shown in figure 3.1.
3. Terrain analysis

Extend flood levels to floodplains

To determine the potential flood volumes, the water levels are extended towards the floodplains. Since the water normally follows the exact course of the river and in case of a flood the more global course, the water level slope has the same pattern. To determine these water levels, curves are drawn starting perpendicular to the main branch and ending perpendicular to the more global river course. Interpolation between these curves than gives an estimate for the potential flood level.

Determine embankment height

The 1 m DEM is used to determine the embankment heights. The difficulty here is that the embankment is irregular: the variation over a 100 m stretch is about 1 m. Drawing a profile over the embankment, as would be a suitable method for a smooth Dutch levee, does not necessarily give the correct values in case of the Belgian Meuse. Therefore a method is used based on a flood fill method to find the correct embankment height:

1. Take an embankment section of roughly 100 metre and divide the river, embankment, and floodplain.
2. Enlarge the water level in small steps, and check which flooded tiles are connected to the river. The area that is below the water level and connected to the river, is the inundated area. As soon as the inundated area intersects the floodplain, we know that the embankment is overflown.
3. Select the point where the embankment is overflown. This point is part of the ‘critical’ embankment; as soon as it is overflown flooding occurs.
4. The points that are flooded if the water level rises further are also needed, so the overflown point is closed by increasing its elevation, and the procedure is repeated to find the next point.
5. As soon as the determined points form a closed line, the embankment height is found.

This method works well for determining the embankment height from a detailed DEM. The problem however is that the DEMs also contain inconsistencies. The surface model contains objects like trees, which interfere with the embankment, while in the terrain model objects like quay walls are filtered out. To solve this problem, the embankment has been determined for both the surface and the terrain model. All embankment segments have been checked manually for inconsistencies.
3.3. Results

### Significant areas

Figure 3.2 shows an elevation map in which the thick black line indicates the outer bounds of the flood-prone areas. All these areas are located along the main branch. There are no larger ones, which could subtract large volumes of water from the river system. The only ‘holes’ in the contours are caused by tributaries flowing into the Meuse. We also see that the region around Namur is vulnerable to flooding in case of a large flood. This region is not considered in the AMICE research.

By looking at the areas below the $Q = 5000$ m³/s flood level, the regions which are important to analyze are located. These areas are shown in figure 3.3. The boundaries of the areas are chosen such that the connectivity in case of a $5000$ m³/s flood wave is not disturbed.

### Area characteristics

For each area, the potential inundation volume and area are determined as described in the last section. Figure 3.4 shows the results for the area around the centre of Liège during $Q = 5000$ m³/s. Left the potential flooding is shown. A large part of the valley floor is inundated during this extreme event. In the right figure...
3. Terrain analysis

the embankment height relative to the flood level is shown. For more figures showing the potential flooding one is directed to the appendix.

Figure 3.3: Map of the Belgian Meuse (blue) with the inundated areas in case of the 5000 m$^3$/s flood level marked red.

Figure 3.4.: On the left, a flood map of the region around Liège, for the $Q = 5000$ m$^3$/s scenario. Note that the region in the upper right corner is not directly connected to the river, but can have significant flooding nonetheless. On the right, the relative water level compared to the embankment level. Red means the embankment is overflown, blue the opposite.

Particularly interesting are the total flood volumes and areas, since they give a quantitative answer to the question whether flooding is important. The numbers are shown graphically in figure 3.5 for each of the scenarios.
3.4. Discussion

The goal of this part of the research is to find out if flooding along the Belgian Meuse is important for the Netherlands. To find this out, it is analysed which areas are prone to flooding, and what the potential storage capacity of these areas is. In this discussion the relevance of flooding for the Netherlands is treated, and the areas and volumes determined in the analysis are placed into context.

3.4.1. Conveyance capacity and mining subsidence areas

As Ogink & Barneveld (2002) discussed, the conveyance capacity of the Meuse is limited to 4600 m³/s. Above this limit substantially large mine subsidence areas could limit the flow capacity of the Meuse, by topping of the wave. This conclusion was based on rough estimates and should, as they stated, be checked with a hydraulic model and detailed DEM. This is done is in this chapter. The results show that there are no substantially large areas to top off the wave. Mine subsidence, which caused the floodplain level to subside around Liège, is not substantial enough to create large storage volumes. Around Maasmechelen this might be the case, as De Wit (2004) describes, but this is outside the area of research. Other areas around the Belgian Meuse have a clear valley shape, and only damp the wave. There is thus no physical limitation to the conveyance capacity of the Meuse.

3.4.2. Quality of the results

Before conclusions on the relevance for the Netherlands can be drawn, the quality of the results is discussed. To do this a comparison with available data is made. As far as known, the only similar data comes from the AMICE project. In this project steady and unsteady simulations have been carried out for four waves with a peak discharge at Eijsden of $Q = 3100$ m³/s, $Q = 3630$ m³/s, $Q = 4100$ m³/s and $Q = 5000$ m³/s. The steady simulations give good similar data, since the idea is similar: checking the maximum volume and spatial limits of a flooding. The unsteady simulations show what the results would be for a ‘real’ flood wave, and are thus more a validation of the terrain analysis and steady simulations in this case.
3. Terrain analysis

Table 3.1 shows a comparison between the volumes calculated in the terrain analysis and the volumes calculated by Dewals et al. (2012). The volumes from the terrain analysis are larger, especially for low discharges. This can be the result from the different method used, which leads to a bit higher water levels, or the higher level of detail from the DEM. The $Q = 4100 \text{ m}^3/\text{s}$ wave gives very similar results, both with the steady as the unsteady simulations. This shows that the used approach gives a good estimate for significant floods.

<table>
<thead>
<tr>
<th>Flood wave</th>
<th>Volume [$10^6 \text{ m}^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Terrain analysis (partly)</td>
</tr>
<tr>
<td>$Q = 3100 \text{ m}^3/\text{s}$</td>
<td>2.3</td>
</tr>
<tr>
<td>$Q = 3630 \text{ m}^3/\text{s}$</td>
<td>10.4</td>
</tr>
<tr>
<td>$Q = 4100 \text{ m}^3/\text{s}$</td>
<td>20.2</td>
</tr>
<tr>
<td>$Q = 5000 \text{ m}^3/\text{s}$</td>
<td>50.2</td>
</tr>
</tbody>
</table>

Table 3.1: Comparison of flood volumes between the terrain analysis and the research carried out by Dewals et al. (2012). Only the volumes of the overlapping areas are summed, to come to a fair comparison.

To see if no areas are overlooked, and thus if the total flood extent is known, the areas from the unsteady AMICE wave with a peak discharge of $Q = 4100 \text{ m}^3/\text{s}$ are compared to the flooded area in the terrain analysis. Figure 3.6 shows that the flooded area is comparable. The AMICE results show more flooding in the centre of Liège, but still everything is within the valley bounds. The terrain analysis did however show areas upstream of the illustrated area to be important too. From Namur onward in downstream direction, there are flood prone areas along the Meuse, but all are located within the a few kilometres from the river axis. The valley limits the flood extent.

Figure 3.6: Map of the Belgian Meuse with the flood extent from the unsteady AMICE simulations compared to the terrain analysis. Legend: Red - AMICE 4100 extent; Blue - Terrain analysis extent.
3.4.3. Relevance of flooding for the Netherlands

Is flooding in Belgium important for the Netherlands? It has become clear that there is no physical maximum to the conveyance capacity. Still, there is however quite a lot damping of the possible. The total flood volumes and areas for the four investigated waves are given in Table 3.2. Note that the numbers in this table are for the full Belgium Meuse, and are therefore higher than the ones in Table 3.1.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Volume [$10^6$ m$^3$]</th>
<th>Area [km$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q = 3100 \text{ m}^3/\text{s}$</td>
<td>5.8</td>
<td>5.4</td>
</tr>
<tr>
<td>$Q = 3630 \text{ m}^3/\text{s}$</td>
<td>17.4</td>
<td>14.3</td>
</tr>
<tr>
<td>$Q = 4100 \text{ m}^3/\text{s}$</td>
<td>32.4</td>
<td>25.8</td>
</tr>
<tr>
<td>$Q = 5000 \text{ m}^3/\text{s}$</td>
<td>78.2</td>
<td>43.8</td>
</tr>
</tbody>
</table>

Table 3.2: Inundated volumes and areas for the four scenarios.

As a reference, we compare the volumes to the volume of the large Dutch retention area Lob van Gennep, which is 20 million m$^3$. The found potential flood area is significant. The potential flooding is calculated with a steady flow simulation of the peak discharge. This is not realistic so in reality the numbers might be a bit lower. The conclusions do however not change, flooding can have a significant result on the extremity of the flood in the Netherlands. Numerical simulations are necessary to quantify this impact more detailed, which is done in the next chapters.

3.5. Conclusions

This chapter tried to answer the question: Which areas along the Belgian Meuse are likely to be flooded? The general answer to this question is: the region around Namur, and from Andenne to the Dutch border. Some regions are more sensitive than other regions. For example, Seraing will flood early and severely due to its low terrain level. The find out what the exact impact of the flooding of these areas is, a numerical model is necessary.

Another important result is that there is no physical maximum to the conveyance capacity of the Meuse. There are no large areas along the river that could store such amounts of water that the discharge is topped of. Mine subsidence around Liège, which created a low-lying area at least at Seraing, is thus not of such scale to limit the conveyance capacity of the Meuse.
4. Flood scenarios

Chapter summary
This chapter describes the steps made to find out what the effect of flooding is on extreme discharges and the hydrograph shape. The result is a new exceedance frequency curve including flooding.

To analyse the effect of flooding, a hydraulic model is set up. The software used is SOBEK 3.4, which has a 'new' way of calculating flooding based on links between a one-dimensional (SOBEK) and two-dimensional model (Flexible Mesh). The main branch has largely been adapted from the current official SOBEK model for the Meuse. This model is converted to 1D-2D model, by adding floodplains at the location of the flood prone areas determined in the terrain analysis.

Important is that the schematization of the official model already has significant storage. To analyse the impact of flooding also a model without any storage is created, which gives us three models: The new 1D-2D flood model, a remake of the current (official) model and the model without flooding.

The results show a significant reduction of the peak discharge compared to the case of no flooding. When compared to the official model, the reduction is less, but still significant. Flooding makes the hydrograph wider and reduces the peak. The effect of flooding starts at discharges above 3000 m$^3$/s and increases up to 5% and 8% for discharges of respectively 4000 and 5000 m$^3$/s. Compared to the current GRADE model, the reduction is roughly one third less.

4.1. Introduction

Narrow urbanized floodplains characterize the Belgian Meuse valley. The valley floor spans 0 to 3 km, after which the terrain quickly rises. Flooding commences in a narrow discharge band: a flood wave with a peak discharge of 3000 m$^3$/s will leave everything dry, whereas a peak discharge of 4000 m$^3$/s will flood most of the floodplains. To investigate flooding along the river, for these extreme scenarios, we will set up an flood model. The goal is to clarify how large the impact of flooding on the Netherlands actually is.

The next sections describe the schematization and validation of the 1D model, 2D model and the river control structures. After this, the performance is discussed.

4.2. Methods: model schematization

4.2.1. Model software

The question is which software should be used to set up a model. In the scope of the research, determining the impact of flooding for the Netherlands, the model should accurately calculate the discharges at the border. To do so, the model should accurately determine the amount of flooding, and the timing of the flooding. We therefor use the following guidelines while setting up the model:
• The embankment height is accurately determined, since it is important for the timing of the inflow.
• The floodplain topography gives the correct flooded volumes, but does not have to be very fine, since the local flood pattern is not a main result.
• We expect that the flow over the floodplains is important in this case, due to the valley shape of the Meuse. The model has to reproduce this pattern.
• The calculation time is under one hour, since a large number of simulations need to be done.

The requirements lead to the choice of a 1D-2D flood model, since this of model is a good compromise between speedy computations and detailed flood modelling. Two 1D-2D models are taken into consideration, the old SOBEK 1D-2D and the new SOBEK 3.4. The terrain characteristics (narrow floodplains, embankment height important) and the added value of working with a new model, lead to the choice of SOBEK 3.4 over the old SOBEK 1D-2D.

SOBEK 3.4 is a new version of SOBEK 3, in which a lateral link between a 1D flow channel and a 2D floodplain can be made. The main branch is quickly calculated with the SOBEK computational core, while in case of flooding of the floodplains this is simulated with a 2D Flexible Mesh model. In the event of flooding occurs, the flow over the links is calculated with a flow over a weir formula.

### 4.2.2. Eijsden and Borgharen

Before describing the model itself, the relation between Borgharen en Eijsden is explained. Eijsden is officially the end of the Belgian Meuse, so the results in this thesis are derived for Eijsden. Potential flooding until Eijsden is analyzed, and not for the river between Eijsden and Borgharen. Most of the used comparison material is available however at Borgharen, so many results are presented for this location. The difference in discharge is mostly around 30 m³/s; the contribution of the Jeker during extreme events.

### 4.2.3. Schematization

The model schematization is shown in figure 4.1. It shows the geographical position of the model and summarizes the most important characteristics. In this section some general notes are given, but for an extensive description of the model schematization and validation, one is referred to appendix A.1.

**Different models used in this thesis**

The 1D-2D model is created by transforming the existing model to an flood model. To do so, the main branch from the existing model is folded along the actual course of the river Meuse. At the position of the flood prone areas, a 2D grid is added. The existing model already contains an amount of storage, since the cross-sections are very wide at the location of the flood prone areas. This is good, since it approximates the true situation better, but it is not a model without storage due to flooding. Figure 4.2 shows an example of a cross-section with storage, this one at Liège.

To know the effect of flooding, it is necessary to compare a flood model to a model without flooding. The original model cross-sections are therefore stripped of all storage, creating a hypothetical case with no potential storage. We also recreate the existing model with the 'new' river course. This model is nearly equal to the official model. Summarized, the following model schematizations are used:

• The one for the existing model, but translated to the new 1D model. This means the original cross-sections, but on the new branch.
4. Flood scenarios

- 2D grid: 18984 triangular cells with a maximum edge length of 100 m
- Topography: average elevation from DEM within polygon around nodes
- Roughness: Land use data translated to characteristic roughness values
- Embankments: Height given by 10th percentile from terrain analysis.
- Each embankment cell is connected to the main branch. The flow is calculated with a weir formulation

The weirs are adapted from the current model, and all changed to a broad crested river weir to avoid oscillations. The movement of the weirs is programmed with a python script.

Cross-sections: Adapted from the existing model. At the locations where there is a 2D-grid present, all storage is stripped from the cross-section. At the others there might be a little left.

The schematization of the main branch is adapted from the current 1D model. The horizontal geometry is adapted from openstreetmap data. The roughness is determined from calibration. The plot below shows the vertical profile.

Chooz: upstream boundary. A time dependent discharge from GRADE, just as the laterals.

Borgharen, downstream boundary. A Qh-relation, determined by measurements.

Figure 4.1: Model schematization of the created flood model
4.2. Methods: model schematization

Figure 4.2.: Left: cross section from existing model located in Liège. Right: Top view of the cross section.

- The one without flooding, so all storage stripped out.
- The new 1D-2D model: For this case, the profiles without storage are used in the areas where a 2D grid is present. On the sections without a 2D grid, the profiles with storage are used. Note that the storage in these last profiles is very small, since all significant flooding is simulated with a 2D grid.

Figure 4.3 shows the three models schematically:

Figure 4.3.: A schematic overview of the existing model and the new models. The frame colours are equal to the colour in which the results are presented.

4.2.4. Hydrograph selection

Because of the computation time involved in running a simulation (on average 25 minutes), it is not possible to simulate all 800 GRADE waves which cause flooding. 100 waves are selected for which the discharge at Eijsden is calculated. Differences in hydrograph shape can cause a larger damping for some of the waves than for others, which will change the return period of the hydrographs. It is expected that the selection has a negligible effect on the result, since some waves are damped more, and others less. The danger is however that the 100 selected hydrographs are all damped more or less, so there is a net effect. To check this, the curvature \( C_2 \) of the waves is used as estimator for damping. Pol (2014) determined that the curvature is a good estimator for the water level downstream, and uncorrelated to the peak discharges. The uncertainty analysis
4. Flood scenarios

will show by checking the deformation of the hydrograph for different hydrographs, that the curvature can also be used to indicate the amount of damping. The curvature can be calculated with formula 4.1.

\[
C_2 = -\frac{Q_{tp-\Delta t} - 2Q_{tp} + Q_{tp+\Delta t}}{Q_p\Delta t^2} \cdot 10^{12}
\]  

(4.1)

In which:

- \(C_2\) = the curvature \([s^{-2}]\)
- \(Q_{tp}\) = The discharge at the time of the peak \([m^3/s]\)
- \(\Delta t\) = Time step, \(24 \times 7200\) seconds

The selection of wave requires: 1) The curvature distributions to be equal; 2) The relation between curvature and discharge to be equal. For the selection, both criteria are shown in figure 4.4. The distributions match very well, and the direction of the linear fit is also similar.

![Figure 4.4](image)

Figure 4.4.: On the left, the empirical distribution function of the curvatures. On the right, a linear fit through the selection and the complete set.

4.3. Results

This section presents several results. First, the results of the flooding for Belgium are presented. After this the results at the downstream border: the exceedance frequency curves and hydrograph shape at Eijsden are presented.

4.3.1. Flooding per area

Figure 4.5 shows the flood pattern for the 50,000 year wave in the area around Liège. The red lines are the embankment. The redder the colour, the higher the water level on top of the embankment. Note that an overflowing embankment does not necessarily means the area behind it is flooded. In many cases the terrain behind the embankment is higher than the embankment itself.

The right figure shows the flooded volume for each of the 100 simulations. Liège, as well as Seraing, are protected relatively well by embankments. The consequence of this is that flooding will start quite suddenly when the discharge is so high that the relatively constant embankment level is overflown. Other areas along the Belgian Meuse are more valley shaped, so the flooded volume increases more gradually. The figures for the other areas are shown in appendix A.4.
4.3. Results

Figure 4.5: Flood map of Liège for the 50,000-year wave. The left figure shows the flooding; the right figure the stored volume for each wave.

4.3.2. Exceedance frequency curve

Figure 4.6 shows the exceedance frequency curves for the 100 simulations done with each of the models. In addition, the current GRADE curve and the HR2006, which is the current method used to determine design discharges, are shown. The black dashed GRADE curve matches the orange curve quite well, which means the constructed model can reproduce the GRADE results. The HR2006 curve is significantly lower for return periods up to 1000 years. This is caused by the different method used to establish the curve, statistical extrapolation. Also, note that the most extreme wave gives a discharge which is significantly higher than the fitted line. This can be simply the result of the scarce observation density of such extreme waves, so it might actually be a once in a 100,000 years wave. This seems a plausible explanation, since the amount of water in the flood is extraordinary large (see appendix), but to know it for with certainty more realizations of this magnitude are needed.

The used plotting positions for the simulations is \((k-0.3)/(n+0.4)\), in which \(k\) is the rank, and \(n\) the number of observations, conform with GRADE (Personal communication with H. Van den Boogaard). The observations with a return period above 500 years are fitted with a Weissman fit (Van den Boogaard et al., 2014), also conform with GRADE.

The percentage of reduction of the peak discharge is shown in figure 4.7. These result are derived from the exceedance frequency curves in figure 4.6.
4. Flood scenarios

Figure 4.6: Exceedance frequency curves and peak discharges from the 100 simulations with the three models. The data location is Borgharen, for comparison.

Figure 4.7: Comparison of the peak discharge with the old model (orange), the case of no flooding (red) and the current practice (black). The data location is Borgharen, for comparison.

The difference between no flooding and the new model is clear: the larger the peak discharge, the larger the damping. Between the current model and the new model, the results are less monotonous. Due to the different implementation of storage in the models, the discharges can become higher in the flood model. This is also the reason for the differences at larger return periods between the remade model and the 1D-2D flood model. Between 200 and 1000 year the amount of flooding is roughly the same, after this the amount of flooding in the 1D-2D model gets larger.

Compared to the current design conditions, the HR2006, the differences are very large for lower return periods. This is the case for GRADE Meuse in general. Taking flooding into account does not lead to a reduction at return periods up to 1500 years. For return periods from 1500 years on there is a reduction in discharge.

4.3.3. Hydrographs

We are also interested in the change of the hydrograph shape. Pol (2014) shows that this can have a large effect on the downstream water levels. This water level effect will be elaborated in chapter 7. Here the effect of the flooding on the shape of the wave (and vice versa) is presented. Figure 4.8 on the left shows
the changing curvature due to flooding. The direction in which the points shift from without towards with flooding is very similar. The larger the flooding, the more the hydrograph changes. We observe that not only the peak discharge changes, but also the hydrograph shape. Wider hydrographs generally give higher water levels downstream, since the effectiveness of retention is reduced.

The right figure shows the largest hydrograph with (blue) and without (red) flooding. It is visible that the flooding removes part of the top of the wave, and adds it to the back. The calculation points used to calculate the curvature are marked with red and blue dots.

Figure 4.8.: Change of the hydrograph shape due to flooding. Curvatures are ‘measured’ at Eijsden.

4.4. Discussion

The subquestion to be answered in this chapter is what the impact of flooding on the downstream discharge and hydrograph shape is. To do this a flood model is set up, and the most extreme GRADE waves have been simulated with it. To find out if the subquestion can be answered with certainty, the quality and results of the model will be discussed here. Note that the results will only be discussed briefly, since the next chapters will focus completely on the sensitivity of the model to uncertainties.

4.4.1. Model schematization

The model schematization is based on an old model, of which no source data are available. Part of the schematization has been checked with open data and the DEM of the SPW. The cross section width has been checked with openstreetmap.org and cross profiles from the DEM, and seemed to be accurate. The bottom level can however not be validated with available data. The calibration corrects for errors in the bottom level, but does not solve the problem. Events outside the calibrated range, which are the ones simulated, cannot be verified.

The exact lay out of the weirs is another point that should be validated. Many of the weirs have a gate that can be lifted in case of a large flood. How far is however not known, and the gate is not present in the model. The width of the weirs does also not always match with aerial photographs. The movement scheme of the weirs is another uncertain point of the weirs, but that will be discussed in the sensitivity analysis.

A third point of inaccuracy is the 2D grid, which consists of triangles with 100 m legs. The computation time is the limiting factor, which leads to this quite coarse grid for the modelled area. A test with a finer grid has not been carried out, due to time and potential problems involved in creating a finer model. The computation time has not been compared to similar software, but it seems to be large for a model with roughly 20,000
2D cells and 1,700 1D cells. It is therefore recommended to compare SOBEK 3.4 (when finished) to fully 2D alternatives like Flexible Mesh, for computation time and accuracy.

Although there are quite some points that indicate inaccuracy of the model, there is enough reason to assume the model performs well. Almost all data used in the schematization is verified and the course 2D grid is constructed from a detailed DEM with detailed roughness data. The influence of an error in the parameters will be verified in the sensitivity analysis.

Finally yet importantly, the model performed well in the verification. The model results for the same waves as in the AMICE project are compared to the results from the AMICE project. The results match well. From that point of view, the model performs well. Besides that, the most interesting information for this research is the discharge in the Netherlands. The coarseness of the grid is thus not that important, as long as the results are correct.

### 4.4.2. The results

The resulting exceedance frequency curve looks show that the amount of damping is a few per cent. This is in agreement with the result from the AMICE project. The gradually increasing reduction compared to the no-flooding case makes sense, due to the valley shape of the basin. The changing hydrograph shape, lower and wider, is also as expected.

### 4.5. Conclusions and recommendations

Flooding along the Belgian Meuse causes a discharge reduction up to roughly 6% for the Netherlands. This is however in comparison to the case of no flooding. The SOBEK model that is used for the hydraulic computations in GRADE also incorporated storage due to flooding, so the comparison should be made to this situation, since it is the current GRADE exceedance frequency curve. In that case, we see a reduction up to 5% for the extremes, but for lower return periods the reduction is lower, sometimes even an enlargement. The hydrograph shape changes significantly due to the flooding. The more peaked the input hydrograph, the lower and wider the hydrograph becomes.

The new SOBEK 3.4 software is suitable to create flood models, as the one created for this thesis. The software still contains bugs, which makes working with it difficult, but the model gives good and reliable results. The option to use Python scripts to control and build the model is ideal, and offers a lot of possibilities. The coupling between the 1D and 2D model is ideal for fast simulations of low flow periods and detailed simulations during floods, but has a lower performance that the model individually. It is recommended to compare SOBEK 3.4 (when finished) to full 2D alternatives like Flexible Mesh for computation time and accuracy. This should clarify if it is useful to use a 1D-2D model in cases where virtually only flooding is simulated, like this research.
5. Sensitivity analysis

Chapter summary

In this chapter, the sensitivity of the hydraulic model to variations in the schematization parameters and boundary conditions is determined. This gives a feeling for the magnitude of the uncertainty around the exceedance frequency curve (determined in the previous chapter) due to variations only in the hydraulic model.

The sensitivity analysis shows that the model is sensitive to variations in some of the model parameters, but the effect on the peak discharge is limited. This is because the total reduction at $Q_p = 4000 \text{ m}^3/\text{s}$ (used as representative wave) is about 4% (160 m$^3$/s), so when the peak damping doubles, it is still only 8% (320 m$^3$/s).

The main channel roughness has the largest influence, since it directly affects the water level and thus also the amount of flooding. The variations in the peak discharge are around 3%, which is in the same order as the total damping. Another influential factor is the timing of the lateral inflow peaks, which can cause a large reduction in the peak discharge. This is the result of the 24-hour time step in the weather generator and HBV model, and will therefore not be taken into account further in this research.

5.1. Introduction

The created model is a simplified representation of reality. To build the schematization of the model, simplifications and assumptions are made. All these can influence the simulations in such a way that the results deviate from what happens if a simulated event would strike the real Meuse basin. Therefore, it is important to first assess these deviations and secondly decide whether to consider them when calculating the resulting exceedance frequency curve.

The question to be answered in the uncertainty analysis is what the impact of uncertainties in the model is on the result. In this chapter the preparatory work to answer this question is done; selecting the important uncertainties.

What kind of sensitivities can be taken into account? The process to determine discharges with GRADE is as follows: Measured rainfall is resampled with a rainfall generator and drained to the river with a hydrological model. Finally the created hydraulic model is used to accurately simulate the genesis of the flood wave as well as potential flooding. The input; resampled rain measurements, are thus translated with the models to the output, the exceedance frequency curve.

The exceedance frequency curve is an uncertainty itself, since it is constructed from the specific rainfall measured in the past $N$ years. These measurements are just realizations of the infinite possibilities of the weather. Would the realisation be different, than would the exceedance frequency curve be different as well. The uncertainty of these possibilities is a statistical uncertainty. The HBV model and SOBEK 3.4 model both add some uncertainty to the outcome too, which is model uncertainty since it comes from the schematization of physical properties.
5. Sensitivity analysis

The uncertainty in the rain series and HBV model are already determined, as described in 2. This research looks at the uncertainty caused by the hydraulic model. Note that the uncertainty around an exceedance frequency curve us determined, while this curve is a model describing an uncertainty itself.

Uncertain factors in the hydraulic model are mainly schematization parameters, but also uncertainties in the boundary conditions, which are the result of the HBV time step. The following factors are assessed:

- Main channel roughness
- Floodplain roughness
- Floodplain topography
- Embankment height
- Weir operation
- Hydrograph shape
- Emergency measures

5.2. Methods

This section describes the different approaches used to determine plausible, non-probabilistic variations for each parameter. The influence of each of the variables is assessed using a flood wave with a peak discharge of 4000 m$^3$/s and an average hydrograph shape. This magnitude is chosen since 4000 m$^3$/s gives significant flooding, but not everything is flooded yet. A different water level or embankment can cause new areas to flood or stay dry.

5.2.1. Main channel roughness

The variations in the main channel roughness are assessed from Chows hydraulic roughness table (Chow, 1959). This table gives a lower and upper limit for the Manning’s $n$ value. With formula 5.1 and an estimate for the bank full water depth, the Chézy values are calculated.

$$C = \frac{1}{n} R^{\frac{1}{2}}$$

In which:

$C$ = Chézy roughness coefficient [m$^{0.5}$/s]

$n$ = Manning’s roughness coefficient [m]

$R$ = Hydraulic radius, cross sectional area divided by wet perimeter [m]

This gives the following upper and lower limits for the main channel roughness. The minimum, average, and maximum are used as scenarios for the sensitivity analysis. Note that his analysis is carried out before the reach from Eijsden to Borgharen is added to the model.
5.2. Methods

5.2.1. Roughness

Chezy roughness value $[m^{1/3}/s]$

<table>
<thead>
<tr>
<th>Reach</th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liege down - Eijsden</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Liege up - Liege down</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Namur - Liege up</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dinant - Namur</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chooz - Dinant</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5.1.: Roughness ranges per river reach based on the values given by Chow. Note that a high Chézy value coincides with a low roughness, or smooth bed. Each reach has different values, while the Manning $n$ values are the same. This is since while calculating the Chézy values, the water depth $h$ ($\approx R$) is used, which differs per reach.

5.2.2. Floodplain roughness

The used floodplain roughness comes from a translation from land use maps to roughness maps. The Dutch LGN6 classification is used as a translation table. Uncertainties in the roughness maps may result from the course Dutch classification, or from the use of too global land use maps. To consider the first error, the Nikuradse roughness length is lowered enlarged with 25 percent. For the second, the urban roughness length is set on 2.5 m instead of the 10 m from the translation table. The land use map we used classifies whole blocks as urban (10 m), neglecting possibilities for the water to flow through and along the buildings, following paths with lower roughness. Expected is that the effect for extreme water levels will be larger, since the overland flow increases. Therefore, the four scenarios will also be simulated for a 20% higher flood wave with a peak discharge of roughly 4800 $m^3/s$ at Borgharen.

5.2.3. Floodplain topography

The floodplain topography (or bathymetry) stems from the DEM with all objects filtered out. The terrain model has a horizontal resolution of 1 m, so an uncorrelated stochastic error will have no effect over large domains, following the central limit theorem: $RMSE/\sqrt{\text{number of tiles}} = 0.12/O(10^5) \approx 0$. A systematic error could cause large deviations. To quantify such error, an overlapping part of the Walloon terrain model with the Dutch ahn2 terrain model is used, as displayed in figure 5.4 on the left. The right part of the figure shows the resulting differences. The systematic error is 8 centimetre. This may result from an error in the ahn2 or the Walloon terrain model. In this analysis, the conservative assumption is made, that it comes completely from the Walloon terrain model. The 8 cm error is thus added to and subtract from topography.
5. Sensitivity analysis

Figure 5.2.: Variations in the DEM. On the left, the area for which the elevation difference is analysed. On the right, a histogram of the differences.

5.2.4. Embankment height

The embankment height is accurately determined from the DEM (see section 3.2.2), which is very accurate. The uncertainty that stems from the data is therefore small. A larger effect is expected to come from the model schematization: the schematised embankment level has one value over the full cell width (100 m). This gives a dilemma between moment of inflow and inflow discharge, as is described in section A.1.2. In the deterministic approach, the 10th percentile embankment height is chosen. In the sensitivity analysis, we take the lowest (0th percentile) and average (50th percentile) value. Figure 5.3 illustrates the length difference between the different percentiles.

Figure 5.3.: On the left, a histogram of heights in one embankment plus the differences between the percentiles and the mean. On the right, a histogram of all percentiles, plus the mean of them.

5.2.5. Weir operation

As shown in section A.1.3, figure A.8, the crest level of the weir can deviate up to 1 metre between the modelled and calibrated scenario. A deviation of 0.5 m over the full range seems a good deviation for the sensitivity analysis. Figure 5.4 illustrates the three scenarios. 0.5 m is low in comparison with the measured
5.2. Methods

deviations, but it is significant since it is applied in all weirs at the same time and over the full discharge range.

Figure 5.4.: Variation in the weir control used for the sensitivity analysis.

5.2.6. Hydrograph shape

Hydrograph shapes are a different matter than the previous variables, since it is a different kind of parameter. It is a boundary condition that determines the quantity of water that enters the model, instead of a parameter that determines what happens with the water in the model. The difference is that inaccuracy in this parameter comes from the hydrological model (HBV), and not from the hydraulic model. The inaccuracy of a boundary condition can come from magnitude, shape or the timing of the laterals. This is the inaccuracy that one certain hydrograph can have and the inaccuracy that is treated in this chapter.

Besides the inaccuracy in a hydrograph, there is also the variety in hydrograph shapes, which comes from the rainfall series. This uncertainty has already been quantified (see section 2), so it is not treated here.

Timing

Before going to the sensitivities of the hydrograph, the hydraulic characteristics of the basin have been analysed. This is described in appendix B.1. Here we list the conclusions:

- The variety in the timing of the laterals is small. For almost all events, all peaks occur in a 1 or 2 day time frame. Differences can be explained by different basin characteristics rather than the rainfall pattern.
- The 24-hour time step used in GRADE is too large for the Meuse basin.

The model uncertainty in the waves comes from the timing of the main and lateral inflows, and the smoothness of the wave. GRADE gives output every 24 hours, so the input peaks can only differ a whole number of days. To assess the influence of this timing, the main and lateral inflows are sorted on magnitude and shifted based on their order. The even numbers are shifted six or twelve hours forward, the uneven numbers backward.

Before the timing is tested with simulations, the time dependence of the GRADE results is analysed. By disaggregating the laterals and the hydrograph at Borgharen, the position of the peak is determined more accurately. A cubic spline interpolation is used to do this, as shown in figure 5.5. There is no physical basis on which this can be justified. For an assessment, it should however suffice.
5. Sensitivity analysis

5.2.7. Emergency measures

In case of an imminent river flood, authorities will likely try to defend the communities against the water with emergency measures. The exact way in which emergency measures play a role is hard to assess, since it depends on administrative factors rather than physics. The embankments of the Belgian Meuse are suitable for placing sandbags. In order to make an estimate of the influence of emergency measures, the river is divided into five equal sections, roughly linked to some large municipalities. The sections are, from upstream to downstream, Namur, Andenne, Seraing, Liège and Vise. One scenario is to place 0.5 m of sandbags over the full length of the river. The other scenarios are defending each section individually, by placing 1 metre of sandbags on the embankment. Note that 1 m is a lot, but here the goal is to prevent flooding in one section, not to give a realistic approach. Note that the Belgian Meuse is a river that reacts much faster to rain events than for example the Rhine. The floods can occur very fast. The threat of flooding might be visible too late for installing sandbags.

5.3. Results

This section gives the results for each parameter variation. The variations are quantified with the discharge, since its magnitude has the most direct consequences downstream (the Netherlands). The hydrograph shape also influences the downstream water levels, but the relation between peak damping and hydrograph deformation is strong, as shown in figure 4.8.

5.3.1. Main channel roughness

The effects of the variations in the main channel roughness are in the same order as the damping due to flooding itself, and might be important. The reason for this large effect on the damping, is that the main channel roughness influences the water depth, and the water depth determines the flooded volume which on its turn influences the amount of peak damping.
Figure 5.6 shows the hydrographs for different roughness.

### 5.3.2. Floodplain roughness

The table below shows the discharges and variations for different roughness. Because the effects are expected to become large when overland flow plays a role, they are also tested for a larger (around once in 50,000 years) wave. The large wave shows a larger effect for the low urban roughness. At such large discharges, the flow over the floodplains is a significant fraction of the total flow, so the roughness will also play a role. The difference in discharge for the $Q = 4000 \text{ m}^3/\text{s}$ case is small, one or two orders of magnitude smaller than the damping due to flooding itself.

<table>
<thead>
<tr>
<th></th>
<th>$Q$ = 4000 m$^3$/s wave</th>
<th>$Q$ = 4800 m$^3$/s wave</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$Q_p$ [m$^3$/s]</td>
<td>Difference [%]</td>
</tr>
<tr>
<td>High roughness</td>
<td>3910</td>
<td>0.05</td>
</tr>
<tr>
<td>Normal roughness</td>
<td>3908</td>
<td>-</td>
</tr>
<tr>
<td>Low roughness</td>
<td>3906</td>
<td>-0.05</td>
</tr>
<tr>
<td>Low urban roughness</td>
<td>3913</td>
<td>0.13</td>
</tr>
</tbody>
</table>

### 5.3.3. Floodplain topography

The effect of 8 cm higher and lower terrain levels on the discharge is small; an order of magnitude smaller than the damping due to flooding. The imposed error of 10 cm is very large, if systematically present over the complete terrain model. The error in the terrain model would need to be unrealistically large for the effect to become significant.

<table>
<thead>
<tr>
<th></th>
<th>$Q_p$ [m$^3$/s]</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>High terrain</td>
<td>3892</td>
<td>0.23</td>
</tr>
<tr>
<td>Normal terrain</td>
<td>3883</td>
<td>-</td>
</tr>
<tr>
<td>Low terrain</td>
<td>3873</td>
<td>-0.26</td>
</tr>
</tbody>
</table>
5. Sensitivity analysis

5.3.4. Embankment height

The table below presents the results for different embankment heights.

<table>
<thead>
<tr>
<th>Embankment Type</th>
<th>Qp [m$^3$/s]</th>
<th>Difference [%]</th>
<th>Volume [hm$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50th p. embankment</td>
<td>3902</td>
<td>-0.15</td>
<td>19.41</td>
</tr>
<tr>
<td>10th p. embankment</td>
<td>3908</td>
<td>-</td>
<td>19.94</td>
</tr>
<tr>
<td>0th p. embankment</td>
<td>3908</td>
<td>-</td>
<td>20.03</td>
</tr>
<tr>
<td>No embankments</td>
<td>3858</td>
<td>-1.28</td>
<td>18.53</td>
</tr>
</tbody>
</table>

The effect of choosing a different percentile for the embankment height is unexpected. Expected is from high to low embankments: increased flooded area and volume, and thus decreasing water levels and discharges. The table shows different behaviour:

- The 50th percentile embankment level has a bit lower discharges than the 10th or 0th percentile. This can be explained by looking at the hydrographs in figure 5.7. The 50th percentile embankments cause a delay in the inflow, so the storage is more optimally used.
- The hypothetical case of no embankments shows indeed a lower peak discharge, but also less inundation.

The varying behaviour shows that the effect of the embankment level is more complicated than expected. This process is further elaborated in the uncertainty analysis.

![Hydrographs at Eijsden for different embankment levels.](image)

5.3.5. Weir operation

Figure 5.8 shows the results for different weir operations. Changing those does not affect the peak discharges much since the weirs are fully open anyway in case of a large flood. This does not mean that the weirs do not affect the water levels in case of a flood: even when fully opened, the weirs are still holding up water by imposing a backwater on the upstream reach. In the high weirs scenario the minimum crest levels are also higher, so the weirs form a bigger obstacle. This extra water level gives more flooding and consequently lower peak discharges.
5.3. Results

<table>
<thead>
<tr>
<th>$Q_p$ [m$^3$/s]</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>High weirs</td>
<td>3699</td>
</tr>
<tr>
<td>Normal weirs</td>
<td>3706</td>
</tr>
<tr>
<td>Low weirs</td>
<td>3710</td>
</tr>
</tbody>
</table>

Figure 5.8: Discharges at Eijsden for a different weir operations.

Important to see is that the exact layout of the weir when fully opened, is important for the discharge. Not only the bottom level, but also the width, lifted gates that interfere with the flow, and floating debris can influence the downstream discharges.

### 5.3.6. Hydrographs: Timing of the lateral peaks

Shifting the lateral inflows can have a significant reducing effect on the peak discharge. This is not caused by the flooding, but by the fact that the composition of the laterals gives a lower and wider wave at Eijsden. This can be seen in the hydrographs in figure 5.9. The reduction is more than 100 m$^3$/s and of the same order as the reduction due to flooding itself. Note that this deviation does not come from a hydraulic model uncertainty, but from the time step in the hydrological model.

<table>
<thead>
<tr>
<th>$Q_p$ [m$^3$/s]</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
<td>3908</td>
</tr>
<tr>
<td>6 h shift</td>
<td>3860</td>
</tr>
<tr>
<td>12 h shift</td>
<td>3767</td>
</tr>
</tbody>
</table>

Figure 5.9: Discharges at Eijsden for a different timings of the laterals.

### 5.3.7. Hydrographs: Smoothness of the wave

Adjusting the smoothness of the hydrographs does not have a large effect on the discharges. The reason that the resulting peak discharges are higher is because the interpolation method adds extra volume to the peak.
5. Sensitivity analysis

![Graph showing discharge over time](image)

Figure 5.10: Discharges at Eijsden for a regular and smooth wave.

5.3.8. Emergency measures

The following table shows the results for the use of emergency measures. Figure 5.11 also shows the flooded volume per area for each scenario.

<table>
<thead>
<tr>
<th>Location of sandbags</th>
<th>Q_p [m^3/s]</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Namur</td>
<td>3914</td>
<td>0.15</td>
</tr>
<tr>
<td>Andenne</td>
<td>3912</td>
<td>0.10</td>
</tr>
<tr>
<td>Seraing</td>
<td>3936</td>
<td>0.72</td>
</tr>
<tr>
<td>Liege</td>
<td>3913</td>
<td>0.13</td>
</tr>
<tr>
<td>Vise</td>
<td>3888</td>
<td>-0.51</td>
</tr>
<tr>
<td>Everywhere</td>
<td>3923</td>
<td>0.38</td>
</tr>
<tr>
<td>Nowhere</td>
<td>3908</td>
<td>-</td>
</tr>
</tbody>
</table>

![Diagram showing flood volumes](image)

Figure 5.11: Flood volumes for different variations on placing sandbags.

The general pattern is; protection an area gives less flooding, so higher discharges at Eijsden. The effects are small: an order of magnitude less than the effect of flooding itself. The flood volumes also show that placing 1 metre of sandbags is often not enough to protect an area against the flood.

Two scenarios jump out: protecting Seraing and Visé:

1. Protecting Seraing has a relatively large influence on the discharge, more than you would expect if you look at the reduced volume. Seraing has low-lying areas protected by quays, similar to a Dutch polder or retention basin. In contrary to other areas along the Belgian Meuse, which are more of the valley...
type, Seraing can top of relatively large but narrow part of the hydrograph when the inflow level is exceeded. Blocking Seraing will thus remove this possibility to occur, and have a negative effect on the downstream discharge.

2. Protecting the region around Visé does not prevent flooding, as is the case for Seraing. It does however delay the flooding, with the result that another part the hydrograph is reduced. The timing is better, so the peak discharge at Eijsden is lower.

5.3.9. Upper and lower bound

As a last case, a combination of model parameters is considered. This is done, to find out if a number of deviated parameters combined lead to higher deviations in the discharge. Two simulations are carried out, one with a low main channel and floodplain roughness, high weirs, 50 cm sandbags, and a high floodplain topography and another with high roughness, low weirs, low embankments and a low topography. The boundary conditions are equal.

<table>
<thead>
<tr>
<th>Time [days]</th>
<th>Water level [m + DNG]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>2600</td>
</tr>
<tr>
<td>5</td>
<td>2800</td>
</tr>
<tr>
<td>6</td>
<td>3000</td>
</tr>
<tr>
<td>7</td>
<td>3200</td>
</tr>
<tr>
<td>8</td>
<td>3400</td>
</tr>
<tr>
<td>9</td>
<td>3600</td>
</tr>
<tr>
<td>10</td>
<td>3800</td>
</tr>
</tbody>
</table>

Figure 5.12.: Discharge at Eijsden for the upper and lower limit.

The resulting hydrographs look much like the ones in case of only a roughness variation. The ranges are not that extreme considering the schematization. The upward deviation is even lower than in case of solely the roughness variation. The embankment height might play a role here. The roughness seems to dominate the total sensitivity.

5.4. Discussion

The question to be answered in this chapter is which uncertainties can have a large influence on the results. The following table shows the effect on the discharge when a variable is varied up or down. Up or down is a bit abstract in this sense, since it can also implicate a different waveform for example. Up means the variation giving higher discharges, down means lower discharges.

<table>
<thead>
<tr>
<th></th>
<th>$Q_p$ [m$^3$/s]</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>High extremes</td>
<td>3741</td>
<td>-2.40</td>
</tr>
<tr>
<td>Normal</td>
<td>3833</td>
<td>-</td>
</tr>
<tr>
<td>Low extremes</td>
<td>3986</td>
<td>3.99</td>
</tr>
</tbody>
</table>

1No embankments causes an error, as will be described in the uncertainty analysis. Therefore we choose the 0th percentile embankment.
5. Sensitivity analysis

5.4.1. The important parameters

<table>
<thead>
<tr>
<th>Variable</th>
<th>Variation down</th>
<th>Variation Up</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main channel roughness</td>
<td>-1.93 %</td>
<td>4.20 %</td>
</tr>
<tr>
<td>Floodplain roughness</td>
<td>-0.05 %</td>
<td>0.13 %</td>
</tr>
<tr>
<td>Floodplain topography</td>
<td>-0.26 %</td>
<td>0.23 %</td>
</tr>
<tr>
<td>Embankment height</td>
<td>-0.15 %</td>
<td>0.00 %</td>
</tr>
<tr>
<td>Weirs</td>
<td>-0.19 %</td>
<td>0.11 %</td>
</tr>
<tr>
<td>Timing of the laterals</td>
<td>-3.61 %</td>
<td>0.33 %</td>
</tr>
<tr>
<td>Smoothness</td>
<td></td>
<td>0.33 %</td>
</tr>
<tr>
<td>Emergency measures</td>
<td>-0.56 %</td>
<td>0.72 %</td>
</tr>
</tbody>
</table>

Table 5.1: Table summarizing all sensitivity variations

In general, the variation in discharge is small. Note however that the reduction at a 4000 m$^3$/s wave is around 4%, so a variation in the discharge of a few percent is equally large as the effect of flooding itself. Two variables jump out in magnitude, the main channel roughness, and the timing of the laterals.

**Main channel roughness**

The main channel roughness has a large influence because of the direct and large effect on the water level. It thus influences the amount of flooding, and the discharge reduction. The chosen variations (varying the Chézy value $10 \, \text{m}^{0.5} / \text{s}$ up or down) are large but realistic. A range as we have calculated is therefore realistic too.

**Timing of the laterals**

The timing of the laterals is important because the travel time of flood waves in the basin is small compared to the GRADE time step (1 day). When the lateral peaks occur in the same 24 hours, they will enter the hydraulic model at exactly the same moment. This mostly leads to a higher wave than would have occurred if the peaks had entered at their own peak moment in the 24 hours. The problem here is that the 24-hour period comes from the hydrological model and the rainfall generator. If the timing is shifted, it creates a mismatch with the time steps in these models, which is undesirable. Secondly, as shifting reduces almost all high peaks, which are produced by an assembly of small peaks, lower peaks might increase in magnitude since the timing becomes 'better'. In this research only the highest waves are simulated, so adding a time shift would add a bias to the result.

This research does not go further than the recommendation to investigate the difference in results, when using a smaller time step in GRADE. The uncertainty will not be taken into account, since this is too elaborate and no longer in the scope of this research.

**Recommendation**: It is recommended to use a smaller time step in the rainfall generator and HBV during the extreme events.

**Emergency measures**

Emergency measures are the next one in the order of magnitude. This is a difficult case, since it depends on human and administrative factors rather than physics. It is however, a scenario that will most likely lead
to higher discharges, and thus an upward shift of the exceedance frequency curve. Emergency measures will therefore be elaborated further in the uncertainty analysis to get a better estimate for the discharge differences involved, as well to have an answer to the question: What if?..

5.4.2. The impact of the used hydrograph

The variations are determined for an average hydrograph with a peak discharge of 4000 m³/s. However, what if the boundary conditions are different? In general a higher peak discharge gives higher water levels and thus more flooding, as can be seen in figure A.25 on the right. The expected results for a wider wave would be less damping, since the peak contains more volume and the storage is probably full before the peak has passed. A narrow wave should show the opposite behaviour, more damping due to the smaller peak volume, which allows a larger part of the wave to be damped. The results are expected to average out when considering all hydrograph shapes. In the uncertainty analysis, the hydrograph shape is treated more detailed.

5.5. Conclusion

This chapter's conclusion is that the main channel roughness is by far the most important parameter that we take with us to the uncertainty analysis. The embankment height is also analyzed a bit more, due to the complex behaviour. The emergency measures will also be treated in the uncertainty analysis, particularly because it is interesting to know its effect. The 24-hour time step of the boundary conditions has a large influence on the peak discharges, but will not be researched further, since it is outside the scope if this project. It is recommendable to refine the GRADE time step for the Meuse, since both the timing and the smoothness have a large influence on the results for this particular river.
6. Uncertainty analysis

Chapter summary

In this chapter, the probabilistic uncertainties are determined of the parameters which have proved to be important in the sensitivity analysis. These are the main channel roughness and emergency measures. Furthermore the effect of different embankment heights is determined better.

The roughness is again the most important variable. A variation in the roughness of from 15 m$^{0.5}$/s total gives a discharge variation of about 250 m$^3$/s max. Again, this is because the total amount of damping is not that big either. The water level range due to the variation is about 2 metres.

The embankment height determines how well an area can top of the hydrograph. This gives that the use of sandbags does not necessarily lead to a higher discharge downstream. On average it is the case, but it heavily depends on the specific configuration.

When the uncertainties are averaged out (taking the expected values), the exceedance frequency curve nearly rises. This is because the uncertainties are small. The hydraulic model is almost a deterministic model.

The analysed uncertainties are only the ones in the hydraulic model, so the contribution of this model to the total uncertainty is small. Combined with the rain series and the HBV model, the uncertainty around the exceedance frequency curve is still large.

6.1. Introduction

As the assessment in the sensitivity analysis appointed some factors that can cause the model results to deviate significantly, this chapter looks closer into these sources. The goal is to find out what the uncertainty around the exceedance frequency curve is. To find this out, the individual uncertainties will be investigated more detailed: What kind of probability distribution do they have, and to what kind of discharge distribution at Eijsden does this lead. The roughness and emergency measures are quantitatively treated. For GRADE Rhine there was some discussion on the correlation of the bottom roughness between Germany and the Netherlands, which is for the Meuse also treated in this chapter. This uncertainty analysis only treats the uncertainties in the hydraulic model. Since the rainfall generator and hydrological model also contain uncertainties, they will be combined with the hydraulic model resulting in the total uncertainty for GRADE for the river Meuse.

6.2. Methods

6.2.1. Main channel roughness

As presented in the results, the main channel roughness turns out to be an influential variable, so we are assessing it more detailed. Note that we split the river into three sections, for which we determine the rough-
ness individually. We do this to take differences in the river geometry and partial correlations into account.

The magnitude and variation of the roughness

Ideally, the roughness is determined with a combination of calibration and theoretical relations. A study with a similar goal was carried out by Warmink, Booij, Van der Klis & Hulscher (2013). They had sufficient sediment and bed form measurements to their disposal, on which they applied several roughness formulations. Unfortunately, such methods cannot be applied, since the specific data is not available. Calibration is therefore the only method to estimate the mean value and variation of the roughness in each channel section. Given the data and the fact that not only the roughness, but also model irregularities are included in the coefficient, it is the best option. Water level and discharge data are fairly complete available for the period from 1999 to 2012. In this time window, about 40 peaks with a discharge at Chooz higher than 400 m³/s occurred. The measured data on these peaks used to calibrate the model. For more information on the calibration one is referred to appendix A.3, the result is shown in figure 6.1.

![Figure 6.1: Roughness ranges per river reach from calibration.](image)

Type of probability distribution

The roughness values generated by the calibrations (34 in total) are not numerous enough to derive with certainty a probability distribution by fitting. Literature and similar research is used to determine the type of probability distribution for the sediment.

Prinsen, van den Boogaard & Hegnauer (2015) choose a parabolic distribution to represent the roughness variations in their uncertainty analysis for GRADE Rhine. The limits of the distribution are given by adding and subtracting 20% to the roughness value. The values and uncertainty bound are chosen from expert opinions and available calibration data, and well substantiated. The form of the probability distribution is not explained in the report, so it cannot be used for that matter.

Research often chooses the lognormal distribution to describe the sediment distribution in a river bed. Middleton (1970) discusses several explanations to link sediment distributions to multiplicative processes, which are described by log-normal distributions. Spencer (1963) concludes that a sediment distribution can be described as a mixture of three or less fundamental populations of log-normal grain sizes: gravel, sand and clay. Based on the above literature we assume the grain sizes are log-normally distributed.

Assuming the roughness is linearly related to the characteristic roughness length $k$, the lognormal sediment distribution can be translated to Chézy values using the White-Colebrook equation:

$$ C = 18 \log \left( \frac{12R}{k} \right) $$

(6.1)
6. Uncertainty analysis

The log in this equation reduces the lognormal distribution to a normal distribution. From a theoretical point of view the roughness distribution should thus be a normal distribution. Some notes should be made with this statement however. Chézy values of 20 m$^{0.5}$/s or 80 m$^{0.5}$/s are very unlikely. For example, a high Chézy means a high smoothness, or small particles. This leads to bed-forms, which result in higher roughness. The spread of the distribution should not be too big (30 to 70 m$^{0.5}$/s), to hypothesize a normal distribution.

From a theoretical point of view, the variation should not differ much between different sections since the bottom does neither. The results do however show quite some difference in the root mean squared errors. We choose to use the calibrated values, since we are not only calibrating the bottom roughness, but also other model uncertainties.

**Correlation between sections**

The reason to use different distributions for the roughness per section in the first place, is that the layout of the river sections do differ significantly in space. Between these different roughness values there is likely a spatial correlation, which needs to be taken into account. An uncorrelated relation might give different results than a fully correlated relation. In this research, correlations are taken into account, since it gives the most realistic representation of reality. The correlations are determined from the calibration. Figure 6.2 shows the results.

![Figure 6.2: Correlations between roughness values.](image)

Now we have determined the uncertainty in the roughness, with the correlations between the reaches. Figure 6.3 summarizes the results.
6.2. Methods

Generating correlated samples

Now the task is to draw correlated samples. A larger sample that fits the chosen distribution is needed. The generated sample should fit both the distributions and have matching correlations. We first determine the correlations in the tails to see which copula we should use when creating our sample. We scale the values to standard normal, and determine the correlation in the quadrants:

The correlations in the southwest and northeast quadrants show no clear relation. There are differences, but they are not consistent for all combinations. A Gaussian copula with no preference for the correlation in one of the tails is the best option. The Clayton and Gumbel copula are also fitted and do not give a different conclusion.

To generate a sample, a large correlated sample is generated from a normal copula. From the distribution for the second river segment, we determine which indices to take so our normal distribution is preserved.
These indices are also taken from the other distributions, which gives roughly the same correlation as the correlation from the large sample. This procedure is repeated many times, until a distribution for the first and last section is found, which also fits a normal distribution well. The result is the following sample, which is used in the calculations.

![Figure 6.5: Generated sample of roughness values](image)

---

### 6.2.2. Embankment height

Discussing the variation in the embankment height will not be done here. After all, the exact height of the embankment does not matter much for the results. The effect of higher or lower embankments is still researched, since the embankment level can be changed by placing sandbags. The water levels on the main channel can also be changed, which has a similar effect. Not exactly similar however, since changing the water level also gives a larger potential flood volume. To investigate the effect of specific heights, the embankment is lifted and lowered 30 or 60 cm, and simulated with three waves.

### 6.2.3. Emergency measures

Estimating the possible emergency measures is difficult. Coordinating operations as placing sandbags in chaotic situations might not succeed, however the scenarios in which placing sandbags is useful, are not that extreme. When an extreme flood hit the Elbe in 2013, 1 metre of sandbags where placed over 100 km length, so we can assume the same could be done in Belgium. The river Meuse reacts fast to a rainfall event so the lead times are small, which decreases the probability of success. For the less extreme floods it however often suffices to 'fill some holes', so not the full river length needs to be protected. The embankments are suited for placement of sandbags.

The river is split into 5 sections as in the sensitivity analysis, but now such that the volumes and lengths are more equally distributed. For each section there is a 50% chance that the placement of sandbags, 50 cm height (a more realistic estimate than 1 m, given the described conditions), is successful. This leads to \((2 \text{ options})^{5 \text{ regions}}\) = 32 possibilities to be simulated. The different regions are shows in figure 6.6.

### 6.2.4. Incorporating uncertainties in the exceedance frequency curve

The uncertainties around the exceedance frequency curve gives the difficulty that there are multiple possibilities for the curve. All these possibilities have a probability, the uncertainty around the exceedance frequency curve. The result is an exceedance frequency curve, which is probability distribution, with an uncertainty...
6.2. Methods

Figure 6.6: Map showing the sandbag locations. From upstream to downstream: Namur, Andenne, Amay, Seraing and Liège, and Visé.

(also a probability distribution) around it. To combine this to one exceedance frequency curve the weighted average of all curves is taken. This gives the expected exceedance frequency for each discharge. This is approach is conform the approach used in VNK (2015). The uncertainties have however not changed by averaging them out. The discussion whether it is the right approach is not conducted in this research. The approach is used to be consistent with the used approach in the Dutch water safety.

Without uncertainties, finding $P(Q > q)$ for each discharge is straightforward, the exceedance probability where the exceedance frequency curve crosses $Q$. With uncertainties it becomes a bit more complicated. With the integral in equation 6.2 the expected exceedance probability for a certain discharge can be calculated. In this integral $P(Q > q|a)$ is the one of the possibilities for $q$ to be exceeded, and $f(a)$ the probability (weighing factor) for this to happen.

$$P(Q > q) = \int_0^\infty P(Q > q|a)f(a)da$$  \hspace{1cm} (6.2)

In which:

- $Q$ = discharge [m$^3$/s]
- $q$ = discharge exceedance level [m$^3$/s]
- $a$ = deviation [m$^3$/s]

Calculating this integral for each discharge gives the expected probability, which can be used as representative value. To do so, a script is written which is based on the MATLAB script developed by Geerse & Wojchiechowska (2015) to solve this integral. A few adjustments have been made to be able to use different distributions on one curve, and use kernel, convoluted or empirical distributions.

The process of weighing the uncertainties works as follows:

First $f(a)$ is calculated on a numerical grid for $a$. This is actually derivative of the exceedance frequency curve itself, an exponentially decreasing function.

For each discharge $q$ (also on a numerical scale) and each $a$, $P(Q > q|a)$ is calculated. This exceedance probability is calculated as $1 - CDF(Q)$. The CDF might need to be interpolated, since the probability distributions of the uncertainties are only given at certain discharges. This interpolation is done by logarithmically interpolating the location of the percentiles of the neighbouring distributions.

The numerical integral is shown in equation 6.3 purely to show the numerical implementation. For a probability grid (in the integral a return period grid for simplicity), the sum of the exceedance probability at the
certain return period, times the probability of the return period gives the exceedance probability.

\[
P(Q > q) = \sum_{T=10}^{\infty} (1 - F_T(Q - q)) \left( \frac{1}{T + \Delta T} - \frac{1}{T} \right)
\]

(6.3)

In which:

- \( T \) = return period [years] Note that \( T > 10 \), for this notation, since otherwise \( P = 1/T \) is not valid.
- \( F_T \) = The cumulative distribution function at the return period

6.3. Results

6.3.1. 1D Roughness

The expected behaviour for the main channel roughness is:

Higher roughness (lower Chézy) ⇒ higher water levels ⇒ more flooding ⇒ lower discharges.

Almost all flood prone areas are located along the downstream half of the Belgian Meuse, the second roughness section. Expected is that the roughness along this section is important, but also the roughness on the Dutch section, since this gives a backwater through the second section. The farthest upstream section is probably less important, since the only flood prone areas are on the very end of the section and thus influenced by the backwater of the second section.

The upper row of figure 6.7 shows the results from the simulations with a flood wave selected on its average hydrograph shape, and a peak discharge of 3500 m³/s at Borgharen.

A clear relation between the roughness in the second reach and the discharge at Eijsden is visible. For the other reaches the correlation is also high. This is due to the correlation between roughness and not due to backwater effect of water level effects. This is clarified in the second and third row of figure 6.7, where the results for respectively none and full correlation are shown. The second row shows that the correlation between the third and first reach and the discharges is not significant (\( \rho \) is less than 0.279). The third row shows the same correlations because the relative roughness and discharges are the same. The results for the different correlations are not the same. There is some effect, but it is negligible.

For low roughness, the discharge reaches an upper limit. This makes sense, since the discharge cannot endlessly be enlarged. At a certain point, a lower water level does not lead to less damping, since no flooding occurs, and there is thus none left to be reduced. When the discharges are higher this limit will not be reached, so there is no upper limit and the tails are equal.

Interesting to know is that the variations in the roughness give a variation in the maximum water level of 1.5 - 2.0 m over the full river.

The calculations for the roughness variations are repeated for several waves with a normal shape. For larger discharges, the uncertainties also get larger. We choose a Generalized Extreme Value distribution to fit the uncertainties around the exceedance frequency curve. This distribution turns out to be one of the best fitting (based on maximum likelihood estimates) for each discharge set, and it can take an upper limit, which the uncertainty around the lower discharges also has. The result is the following exceedance frequency curve with uncertainties.
Figure 6.7: Relation between roughness in each section and discharge at Eijsden for $Q = 3500$ m$^3$/s. The histogram on the right shows the distribution of the resulting discharge at Eijsden. The circled zigzag is caused by the embankments, of which the behaviour is explained in the next subsection.
6. Uncertainty analysis

Figure 6.8.: Above: Exceedance frequency curve with variations due to the main channel roughness. Below: histograms of maximum discharges at Borgharen, for each set of simulations.

The variations in discharge have also been checked for the case without flooding. This gave a bandwidth of 20 m³/s in which the discharge varied. This range is more or less constant for different peak discharges. We can thus conclude that the variation in discharge is mainly caused by flooding.

6.3.2. Embankments

Before showing the results from the emergency measures, this section describes the behaviour of the model to variations in the embankment height. These are not taken into account in the total uncertainty, due to their small size. They are however important for understanding the behaviour of the model.

The expected behaviour when changing the embankment height (or changing the water level in the river the opposite way) is:

\[ \text{Higher embankments} \Rightarrow \text{less flooding} \Rightarrow \text{higher discharges}. \]

Simulations show a more complicated picture, there even seems to be an inverse relation between embankment level and discharges for certain ranges. Figure 6.9 shows the result for a number of simulations with 3 discharges and different embankment height.
6.3. Results

Discharge [m³/s]

-0.6
-0.4
-0.2
0.0
0.2
0.4
0.6

embankment height [m]

Figure 6.9. Behaviour of the model to variations in the embankment height.

The variations in discharges are generally not that big, the discharges gradually increase when the embankments are lifted. In certain ranges however, there is a relatively large damping. What causes this large reduction in discharge, for higher embankments, we explain by looking at the wave shown in figure 6.10 on the left. The height of the embankment determines the volume that can be stored. High embankments give lower volumes, but at the same time, they delay the starting moment of inflow. As illustrated in the left figure, later inflow can cause the peak of the wave to be stored, which reduces the discharge. For different discharges and hydrograph shapes, the water levels are different, and the areas that optimally reduce the peak too. The reduction we see at the 3500 m³/s wave is thus not the same as we the one at the 4000 m³/s wave.

Figure 6.10. Example showing the that higher embankments can lead to lower discharges

The second effect that can be observed, is that the peak discharges reduce for lower embankments. This makes sense, as the lower embankments give more room to the water, giving a higher overall damping. The model schematization or software does however also contain a bug, which reduces the total amount of water in case of low embankment. This effect will also reduce the discharges, so one should watch out with drawing conclusion on this effect. Appendix A.5 gives more information on this error. It also shows that normal embankments produce correct results, while no embankments create a water deficit. We are only working with the first case, so the model remains valid.

6.3.3. Emergency measures

The variations in discharges due to the placement of sandbags are mostly positive, but not that large. This might be surprising, since it looks like a drastic measure. Placing half a metre of sandbags has roughly the
same effect as a higher roughness (Not precisely the same, since the floodplains are not lifted). For the roughness we saw a water level difference of 1.5 to 2.0 m resulting in 200 m$^3$/s variation in discharge. It is therefore not strange that 0.5 m sandbags give $\approx 50$ m$^3$/s variation. Figure 6.11 shows the histograms of the discharges. No probability density function is fitted to the histograms. After all, the input is defined by some discrete possibilities, not a probability distribution, so considering extrapolating for the extremes is not necessary, they are in the sample already. An empirical distribution function is used to adjust the exceedance frequency curve to uncertainties.

Figure 6.11.: On top, an exceedance frequency curve with variations due to emergency measures. Below, a histograms of maximum discharges at Borgharen, for each set of simulations. The black dashed lines show the reference (no sandbags) discharge.

For the larger waves, the histograms show multiple modes. We will consider the flood wave with $Q_p = 4500$ m$^3$/s to find out what causes this. Table 6.1 shows the average discharge with or without sandbags, for each region. For this particular wave, protecting Namur and Seraing-Liège has a relatively large effect. For Namur higher embankments means a more efficient peak damping, for Seraing and Liège the opposite. The embankments are thus causing most of the spread in the resulting discharges, more than the on average higher embankments.

<table>
<thead>
<tr>
<th></th>
<th>Namur</th>
<th>Andenne</th>
<th>Amay</th>
<th>Seraing-Liège</th>
<th>Vise</th>
</tr>
</thead>
<tbody>
<tr>
<td>With</td>
<td>4365 m$^3$/s</td>
<td>4352 m$^3$/s</td>
<td>4351 m$^3$/s</td>
<td>4342 m$^3$/s</td>
<td>4352 m$^3$/s</td>
</tr>
<tr>
<td>Without</td>
<td>4340 m$^3$/s</td>
<td>4353 m$^3$/s</td>
<td>4354 m$^3$/s</td>
<td>4364 m$^3$/s</td>
<td>4353 m$^3$/s</td>
</tr>
</tbody>
</table>

Table 6.1.: Average peak discharge when protecting or not protecting a region with sandbags.

Although the use of emergency measures has a varying effect on the downstream discharges, this is of course
not the goal of emergency measures: protecting the urbanized floodplains against the flood. As most the areas will flood anyway, the placement of sandbags is not effective to prevent flooding but it does reduce the impact. When the mean flooded area per number of defended sections is compared to the flooded volume, an almost linear relation is visible. Table 6.2 shows this.

<table>
<thead>
<tr>
<th>$Q_{ref}$</th>
<th>Flooded area [km$^2$] per number of protected regions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>None</td>
</tr>
<tr>
<td>3004 m$^3$/s</td>
<td>2.2</td>
</tr>
<tr>
<td>3454 m$^3$/s</td>
<td>8.5</td>
</tr>
<tr>
<td>3882 m$^3$/s</td>
<td>19.3</td>
</tr>
<tr>
<td>4316 m$^3$/s</td>
<td>30.2</td>
</tr>
<tr>
<td>4731 m$^3$/s</td>
<td>37.9</td>
</tr>
</tbody>
</table>

Table 6.2: Average flooded area per number of protected regions. The reference discharge is the discharge without sandbags at Eijsden.

In contrary to the downstream discharge, the flooded volume is not strongly relation to protecting a certain area. Placing sandbags at one of the sections is generally not much more effective than placing them at another.

6.3.4. Combining the uncertainties

The uncertainties from the rainfall and HBV-runoff are combined with the uncertainties from the hydraulic model to get a complete picture of the uncertainty, but also to put the uncertainty from the hydraulic model into context. The use of emergency measures is not taken into the total uncertainty, since it is a different kind of uncertainty: It comes from human behaviour, not a physical or statistical factor that is estimated. It is therefore not desirable to add this uncertainty to the frequency curve.

For combining the uncertainties, we assume they are independent. For the sources of the uncertainties this is true, but the might not be case for the resulting discharges. A higher discharge from the run-off will give higher water levels, and thus lead to a different behaviour when varying the roughness. Because the confidence bounds are quite constant, this dependence is neglected.

The convolution integral is used to combine the two sources of uncertainty, since the PDF of the sum of two independent PDFs is given by the convolution integral. With the convolution integral we create a new exceedance frequency line, which is shown below.
Figure 6.12 shows the total exceedance curve with all uncertainties. The grey curves are the current curve and confidence bounds for GRADE. Note that the span of the bounds does not become much larger when the hydraulic uncertainties are added. This is because of the assumption that the uncertainties are independent. It is easiest to comprehend for the sum of independent random variables from a normal distribution, which is a normal distribution in which the standard deviation is the square root of the sums of the squares (as in the Pythagorean Theorem). As one of the uncertainties becomes much larger, the contribution of the other becomes smaller, as is illustrated below. If the uncertainties from the emergency measures had been added too, the result would not differ significantly, since the spread from the emergency measures was relatively small.

Note that the curve with incorporated uncertainties deviates significantly from the average curve. This is a consequence of the large uncertainties.

### 6.4. Discussion

The goal of this chapter is to find out what the magnitude of the uncertainty around the exceedance frequency curve is. To find this out, the uncertain factors are detailed quantified. This is mainly the hydraulic roughness of the main channel, since this is the most important factor, and the only hydraulic uncertainty added to the exceedance frequency curve.
6.4.1. Main channel roughness

The main channel roughness has purely been determined by calibration. The calibration results from openDA are checked, and generally fit the measurements well. The higher peaks better than the lower, but these are also necessary to have a large enough sample, 34 peaks. Flood waves which clearly show a measurement error have been filtered out. Although the conditions for the calibration could be better (more data, more stations) the results are consistent, and there is no need of expert judgement. It might however still be good to compare the found values with literate and similar research. The next list shows Chézy values for the Meuse found in literature.

- The values for the roughness from Chow (1959) are roughly between 43 and 59. For the first two reaches this seems to be a plausible range, the third reach shows lower values. The river does however also get less canalized and 'clean' in the Netherlands, so a range from 35 to 47 m²/s (one step less 'clean') would be more plausible. This matches our values for the last reach.
- Paalvast (1993) gives a Chézy value of 40 m²/s, for the Moyenne Meuse, which is the Meuse upstream of Belgium.
- Rijkswaterstaat/RIZA (1994) gives a Chézy value of 46 m²/s for the Grensmaas (from Borgharen to Linne, so downstream of investigated reach).
- Dewals et al. (2012) use a Strickler coefficient of 30 at Chooz to 40 at Lixhe m²/s. Translating this to Chézy values gives roughly 43 m²/s at Chooz to 56 m²/s at Lixhe. These are the only values found for the Belgian Meuse. The source of these values is however unknown.

The general trend found in literate, is from rough upstream to smooth downstream. Compared to the calibrated values there are quite some differences, however not large enough to conclude the calibration produced incorrect values. Remember that the calibration also corrects for schematization errors, not only the bottom roughness.

The main channel roughness leads by far to the largest uncertainty in the discharge, with the 95% confidence bounds spanning roughly 200 m³/s. We saw that this is caused by the roughness in the section from Namur to Liège. The deviation of the roughness in this section is rather wide (10 m²/s up and down), so our confidence bounds are probably on the conservative side.

6.4.2. Influence of the hydrograph shape

As already discussed in the sensitivity analysis, the hydrograph shape can influence the uncertainty bounds. Peaked waves can be damped more than wide waves. To find out how this relation works more detailed, the amount of damping is investigated for a number of different hydrographs. A selection of 50 different hydrographs is scaled such that the peak discharge at Eijsden is 4000 m³/s in the case of no flooding. The waves have been simulated with the flood model with low, normal, and high roughness. The result is shown in figure 6.13.

The red dots indicate the curvature and discharge at Eijsden without flooding, the blue ones with flooding. The relation between the blue dots is linear (a second order polynomial does not give a different line). By using a hydrograph with an average curvature, also an average amount of damping is achieved. The grey dots and lines show the results for a high and low roughness. This shows that the uncertainty also increases with the amount of damping, which makes sense. This effect does not average out completely by taking the average curvature, since high uncertainties contribute more to a higher exceedance frequency curve than low uncertainties, when averaging out the uncertainties. On the total uncertainty, the effect is probably negligible. To know it with accuracy, more simulations are necessary.
6. Uncertainty analysis

Figure 6.13: Relation between curvature and damping for different roughness. A higher curvature (peaked wave) gives more damping and a larger uncertainty band. The differing line directions for low curvatures are the result of calculation of the curvature, which works less well for wide waves.

6.4.3. Correlation between sections

The results show that the amount of damping is caused by the presence of floodplains and the roughness in the particular section. The downstream roughness has no demonstrable effect on the damping in a river section. The upstream roughness does neither influence the damping, but it can influence the discharge (and water levels) due to upstream damping. A positive correlation of the roughness with the upstream reach leads to lower water levels in the downstream reach in case of a high roughness (more damping), and higher water levels in case of low roughness (less damping). For the upstream reach, nothing changes due to the correlation given that there is no damping even further upstream.

When this knowledge is applied to the relation between Belgium and the Netherlands, the correlation between the roughness on both sides of the border is not important for Belgium, but it is for the Netherlands. When a once in X year load is determined for the Dutch levees with uncertainties for the water level and the incoming discharge, it is recommendable to take a correlation between the two into account. Not doing so leads to an overestimation of the water level, for the reasons described above. den Bieman (2015) demonstrates this and quantifies the difference between no and full correlation for the Rhine as 0.3 m. This relation (not the same value) also applies on the Meuse.

Ideally, the GRADE model would cover the Dutch Meuse too. This would solve the difficulties with correlations and transitions between models, and also give better estimates for the lateral discharges on the Meuse in Limburg. Given the fact that the timing of the laterals in French and Belgium is important for the genesis of the wave (Ogink, 2006), it could be that the Dutch laterals contribute to this process too. If a GRADE model of the complete Dutch Meuse could give accurate predictions for the water level, the GRADE waves could directly be translated to water level exceedance frequency curves, which in the end determine the load on the flood defences.

6.4.4. Emergency Measures

The use of emergency measures turns out to be ineffective for preventing flooding. Defending a region displaces the problem downstream, so to be effective all vulnerable embankments should be protected. The range of discharges for which placing 0.5 m of sandbags keeps out the water, is small.
6.5. Conclusion

The choices made for schematising the use of embankments are not well based. No is taken no account of which areas need to be protected, and the division into regions is quite arbitrary. The results show that this does not matter that much. Due to the specific response to the embankment height, and the fact that mostly the problem is only displaced, placing sandbags does not show a predictable response.

The choice of an empirical distribution to represent the effect on the discharge, can also be criticized, since the effect for discharges in between the simulated ones can probably not be represented by interpolating. However here again, the influence of a different distribution does not matter that much for the results.

6.4.5. Incorporating all uncertainties in the exceedance frequency function

The total uncertainty around the exceedance frequency curve is small, so small that the model gives almost deterministic results. This is however only the hydraulic model. There is still a lot more uncertainty in GRADE, from the hydrological model and rain series. When these factors are added, the total uncertainty gets much larger. When combined, the contribution of the hydraulic uncertainties is very small. The total effect of taking flooding with uncertainties into account is a lower exceedance frequency curve.

Averaging all uncertainties in the exceedance frequency function gives a significantly higher curve. Again this is mainly the effect of the rainfall and run-off uncertainties, which are large. The curve with flooding and uncertainties is still higher than the old HR2006 curve, because uncertainties are incorporated in GRADE.

6.5. Conclusion

The research question to be answered from the last two chapters is:

What is the uncertainty in the flooding parameters and what is the total uncertainty around the impact of floods on discharge and hydrograph shape? The uncertainty analysis shows that the main channel roughness is by far the most influential model variable for the result. The variation in roughness values between Namur and Eijsden, gives a variation in the discharge of 200 m³/s between the 95% confidence bounds. Given the discharges at Chooz and the laterals, the model can thus predict the discharge at Eijsden with roughly 100 m³/s: an accuracy of roughly 2.5%.

Emergency measures has a small and varying effect on the peak discharge. Due to the specific properties of the embankments, the result of placing sandbags can be both a lower or a higher discharge. Sandbags are generally ineffective since the water level exceeds the embankment with more than a few decimetres. There is however a clear negative relation between the number of protected regions and the flooded area.

Incorporating the hydraulic uncertainties in the exceedance frequency function gives approximately the same discharges, since the uncertainties are so small. When all GRADE uncertainties are incorporated, the curve does rise significantly. The addition of the hydraulic uncertainties is however very small, which makes that the adjusted exceedance frequency curve is still below the one for the current GRADE curve. Considering flooding with uncertainties gives physically and statistically more sound estimate for extreme floods on the Meuse. Is it thus recommended to use the GRADE exceedance frequency curve with flooding, for estimating extreme floods on the Meuse.
7. Impact in the Netherlands

Chapter summary

In this chapter, the impact of the upstream flooding for the Netherlands is determined. As the results in chapter 7 showed, considering flooding gives lower discharges, but compared to the HR2006 the discharges rise.

The water levels in the Netherlands are determined by comparing the results, the peak discharge, and hydrograph shape, with reference WAQUA simulations. This way the water level differences between flooding and no flooding are determined as well as the differences between GRADE with flooding and the HR2006.

The discharge reduction has a reducing effect on the water levels in the Netherlands. The hydrograph shape compensated this, since retention functions worse for wider hydrographs. The water level reduction is roughly 2 cm per percent damping, so more for the extreme flood waves (10-30 cm), and vice versa. Compared to HR2006, the water levels will rise for all return periods.

The hydrograph shape, of which the influence is less obvious and clear than the discharge, is very important from Roermond on. If the hydrograph shape would not change, the reduction would be twice as big.

7.1. Introduction

This last research chapter presents the impact in the Netherlands of the results found in the previous chapters. A selection of the results that are important for the Netherlands are repeated here. The flooding turned out to have a reducing effect on the discharges at Eijsden. However, compared to the current design conditions the effect is an increase in discharge. Figure 7.1 shows the HR2006, and the GRADE curves. Compared to the statistical extrapolation that forms the HR2006, the GRADE curves give higher discharges at return periods up to 1000 years, and lower discharges for higher return periods. Flooding has a positive effect on this: even lower discharges for high return periods. The choice to adjust the curves with the uncertainties, which is not done for the HR2006, gives higher discharges over the full range of return periods. For the 100 year, 300 year, 1000 year etcetera events the discharges at Eijsden are given in table 7.1.
7.1. Introduction

Figure 7.1.: HR2006 (solid red), and GRADE curves. The black curves are the current GRADE curve (dashed) and the current GRADE curve adjusted with uncertainties (solid). Similar for the case with flooding in blue.

<table>
<thead>
<tr>
<th>Return period</th>
<th>( Q_p ) [( m^3/s )]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>HR2006</td>
</tr>
<tr>
<td>100</td>
<td>2984</td>
</tr>
<tr>
<td>300</td>
<td>3339</td>
</tr>
<tr>
<td>1000</td>
<td>3728</td>
</tr>
<tr>
<td>3000</td>
<td>4083</td>
</tr>
<tr>
<td>10000</td>
<td>4472</td>
</tr>
<tr>
<td>30000</td>
<td>4827</td>
</tr>
<tr>
<td>100000</td>
<td>5216</td>
</tr>
</tbody>
</table>

Table 7.1.: Table with discharges for HR2006 and GRADE with and without flooding at the design return periods. Note that these discharges do not translate one-on-one to design conditions. For example the wind might also contribute to levee failure.

The different discharges are not the only factor important for the Netherlands. The hydrograph shape also changes due to flooding. Pol (2014) researched that a different hydrograph can have a large influence on the water level, which is the relevant quantity for the Netherlands. With Pols research, the water levels along the Dutch river Meuse will be calculated.

In the coming sections the impact in the Netherlands will be elaborated. Two versions of ‘the impact’ are presented:

1. Firstly the effect of flooding in Belgium itself, so the direct result from this research. This will be the difference between the results from the flood model, and the model without any storage.
2. Secondly the difference between the HR2006 and GRADE with flooding. This represents the effect (on the water levels) when the switch from the current design conditions to GRADE with flooding would be made.
7. Impact in the Netherlands

7.2. Methods

The effect on the Netherlands

To determine the water levels in the Netherlands the research of Pol (2014) is used. He determined relations between the discharge and hydrograph shape at Borgharen, and the water levels along the rivers. The elegance of this method is that not only the discharge, but also the hydrograph shape is taken into account. With the discharge and a characteristic parameter for the hydrograph shape, the water levels along the river can be determined. Different lateral inflows could change the water levels, but this does not matter since the difference between the water levels needs to be acquired, not the specific water level itself.

Pol carried out 25 WAQUA simulations with roughly 5 different discharges and 5 different hydrograph shapes. The hydrograph shape is quantified by the curvature, which is uncorrelated to the discharge. For each location along the WAQUA channel axis a relation as shown in figure 7.2 can be derived. The values between the 25 points are determined using a cubic spline, which gives the smoothest interpolation.

![Figure 7.2.](image)

Figure 7.2.: Example of a (discharge-curvature)-stage relation determined by Pol.

For each point along the channel axis, the water level can be determined with these relations. Each location thus has its own relation, but the general form is equal: Higher discharges give higher water levels (blue line in figure 7.2) and higher curvatures give lower water levels (red line in the figure).

7.3. Results

Figure 7.3 shows the topographical information on the used WAQUA schematization. The model spans the Meuse from Borgharen to Keizersveer. The results between Eijsden and Borgharen are added with the Belgian flood model.

7.3.1. Difference between with and without flooding

First the reduction in water level resulting from the flooding is calculated. By entering all discharges and hydrograph shapes which result from the 100 simulated GRADE floods, in the relations described in the
7.3. Results

Maasplassen, first large retention area. Upstream of this region, the hydrograph shape has no effect on the water level. Downstream it does, since retention works generally better for peaked hydrographs.

Borgharen, upstream end of the WAQUA model. The section from Eijsden to Borgharen is taken from the Belgian flood model.

Methods, the water levels are determined. Subtracting the water levels with and without flooding from each other gives the reduction in water level shown in figure 7.4 along the river branch. For some of the discharges (mostly the biggest) interpolation was not possible and extrapolation was used, which gives less reliable results. These lines are dashed.

The lines show that the reduction can be up to 30 cm for the most extreme GRADE waves, but the reduction strongly varies between reaches. In the first part of the Meuse, the fluctuations are largest. When the flood extent on the map of figure 7.3 is compared to the reduction, it is visible that the narrow sections have more reduction. This makes sense since the less discharge does not have to be spread over a wide river, so the water level effect is large. At river kilometre 150, the lines fluctuate quite a lot. This is probably due to the retention around that point, which works specifically well for flood waves with a certain hydrograph shape. Figure 7.5 illustrates this.
7. Impact in the Netherlands

Figure 7.4.: Water level reduction along the Dutch Meuse due to lower discharges and wider hydrographs. The part between Eijsden and Borgharen is determined with the flood model and from Borgharen on with the discussed stage-(discharge and shape) relations.

Figure 7.5.: Retention works optimally for a wave with a certain shape and magnitude, so the flooding can change this such that the hydrograph gets closer to the optimum (more reduction) or further away from the optimum (less, even negative, reduction). This figure shows the position of the red dots, which is lower than the position of the green dots. For this selection, the damped and wider wave causes higher water levels downstream, instead of lower.

It is interesting to see which part of the reduction is caused by the change in hydrograph shape, and which is caused by the reduced discharge. To do so, for a representative wave the contributions of the two are split up. Figure 7.6 shows the contributions for each locations along the channel axis. The blue line is determined purely with a constant hydrograph curvature, where the red line is determined with a constant discharge and changing curvature.
7.3. Results

Figure 7.6.: Contribution of the discharge and hydrograph shape to the water levels along the Meuse.

Until Roermond Lateraalkanaal (kilometre 68), the effect of the hydrograph shape is negligible. The large retention areas, which can damp the peaked waves, are located downstream of this point. When flooding in Belgium already do this the reduction has already partly taken place, and the water level reduction is smaller.

7.3.2. Difference between HR2006 and GRADE with flooding

The effect of GRADE with flooding on the water level is also compared to the HR2006. Since it is not clear which hydrograph shape to take for both, the same standard hydrograph shape ($C_2 = 12.65 \text{ s}^{-2}$) is used. This means that the effect of a different hydrograph is not looked at, only the discharge. The same return periods as in the introduction (100, 300, 1000, ...) are used. The results are shown in figure 7.7. As expected from the difference with the exceedance frequency curve, all return periods lead to higher water levels. The lower return periods have a larger increase than the higher, this is the direct result of the difference between the exceedance frequency curves.

Figure 7.7.: Water level difference when the situation with flooding is compared to the current method with which extreme discharges are determined.
7. Impact in the Netherlands

7.4. Discussion

The goal of this chapter is to find out what the impact of the flooding along the Belgian Meuse in the Netherlands is. To do this, the discharges at Eijsden and water levels along the river are compared. Firstly, a comparison is made between flooding and no flooding, the direct results of this research. Secondly, a comparison between HR2006 and GRADE with flooding is made, which illustrates the impact when switching from the current method to the derived results in this research.

7.4.1. Determining the water levels

The effect of flooding is determined using the ‘rating curves’ from the research of Pol (2014) with which also the hydrograph shape is taken into account. The addition from the hydrograph leads to a lesser reduction due to flooding, than when only the discharge difference would be taken into account. This makes sense, since the Belgian flooding damps the hydrograph by displacing some water from the top to the back. This leads to a larger volume in the top of the wave in the Netherlands, so a larger retention is needed to damp the wave. However, in some cases the changing hydrograph shape seems to overcompensates the discharge reduction. From a physical point of view this is strange, but it could be the effect of retention. To know this with more certainty, more reference WAQUA simulations are needed. The method is quite neat, since it takes the discharge and the shape into account and makes use of detailed WAQUA simulations. The number of WAQUA simulations is limited, whereas the retention basins can give some abrupt changes in the relations, but for estimating the impact on the Netherlands the method is good enough.

7.4.2. The effect of flooding and switching to GRADE

The effect of flooding in Belgium leads in almost all cases to a reduction in the water level, when compared to GRADE without flooding. The discharges are lower, which is partly compensated by wider hydrographs, but still there is a reduction. When compared to the HR2006 the water levels rises for all return periods. The fact that GRADE gives a lower estimate for extreme events than the HR2006 is fully compensated by the choice to take uncertainties into account, which is not done for HR2006.

Even though the models in GRADE have a lot of uncertainty, the estimate for high return periods is better substantiated than statistical extrapolation used in HR2006. The statistical and model uncertainties are quantitatively estimated and can be incorporated into the exceedance frequency curve. GRADE is for these reasons preferable over the HR2006, as also advised by ENW (2015). Taking flooding into account is a better substantiated representation of reality, and is recommended to be taken into account when estimating extreme discharges for the Meuse.

7.5. Conclusion

Taking flooding along the Belgian Meuse into account, gives a water level reduction up to 30 cm for the most extreme GRADE flood waves. The effect of only the lowered discharges is about twice as big, but the changing hydrograph shapes reduce this. For the part upstream of Roermond, the hydrograph shape is not important. However, compared to the HR2006 the water levels will rise, which is the consequence of the higher exceedance frequency curve.

When calculating the water levels in the Netherlands with upstream flooding, it is recommended to use a wider hydrograph. Otherwise the water level reduction downstream of Roermond is overestimated.
8. Conclusions and recommendations

The goal of this thesis is to determine the effect of flooding along the Belgian Meuse on the water levels in the Netherlands. The conclusions from the subquestions are summarized first, after which the main conclusion is stated.

Potential flood extent

The terrain analysis shows that there are no large areas along the Belgian Meuse that could store such amounts of water that the conveyance capacity of the Meuse has a physical maximum. The effect of the mine subsidence areas around Liège is not large enough to top of the waves. The valley shape of the Meuse limits the potential flood extent to a few kilometres from the river axis. The flood prone areas are located from Namur downstream on to the Dutch border.

Effect on the exceedance frequency curve and hydrograph shape

Compared to the hypothetical case of no flooding, the peak discharge reduction due to flooding increases gradually from nothing at a return period of 50 years to 7% at a return period of 50,000 years. The hydrograph shape also changes due to the flooding. The peak gets topped of, resulting in a wider wave.

The new SOBEK 3.4 software is suitable to simulate flooding in river valleys as the Belgian Meuse.

Recommendation:

- Compare SOBEK 3.4 to full 2D alternatives like Flexible Mesh, for computation time and accuracy.

Uncertainty around the exceedance frequency curve

The uncertainty around the exceedance frequency curve is mainly the result from the hydraulic roughness in the main channel. Th 24-hour GRADE time step is too small to accurately simulate the genesis of flood waves in the Meuse basin. The response of the model to placing sandbags is variable due to the specific peak reduction a region can cause, on average there is only a small rise in peak discharges. The influence of other model variables is negligible.

Averaging the uncertainties in the exceedance frequency curve does not change the discharges or return periods, when only the hydraulic uncertainties are taken. If all GRADE uncertainties are considered, the expected magnitude of the same events (same return period) becomes significantly higher.
8. Conclusions and recommendations

Recommendations:

- Use the exceedance frequency curve with flooding and uncertainties to determine extreme discharges in the Netherlands.
- Use a smaller GRADE time step for the Meuse, for example 1 or 3 hours, to more accurately determine the wave genesis and shape during extreme events.

The impact of flooding on the water levels in the Netherlands

The lower discharges lead to lower water levels along the Dutch Meuse. The reduction due to flooding in Belgium is up to 30 cm for the most extreme GRADE waves, compared to the hypothetical case of no flooding. The hydrograph shape becomes wider due to flooding. This has a significant negative effect on the water level reduction in the Netherlands, due to the reduced effectiveness of retention.

Compared to the current design standards, the HR2006, the water levels become higher when considering GRADE with uncertainties. Considering flooding gives a smaller increase than not considering it.

Recommendation:

- Consider an adjusted hydrograph shapes when calculating the water levels in the Netherlands with flooding along the Belgian Meuse.

Main conclusion

From these findings it can be concluded that taking flooding into account decreases the discharges at Eijsden and the water levels along the downstream river. This effect starts to count from once in 500 years events floods and larger. Events of these magnitudes cause significant flooding along the Belgian Meuse from Namur to Eijsden. Considering GRADE with flooding and with uncertainties is recommended, since it is a physically and statistically more sound estimate for extreme discharges than the current GRADE line or the HR2006.
Bibliography


A. Model schematization and results

This appendix elaborates the set-up of the model, the calibration, the result and a found error. The first section describes the schematization of subsequently the 1D main branch, 2D floodplains and weir-schematization. The second section presents the calibration of the main channel hydraulic roughness. The third section presents additional flood maps from the simulation with the largest GRADE wave. The last section elaborates on an error found in the schematization or software.

A.1. Schematization

A.1.1. 1D main branch

Existing model

The hydraulic model which is currently used for the hydraulic GRADE computations, is a SOBEK-RE model. This model is converted to SOBEK 3, and can be used as base model for schematization of the one dimensional main branch. The existing model also contains an RTC-module (Real Time Control) with which the river weirs are controlled.

Main branch

For a 1D model the course of the river is not important, as long as the length and slopes are correct. However when coupling a 2D model to it, the horizontal geometry should be correct. To create the main branch, the existing model is folded along the actual location of the river (extracted from openstreetmap.org). The length of the total river in the model is almost equal to the true course, but not completely, so the branch is scaled between known locations such as weirs and lateral inflow points. Only the main river branch is schematized in the model. Other branches are modelled as lateral inflow directly on the main branch.

The computational grid points in the existing model are located one at each cross section. The advantage of this approach is that there is no use of interpolated cross-sections in the computation, so every detail in the cross sections is used. The cross-sections however are spaced with such irregularity that the grid is not smooth. This causes a lower accuracy of the solution (larger numerical errors), and a decreases computational time step, resulting in longer computation times. The grid spacing in the new model has been made with the idea that:

1. The links between 1D and 2D are one-on-one, so the grid spacing in the main branch is at the position of the 2D grid equal to its grid spacing. This is 100 m in our case.
2. The grid should be orthogonal. The ratio between consecutive cells should preferably be smaller than 1.2.
3. Each cross-section should have a grid point, so all the cross sectional detail is taken into account.
A. Model schematization and results

![Figure A.1](image-url):
Top: cell lengths along the river axis. Bottom: cell length ratios. Liège up and Liège down refer to up and downstream of the bifurcation.

The one-dimensional grid has been generated by SOBEK with the given preferred length of 100 metre. The locations where cross sections where close together are the point where the river changes abrupt, for example at weirs. The distance between cross sections had been enlarged at these few locations from 0-50 m tot 50-100 m, which is not much, and not unreal (a structure with a weir and navigational lock is often longer than 100 m). The resulting cell lengths and length ratios are shown in figure A.1.

It is not necessary to aim at 100 m cells at the locations where there are no 2D grid areas connected to the branch, but since the computation time mainly goes to the 2D calculations, and for the sake of simplicity, the aimed length is 100 m everywhere.

**Cross sections**

The cross sections in the existing model are defined as a depth-width table. For a number of depths the width is given, from which a cross-section is determined. Since the model is rescaled, the location of the cross-sections are shifted slightly. All cross-sections are therefore checked with aerial photographs, openstreetmap.org and DEMs, to validate and if needed change their shape. The depth of the river could not be checked: inconsistencies will be corrected with the bottom roughness during the calibration.

**Boundary conditions**

The inflow boundary conditions differ per wave scenario. The time step is 1 day, in accordance with GRADE. The magnitude and form of the upstream inflow condition, which is a hydrograph, differs per flood wave. The same counts for the lateral discharges. These different inflow conditions, which come from the rainfall generator and the hydrological model, can statistically be seen as an empirical distribution which forms the exceedance frequency curve, when calculated with the model.

There are two outflow boundary conditions, Borgharen and the Albert Canal. The downstream boundary condition is modelled as a stage-discharge relation. In the real-life situation there is a weir which regulates the water level during low flow. We derive the rating curve by curve-fitting a formula of the form $Q = ah^b$, in which $a$ is a coefficient representing the roughness, width and slope of the channel, and $b$ represents the shape of the cross-section. The fit and the measured series are shown in figure A.2. Note that the black line
A.1. Schematization

does, not fit the red points. This is done to avoid oscillations. The other downstream boundary conditions, the Albert Canal, shows a less logical relation when being fitted. On average the discharge decreases when the discharge on the main branch increases, but there is a lot of variation as shown in figure A.2. We choose a conservative 20 m$^3$/s for the outflow at this point.

![Figure A.2: Downstream boundary conditions: On the left the stage-discharge relation at Borgharen. The black line is not horizontal for low flow, cause this creates oscillations. On the right the observed, fitted and chosen discharge in the Albert Canal.](image)

Validation

Initially the roughness values from the existing model were adapted. The slope and profile changes are so minor that the calibration still gives the same outcome: The discharge peaks fit well. For low discharges, the weir operation disturbs the calibration.

In the uncertainty analysis, the calibration is repeated to deduce a probability distribution for the roughness. The model has later been adjusted to these values. The calibration for the uncertainty analysis is described in section A.3.

A.1.2. 2D floodplains

The numerical calculation of the flow in the floodplains is done with Flexible Mesh. The grid consist of triangles with a maximum edge length of 100 metre. The extent of the grid is chosen at the limit of the potential flood area from the 5000 m$^3$/s wave in the terrain analysis, plus a buffer of roughly 100 metres. The aim is that this buffer does not flood, so the grid will span the full flood-prone area and no areas are overlooked.

Embankments

To model the interaction between 1D and 2D correctly, the height of the embankments is required. SOBEK 3.4 calculates the discharge over the embankments with a flow over a weir formula. Depending on the water level on both sides of the embankments a free weir flow or drowned weir flow formula is used. For the exact implementation one is referred to the Technical Reference Manual of SOBEK 3 (Deltares, 2015).

The height of the embankment which SOBEK uses for its overflow is taken at the intersection of the embankment and the flow-link. This height is assumed over the complete width of the computational cell, so the water flows over the complete width. The consequence of this method is that variations in the embankment height over the computational cell are ignored. This creates a dilemma for the schematization: Taking the lowest value will simulate the correct moment of start of the inflow, but gives a too high discharge. Taking an
A. Model schematization and results

average value will approximate the discharge better, but the start of inflow will be delayed. A compromise between these is made by taking the 10th percentile value of the embankment height values calculated in the terrain analysis (chapter 3) along the computational cell.

Since the 10 percentile value is below average and the topography at the embankment coordinates is calculated towards the floodplains, it can occur that the embankment height is lower than the terrain. In that situation the embankment height is set equal to the height of the terrain. That is, the average of the 2 nodes which confine the piece of embankment. The resulting weir level is equal to the value of the terrain level in the flow over weir formula.

Figure A.3 illustrates the process of determining the embankment heights. The choice of the 10th percentile for the embankment height is also discussed in the sensitivity analysis, section 5.2.4.

Roughness

To determine the roughness in the floodplains, we use Walloon land use maps. With a so called ‘spatial query’, we can request land use data from the Service Public de Wallonie (SPW). The data consist of land use types like forest, residential and agricultural areas and is subdivided into several levels of detail. A good translation between the land use types and hydraulic roughness was not found, so we estimated the translation from the slightly different Dutch LGN6 division. Each land use type is linked to its Dutch equivalent and an LGN6 value is coupled to it. The division is given in table A.1. Note that the values are quite high (compare them to metres). This is because the obstacles are not present in the topographic map.
### Table A.1: Walloon and corresponding Dutch land use types with roughness.

<table>
<thead>
<tr>
<th>Walloon name</th>
<th>Corresponding Dutch type</th>
<th>White Colebrook, $K_n$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Urban areas</td>
<td>Urban areas</td>
<td>10</td>
</tr>
<tr>
<td>Roads and railways</td>
<td>Roads and railways</td>
<td>1</td>
</tr>
<tr>
<td>Agricultural areas</td>
<td>Cultivated land</td>
<td>0.4</td>
</tr>
<tr>
<td>Abandoned agricultural land</td>
<td>-</td>
<td>1.1</td>
</tr>
<tr>
<td>Forest</td>
<td>Forest</td>
<td>10</td>
</tr>
<tr>
<td>Shrub and herbaceous vegetation</td>
<td>(Densely vegetated) Heath</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Since Flexible Mesh, the 2D numerical model, solves its equations on a staggered grid, the roughness is defined in the flow links, which are located roughly in the middle of each cell edge. Polygons are drawn around the flow links using a Voronoi algorithm, and the average roughness value is taken within each polygon. Whether the average is the right value to take can be debated. To get an answer a comparison with a much finer grid should be made, but this is outside the scope of this research. Figure A.4 illustrates the way in which the roughness is determined.

**Topography**

The topography has been determined from the DEM. Similarly to the roughness, the average value within a polygon is calculated. For the topography the polygons are drawn around the cell nodes on which the bed level is defined.

Taking the average of a triangular cell with 100 metre legs can smooth out topographical features. For example line elements can be averaged, leading to extra flood areas. At one location next to an open mine the topography is adjusted such that the mine does not flood. This is confirm the flood extent found in the terrain analysis.
A.1.3. River control structures

The structures

The water control structures in the Belgian Meuse consist of a weir and a navigational lock, as can be seen in figure A.5. The weirs in the model are modelled as a composite structure with a movable river weir and a structure to simulate the scenarios where water flows around the weir (flooding).

The schematized weirs in the original model were gated structures. These structures cause oscillations during the transition from free to submerged flow. The ‘river weir’ does not, so we chose to change all gated structures to river weirs. The results might differ a few cubic metres per second. This can be the consequence of the no longer present oscillations, or the different numerical implementation.

Validation of the weirs

For two weirs, Anseremme and Visé, it is possible to accurately validate the movement of the weirs. These weirs have a water level and discharge gauging station just upstream of the weir, and a water level or stage-discharge relation just downstream. The weir is modelled with a small river section, with the discharge measurement as input. For each measurement in this discharge series, the crest level of the weir is adjusted such that the water level in the observation point matches with the water level in the measured series. Figure A.7 shows the model schematization for these tests.

The results of the weir movement check are shown in figure A.8. Each figure illustrates two things:

1. Blue line: The control scheme of the weir, as it is implemented in SOBEK. The model works such that for each time step the crest level is adjusted to the level which matches with the measured discharge in an upstream observation point.
2. Grey dots: The level of the weir necessary to make the measured water level match with the simulated water level, at a given discharge.

Note that during the simulation, most weirs are adjusted with a time lag of 1 to 4 hours. The measured data however are on a daily base, so the time lag is neglected. The measurements are assumed to be taken at the same moment.
A.1. Schematization

Figure A.7: Location and schematization of the weirs which are validated. Left the Anseremme weir, right the Lixhe weir, and in the middle their locations along the Meuse. The $h$ are water level stations, the $Q_h$ is a stage-discharge relation and the $Q_{in}$ is the inflow boundary.

Figure A.8: Crest level for the Lixhe (left) and Anseremme (right) weir deduced from measured data (point cloud) and the implemented weir control (blue line)
Simulation of the movement

RTC, meaning real time control, is a Deltashell module to manage water control structures. The existing model of the Belgian Meuse contains this module, with which the movement of the weirs is simulated. The problem with the RTC-module is that it does not work yet in combination with a 1D-2D model. For now the RTC has therefore been programmed in the model using a Python script which is a possibility in SOBEK 3.4. The Python script carries out the following actions between each large computation step (30 minutes).

1. Get the discharges at the observation points from the model results, taking into account the time lags.
2. Look up the new crest levels of the weir from the control regimes.
3. Adjust the new crest levels in the model schematization.

The results have been compared with a similar 1D+RTC model (which does work). Overall the results are very good, as is shown in figure A.9.

![Figure A.9: Comparison between the built in RTC module, and the scripted weir movement. The blue line shows the difference in the average water level, which is negligible. The red line shows the discharge at Eijsden during the simulation.](image-url)
A.2. Verification

Since there are no data available to calibrate the 2D model, only a verification is carried out. The result from the University of Liège are used as verification data. Sufficient input and output are known to compare the models. Since the model from ULg has a 5 m grid, its result are assumed to be precise. The validity can however not be verified.

A.2.1. Input and schematization

The university of Liège has simulated four flood waves with peak discharges of $Q = 3100\text{ m}^3/\text{s}$, $Q = 3630\text{ m}^3/\text{s}$, $Q = 4100\text{ m}^3/\text{s}$ and $Q = 5000\text{ m}^3/\text{s}$. The first, third and fourth are verified in this chapter. The first wave gives almost no flooding, so can be used as extra validation for the 1D model. The second wave is left, since it gives only a bit of flooding. The third wave gives more flooding, and is modelled on the precise $5\text{ m} \times 5\text{ m}$ grid. The fourth waves gives a lot of flooding, but is modelled on a $20\text{ m} \times 20\text{ m}$ grid. Figure A.10 shows the input waves for $Q = 3100\text{ m}^3/\text{s}$, $Q = 4100\text{ m}^3/\text{s}$, $Q = 5000\text{ m}^3/\text{s}$. The average wave at Borgharen used in the uncertainty analysis is also shown, to show the relative width of the waves. The waves simulated by the ULg have a more sharper peak and wider tails than the average GRADE wave between 3800 and 4000 m$^3$/s.

![Figure A.10](image_url)

Figure A.10: Hypothetical example explaining how the input embankment for SOBEK is determined: The maximum of the 10 percentile value from the terrain analysis height and the grid level is taken.

The model of the ULg simulates the Meuse from Ampsin to Eijsden, including a part of the Ourthe. This is different from the created model, which goes from Chooz to Eijsden, but simulates the Ourthe not as a branch but as a lateral inflow.

A.2.2. Results

The results are presentes for each wave individually. The results contain water levels along the reach, water levels and discharges at Eijsden and comparison of the inundated area and volume.

Wave with $Q_p = 3150\text{ m}^3/\text{s}$

The first simulated case is a wave with a peak discharge of $Q = 3100\text{ m}^3/\text{s}$. Since this wave gives almost no flooding, it is used to validate the water levels. The comparison between the waterlevels is shown in figure

77
A. Model schematization and results

A.11.

The first part of the reach matches nicely, but especially along the bifurcation at Liège, there are large differences, up to 70 cm. There are several schematization factors which could cause this difference. Another possibility is that the 2D-modeling of the inflow of the Ourthe gives higher water levels, than the modeling of a one-dimensional lateral inflow.

The discharges at Eijsden match well for the two models. The 1D-2D model has slightly higher discharges and a broader peak. The tail from the ULg-model is lower, which cannot be explained from the input.

**Wave with $Q_p = 4100 \text{ m}^3/\text{s}$**

The second simulated case is a wave with a peak discharge of $Q = 4100 \text{ m}^3/\text{s}$. The peaks of the hydrographs at Eijsden, shown in figure A.13, do coincide well. Again the tails deviate, which has to do with the boundary conditions. The flooded area deviates significantly in the center of Liège. Different areas are flooded, which is the result from the different water levels, as shown in figure A.11 (The pattern is equal for $Q = 4100 \text{ m}^3/\text{s}$ and $Q = 5000 \text{ m}^3/\text{s}$). Table A.2 shows that the flooded volume and area does not differ
A.2. Verification

<table>
<thead>
<tr>
<th></th>
<th>SOBEK 3.4</th>
<th>AMICE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area [km²]</td>
<td>13.49</td>
<td>15.0</td>
</tr>
<tr>
<td>Volume [million m³]</td>
<td>17.55</td>
<td>15.3</td>
</tr>
</tbody>
</table>

Table A.2.: Comparison of flooded volume and area between SOBEK 3.4 and AMICE for \( Q = 4100 \text{ m}^3/\text{s} \)

that much. So even though the specific location of the areas deviate, the sum is approximately equal. For the discharge at Eijsden, which is the most important parameter for this research, the result is very good.

![Figure A.13. Comparison of the hydrographs at Eijsden between SOBEK 3.4 and AMICE for the \( Q = 4100 \text{ m}^3/\text{s} \) wave.](Image)

Wave with \( Q_p = 5000 \text{ m}^3/\text{s} \)

For the largest wave, with a peak discharge of 5000 m³/s, the results are different. Now the flood pattern very much alike: everything is flooded. Again the ULg model shows more flooding around the Ourthe, and the

![Figure A.14: Comparison of the flooded area during the \( Q = 4100 \text{ m}^3/\text{s} \) flood event, between the model of the university of Liège and the SOBEK 3.4 flood model.](Image)
A. Model schematization and results

<table>
<thead>
<tr>
<th></th>
<th>SOBEK 3.4</th>
<th>AMICE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area [km²]</td>
<td>26.43</td>
<td>31.92</td>
</tr>
<tr>
<td>Volume [million m³]</td>
<td>45.10</td>
<td>40.17</td>
</tr>
</tbody>
</table>

Table A.3.: Comparison of flooded volume and area between SOBEK 3.4 and AMICE for $Q = 5000 \text{ m}^3/\text{s}$

SOBEK model more just downstream of the island. The flooded volume in SOBEK is quite a bit larger, while the flooded area is smaller. The cause of this is that in the SOBEK model the choice is made to use the terrain model, where in the ULg model (as far as known) the surface model is used, because of the fine grid. This makes that the potential flood volume in the SOBEK model larger is. The peaks of the hydrographs match well again, however the ULg model had a more peaked hydrograph. This could be the result of different starting moment of the flooding, due to the difference in schematization.

![Comparison of hydrographs at Eijsden between SOBEK 3.4 and AMICE for the $Q = 5000 \text{ m}^3/\text{s}$ wave.](image)

Figure A.15.: Comparison of the hydrographs at Eijsden between SOBEK 3.4 and AMICE for the $Q = 5000 \text{ m}^3/\text{s}$ wave.

![Comparison of the flooded area during the $Q = 5000 \text{ m}^3/\text{s}$ flood event, between the model of the university of Liège and the SOBEK 3.4 flood model.](image)

Figure A.16.: Comparison of the flooded area during the $Q = 5000 \text{ m}^3/\text{s}$ flood event, between the model of the university of Liège and the SOBEK 3.4 flood model.
A.3. Calibration of the main channel roughness

Initially the calibrated roughness values from the existing model is adapted. There are only minor changes in the 1D branch schematization, and when the calibration simulation was carried out again, the results were similar. After the sensitivity analysis indicated the main channel roughness as most sensitive parameter, the calibration is used as method to determine the roughness distribution. To do so, all peaks between 1999 and 2012 above 400 m³/s at Chooz are calibrated. This gives in total 34 successfully calibrated flood waves. The calibration is done with OpenDA, an open source calibration and optimization tool.

A.3.1. Topographical overview of calibration data

To calibrate the model, discharge and water level data from the SPW is used. Figure A.17 shows an overview of the basin with the locations of the used discharge and water levels stations indicated. Almost all side branches have a discharge station, except for the very small. Luckily there are also discharge measurements on the branch itself available, which can be used to correct discharges or supplement missing data.

The water level locations are chosen such, that they are not located just upstream of a weir. If a gauging station is too close to a weir, the weirs backwater will determine the water level and not the bottom roughness. The weirs do however still effect the water level at low discharges, so they have been calibrated first. This calibration is similar to the calibration described in section . In total there are 6 water level stations used for the calibration, 2 per river section for which a roughness value is calibrated.

Figure A.17: Map showing the location of discharge and water level stations used for the calibration. The Q’s on the main branch indicate from upstream to downstream the discharge stations at Chooz, Waulsort, Amay and Visé.
A. Model schematization and results

A.3.2. Data availability

For the period between 1999 and 2012 there is sufficient data available for the calibration. Before 1999 the water level data is not sufficient, and after 2011 the data availability stops, perhaps it is no longer uploaded on the SPW website.

Each peak at Chooz above 400 m$^3$/s is used as calibration data. Figure A.18 shows the discharge series for Chooz, and the data availability of the other sources in this period.

![Figure A.18: Above: discharge series at Chooz. Below: data availability during this period for all stations.](image)

Note that Waulsort and Sambre do not span the full 12 years. For Waulsort this is not a problem: The station measures the discharge on the main branch, and is only used to determine a minor inflow between Chooz and Waulsort. For the Sambre this is problematic, since it is an important side branch with a water level station just downstream of the confluence. To solve this problem, the contribution of the Sambre to the discharge on the main branch is calculated in the period where data for the Sambre is available. This gives an average of 68% during flood peaks. Figure A.19 shows this fraction, for the peak discharges. The other 32% is assigned to smaller lateral inflows.
A.3. Calibration of the main channel roughness

Figure A.19.: Contribution of the Sambre to the discharge. The grey line (total difference $\times 0.68$), matches quite well with the red line. Note that on the horizontal scale the total peak time is used. All low flow periods are cut out, and the peaks are glued together.

A.3.3. Calibration with OpenDA

The calibration itself is done with OpenDA. This tool can calibrate models automatically if it is given observation points and data to be fitted. Several options can be chosen to calibrate the model. More information on OpenDA can be found on its website: http://www.openda.org/, and more information on how to implement it for SOBEK 3.4 in Deltares (2015). A result of the calibration of a peak is shown in figure A.20. Most calibrations have a similar quality, the differences between measured and simulated are in the order of decimetres.

Figure A.20.: Example of a roughness calibration result. The incoming discharge at Chooz is 700 m$^3$/s for this wave. The calibration shows small differences, order of decimetres, between the observed and calibrated values.
A. Model schematization and results

A.4. Flood maps

Figure A.21: Flood map of Namur for the 50,000 year wave. The left figure shows the flooding, the right figure the stored volume for each wave.

Figure A.22: Flood map of Andenne for the 50,000 year wave. The left figure shows the flooding, the right figure the stored volume for each wave.
A.4. Flood maps

Figure A.23: Flood map of Anay for the 50,000 year wave. The left figure shows the flooding, the right figure the stored volume for each wave.

Figure A.24: Flood map of Seraing for the 50,000 year wave. The left figure shows the flooding, the right figure the stored volume for each wave.
A. Model schematization and results

Figure A.25: Flood map of Visé for the 50,000 year wave. The left figure shows the flooding, the right figure the stored volume for each wave.

A.5. Model error

This appendix describes the error found in SOBEK 3.4, and the influence on the results. We will illustrate the error with two simulations. A simulation in which we use the 10th percentile embankment height, and a simulation in which all artificial embankments are removed. Artificial embankments are the locations that have an embankment height larger than the average of the two grid nodes that enclose the embankment. Removing thus means that the embankment is put on the level of the terrain.

Figure A.26: [Above] Hydrograph at Eijsden for both waves. [Below] Difference in discharge. Note that the one without embankments is almost always below the one with embankments.

The hydrograph without embankments is at every time below the hydrograph with embankments, while the second could store more water, and should be the lower one. Apparently embankments cause extra water in the model.
The next thing we do is compare the amount of water passing Borgharen to the case of no flooding. The next thing we will do is plot the discharge at Eijsden, and the amount of water stored in the floodplains. The sum of the deficit at Borgharen and the stored volume should be 0.

We see that in the case of embankments the sum of the deficit at Borgharen and the stored volume are equal, while this is not the case for no embankments. So in some way water is disappearing.

The water passing Borgharen in case of embankments is 2.384 Gm$^3$. In the case of no embankments, it is 2.372 Gm$^3$, a difference of roughly 0.5%.

Does this compromise our model? In general it does not, since the model functions right in case of embankments. We should however look out that the water balance is closed, when we are varying the embankment height.

Figure A.27.: [Above] Embankments, [Below] No embankments. The green line shows the cumulative difference of the discharge at Borgharen, with a no flooding situation. The orange line shows the stored volume. The sum of both lines should eventually be zero, meaning no water is added or lost.
B. Uncertainty analysis

B.1. Hydraulic analysis of the basin

we make a small detour in which we analyse the hydraulic characteristics of the basin. We want to know how important the timing of the laterals is for the genesis of an extreme flood wave, and what the travel times of the waves through the basin are. We will also look at some extreme hydrographs, to see how they generally look.

Timing

To see how important the timing of the laterals is for the occurrence of an extreme flood, we look at the relation between the sum of the lateral peaks, and the maximum discharge at Borgharen. The results are shown in figure B.1. We see that generally the peak discharge at Borgharen will be as high as 90% of the laterals. Fitting a polynomial shows that this percentage slightly increases for higher discharges, but the pattern is also clearly visible at low discharges. Timing looks to be important to get high discharges, but also inevitable.

Two other things which are interesting are the timlags between the lateral peaks and the Borgharen peak, and the shape of the extreme waves. To do so, we disaggregate the waves as described in section 5.2.6, this enables us to analyze onto a finer scale than daily. We compare the time lag between the peaks at the lateral and Borgharen.
The limits of the time lag are small, and do not differ much between the extremes (3000 m³/s plus, red), and the less extreme waves (1500 m³/s plus, green). The timespan in which the peaks enter is small, some basins respond a bit quicker, some a bit later. Remarkable is that the sub-basins are often earlier than the wave from France. Thinking about it this makes sense, since the extreme peaks are probably caused by the same large rainfall event, but the traveled distance for the water also increases, and the river is less steep.

**Hydrograph shape**

Based on these findings, we can conclude that a measure to reduce the peaks, could for example be to speed up some of the basins and slow down some others, the peaks should however be not too wide for this to succeed.
B. Uncertainty analysis

Figure B.3: 10 Waves with a peak discharge around 4000 m$^3$/s disaggregated. Here it is also visible that the smaller basins are often earlier than the wave from Chooz.

Conclusion

This quick analysis shows that the Meuse basin reacts very quick to a large rainfall event. The timing of the basins, depends on the basin characteristics, but does not show much variety. There is no clear trend difference between extreme and less extreme events, almost all give a peak discharge at Borgharen which is almost equal as the sum of the lateral peaks.

The GRADE timing of 24 hours seems to course to describe the genesis of the wave detailedly. It is recommended to analyze the hydraulics when using a finer time scale, and whether the behavior is significantly different.