Flood Safety Clarence Valley

Flood Safety in the Clarence Valley

Feasibility study into flood mitigation measures to make `Room for the River'

March 2018

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TU Delft

MONASH University

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Feasibility study into flood mitigation measures to make ‘Room for the River’

by

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An electronic version of this project report is available at
http://repository.tudelft.nl/

The cover image is retrieved from the Clarence Valley Council database.

Delft University of Technology

Preface

In April 2017, the decision was made to participate in the course CIE4061-09, Multidisciplinary Project. The main goal of this project is to combine different master disciplines while researching a real-life problem. A challenging project was found in Grafton (Australia), which offered us an interesting combination between Structural and Hydraulic disciplines of Civil Engineering. Six students of the Delft University of Technology, with backgrounds in the above mentioned expertises, were working on this project in order to seek possible solutions.

During this project we have learnt to work in a country and environment unknown to us and working with software we did not use before. Besides, we have also learnt to combine our different Civil Engineering backgrounds in one final report.

A special thanks is directed to the Clarence Valley Council for the opportunity to work on a project in the Clarence Valley. Moreover, their hospitality and their effort to arrange our stay is highly appreciated. We want to thank BMT WBM for receiving us at their office in Brisbane for a 2-day TUFLOW tutorial and the Civil Engineering Department of the Monash University in Melbourne for providing a work space for six weeks. Furthermore, we also would like to thank our supervisors Prof. dr. ir. S.N. Jonkman, Ir. E.C. van Berchum, Ir. L.J.M. Houben, A/Prof. dr. ir. V.R.N. Pauwels and K. McAndrew for launching the project, providing us with feedback and their availability for questions. Also, we highly appreciate the help of the companies and organisations who supported this project: Arcadis, TWD, DIMI, Iv-Groep and BMT WBM.

We would like to explicitly mention our hosts during the Clarence Valley field visit. The hospitality of Erica & Mark, Shane and BeeJay on providing a location to sleep and showing us around Grafton is highly appreciated. Finally, we want to thank Kieran, Frank and Matt of the Clarence Valley Council for their efforts on constructing a field visit itinerary and for providing us with detailed insights in the Clarence Valley.

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Melbourne, March 2018

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During this project, we had a lot of support from several organisations and authorities. We want to express our appreciation to the people who and institutions which contributed to this project.

Clarence Valley Council	
Kieran McAndrew	Floodplain Engineer & Emergency Officer
Frank Rasborsek	Floodplain Engineer
Matt Foley	Water Efficiency Consultant

BMT WBM	
Chris Huxley	Senior Engineer & Software Business Development Lead
Barry Rodgers	Senior Flood Risk Consultant

Monash University	
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Arcadis	
Phillipe Vienot	Associate Technical Director
Ian Rath	Principle Drainage Engineer

Others	
Geoff Duckworth	Farmer & Member of the Swan Creek Committee
Paul O'Halloran	Member of Flood Risk Management Committee
Vince Castle	Local Sugarcane Farmer
Des Harvey	Member of Flood Risk Management Committee
Robin Knight	Port of Yamba Historical Society Member

Partners

Multiple organisations supported the Flood Safety in the Clarence Valley project. We would like to thank the following institutions, organisations and companies for helping us organising and executing this project (in alphabetical order).

- Arcadis
- BMT WBM
- Clarence Valley Council
- Deltas, Infrastructures & Mobility Initiative (DIMI)
- Delft University of Technology
- IV-Groep
- Monash University
- TWD

Summary

The Clarence River catchment is located in the state of New South Wales (NSW), on the east coast of Australia. The lower Clarence Valley is an area covering approximately 1,000 km^2 and is located on the downstream part of the Clarence River. In the remaining of this summary, the lower Clarence Valley is referred to as the Clarence Valley. Due to heavy rainfall, the Clarence River discharge can increase from an average of 160 m^3/s to a staggering 20,000 m^3/s. As a result, water levels in the Clarence River rise significantly, leading to severe floods in the Clarence Valley. The main urban areas in this region, Grafton, South Grafton and Maclean, are located in narrowing river bends which makes them particularly vulnerable to flooding during high water levels. Both towns are surrounded by extensive levee systems, protecting them from high water levels.

Despite the extensive levee systems, the cities of Grafton, South Grafton and Maclean have a long history of flooding, the largest of which was recorded in 2013. Until now, all flood mitigation studies mainly focused on strategies like heightening the levees. However, the local government (Clarence Valley Council) has shown interest into expanding these strategies to the following principle: reducing the water levels during high river discharges by increasing the storage capacity of floodplains. As this principle is very much alike the Dutch flood mitigation strategy called 'Room for the River', the Council and six students from the Delft University of Technology agreed upon a collaboration to assess the feasibility of applying this approach to the Clarence Valley.

The main goal of this report is to present flood mitigation measures to reduce the impact of flooding in the urban areas of the Clarence Valley, based on the newly formulated strategy. Consequently, the following research question was formulated:

> **How can the impact of flooding on the urban areas in the Clarence Valley be reduced by increasing the storage capacity of the floodplains?**

In order to answer the research question, the following project approach is applied. Six areas were identified, based on a field visit and an extensive preliminary study, to implement flood mitigation measures and assess existing flood defences. Part of these flood defences are the Swan Creek Floodgate and the reinforced concrete levee wall of Maclean, which will be investigated on their performance. A fully calibrated numerical flood model provides input for the hydrological analysis. The model represents the current situation in the Valley, referred to as the reference situation.

Eighteen scenarios are formed by applying topographic adjustments to the appointed areas, mainly focusing on the addition, removal or lowering of levees. The scenarios are based upon the 'Room for the River' principle and expert judgement. The new scenarios are implemented into the numerical model and the outcomes are compared to the reference situation for the 5, 20 and 50 year Average Recurrence Interval (ARI) flood events in order to assess their effectiveness on flood mitigation in urban areas.

The reference situation shows that no urban areas inundate during a 5 year ARI flood event. Furthermore, a 20 year ARI flood event inundates Grafton partly and a 50 year ARI flood event inundates nearly the whole of Grafton, parts of South Grafton and a part of Maclean. Topographic changes in Baker's Swamp, Southampton Floodplain and Clarenza Floodplain show reduction of flood impact on urban areas. Topographic changes made in these scenarios can be seen in Figures 1 and 2 (orange, red or green lines).

Results indicate that inundation of Grafton, during a 20 year ARI flood event, can be prevented by increasing the storage capacity of the Southampton Floodplain only (see Figure 1). This figure shows the difference in maximum peak flow levels between the reference situation and the simulated scenario. In this scenario, the Waterview Levee is lowered (orange line) and water flows in the Southampton Floodplain (dark blue). The red colours in Grafton indicate that the area was wet and is now dry, as a result of the applied measures.

During a 50 year ARI flood event, inundation of Grafton, South Grafton and Maclean can only be reduced in magnitude by increasing the storage capacity of floodplains. The reduction of the water levels, during a 50 year ARI flood event, in Grafton and South Grafton is a result of an increase in storage capacity of Baker's Swamp, the Southampton Floodplain or the Clarenza Floodplain or a combination of the three areas (Figure 2). This figure shows the combination of adjustments of the levees in the three areas (orange, green and red lines), which results in a reduction of inundation depths in Grafton (red areas). This combination also shows downstream effects, as Maclean inundates to a lesser extend.

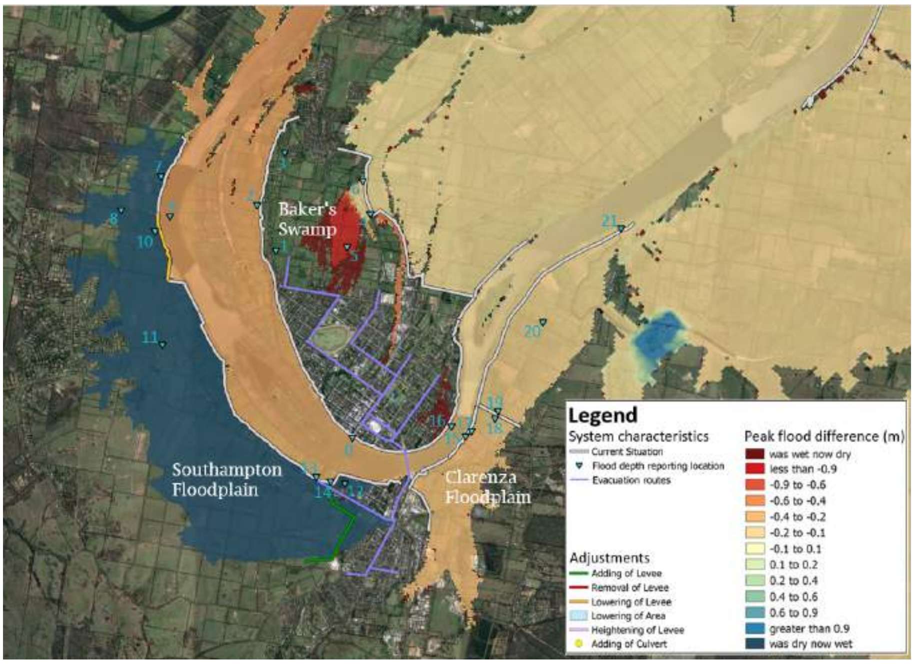


Figure 1: A scenario where the Waterview Levee is lowered (orange line) and a levee at South Grafton is constructed (green line). Southampton Floodplain inundates (dark blue area), Grafton (red area) stays dry.

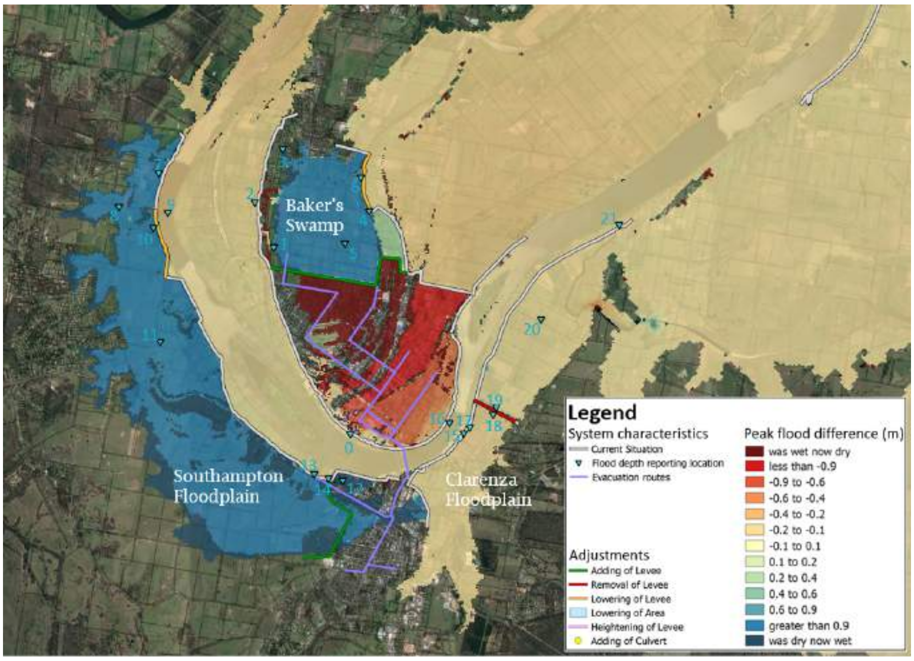


Figure 2: A combination of measures in Baker's Swamp, Southampton Floodplain and Clarenza Floodplain. This leads to inundation of Southampton Floodplain (dark blue area) and to decrease in inundation depths in Grafton (red areas).

The Swan Creek Floodgate in the Clarenza Floodplain suffers from erosion problems, possibly due to piping. This leads to tilting of structural parts. The reinforced concrete levee walls in Maclean also show possible piping problems.

To conclude, inundation of urban areas can be prevented by only increasing the conveyance capacity of floodplains in case of a 20 year ARI flood event. However, for a 50 year ARI flood event, neither Grafton nor Maclean can be protected from flooding by only using the 'Room for the River' principle.

Besides, the flood defences in the Clarenza Floodplain (Swan Creek Floodgate) and city of Maclean (reinforced concrete levee walls) require more attention. For the Swan Creek Floodgate, this requires research into finding the origin of the displacements of the outlet walls. In order to retain the function of the floodgate, one could subsequently apply erosion protection, anchors or sheet piles. For the Maclean levee walls, piping problems are identified, which could lead to flooding of Maclean during high water.

By making use of the proposed floodplains and improving the performance of existing flood defences, the flood defence system of the Clarence Valley can be extended. It can be concluded that it is possible to reduce the impact of flooding in the urban areas of the Clarence Valley by increasing the storage capacity of floodplains around Grafton. Therefore, the usage of a 'Room for the River' strategy can be a solution to the problems the Clarence Valley is facing, and possibly might be applicable to more flooding-vulnerable areas in Australia.

It is suggested to investigate the heightening of the levees around Grafton in combination with the increase of storage capacity of the appointed floodplains for a 50 year ARI flood event. Also, further detailed research is required to present:

- the effect of 'Room for the River' measures on infrastructure, mainly evacuation routes;
- the effect of 'Room for the River' measures on all of the influenced stakeholders;
- the effect of 'Room for the River' measures on the (possibly even bigger) flood events;
- the performance of the investigated flood defences.

Disclaimer

The presented report is the result of a multidisciplinary project, written by six students in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering at Delft University of Technology. This report outlines an overview of the problems the Clarence Valley is facing. Multiple suggestions have been proposed to possibly mitigate these problems. The reader has to keep in mind that the report has been written by an independent group of students, none of which have lived in the project area. This implies that no 'best solution' is given, as this is subject to many factors that require much more local knowledge. The project was supervised by staff from Delft University of Technology, Monash University and the Clarence Valley Council, and supported by IV-Groep, Delft Infrastructure & Mobility Initiative (DIMI), TWD, BMT WBM and Arcadis. However, the conclusions and statements do not necessarily represent the view of these organisations and companies. Moreover, none of the outcomes are checked on their correctness by any involved second party.

Note

This multidisciplinary project is concluded by presenting the following deliverables:

	Deliverable	Format
1	Technical report including appendices	PDF file
2	Presentation	MS PowerPoint
3	Flood animations	AVI file
4	SCIA report of the Swan Creek Floodgate	PDF-file
5	Scripts to process raw TUFLOW simulations output data	Python scripts
6	Output data from TUFLOW simulations	MS Excel
7	Hand calculations Floodgate	MS Excel
8	Hand calculations Piping - RC Levee Maclean & Swan Creek Floodgate	MS Excel

The outcomes of the project are transferred using presentations and animations. Calculations included in this report have been performed using SCIA Engineer, TUFLOW, Python and Microsoft Excel. From the above-mentioned deliverables, items two to seven are not attached to this report but can be send upon request. One can retrieve them by contacting the project group on: mdpstreep@gmail.com

Contents

Acronyms and Abbreviations

Acronym / abbreviation	Description
1D	One Dimensional
2D	Two Dimensional
ALS	Airborne Laser Survey
ARI	Average Recurrence Interval
BC	Boundary Conditions
CVC	Clarence Valley Council
CVFM	Clarence Valley Flood Model
DEM	Digital Elevation Model
DTM	Digital Terrain Model
e.g.	Example given
FEM	Finite Element Method
FRP	Fibre Reinforced Plastics
GIS	Geographical Information System
hr	Hours
mAHD	Australian Height Datum in meters
NSW	New South Wales
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PO	Point Output (Depth reporting locations / digital gauges)
QGIS	Quantum Geographic Information System
RC	Reinforced Concrete
SCIA	Scientific Applications
SWE	Shallow Water Equations
TUFLOW	Two-dimensional Unsteady FLOW
UC	Unity Check

List of Symbols

Parameter	Description	Unit
A	Cross-sectional area	m^2
B	Width of Flow	m
c_f	Coriolis force coefficient	-
C_B	Bligh's constant	-
C_L	Lane's constant	-
C_r	Courant number	-
D	Stiffness matrix	-
E	Young's Modulus	N/m^2
$E_{d,stb}$	Normative load combination	kN
E_{us}	Earthquake load	kN
f	Friction coefficient	-
f_1	Form (energy) loss coefficient	-
f_{cd}	Compressive strength	N/mm^2
f_{ctm}	Tensile strength	N/mm^2
$f_{k,rep}$	Representative strength	kN/m
F_G	Self weight	kN
F_H	Horizontal force	kN
F_{HW}	Horizontal water force	kN
F_S	Forces due to sand weight	kN
F_{UW}	Upward water pressure force	kN
F_V	Vertical force	kN
F_{VW}	Vertical water force	kN
F_x	Sum of external forces in x-direction	-
F_y	Sum of external forces in y-direction	-
g	Gravitational acceleration	m/s^2
G	Dead load	kN
G_{shear}	Shear Modulus	N/m^2
H or h	Depth of water	m
h_{creek}	Waterdepth in creek	m
h_{river}	Waterdepth in river	m
k	Energy loss coefficient	-

*continues on next page

Parameter	Description	Unit
L	Total seepage length	m
L_{vert}	Seepage length in vertical sense	m
L_{hor}	Seepage length in horizontal sense	m
M	Moments	kNm
M_{xD}	Bending moment	kNm/m
n	Manning coefficient	-
p	Pressure	kN/m^2
p'_{max}	Maximum bearing capacity	kN/m^2
q_{maxb}	Shear force	kN
Q	Live load	kN
R	Hydraulic radius	m
$R_{a,d}$	Design value of strenght	kN
$R_{a,k}$	Characteristic value of strenght	kN
$R_{a,min}$	Minimum value of strenght	kN
S_u	Imposed action loads	kN
t	Time	s
u	Depth and width averaged velocity in x-direction	m/s
v	Depth and width averaged velocity in y-direction	m/s
W	Section modulus	m
W_u	Wind load	kN
x	Distance in x-direction	m
y	Distance in y-direction	m
ΔH	Differential head across structure	m
Δt	Computational timestep	s
Δx	Cell size in x-direction	m
Δy	Cell size in y-direction	m
ρ	Density	kg/m^3
σ	Stress	kN/m^2
σ_{max}	Maximum stress	kN/m^2
$\sum$	Sum	-
γ	Safety/Partial factor	-
ϵ	Strain	-
ϵ_a	Reduction value	-
μ	Horizontal diffusion of momentum coefficient	-
ν	Poisson’s Ratio	-
ζ	Water level	m

1
Introduction

The introduction describes the area of interest, the problem statement and objectives, the research questions, the project scope and the study approach.

1.1. Area of Interest

The Clarence River catchment is located in the state of New South Wales (NSW), on the east coast of Australia (see Figure 1.1). This catchment is about 22,000 km^2 and includes the upper Clarence Valley and the lower Clarence Valley. The Clarence River passes several urban areas in the lower Clarence Valley, with a total population of around 52,000 inhabitants, such as Grafton, South Grafton, Ulmarra, Maclean, Lawrence, and Yamba (Clarence Valley Coucil, 2018). The lower Clarence Valley often floods due to high water levels in the Clarence River, which has a total length of 400 km.

Figure 1.1: Location of the Clarence River Catchment (red marking within red circle) in Australia (GoogleMaps, 2018).

Grafton is located at the Clarence River and is the largest urban area and administrative centre of the lower Clarence Valley. Besides, Grafton acts as the main infrastructure hub of NSW. Here, the Pacific Highway meets the Summerland Way and the Gwydir Highway (see Figure 1.2), these are crucial road connections in the region. The Pacific Highway links the cities of Brisbane and Sydney. The Summerland Way is an alternative route for the Pacific Highway. Inundation of Grafton would result in large economic damage and severely disrupt the important road connections between Sydney and Brisbane.

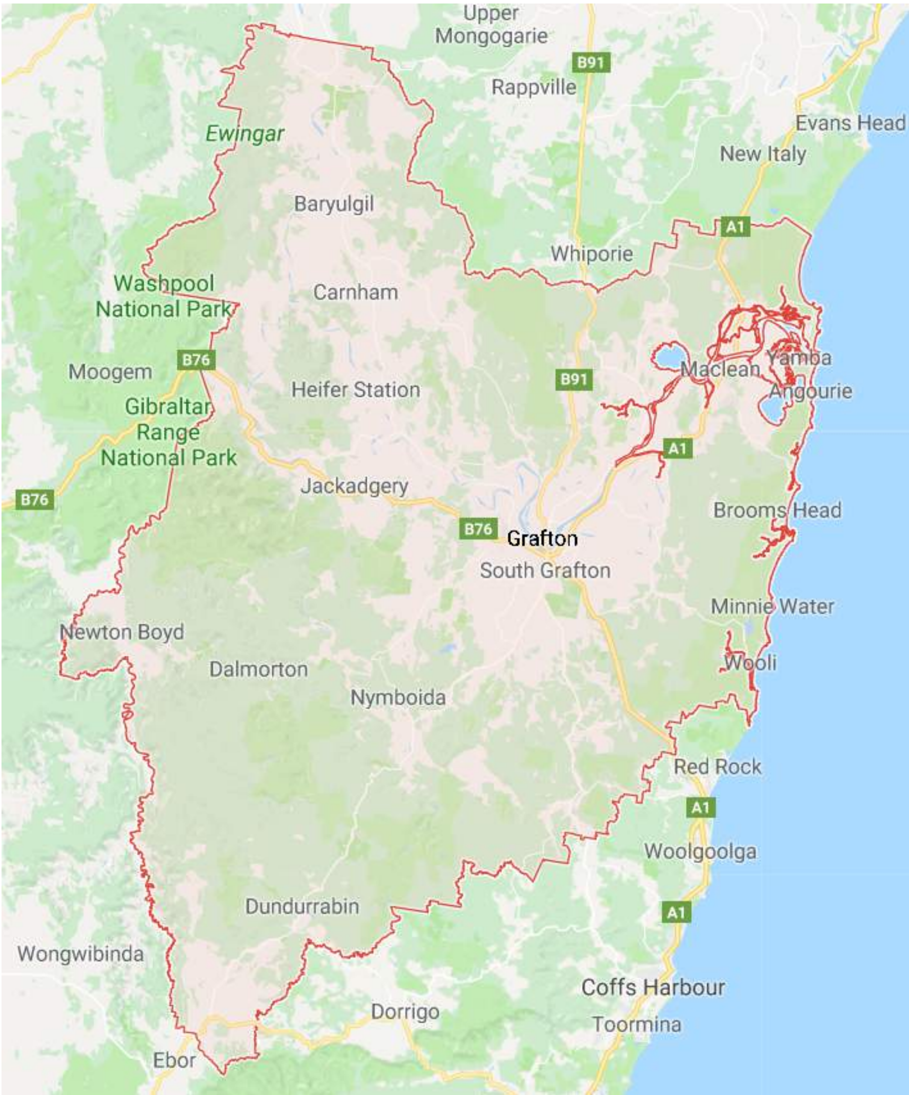


Figure 1.2: An overview of the Clarence River catchment with its main roads: Gwydir Highway (B76), Summerland Way (B91) and the Pacific Highway (A1) (GoogleMaps, 2018).

1.2. Problem Statement and Objectives

The climate in the Clarence River catchment is subtropical and this causes heavy rainfalls on regular basis. Moreover, the large catchment area and relatively short river length contributes to the enormous rise of river discharge. In case of extreme weather events (e.g. cyclones), the relatively small discharge of 160 m^3/s can grow to as much as 20,000 m^3/s, causing major flooding in the lower Clarence Valley as a result of high water levels. The area of the lower Clarence Valley is around 1,000 km^2 and the length of the Clarence River in this section is approximately 80 km. In the remainder of this report, the lower Clarence Valley is referred to as the Clarence Valley.

The raise in discharge and thus the increase of water levels, results in storage issues for the Clarence River. The river overtops its banks on multiple locations leading to flooding of the floodplains but also of urban areas. Grafton is located at a narrowing river bend, making it vulnerable for rising discharges and water levels. Maclean is also located near a narrowing of the river, resulting in the same vulnerability as Grafton. The last example of a major flood is from January 2013. The flood reached a peak of 8.08 $mAHD$. More than 2,100 residents were ordered to evacuate. After the 2013 flood, the Clarence

Valley is officially declared as a disaster zone. This declaration provides funding for those who have suffered from property damage, including residents, councils, business owners and primary producers.

The Clarence River is regulated by a large amount of flood defences (e.g. levees). Recently, the Clarence Valley Council has shown renewed interest in improving the flood mitigation strategy in the urban areas of the Clarence Valley. The council aims to reduce the water levels during high river discharges by increasing the storage capacity of floodplains. This report will investigate the feasibility and effectiveness of different flood mitigation measures mainly focusing on floodplains.

The main objective of this project is to support the Clarence Valley Council with additional insights into increasing the storage capacity of floodplains, according to the 'Room for the River' principle. The current flood protection policy focuses on heightening the levee system, especially around Grafton. However, earlier heightening efforts have shown that these measures only prevent inundation locally. In this multidisciplinary project, the goal is to investigate whether the flood protection system can be improved by making use of floodplains. This project will mainly focus on the cities of Grafton, because of the central economic and infrastructural function within the region, and Maclean, for the number of affected residents in case of a flood.

1.3. Research Questions

The main question which this multidisciplinary project aims to answer is:

How can the impact of flooding on the urban areas in the Clarence Valley be reduced by increasing the storage capacity of the floodplains?

Several subquestions are formulated to be able to answer the main research question:

- Which analysis methods can be used to qualify the impact of flooding in the Clarence Valley?
- How can the Clarence Valley Flood Model be used to validate and quantify new flood mitigation measures?
- To what extent can the floodplains in the appointed potential locations influence the impact of flooding in urban areas in the Clarence Valley?

1.4. Project Scope

The scope of the project is directly related to the main research question which focuses on reducing the impact of flooding in urban areas in the Clarence Valley. In this research, the villages Grafton, South Grafton and Maclean will actively be taken into account. Other, smaller, villages are outside the project scope, but can be taken into account qualitatively or mentioned in discussions.

The impact assessment will be based on a flood model, which has been created for the Clarence Valley. The boundary conditions of the Clarence River are imposed about 10 km upstream from Grafton and near the city of Yamba. The floodwave in the Clarence River will be simulated from 0 till 150 hours after initiation. The following design floods will be investigated in this study:

- 1 in 5 year flood;
- 1 in 20 year flood;
- 1 in 50 year flood.

A 1 in 5 year flood is the flood that occurs with a yearly probability of 1/5. The renewed interest in improving the flood mitigation strategy suggests the increase use of storage capacity of floodplains, during floods, in the Clarence Valley. Improvement or adjustment to existing flood defences are incorporated in the scope. Only first order estimations are taken into account in the study. In other words, no optimisation for the proposed solutions are taken in to consideration. Only permanent solutions for the problem will be examined. Climate change is not incorporated into the research scope, the design floods are sustained to any changes. Adaption of surface areas, change in Manning coefficient and ongoing- and future projects will not be included. Flood defences in the form of a dam or a bypass floodway are excluded from the project scope on request of the Clarence Valley Council.

1.5. Study Approach

BMT WBM created a numerical model, called the Clarence Valley Flood Model (CVFM), for the Clarence Valley. BMT WBM is a leading edge consultancy in mechanical, environmental, maritime and water engineering located in Australia. Simulations of the free-surface water flow in the Clarence Valley is done using the CVFM. These simulations provide results which will be assessed in this study. The model is necessary for the study because of the size and complexity of the study area.

The CVFM offers an opportunity to implement topographic changes to the area and show the consequences of these changes. A brief training at the office of BMT WBM in Brisbane functions as a kickoff for the project. A three days field visit is organised by the Clarence Valley Council. The field visit contributes to the understanding of the flood defence system in the Clarence Valley. Research questions, objectives and scope are discussed with a delegation of the Council.

At the Monash University in Melbourne, further research has been done. The software package needed to run the CVFM is provided by BMT WBM. Aerial information, mapping, local sources and previous studies are provided by the Monash University and the Clarence Valley Council. The duration of the project is seven weeks, one of which is dedicated to a field visit in the Clarence Valley. The study contains a feasibility study, simulations of proposed solutions and recommendations to improve the current situation.

2

Preliminary study

The preliminary study is the base of this report, the study will zoom in on the project area. First, a general description of the infrastructure analysis, stakeholder analysis and the hydrological analysis are given. Then a more detailed view of the six areas of interest is provided and a problem definition is stated per area. Appendix A provides a more detailed area analysis for the Clarence Valley in its entirety. A full area map can be found in Figure A.3.

The preliminary study answers the first subquestion:

> *Which analysis methods can be used to qualify the impact of flooding in the Clarence Valley?*

2.1. Infrastructure Analysis

The Clarence Valley contains several important infrastructural links for traffic between cities at the east coast of Australia. The most important road- and railway connections run from Brisbane in Queensland to Sydney in NSW. The Pacific Highway passes the Valley near Grafton and Maclean. The Sydney-Brisbane rail corridor stops at the Grafton railway station, located in South Grafton.

The Pacific Highway together with the Gwydir Highway and the Summerland Way, form a network for the regional traffic in the Clarence Valley and intersect in South Grafton, see Figure 1.2. The Gwydir Highway originates in South Grafton and continues inland. It is one of the main east-west routes on the east coast of Australia. The Summerland Way crosses Grafton and ends at the border between New South Wales and Queensland. Major construction works are currently ongoing to improve the Pacific Highway. The alignment is changed and a new river crossing is constructed. Research has proven that this highway has minor effect on changes in water levels upstream. More information about the new Pacific Highway can be found in (Roads and Maritime Services, 2017).

2.2. Stakeholder Analysis

In the process of proposing new mitigation measures for the Clarence Valley, it is important that all relevant stakeholders are known. Every stakeholder has its own opinion and should be approached accordingly. A distinction between authorities and citizens has been made. In Figure 2.1, the influence and the power of the stakeholders are visualised. An explanation of the different stakeholders, their influence and their power can be found in Appendix A.

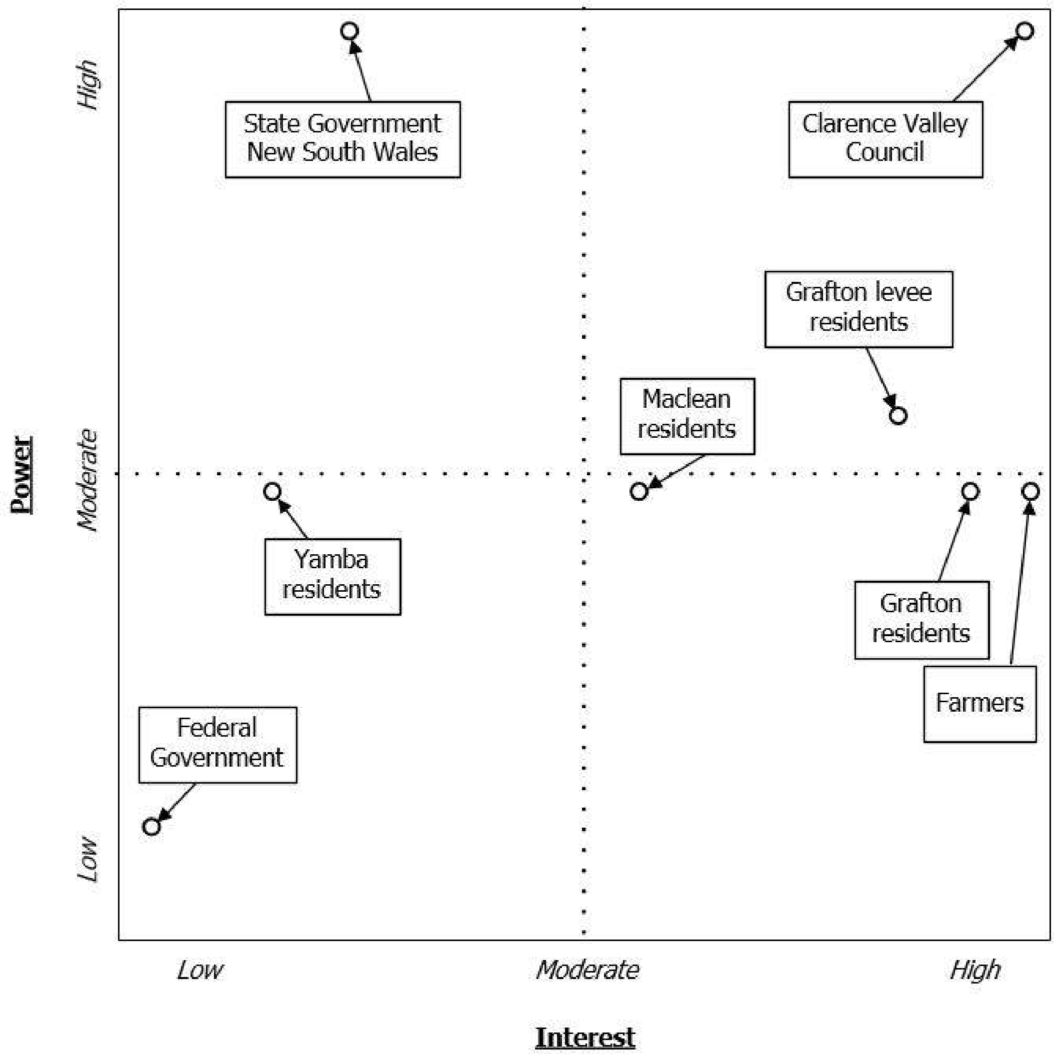


Figure 2.1: The influence of the relevant stakeholders in the Clarence Valley. Their position in the figure explains their power and interest in the proposition of mitigation measures.

2.3. Hydrological Analysis

The Clarence Valley is a complex system consisting the following key features:

- primary inlet (The Clarence River);
- several secondary inlets or tributaries;
- lakes;
- flood defences (e.g. levees, floodplains, culverts, floodgates and various drains - either man-made or existing creeks);
- tides;
- local rainfall.

As stated before in Chapter 1, the Australian east coast suffers from cyclones regularly, resulting in severe rainfall. The excessive water due to local rainfall in the upper catchment is drained towards the coast, leading to a massive discharge increase, which is visualised by Figure 2.2.

The Clarence River has been monitored by gauges for decades, providing reliable information on water levels and recurrence intervals. The Prince Street gauge and the Maclean gauge function as reference points and provide most of the historical data, see Figure 2.3 for their locations. The water levels in the Clarence River can rise up to 7.9 $mAHD$ at the Prince Street gauge, before flooding occurs in the surrounding urban areas. At Maclean, the water level can rise up to 3.4 $mAHD$ before causing problems (Farr and Huxley, 2014). In this analysis, the 5, 20 and 50 year Average Recurrence Intervals (ARI) are used. A 5 year ARI means that a hypothetical flood event is likely to happen once every five years. The probability of occurrence is 20% for any given year in that case.

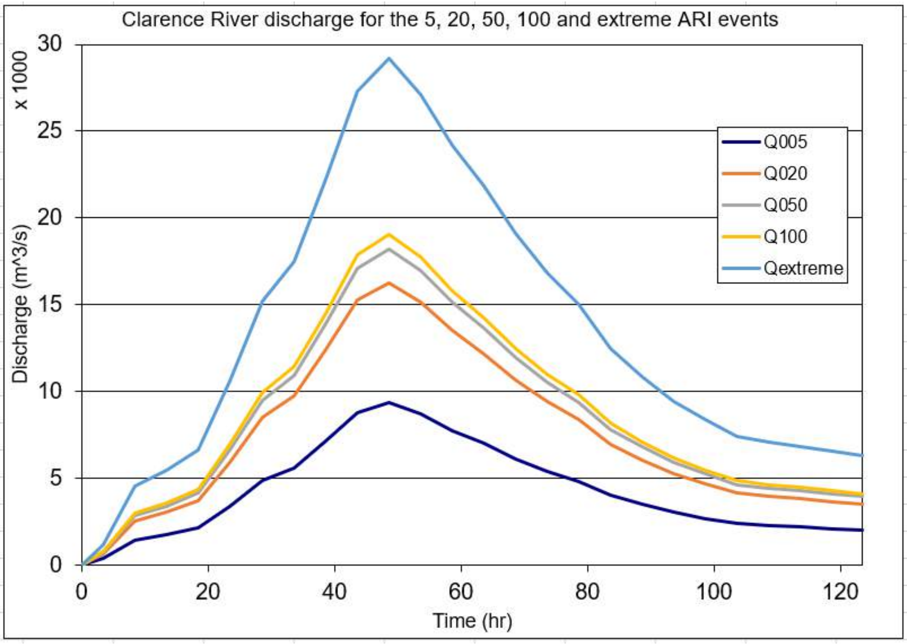


Figure 2.2: Discharges of the Clarence River inflow boundary for the 5, 20, 50, 100 and extreme ARI flood events. 100 & Extreme ARI flood events are only visualised to present the magnitude of the floodwaves, but are outside of the project scope. No return period can be coupled to the Extreme ARI flood event, see Appendix A.4.

The total water discharge by the secondary inlets is about 20% of the primary inlet (Clarence River) discharge. An overview of all the inlets of the Clarence Valley is given in Figure 2.3 below. This figure includes the primary inlet, secondary inlets, downstream (tidal) outlet and smaller local rainfall catchments. Lake Woolooweyah discharges its local rainfall directly to the outlet boundary. The tide of the Pacific Ocean has influence on the water level of the river up to Copmanhurst, a town about 40 km upstream of Grafton. The tidal effect is at its largest at the outlet of the river and decreases upstream. This tidal effect is included in the CVFM and therefore taken into account in this study.

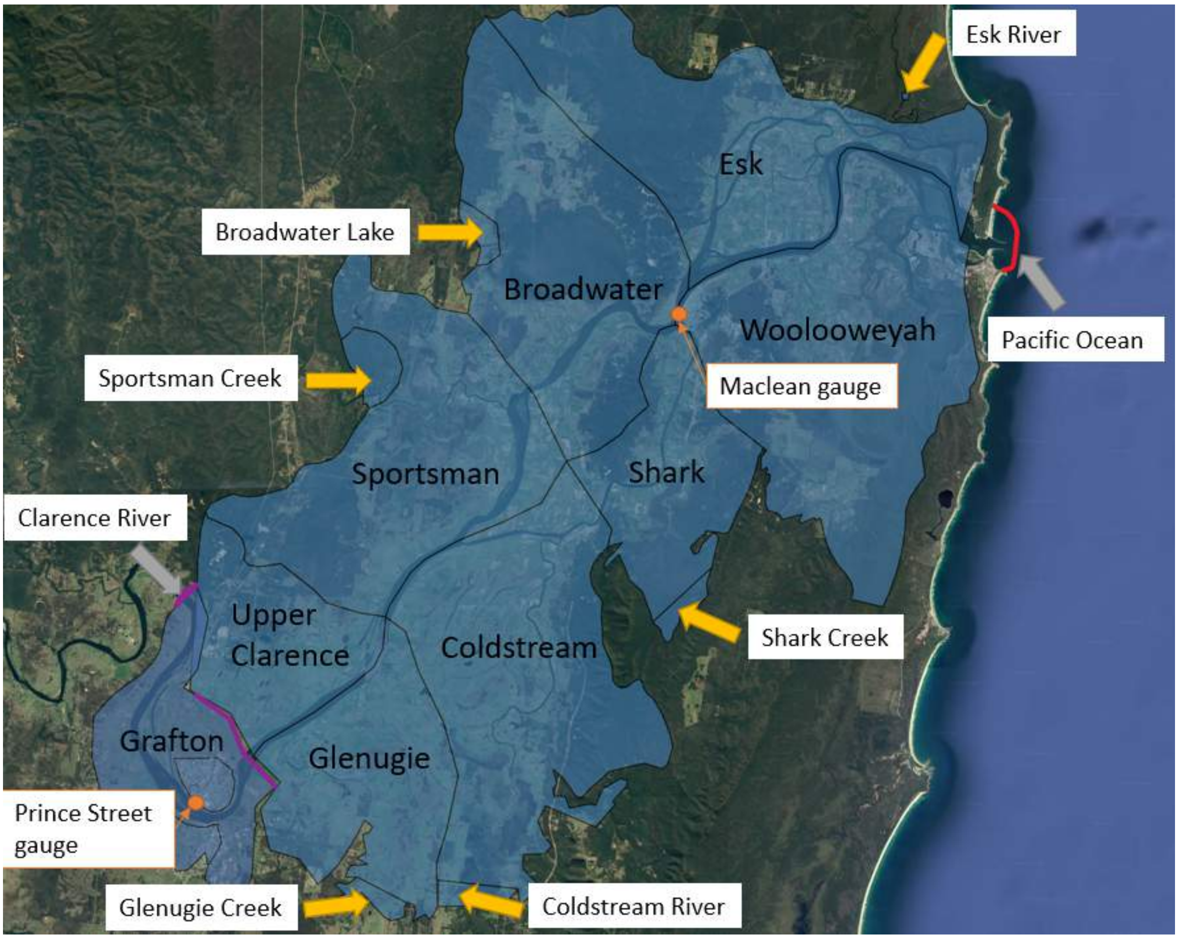


Figure 2.3: Clarence Valley hydrological system containing the primary inlet (grey arrow & purple line), secondary inlets (yellow arrow), downstream outlet (grey arrow & red line), catchment domains (light blue polygons) and important gauges (orange dot).

In the figures on the next pages, the consequences of the design ARI flood events are shown for Grafton (see Figures 2.4) and Maclean (see Figures 2.5).

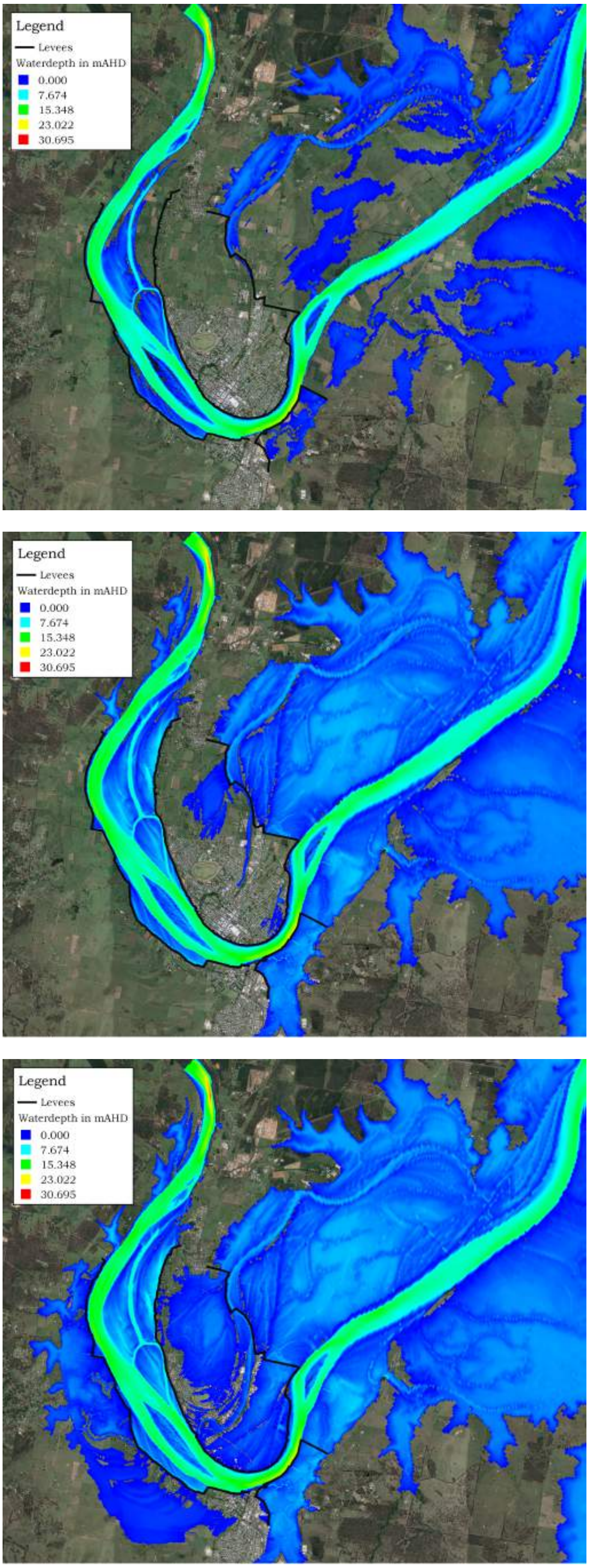


Figure 2.4: Overview of the 5 year (top), 20 year (middle) & 50 year (bottom) ARI flood events around Grafton.

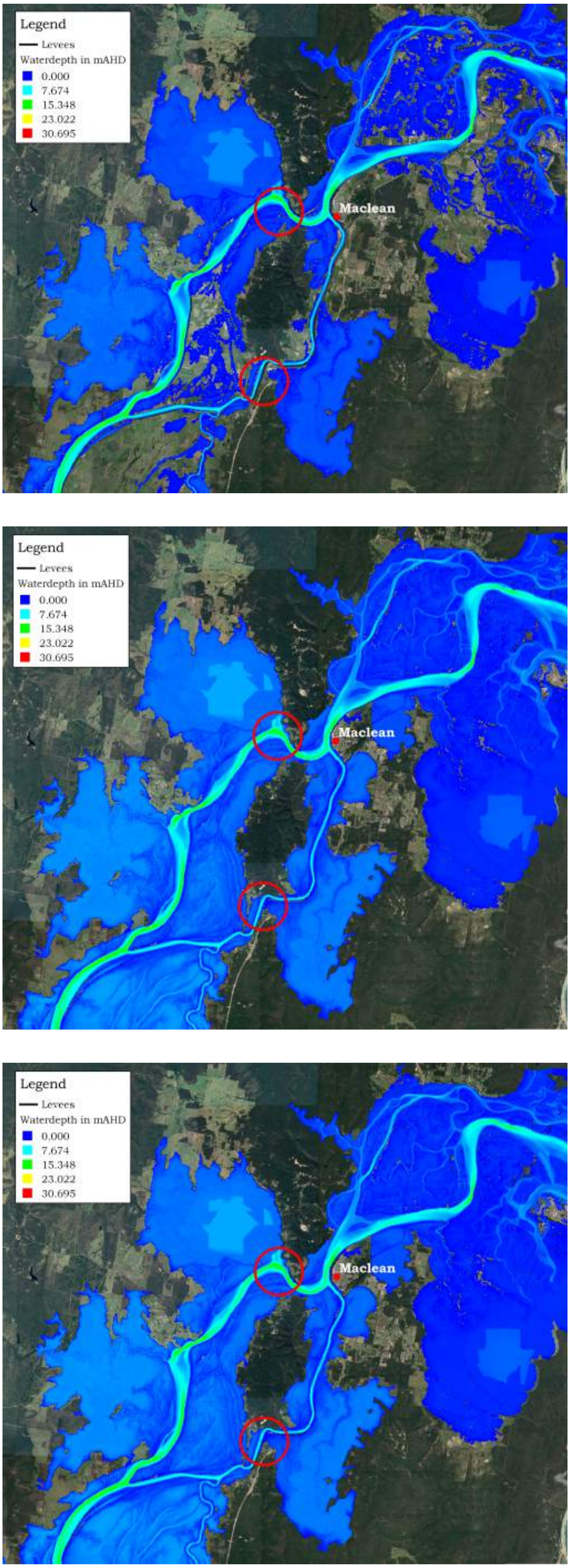


Figure 2.5: Overview of the 5 year (top), 20 year (middle) & 50 year (bottom) ARI flood events around Maclean.

2.4. Areas of Interest

The preliminary study and the field visit (see Appendix B) contributed to the general understanding of the flood defence system of the Clarence Valley. This resulted in six areas of interest which could potentially lead to improving the flood mitigation in the urban areas in the Clarence Valley. The areas of interest contain a bottleneck near urban areas or the storage capacity of the floodplains is not fully used during a flood. By changing the topography of the area, a new situation can be created. In this chapter, the six areas are assessed and the following subjects are examined in detail:

- Topography;
- Presence of floodplains;
- Urban areas;
- Flood defences;
- Impact analysis.

The impact analysis explains for which flood events urban areas flood, levees overtop and the storage capacity of the floodplains could potentially be increased. The aspects mentioned in the impact analysis will be evaluated by means of a scenario in the numerical modelling software, TUFLOW. The areas need to be analysed in a qualitative and quantitative way, to decide which problems will be examined for possible solutions. This is done in Chapter 4. The six areas, and their rough boundaries, taken into account can be seen in Figure 2.8. The six areas are chosen based upon the preliminary study, the field visit, the area analysis and the CVFM.

An overview of the levees around Grafton and Maclean are given in Figures 2.6 & 2.7. These Figures will be referenced multiple times in the upcoming sections.

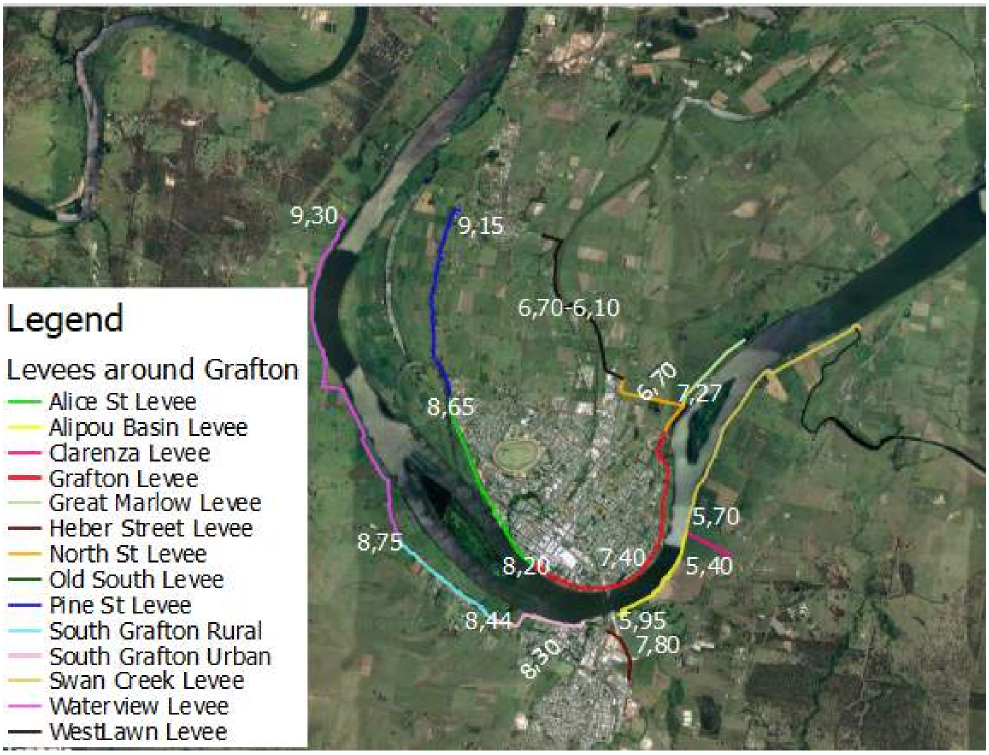


Figure 2.6: Overview of the levees around Grafton, heights of the levees given in $mAHD$.

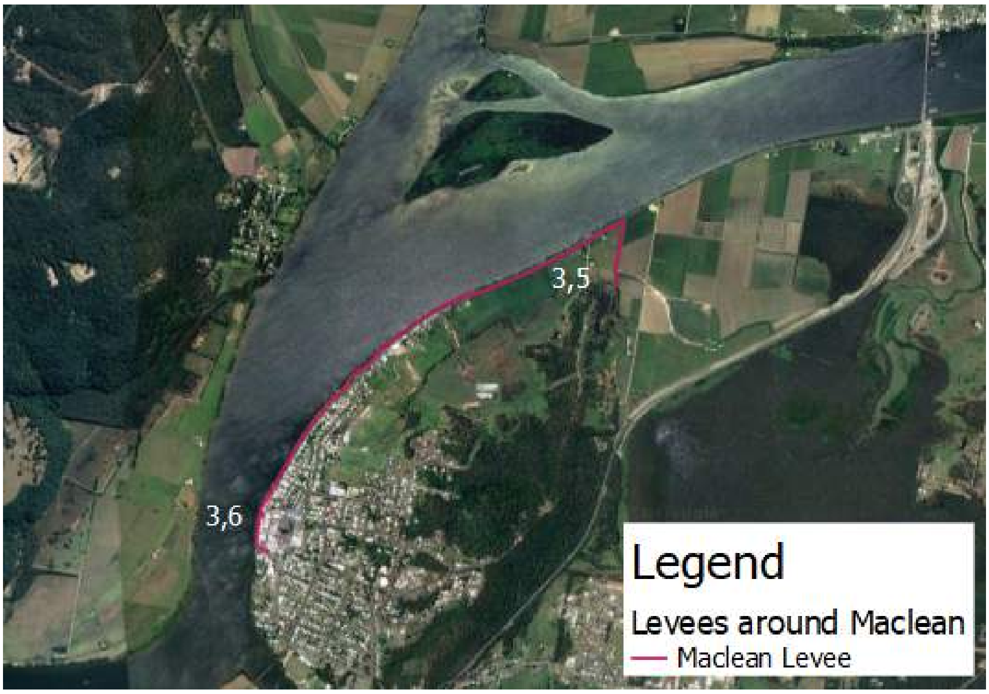


Figure 2.7: Overview of the levees around Maclean, heights of the levees given in $mAHD$.

The numbers presented in Figure 2.8 are linked to the following areas:

- Area 1: Baker's Swamp;
- Area 2: Southampton Floodplain;
- Area 3: South Grafton;
- Area 4: Clarenza Floodplain;
- Area 5: Hill Ridge;
- Area 6: Maclean.

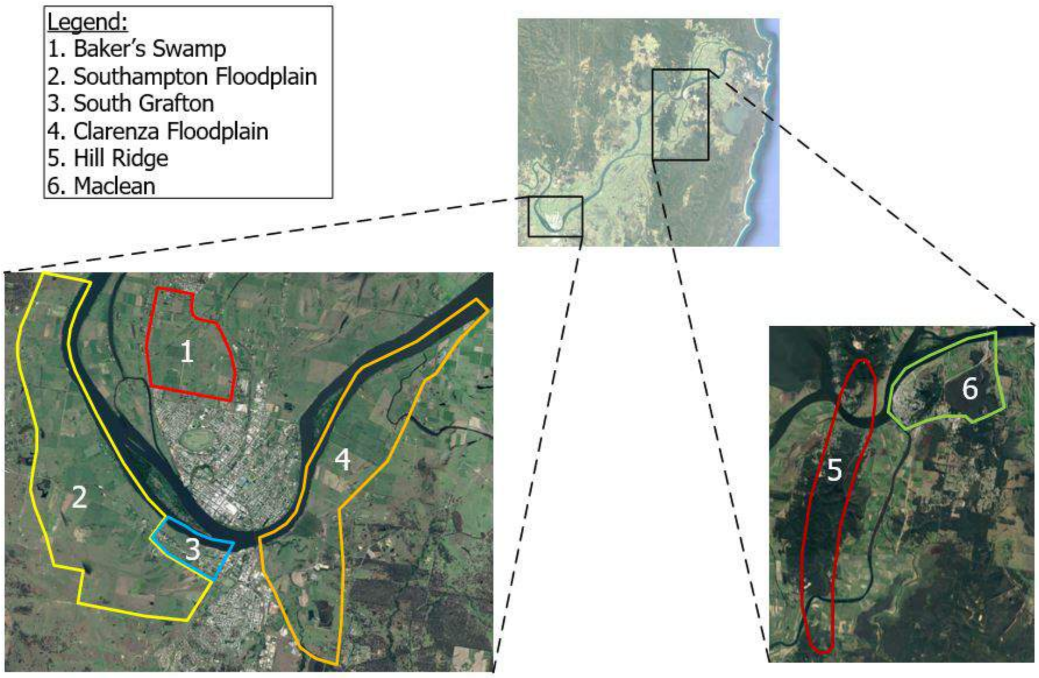


Figure 2.8: Areas of interest in the Clarence Valley where mitigation measures might reduce flood impact in urban areas.

2.4.1. Area 1: Baker's Swamp

Baker's Swamp is a low-lying area north of Grafton. The area is enclosed by the Pine Street Levee on the west, Junction Hill on the north and the Westlawn Levee on the east. In Figure 2.8 one can see the established boundaries and in Figure 2.6 one can see the levees surrounding the area with their given height in $mAHD$.

Pine Street Levee is a combination of a levee and a railway. The railway is constructed on a higher elevation and did not flood in history so far. As mentioned in Appendix A.3, maintaining the railway connection between Brisbane and Sydney is of high importance. When the Pine Street Levee overtops, culverts are used to allow the water to flow into Baker's Swamp to prevent the railway from flooding (see Figure A.2).
When running the design flood events in the CVFM (see Figure 2.4) the following consequences can be seen:

Table 2.1: Consequences of the 5, 20 and 50 year ARI flood events in Baker's Swamp.

ARI flood event	Consequences
5 year	Pine Street Levee & Westlawn Levee do not overtop, Baker's Swamp stays dry;
20 year	Westlawn Levee overtops, part of Baker's Swamp with low elevation inundate;
50 year	Pine Street Levee overtops, culverts will allow water to stream into Baker's Swamp; Westlawn Levee overtops, Baker's Swamp completely inundates.

Impact analysis
Baker's Swamp inundates only partly in a 20 year ARI flood event, while it can store more water. No flood defence is present between Grafton and Baker's Swamp and therefore water can flow into the town during a 50 year ARI flood event.
Alumy Creek flows from the Westlawn Levee into Grafton and during a flood it stores too much water so it overtops its banks in Grafton. This is mostly due to local rainfall. Local rainfall is not the main subject in this report and therefore no adjustments will be made concerning the Alumy Creek.
The culverts underneath the railway also allow water to flow into Baker's Swamp. This is neglected, as the volume of water flowing into Baker's Swamp from the railway culverts is negligible compared to the volume that flows from the Westlawn Levee.

2.4.2. Area 2: Southampton Floodplain

Southampton Floodplain is an area on the west side of Grafton. Southampton Floodplain is enclosed by South Grafton on the east side and higher grounds on the west (see Figure 2.8). Southampton Floodplain is located near the Clarence River and therefore also contains levees. The Waterview Levee and the South Grafton Rural Levee seperate the Southampton Floodplain area and the Clarence River, as can be seen in Figure 2.6. The Gwydir Highway originates in South Grafton (see Figure 1.2) and crosses Southampton Floodplain to make access inland. The consequences of the design flood events are the following (see Figure 2.4):

Table 2.2: Consequences of the 5, 20 and 50 year ARI flood events in Southampton Floodplain.

ARI flood event	Consequences
5 year	Southampton Floodplain stays dry;
20 year	Southampton Floodplain stays dry;
50 year	Southampton Floodplain floods partly due to overtopping of the Waterview Levee, the Gwydir Highway inundates.

Impact analysis
The Southampton Floodplain area only floods during a 50 year ARI flood event. During this flood, also

one of the main roads, the Gwydir Highway, floods. No flood defence is present between Southampton Floodplain and South Grafton. In a high ARI flood event this can cause problems. For tributaries downstream, local rainfall is taken into account. For Southampton Floodplain the local rainfall is not taken into account, because the rainfall in this area is added to the inflow boundary of the TUFLOW numerical model. This means that the inundation depth could possibly increase in case of a flood event in combination with (earlier) local rainfall. The presence of this possibility is acknowledged but not taken in to account, due to model restrictions.

2.4.3. Area 3: South Grafton Urban

The third area that is being assessed is the area just upstream of South Grafton. This area includes the South Grafton Urban Levee as can be seen in Figure 2.6. This levee protects the South Grafton urban area from flooding. The levee has a sharp narrowing where the floodplain ends downstream. In case of a flood, when the river makes use of this floodplain, the river encounters a sudden restriction in width.

Table 2.3: Consequences of the 5, 20 and 50 year ARI flood events in South Grafton.

ARI flood event	Consequences
5 year	Floodplain contains water, levee does not overtop;
20 year	Floodplain contains water, levee does not overtop;
50 year	Floodplain contains water, levee overtops at the upstream end.

Impact analysis

In case of a flood, the narrowing causes a bottleneck in the river because of a reduction of the river's width. The sharp corners of the narrowing in the levee might cause extra resistance for the streamline of the water flow. Subsequently, this will result in an increase of flow velocity.

2.4.4. Area 4: Clarenza Floodplain

Area 4 is located adjacent to the Clarence River and is surrounded by natural higher ground. The area contains a levee system, a part of the Clarenza Floodplain and parts of the Pacific Highway. The alignment of the Pacific Highway is located parallel to the Clarence River (see Figure A.5) and mainly on higher ground. Only the part before the crossing with the Herber Street Levee (see Figure 2.6), will flood during the 5 year ARI flood event. The Swan Creek crosses the Clarenza Floodplain. The creek emerges in the Clarence River by means of the Swan Creek Floodgate. The Swan Creek Floodgate drains and irrigates the Clarenza Floodplain and Swan Creek Floodplain. The levee system downstream of South Grafton assists the water through the riverbend and controls flooding of the Clarenza Floodplain. The Alipou Basin Levee, with a rock armour protection, prevents erosion on the outer bend of the river.

Table 2.4: Consequences of the 5, 20 and 50 year ARI flood events around the Clarenza Floodplain.

ARI flood event	Consequences
5 year	The Alipou Basin Levee & Swan Creek overtop and inundate Clarenza Floodplain partly; including a part of the Pacific Highway.
20 year	The Alipou Basin Levee & Swan Creek overtop and inundate Clarenza Floodplain. Also parts of the Pacific Highway are flooded.
50 year	The Alipou Basin Levee & Swan Creek overtop and inundate Clarenza Floodplain. Also parts of the Pacific Highway are flooded.

Sequence of flooding of the Clarenza Floodplain for a 20 or 50 year ARI flood event. In case of a 5 year ARI flood event only the first four steps apply :

1. Overtopping of the Swan Creek into the Clarenza Floodplain.

2. Overtopping of the Alipou Basin Levee.
3. Filling of the basin between the Alipou Basin Levee and the higher ground, captured between the Heber Street Levee and the Clarenza Levee.
4. Water flowing over the Pacific Highway, around the Heber Street Levee.
5. Overtopping of the Clarenza Levee.
6. Filling of the Clarenza Floodplain adjacent to the Clarence River and flooding parts of the Pacific Highway.

Impact analysis
The higher flow rate in the outer bend leads to erosion of the river bed. A high level of maintenance of the Alipou Basin Levee is needed compared to other levees in the system. The maintenance is intensive due to erosion of the rock protection present, based on Appendix B. The restricted cross section due to the Alipou Basin Levee creates a bottleneck for the river which stimulates overtopping downstream of the bend. The Grafton Bridge, which is located just before the area of interest creates another bottleneck. The riverwidth is restricted by the embankments of the bridge, consequently, downstream adjustments will be influenced. The current state of the Swan Creek Floodgate could be problematic in the future. The side walls located at the river side show displacements. Erosion of the riverbed has occurred by water flowing through the floodgate (see Appendix C). The supporting reaction and stability of the side walls are negatively influenced. The floodgate has large impact on the Swan Creek Floodplain and the Clarenza Floodplain located behind. The performance of the floodgate will be checked in this study.

2.4.5. Area 5: Hill Ridge
A hill ridge crosses the Clarence Valley and the Clarence River. The Clarence River splits at the village Brushgrove, 50 km from the river mouth. The splitted rivers meet again 30 km from the river mouth at Maclean. Woodford Island is a 100 km^2 island and is enclosed by the two river arms. The hills running over Woodford Island rise up to 200 m and form an obstacle for the Clarence River.

Figure 2.9 shows the elevation of the hill ridge compared to the surrounding area. The Clarence River bed has a negative elevation and is illustrated in blue.

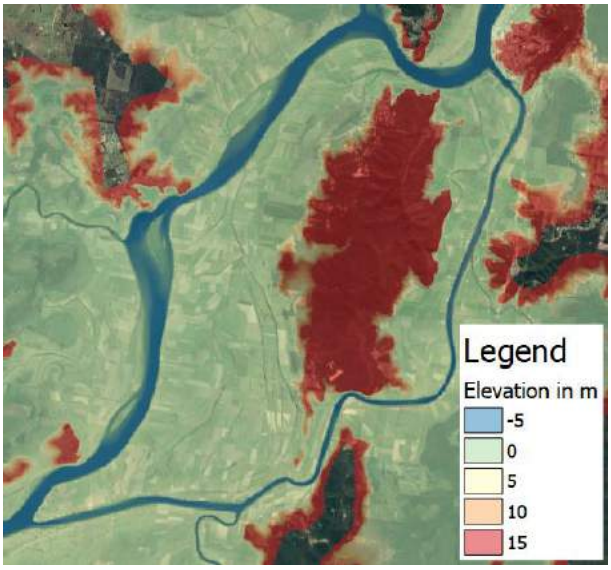


Figure 2.9: Elevation data of the Hill Ridge over Woodford Island. One can see the obstacle formed by the Hill Ridge for the Clarence River.

Table 2.5: Consequences of the 5, 20 and 50 year ARI flood events around the hill ridge.

ARI flood event	Consequences
5 year	Areas upstream of Hill Ridge flood.
20 year	Areas upstream of Hill Ridge flood.
50 year	Areas upstream of Hill Ridge flood.

The consequences of the different flooding events are differences in flooding area and inundation depths. Figure 2.5 shows consequences of different design ARI flood events. One can see that the natural high grounds create narrow passages for the river (red circles in Figure 2.5).

Impact analysis
The flow capacity of the Clarence River is limited by the width of the passages through the hill ridge. The two bottlenecks, north and south of Woodford Island, result in backwater curves upstream of the hill ridge. These backwater curves result in an increase in waterdepth in the upstream areas.

2.4.6. Area 6: Maclean

The last assessed area includes the city of Maclean, its levee system and its local catchment area. The area is enclosed by the Clarence River and higher grounds (see Figures 2.8 & 2.10). The Maclean Levee (Figure 2.7) and the natural higher ground protect the natural lower grounds of Maclean against flooding. The Maclean Levee is mainly an earth levee, but small parts are concrete walls. The concrete walls are connected to a sheet pile wall which extends to about 20 m into the ground.

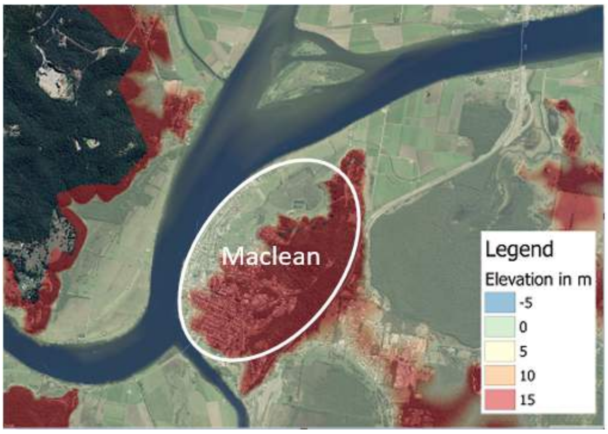


Figure 2.10: Elevation map of Maclean and the surrounding areas.

Table 2.6: Consequences of the 5, 20 and 50 year ARI flood events around Maclean.

ARI flood event	Consequences
5 year	Maclean stays dry;
20 year	Maclean stays dry;
50 year	Part of Maclean Levee overtops and water flows into town.

Impact analysis
The city of Maclean suffers from minor flooding in a 50 year ARI flood event, as water overtops a small section of the Maclean Levee. The reinforced concrete (RC) part of the Maclean Levee Wall is placed on top of a sheet pile wall and connected to the earth wall. During a flood, water boils up from the ground behind the levee wall, according to Appendix B. The event of water appearing behind the levee is assessed in Appendix D because it has impact on the flood safety of Maclean. Due to the fact that only small flooding of Maclean occurs during a 50 year ARI flood event, the decision is made to not construct any scenarios concerning Maclean. However, measures upstream might have influence on

the water levels downstream. Therefore, considered scenarios in area one to five might solve the minor overtopping at Maclean. For Maclean a structural assessment is made concerning seepage around the concrete levee walls.

2.5. Discussion

The preliminary study contains a infrastructural analysis, a stakeholder analysis, a hydrological analysis and an area analysis with an impact analysis. The preliminary study gives an overview of the current situation in the Clarence Valley.

The preliminary study has set the boundaries to the protection of urban areas only. If the project duration took sufficiently long, the flood protection of other smaller communities could have been taken into account.

The main infrastructure is found of national importance and will be taken into account in the flood assessment. The interest and power of stakeholders could conflict with the vision of the project which focuses on urban areas.

The hydrological analysis is based on measurements over the latest years. The design floods for the study are limited on advice of the council to a 5, 20 and 50 year ARI flood event. Other flood events have not been taken into consideration and might give more information about inundation of the Clarence Valley. Due to this restriction, effects of different floods in the system might be overlooked.

The six areas analysed in detail have been chosen based on the councils suggestions, preliminary study and the field visit. According to the reference system in the CVFM, a impact analysis is described per area. Also the sequence of inundation is of importance. Other areas which could improve flood mitigation might be outside of the chosen six areas. The vision of this study could be in conflict with the interest of stakeholders.

3

Evaluation Clarence Valley Flood Model

This chapter will contain an elaboration on the numerical model (TUFLOW), that was delivered by BMT WBM. Moreover, this chapter discusses key considerations that should be addressed during a modelling study.

The second subquestion can be answered after this chapter:

> *How can the Clarence Valley Flood Model be used to validate and quantify new flood mitigation measures?*

3.1. General Background

TUFLOW stands for Two-dimensional Unsteady Flow and is a numerical software program that is developed for modelling two-dimensional (2D) flow. TUFLOW can simulate depth-averaged free-surface flows, for instance long waves like floods and tides. Originally, TUFLOW was developed for two-dimensional modelling only. Nowadays, it is capable of solving one-dimensional (1D) and 2D free surface flows parallel. For this study, TUFLOW Classic 2012-AD is used.

TUFLOW solves the 1D Saint-Venant equations, including the inertia terms. In the 1D situation (e.g. culverts), this means that the continuity and momentum equations are solved (Appendix E). The two equations that have to be solved contain two unknowns, the depth and width averaged velocity (u) and the water level (ζ). Both these equations are first order differential equations and need one Boundary Condition (BC) for a solvable system.

For the 2D case (e.g. floodplains), the depth averaged Shallow Water Equations (SWE) are used. Compared to the 1D Saint-Venant equations, these include additional cross momentum and sub-grid scale turbulence terms. The equations are derived using the assumption that a hydrostatic pressure distribution is present. This assumption is valid when the wave length is much grater than the depth of water. The SWE that are solved can be found in Appendix E. The velocity parameter is dissolved into a x and y component which results in a system of three equations and three unknowns. The water level (ζ), the depth averaged velocity component in x-direction (u) and the depth averaged velocity component in y-direction (v). As two out of three equations are second order differential equations, two BC's are needed for a solvable system. The above-mentioned equations are solved for every grid cell of the TUFLOW model.

TUFLOW does not have its own interface for adjustments on input and output. The TUFLOW software makes use of Geographical Information System (GIS) software and other third-party software to create, manipulate and view data. The use of a GIS software offers the following benefits (*TUFLOW User Manual*, 2016):

- Tools are available for data management, manipulation of data and presentation of data;
- The input data can immediately be geographically referenced;
- Illustrations can be extracted for reports, displays e.g.;
- Multiple GIS packages can be chosen;
- Input data can be transferred to other hardware, depending on user preference.

Furthermore, text files are used to control the simulations and set the right simulation parameters. These files can be used for calibration studies, design studies or scenario studies (used for investigation of flood mitigation options). In order to do so, multiple input datasets need to be specified. All kinds of input data and extensions are explained in Appendix E.

The applied software for manipulation of the input data for this research are the following:

- Text-Editor: Notepad ++;
- GIS-package: QGIS;
- Spreadsheet-software: Microsoft Excel.

3.2. Model Set-Up

In this section, the CVFM is discussed. BMT WBM set up this model to assess the flood behaviour within the Clarence Valley. The model is fully calibrated and the results should assist the Clarence Valley Council (CVC) in their floodmanagement strategies. Moreover, assumptions, attributes and elements of the model are discussed.

3.2.1. Construction

Topography

The model makes use of different sources to provide an overview of the ground elevation in the most detailed way. The council provides ground survey data to visualise show the height and state of the levees. More general topography data of the area is provided by a Digital Elevation Model (DEM). The used DEM in this model has been adjusted over the last few years. In first instance, the DEM consisted out of multiple sources, but lately (2013) the input for the DEM changed to an Airborne Laser Survey (ALS) (Farr and Huxley, 2013). This survey has a 1-meter resolution and can be considered as an accurate data source.

Land use Delineation

To determine the hydraulic roughness of the Clarence Valley, a land use parameter is included. By adding the Manning coefficient, one creates differences in the run-off velocity of for example roads and forest land types. For this model, nine different land use types are specified, which can be seen in Table 3.1. These Manning coefficients have been calibrated in the events which are discussed in Section 3.2.4. A visualisation of the Manning coefficients in the area of interest are given in Appendix E.

Table 3.1: Land use type with specified Manning coefficient (Farr and Huxley, 2013).

Land Use Type	Manning coefficient
River Bank	0.08
River	0.025
Island Vegetation	0.08
Minor Tributary	0.035
Pasture	0.08
Sugar Cane	0.15
Crops	0.1
Forest	0.2
Urban Blocks	0.3
Parks	0.04
Roads	0.02

Model Boundaries
To provide for a stable and solvable model, BC's need to be imposed. The following BC's (see Figure 2.3) are imposed:

- Inflow: discharge of the Clarence River at Mountain View (primary inlet);
- Inflow: discharge of the tributaries downstream of Mountain View (secondary inlets);
- Local rainfall runoff from the floodplains;
- Outflow: water levels (tidal influence) of the Pacific Ocean.

3.2.2. Resolution

A models efficiency and precision is determined by the cell sizes of the 2D domain. The cells need to be sufficiently small to present the correct hydraulic behaviour, but large enough to minimise the simulation times (*TUFLOW User Manual*, 2016).

Three different cell sizes are applied in the CVFM. In Appendix E, the effect of different cell sizes on flood modelling is presented. Due to the fact that the total model is very extensive, a 60-meter grid is used for the non-urban parts. A 30-meter grid is used around critical areas like Grafton and a 10-meter grid is used in the city centre itself. A visual presentation can be seen below in Figure 3.1. One can see the coarsening of the grids from left to right.

Figure 3.1: Example of the grid sizes used in TUFLOW around Grafton.

3.2.3. Computational Timestep

The accuracy and stability of a model prediction is determined by the chosen timestep. The runtime is directly influenced by this timestep, because a smaller timestep will induce more calculations and therefore a longer simulation time. The limit timestep for stability is called the Courant number (see Appendix E). It is possible to state different timesteps for the 1D and 2D features within a model. It has to be noted that the 2D model should have the preference for controlling the timestep. The 1D domain only takes around 1% of the computational effort (*TUFLOW User Manual*, 2016). For this project, the timesteps being used are the following:

Table 3.2: The timesteps used in TUFLOW for the different grids.

Gridsize (meter)	Timestep (seconds)
60	12
30	4
10	4

3.2.4. Calibration

Over the last decades flooding has occured regularly in the Clarence Valley. In Appendix E, one can find the recorded flood data since 1839. All peak flood levels have been measured at the Prince Street Gauge and the unit is $mAHD$. The more recent events have shown to be the largest events, almost resulting in overtopping of the levees. The CVFM has been calibrated by BMT WBM on the following flood data (Table 3.3):

Table 3.3: Flood Model Calibration Events (Farr and Huxley, 2013).

Historic Flood Event	Peak Flood Level ($mAHD$)	Approximate ARI flood event Equivalent
Jun 1967	7.55	25 year ARI
Jan 1968	6.17	8 year ARI
May 1980	6.35	7 year ARI
Apr 1988	6.73	9 year ARI
May 1996	7.03	10 year ARI
Mar 2001	7.70	14 year ARI
May 2009	7.33	12 year ARI
Jan 2013	8.09	27 year ARI

These events have been selected on the following factors:

- **Data Availability:** There is a good spatial coverage of the catchment, concerning the recorded rainfall and water level data;
- **Event Magnitude:** Events of different magnitude have been selected. This makes sure that the model behaves correctly in major and minor events;
- **Event Recency:** The river is a dynamic system, to select more recent events it is ensured that the most recent catchment conditions are represented.

3.3. Added value of the Clarence Valley Flood Model

With a total of 542496 grid cells in the CVFM (solving the presented equations for every single inundated cell), the use of a numerical model in this research is obvious. The CVFM is being used to analyse the present system and to test mitigation measures (validation). Six areas are assessed on potential flood mitigation measures, as presented in Chapter 2. The following design floods are considered in this study:

- 1 in 5 year flood;
- 1 in 20 year flood;
- 1 in 50 year flood.

The present system simulations are considered as reference or base scenario, these can be seen in Figures 2.4 and 2.5. These simulations has shown to be an accurate representation of the reality (Farr et al., 2014). New scenarios will be created based upon the area analysis in Chapter 2 and are presented in Chapter 4. These scenarios may consist of lowering a levee or constructing a levee and are implemented in the model. The output of those scenarios will be compared to the calibrated reference scenario. By using this method, a quantitative analysis between the existing and newly created scenario can be presented, also known as a validation of results.

3.4. Discussion

The CVFM gives insights in the following values: water depths (peak level differences and inundation depths); evacuation times; inundation times and flow velocity. These can be used to quantify the impact of flooding in the Clarence Valley.

This chapter explained the calibration and general set-up of the used numerical flood model. This chapter also elaborates on how to implement modifications in the model and how to generate the desired output data.

The modifications made in the model are basic modifications. With just a two-day tutorial as training, experience with modelling in TUFLOW is limited. Due to limited knowledge of the model and the software, not the full potential of the model has been used. Other different mitigation measures than creation, removal or adjustments of a levee have not been applied.

The 2012-version of TUFLOW is used. The newest version of TUFLOW has been released in 2016 and has for instance new output features for QGIS. A lack of these new output features resulted in extra output processing time.

4 Scenarios

In Chapter 2, the areas which could potentially lead to improving the flood mitigation in the urban areas in the Clarence Valley are appointed. In this chapter, possibilities within these areas are translated into scenarios and implemented in the CVFM. Visualisations of all the adjustments in every scenario are given in Appendix F.

4.1. Area 1: Baker's Swamp

Based on the area analysis, three different scenarios for Baker's Swamp are constructed. These scenarios are presented in Table 4.1 below. The aim for this area is to increase the storage capacity of the Baker's Swamp.

Table 4.1: Scenarios for Baker's Swamp, which might positively influence floods in the urban areas.

Area 1: Baker's Swamp		
Scenario 1.1	Lowering Westlawn Levee	Figure F.1
Scenario 1.2	Construct levee north of Grafton (Carrs Street)	Figure F.2
Scenario 1.3	Combination of both scenario 1.1 & 1.2	Figure F.3

In scenario 1.1, the Westlawn Levee north of Grafton is lowered to 5.0 $mAHD$. This enables controlled overtopping of the Westlawn Levee, resulting in an increase of storage capacity of Baker's Swamp. Scenario 1.2 simulates the construction of a levee north of Grafton (7.5 $mAHD$). This levee is constructed to prevent water inflow from Baker's Swamp into Grafton. Scenario 1.3 contains both measures from scenario 1.1 and 1.2. This combination aims to increase the storage capacity of Baker's Swamp and at the same time protect Grafton from the increase in stored water.

4.2. Area 2: Southampton Floodplain

Four different scenarios are constructed for Southampton Floodplain and presented in Table 4.2 below. The aim for this area is to increase the storage capacity of Southampton Floodplain.

Table 4.2: Scenarios for Southampton Floodplain, which might positively influence floods in the urban areas.

Area 2: Southampton Floodplain		
Scenario 2.1	Lowering Waterview Levee (upstream)	Figure F.4
Scenario 2.2	Scenario 2.1 + construct levee in front of Gwydir Highway	Figure F.5
Scenario 2.3	Scenario 2.1 + construct levee in South Grafton	Figure F.6
Scenario 2.4	Scenario 2.3 + construct Gwydir Highway embankment and culverts	Figure F.7

In scenario 2.1, the Waterview Levee is lowered with 2.0 m over a length of 1.5 km. During a flood, the water is able to flow into the Southampton Floodplain, if the water level in the river exceeds 7.0 $mAHD$. Scenario 2.2 includes the adjustment of scenario 2.1 and a newly constructed levee of 5.0 $mAHD$ in front of the Gwydir Highway. This levee should protect the Gwydir Highway from flooding and therefore maintaining the East-West connection. Scenario 2.3 includes the adjustment of scenario 2.1 and a newly constructed levee in front of South Grafton of 9.5 $mAHD$. In this case, the highway might inundate, resulting in possible traffic diversion to alternative routes. Scenario 2.4 includes the adjustments of scenario 2.3 and heightens the Gwydir Highway on lower elevations, where it might inundate, to 6.5 $mAHD$. Culverts are added on those locations to allow a water flow underneath the highway.

4.3. Area 3: South Grafton

One scenario is constructed for the South Grafton Urban Levee, see scenario 3.1 in Table 4.3. The aim for this area is to get rid of possible bottlenecks, in the form of a sudden river narrowing.

Table 4.3: Scenarios for South Grafton, which might positively influence floods in the urban areas.

Area 3: South Grafton		
Scenario 3.1	Streamline South Grafton Levee	Figure F.8

This scenario is constructed by cutting off the corners of the South Grafton Urban Levee, creating a smooth transition of 8.5 $mAHD$. Prevention of a sudden narrowing may decrease sudden increase in flow velocity or water levels in this area.

4.4. Area 4: Clarenza Floodplain

Five scenarios are constructed for the Clarenza Floodplain. Table 4.4 lists the scenarios. The aim is to increase the storage capacity of the Clarenza Floodplain to make 'Room for the River' in this narrow river bend section, which extends from the Grafton Bridge to Elizabeth Island.

Table 4.4: Scenarios for Clarenza Floodplain, which might positively influence floods in the urban areas.

Area 4: Clarenza Floodplain		
Scenario 4.1	Remove Control Levee	Figure F.9
Scenario 4.2	Scenario 4.1 + widen river at the narrowing section	Figure F.10
Scenario 4.3	Scenario 4.1 + move Swan Creek Levee and Alipou Levee to Pacific Highway	Figure F.11
Scenario 4.4	Scenario 4.1 + construct secondary channel	Figure F.12
Scenario 4.5	Scenario 4.1 + remove Alipou Levee and Swan Creek Levee	Figure F.13

Scenario 4.1 considers the removal of the Clarenza Control Levee. This levee is diminishing free inflow of water into the Clarenza Floodplain. Scenario 4.2 includes the adjustment of scenario 4.1 and widens the river to the same width between the embankments of the Grafton bridge over the complete narrow section. Scenario 4.3 includes the adjustment of scenario 4.1 and diverts the Swan Creek Levee (6.0 $mAHD$) towards the Pacific Highway over the complete narrow section, creating an 'easier-to-access' floodplain for the river in this area. Scenario 4.4 includes the adjustment of scenario 4.1 and only slightly differs from scenario 4.3. Instead of removing the Swan Creek Levee, only small upstream and downstream sections are removed, simulating the construction of a secondary channel. The same Pacific Highway Levee (6.0 $mAHD$) as in scenario 4.3 is implemented as well. Scenario 4.5 includes the adjustment of scenario 4.1 and removes the entire Alipou Levee and the Swan Creek Levee. The Clarenza Floodplain and the Swan Creek Floodplain are not protected by any levee in this scenario.

4.5. Area 5: Hill Ridge

Three scenarios are created for area 5, see Table 4.5. The aim for this area is to get rid of the possible bottlenecks, in the form of narrow river passages.

Table 4.5: Scenarios for the Hill ridge, which might positively influence floods in the urban areas.

Area 5: Hill ridge		
Scenario 5.1	Remove part of northern ridge North of the river	Figure F.14
Scenario 5.2	Remove part of southern ridge North of the river	Figure F.15
Scenario 5.3	Combination of both Scenario 5.1 & 5.2	Figure F.16

Scenario 5.1 simulates the situation where a part of the northern Hill Ridge is lowered to the height of the surrounding floodplain. Scenario 5.2 simulates the situation where a part of the southern Hill Ridge is lowered to the height of the surrounding floodplain. Scenario 5.3 is a combination of both scenarios 5.1 and 5.2.

4.6. Area 6: Maclean

For the Maclean area, no scenarios are constructed as explained in the area analysis of Chapter 2. The possible positive effects of upstream mitigation measures on this particular area are shown in Chapter 5.

4.7. Discussion

New flood mitigation measures have been composed and explained in this chapter.

Due to the limited project duration and long computational modelling time, a limited number of scenarios have been run and not every possibility per area have been taken in to account. Other scenarios might come up with different mitigation possibilities, which have a bigger impact on the flood mitigation in the Clarence Valley. Also only two combinations of simulations have been run, all other scenarios are run independently.

In the past, BMT WBM did some research on possible mitigation measures in the Clarence Valley (Farr and Huxley, 2014). It might be possible that some applied mitigation measures have been examined by BMT WBM before.

5
Results

This chapter contains results of the hydraulic assessment on the constructed scenarios (see Chapter 4), as well as results of the structural assessment on the flood defences. The results of the hydraulic assessment are presented in the following sections:

- Section 5.1, presentation of the most relevant scenarios regarding flood mitigation in the urban areas of the Clarence Valley;
- Section 5.2, a multi-criteria analysis on the simulated scenarios;
- Section 5.3, presentation of two interesting combinations consisting of promising scenarios (based on the multi-criteria analysis).

The results of the scenarios and combinations in Section 5.1 and 5.3 will be presented in the following format:

- Visualisation of the peak flood difference in water depth. The peak flood difference visualisations are all representing the peak water levels of the simulated scenario minus the peak water levels of the reference simulation;
- Additional information resulting from the applied measurements: water level and discharge differences; flow velocity and time of first inundation.

The results of the structural assessment are presented in Section 5.4. Here, first the results of the Swan Creek Floodgate are presented. Finally, the RC levee walls of Maclean are elaborated on.

After this chapter, the third subquestion can be answered:

> *To what extent can the floodplains in the appointed potential locations influence the impact of flooding in urban areas in the Clarence Valley?*

5.1. Scenario results

In total, sixteen scenarios (or 48 simulations as every scenario includes the 5, 20 and 50 year ARI flood events) are analysed. The most relevant scenario simulation results for flood mitigation in urban areas are presented in the following subsections. These scenarios are situated in Baker's Swamp, the Southampton Floodplain and the Clarenza Floodplain (area 1,2 and 4). No relevant scenarios were found for South Grafton and the Hill Ridge (area 3 and 5). Moreover, no scenarios were simulated for Maclean (area 6). All other scenario simulation results, which are not presented in the following sections, are presented in Appendix G.

5.1.1. Scenario 1.3 - 50 year ARI flood event

Scenario 1.3 consists of the lowering of the Westlawn Levee and construction of a levee north of Grafton.

Peak flood difference

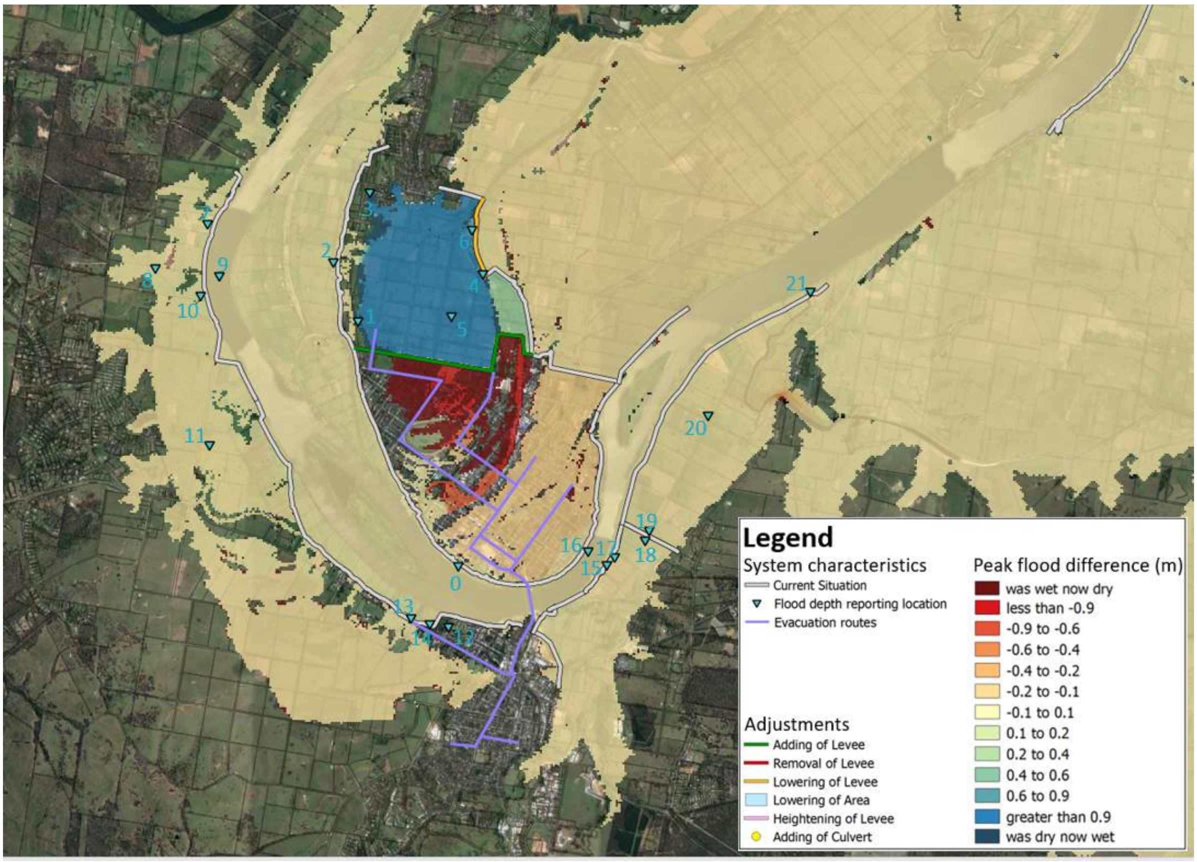


Figure 5.1: Peak Flood Difference for scenario 1.3 for the 50 year ARI flood event.

In Figure 5.1, the peak flood difference for this scenario is shown. As can be seen, Baker's Swamp gets inundated to greater extend during a 50 year ARI flood event. The water level increases by 1.2 m compared to the reference simulation, according to Table G.2. The northern part of Grafton shows a dark red colour which indicates that the certain area was wet in the reference situation and with the measures taken it will stay dry. The orange colour in the southern part of Grafton indicates that the inundation depth will reduce with roughly 0.3 m. It is shown that the storage capacity of Baker's Swamp can be used efficiently in a 50 year ARI flood event, still keeping the north of Grafton dry. Note, in this scenario, it would only be possible to evacuate via South Grafton. Junction Hill is no longer accessible.

Additional information

From the water level differences, as outlined in Appendix G.4.1, it can be seen that the applied adjustments only affect water levels in this particular area. Baker's Swamp is used more efficiently as the water level rises during a 50 year ARI flood event, while the north of Grafton stays dry. This is supported by the discharge output (see Appendix G.4.1), the lowering of the Westlawn Levee allows significant more water to flow into Baker's Swamp where the North Grafton Levee protects Grafton from water flowing in from the north. This scenario also has a positive effect on the time of first inundation as explained in Appendix G.6. 150 Hours of the flood wave is simulated starting from the Clarence River inlet boundary at x = 0 and t = 0. Where the inundation of the north of Grafton starts at 55 hours from the beginning of the simulation in the reference scenario, north Grafton is not inundated at all in scenario 1.3. Furthermore, South Grafton is being inundated six hours later than before (at 102 hours from the beginning of the simulation instead of 96). This scenario has almost no effect on changes in flow velocity.

5.1.2. Scenario 2.3 - 20 year ARI flood event

The 20 year ARI flood event results of scenario 2.3 are presented here. In this scenario, the Waterview Levee is lowered to about 7 $mAHD$ over a length of 1.5 km together with construction of a levee in front of South Grafton of around 9.5 $mAHD$.

Peak flood difference

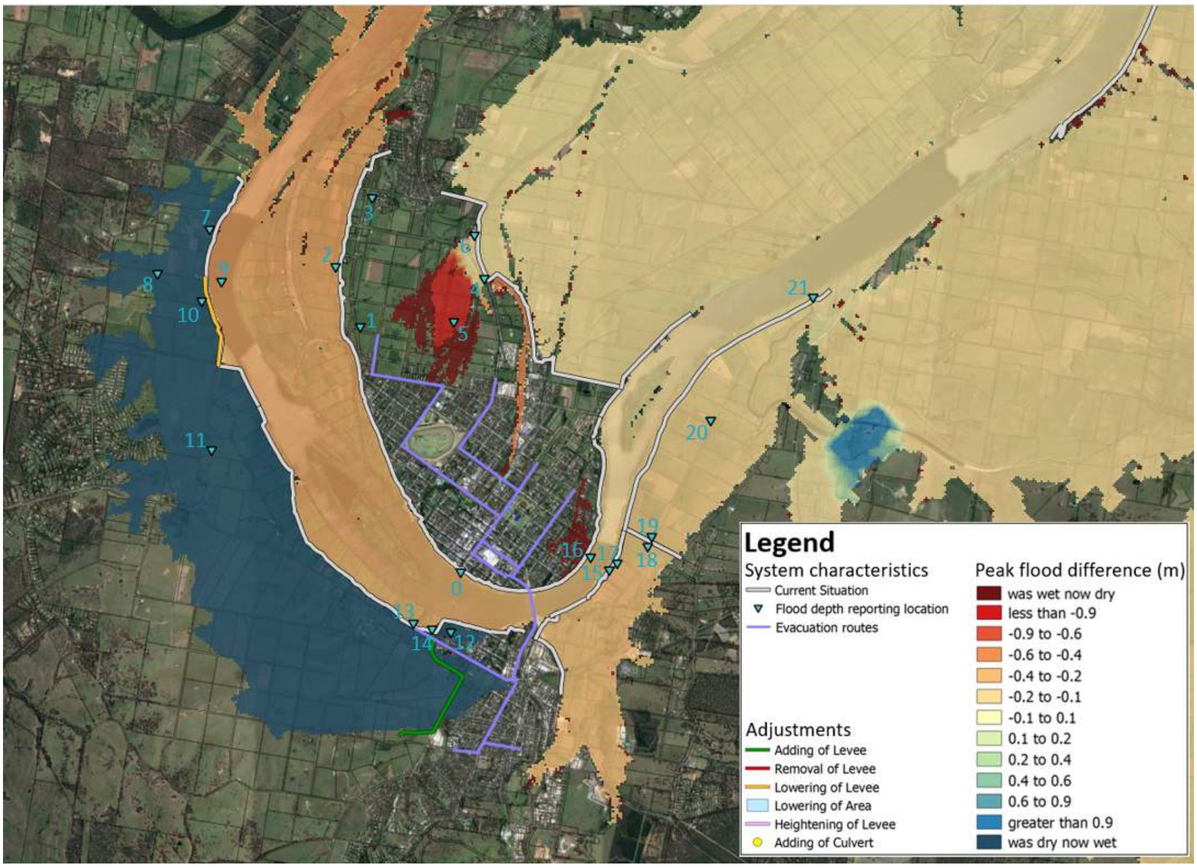


Figure 5.2: Peak Flood Difference for Scenario 2.3 for the 20 year ARI flood event.

Figure 5.2 shows the peak flood difference for scenario 2.3 during a 20 year ARI flood event. The whole of the Southampton Floodplain is inundated, where it was dry before adjustments were made. As a result, Grafton is not inundated at all during a 20 year ARI flood event. Moreover, Baker's Swamp is inundated to a lesser extend than in the reference situation. However, South Grafton gets inundated partly, despite of the implemented levee of 9.5 $mAHD$. This scenario shows that the storage capacity of the Southampton Floodplain can be used as an efficient measure to mitigate flooding in Grafton. Note, this scenario is only relevant if South Grafton remains dry, which means an optimisation of the measures (fixing of modelling errors) in the Southampton Floodplain is needed.

Additional information

Again, the water level and discharge differences in Appendix G.4.2 support the positive effect of this mitigation measure in the 20 year ARI flood event. This scenario affects the water levels in areas 1, 2 and 3. The Southampton Floodplain clearly receives a lot more water (approximately 1,600 m^3/s) as the inundation depth rises to 6 m, due to the lowering of the Waterview Levee. Note, as outlined in Appendix G.5, the increase of flow velocities around the lowered levee is significant. This might lead to problems concerning erosion and eventually levee failure and more research needs to be performed on this particular topic. Also, a decrease in water level of around 20 cm at the Prince Street Gauge is found, which leads to zero discharge at the Lower Grafton Levee. This results in no inundation of Grafton, where it was inundated 48 hours from the beginning of the simulation in the reference situation (see Appendix G.6). However, South Grafton is inundated 55 hours from the start of the simulation, where it was not inundated before. This is probably a modelling input error and needs to be solved to fully utilise the positive effects of this scenario.

5.1.3. Scenario 2.3 - 50 year ARI flood event

Besides the 20 year ARI flood event results of scenario 2.3 presented in the previous subsection, the 50 year ARI flood event results of the same scenario are also relevant to display.

Peak flood difference

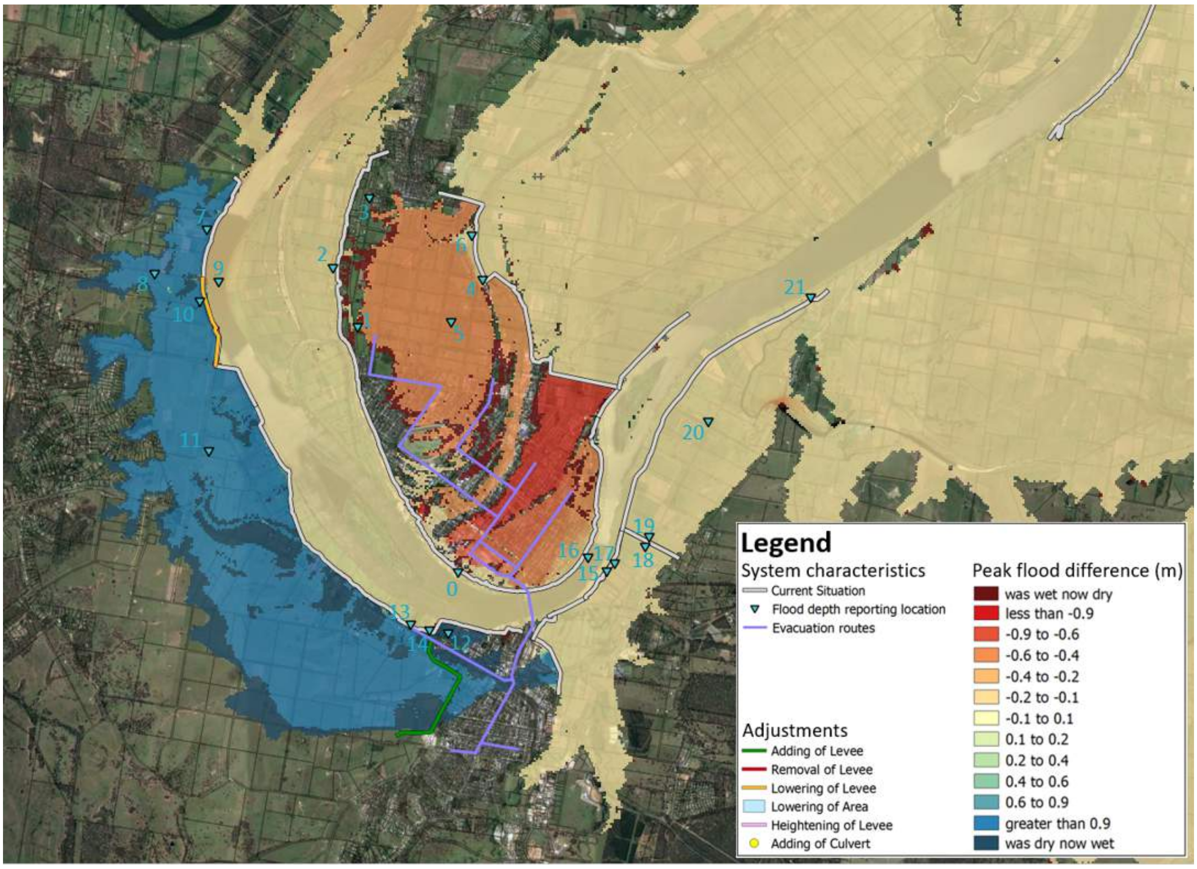


Figure 5.3: Peak Flood Difference for Scenario 2.3 for the 50 year ARI flood event.

In Figure 5.3, the peak flood difference for scenario 2.3 during a 50 year ARI flood event is shown. The Southampton Floodplain gets inundated to a greater extend than in the reference simulation. This results in significant lower water levels in Grafton, while it is still inundated from both the north (Baker's Swamp) and the south (Grafton Levee). However, just as in the 20 year ARI flood event, South Grafton get inundated as well. The added 9.5 $mAHD$ levee and the Heber Street Levee are overtopped. As a result, the use of the storage capacity of the Southampton Floodplain has less effect for the 50 year ARI flood event than it has for the 20 year ARI flood event. But, a positive side effect of this adjustment is the decrease of inundation depths downstream. For example, Maclean inundates to lesser extend as shown in Figure 5.4. Note again, also for the 50 year ARI event an optimisation of the measures in the Southampton Floodplain is needed as South Grafton inundates.

Additional information

For the 50 year ARI flood event, scenario 2.3 only has effect on water levels of areas 1 and 2 as can be seen in Appendix G.4.2. During this kind of flood event, water levels in the Southampton Floodplain increase even more. Moreover, the discharge through the lowered part of the Waterview Levee increases, leading to even greater velocities as seen in Appendix G.5. Grafton and Maclean get inundated 4 hours later than they used to inundate due to the measures of this scenario, which is a positive feature (see Appendix G.6). Nevertheless, where South Grafton was inundated at 102 hours from the start of the simulation, it is inundated at 50 hours after the start now. This is due to the fact that the levee of 9.5 $mAHD$ is not sufficient. Further optimisation should be performed.

Downstream influence on Maclean

Maclean also benefits from the adjustments made in scenario 2.3. For the 50 year ARI flood event, the effects of scenario 2.3 on Maclean are presented below. The inundation depths in Maclean are reduced.

Peak flood difference

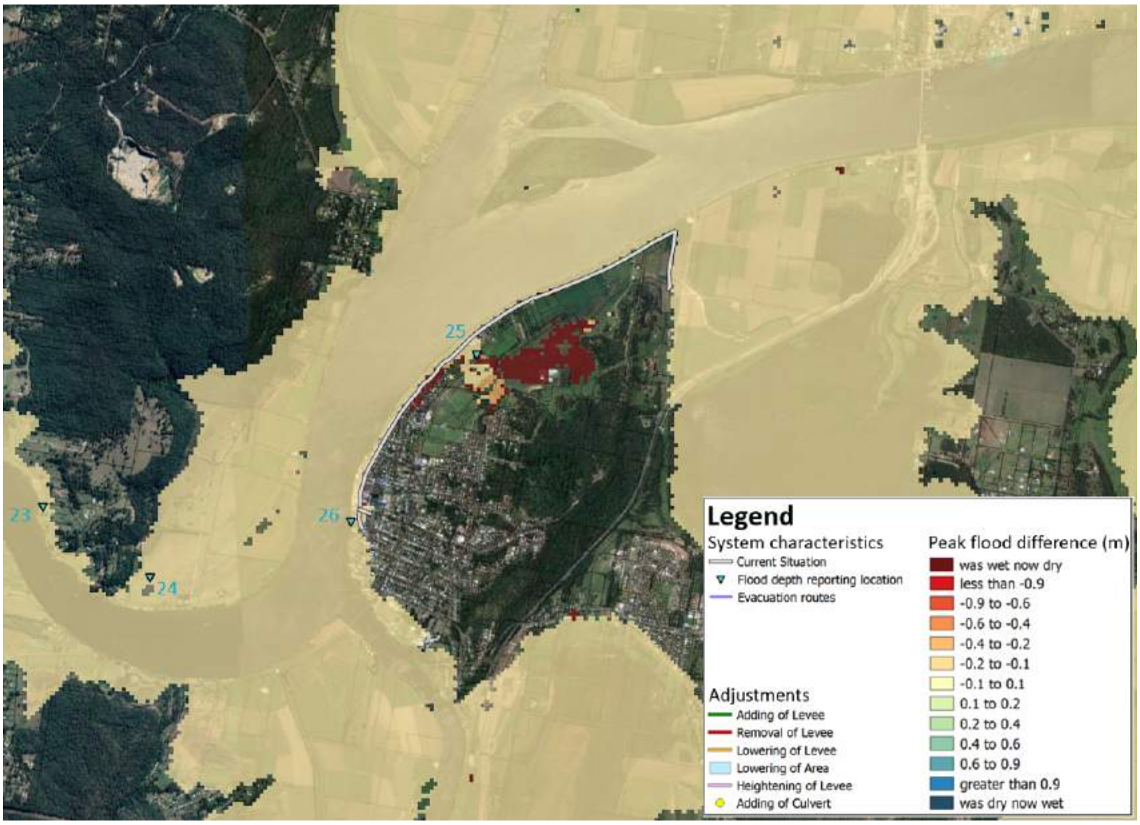


Figure 5.4: Peak Flood Difference for Scenario 2.3 for the Maclean area for the 50 year ARI flood event.

The above presented figure shows less to no inundation of some parts of Maclean during a 50 year ARI flood event as a result of the use of the storage capacity of the Southampton Floodplain upstream. This adjustment results in Maclean being almost fully protected against floods up to 50 year ARI flood events. In order to keep the Maclean Levee completely from overtopping during a 50 year ARI flood event, further research is needed.

Additional information

As there were no scenarios created for the area of Maclean, there is no change in flow velocities. Changes in discharge, water levels and first time of inundation are explained at scenarios in upstream areas.

5.1.4. Scenario 4.1 - 5 year ARI flood event

For scenario 4.1, only the Control Levee in the Clarenza Floodplain was removed.

Peak flood difference

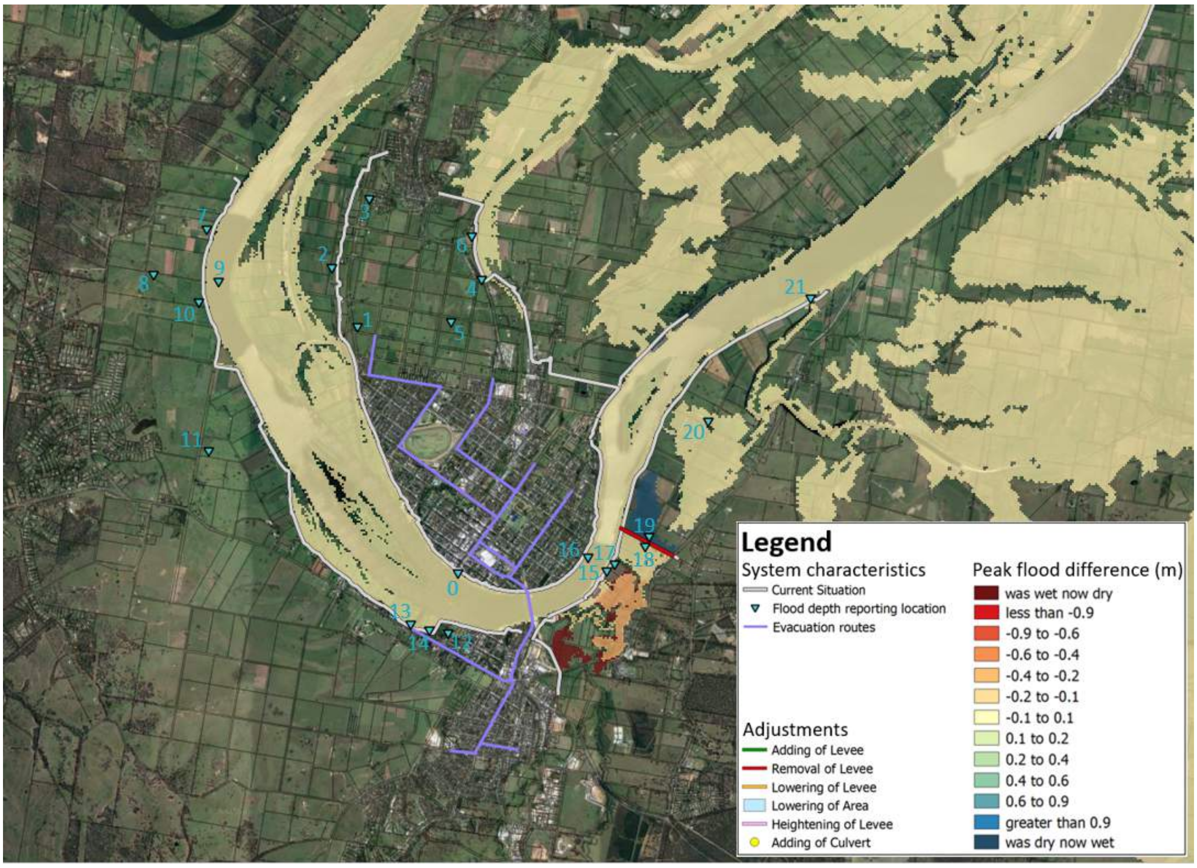


Figure 5.5: Peak Flood Difference for Scenario 4.1 for the 5 year ARI flood event.

Figure 5.5 shows the peak flood difference for scenario 4.1 during a 5 year ARI flood event. As can be seen, the water overtopping the Alipou Basin and Swan Creek Levee can flow freely north-eastwards. This results in a dry Pacific Highway, where it was inundated in the reference situation. This adjustment has no influence on any peak water level reduction in urban areas.

Additional information

This scenario only has a minor effect on water level and discharge differences as outlined in Appendix G.4.3. Moreover, it has no effect on times of inundation nor significant changes in flow velocities.

5.1.5. Scenario 4.5 - 20 year ARI flood event

The 20 year ARI flood event results of scenario 4.5 are presented here. In this scenario, the Control Levee, the Alipou Basin Levee and the Swan Creek Levee were removed.

Peak flood difference

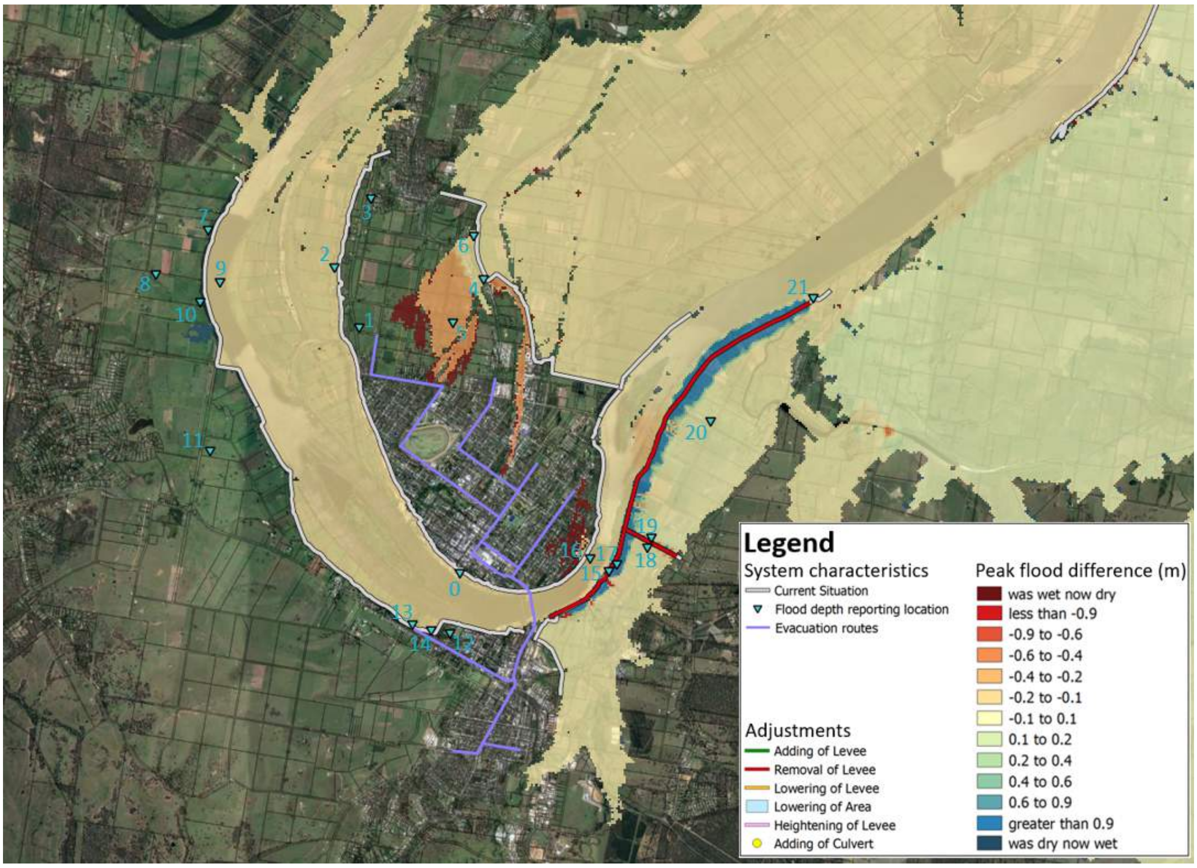


Figure 5.6: Peak Flood Difference for Scenario 4.5 for the 20 year ARI flood event.

In Figure G.22, the peak flood difference for scenario 4.5 during a 20 year ARI flood event is shown. By removal of the the total levee system downstream of South Grafton, the storage capacity of the Clarenza Floodplain can be used to greater extend. This results in almost no overtopping of the Grafton Levee, almost keeping Grafton dry. Moreover, Baker's Swamp is significantly less inundated than in the reference situation. The increase in storage capacity of the Clarenza Floodplain has a positive effect on the inundation depths of the urban areas.

Additional information

For the 20 year ARI flood event, this scenario shows little influence on water levels as seen in Appendix G.4.4. However, as seen in the discharge table on this scenario in Appendix G.4.4, a lot of water flows more freely over the Clarenza Floodplain. This lowers the water levels equally, leaving no significant visible differences at depth reporting locations. Furthermore, it is clearly visible that less water is discharged over the Lower Grafton Levee, leading to almost no inundation of Grafton. As visualised in Appendix G.5, the flow velocity increase over the Clarenza Floodplain in this scenario is significant and needs further consideration in any future research. This scenario results in an inundation delay time of two hours at Grafton compared to the reference situation, as suggested in Appendix G.6.

5.1.6. Scenario 4.5 - 50 year ARI flood event

Besides the 20 year ARI flood event results of scenario 4.5 presented in the previous subsection, the 50 year ARI flood event results of the same scenario are also relevant to display.

Peak flood difference

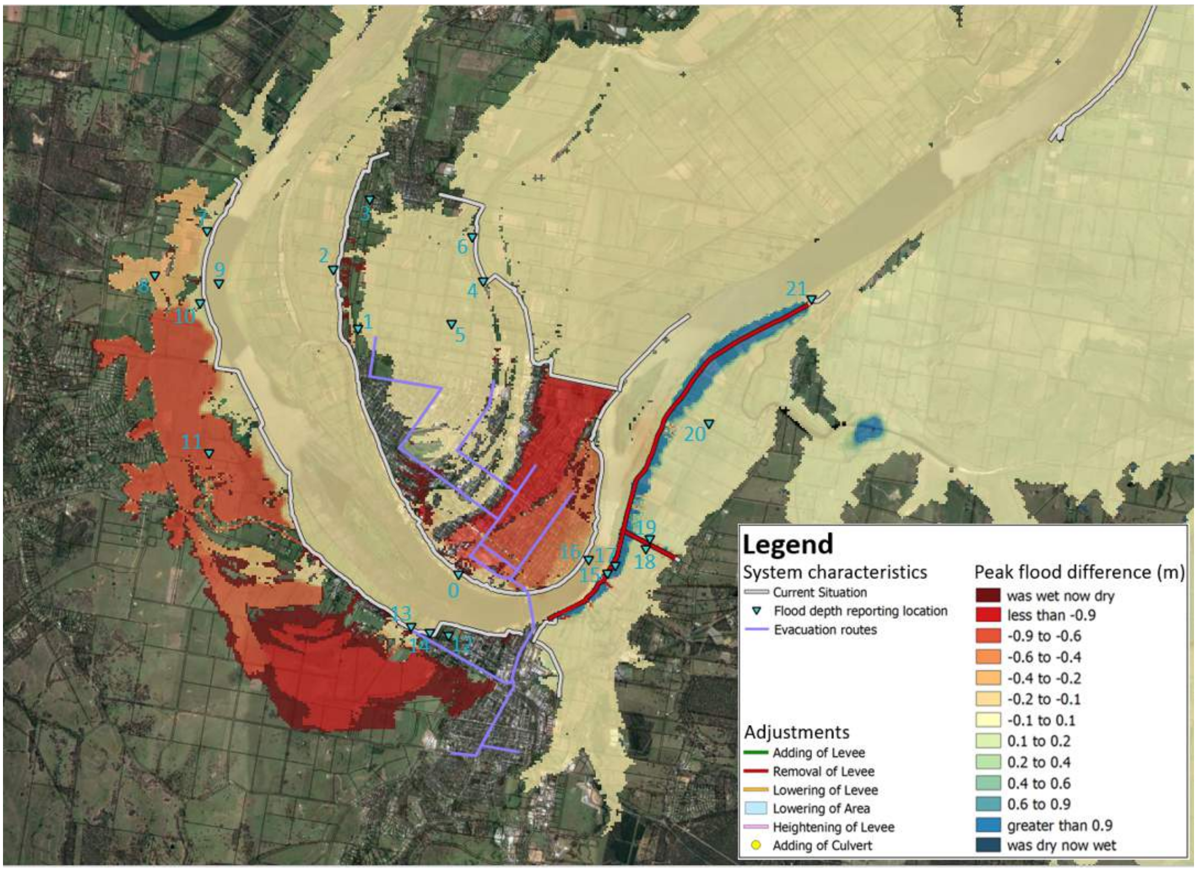


Figure 5.7: Peak Flood Difference for Scenario 4.5 for the 50 year ARI flood event.

Figure 5.7 shows the peak flood difference for scenario 4.5 during a 50 year ARI flood event. It can be seen that the adjustments for this scenario result in lower inundation levels in both Grafton and the Southampton Floodplain. As a result of the lower inundation of the Southampton Floodplain, South Grafton remains dry in this scenario. The adjustment has no effect on the inundation depth of Baker's Swamp. However, the lower inundation depth of the Southampton Floodplain and Grafton has a negative effect on the inundation depths downstream of Grafton. For example, Maclean inundates to greater extend, see Figure G.43. As a result, the increase in storage capacity of the Clarenza Floodplain in the 50 year ARI flood event has a debatable outcome in terms of effectiveness.

Additional information

For the 50 year ARI flood event, again little influence on water levels is observed. From the discharge table presented in Appendix G.4.4, it can be seen that more water is overtopping the Maclean levee. Flow velocities on the Clarenza Floodplain increased even more as visualised in Appendix G.5. Appendix G.6 shows that the inundation of Grafton is delayed by one hour in this scenario. South Grafton stays dry where it was inundated first and Maclean is inundated three hours earlier than before.

5.2. Multi-Criteria Analysis

This section contains a multi-criteria analysis to determine which combination of scenarios is useful to investigate. The water level difference is not the only criterion which determines the usefulness of mitigation measures. The following objectives are set to assess the usability of every scenario, which is expressed in scores:

- Scale of the implemented mitigation measure (size of the scenario adjustment e.g. length of removing or lowering a levee and number of adjustments. This indirectly takes the costs of measurements into account as well as the effect on stakeholders and infrastructure);
- Overall effect on water level changes (difference in peak flood due to the implemented scenario measures);
- Effect on urban areas locally (effect of the implemented adjustments on the stakeholders and infrastructure locally);
- Effect on urban areas downstream (as a result of local measures to the system, effect on the stakeholders and infrastructure).

In Table 5.1, the combined scores (summing up the scores on the 5, 20 and 50 year ARI flood events) of the full multi-criteria analysis presented in Appendix G.2 is shown. The four objectives presented above are scaled with a '+' or '-', which is outlined in Appendix G.2 as well.

Table 5.1: Multi-criteria analysis on the modelled scenarios constructed for area 1 to 5. The green rows are taken into consideration, the orange rows have been rejected in the remainder of this study.

	Scale of the implemented mitigation measure	Overall effect on water level changes	Effect on urban areas locally	Effect on urban areas downstream	Total score
Scenario 1.1	++++++	++	- - - -	0	++++
Scenario 1.2	+++	0	+	0	++++
Scenario 1.3	0	++	+++	0	+++++
Scenario 2.1	++++++	++++	- - - -	++	++++++
Scenario 2.2	0	++++	- - - -	++	++
Scenario 2.3	+++	++++	++++	++	+++++++++++++
Scenario 2.4	0	++++	++++	++	++++++++++
Scenario 3.1	+++	0	0	0	+++
Scenario 4.1	++++++	+	0	0	+++++++
Scenario 4.2	0	++	- - - -	0	- -
Scenario 4.3	0	+	-	0	0
Scenario 4.4	0	+	- -	0	-
Scenario 4.5	0	+++++	++++	-	++++++++
Scenario 5.1	0	0	0	0	0
Scenario 5.2	0	0	0	0	0
Scenario 5.3	0	0	0	0	0

The multi-criteria analysis shown in Table 5.1 results in six scenarios with a total score higher than four '+'-signs (marked green and orange). However, only four scenarios (marked green) will be considered when combinations are formed. Both scenario 2.1 and 2.4 are rejected (makred orange), despite their score of six and ten '+'-signs respectively. The reasons for this decision are presented below (following the four presented objectives):

Scenario 2.1:

- The scale of adjustments to the system is limited, only a part of the Waterview levee is lowered;
- Overall, due to this measurement, a lot of water levels change;
- During a 50 year ARI flood event, Maclean (downstream) remains almost dry;

- However, as South Grafton is not protected, it will inundate almost completely. Scenario 2.3 has the same positive features as mentioned above, plus a constructed levee in front of South Grafton (which needs to prevent South Grafton from inundation). Therefore, it is decided not to take scenario 2.1 into account, but rather focus on scenario 2.3.

Scenario 2.4:

- The scale of adjustments to the system are bigger. South Grafton is protected by a levee (which needs to prevent South Grafton from inundation);
- Overall, due to this measurement, a lot of water levels change;
- During a 50 year ARI flood event, Maclean (downstream) remains almost dry;
- However, the Gwydir Highway is embanked and culverts are added to the system. These extra measures, accompanied with extra costs, have no contribution to water regulation as too much water flows into the Southampton Floodplain. Therefore, it is decided not to take scenario 2.4 into account, but rather focus on scenario 2.3.

It has to be noted that the selected four scenarios are only products of a first feasibility assessment. No optimisation on the applied measures is done, they are solely selected on their potential to solve the problems for urban areas in the Clarence Valley. For instance, the measures taken in scenario 2.3 are not sufficient to protect South Grafton from being inundated (modelling input error). However, this scenario is fully scored on the 'effects on urban areas locally' in Table 5.1, provided that this problem is solved. As the evacuation of Grafton and South Grafton uses routes through South Grafton, it is of primary importance that this area remains dry to prevent the area from isolation (see (Farr, 2014)).

5.3. Combinations

Based on the multi-criteria analysis, two combinations have been constructed. In an attempt to utilise the full potential of the proposed flood mitigation measures, these two combinations use the best scoring scenarios. The best scoring scenarios, 1.3, 2.3, 4.1 and 4.5, are applicable to three areas (Baker's Swamp, the Southampton Floodplain and the Clarenza Floodplain). It is decided to construct combinations with three scenarios from different areas in order to see their maximal effect on the system. The first combination consists of the following scenarios:

- scenario 1.3, lowering of Westlawn Levee & construction of levee north of Grafton;
- scenario 2.3, lowering of Waterview Levee & construction of levee near South Grafton;
- scenario 4.1, removal of Clarenza Floodplain Control Levee.

The second combination consists of the following scenarios:

- scenario 1.3, lowering of Westlawn Levee & construction of levee north of Grafton;
- scenario 2.3, lowering of Waterview Levee & construction of levee near South Grafton;
- scenario 4.5, removal of all levees downstream of South Grafton.

For the 5 year ARI flood event, no urban areas are inundated. For the 20 year ARI flood event, scenario 2.3 already prevents urban inundation, if the inundation of South Grafton is solved. As for this reason, it is decided to only show the combination results of the 50 year ARI flood events. The combination results of the 5 and 20 year ARI flood events can be found in Appendix G.3.

5.3.1. Combination 1

This section provides the results of the 50 year ARI flood event of combination 1, where scenario 1.3, 2.3 and 4.1 are merged.

Peak flood difference

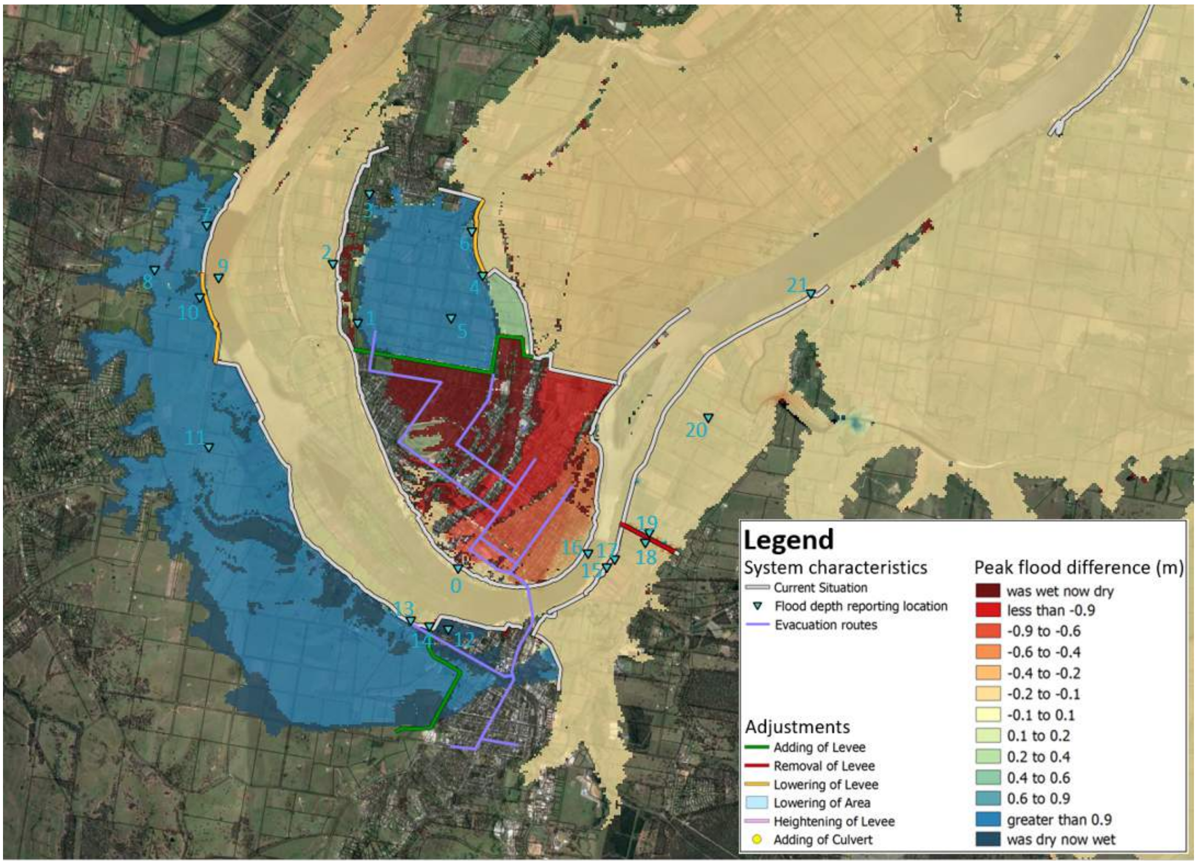


Figure 5.8: Peak Flood Difference for combination 1 for the 50 year ARI flood event

In a 50 year ARI flood event, the first combination shows inundation of the Baker's Swamp and Southampton Floodplain. Together with this, one can see that the inundation depth drops significantly in Grafton, the northern part even stays dry. Note, the South Grafton Levee is not sufficient at the moment and needs to be adjusted properly. This combination has a positive effect on downstream areas like Maclean. The inundation area of Maclean decreases as an effect of scenario 2.3, which is included in this combination.

Additional information

By combining scenarios, a lot of areas are influenced on their water levels. Non-urban areas receive more water, while urban areas receive less as can be seen in Appendix G.4.5. It is also seen that the effect of the applied mitigation measures becomes less when the flood event becomes bigger, as less water level differences are being effected. Contrarily, more discharge differences become obvious as more levees are overtopped during bigger events. This combination has the same effect on inundation delay times as scenario 2.3 for both the 20 and 50 year ARI events as seen in Appendix G.6.

5.3.2. Combination 2

This section provides the results of the 50 year ARI flood event of combination 2, where scenario 1.3, 2.3 and 4.5 are merged.

Peak flood difference

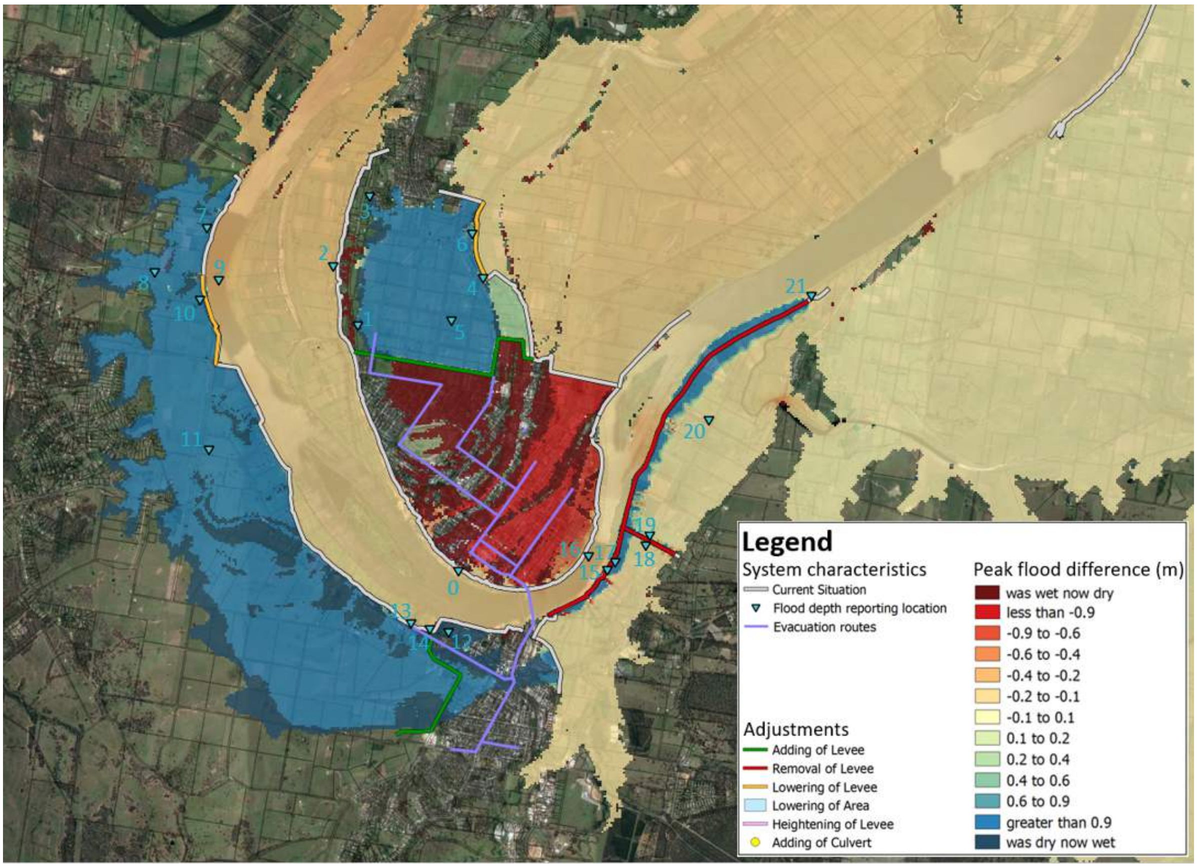


Figure 5.9: Peak Flood Difference for combination 2 for the 50 year ARI flood event

The second combination shows approximately the same behaviour as the first combination. For this combination, the situation in Maclean does not improve. Scenario 2.3 results in a decrease of flooding in Maclean, where scenario 4.5 diminishes this effect.

Additional information 50 year ARI

The same general notions for combination 1, on water level and discharge differences, can be made for combination 2. These differences can be found in Appendix G.4.5. From Appendix G.6, it can be seen that the time before inundation of Grafton is delayed by another six hours compared to the reference situation. South Grafton still inundates, where it did not inundate in the reference situation. The time of first inundation of Maclean increased with two hours compared to the reference situation.

5.3.3. Combination differences

Here, only combination differences on the 50 year ARI flood events are considered. The main difference between combination 1 and 2 is that a larger area of Grafton stays dry for combination 2, when looking at the peak flood difference. Moreover, the inundation depths of the southern part of Grafton show an even bigger decrease for combination 2 compared to combination 1. This is accompanied by a larger interventions resulting from scenario 4.5 in combination 2 compared to combination 1. However, the positive effects of combination 1 on the downstream areas are diminished for combination 2. As a result, combination 2 has a less negative effect on the inundation depths of South Grafton when compared to combination 1, as more water flows to the downstream parts. This negative effect can be solved for both combinations if the modelling input error for scenario 2.3 is solved.

The time of inundation of Grafton is delayed by two hours in favour of combination 2 when comparing both combinations. Moreover, South Grafton is inundated one hour later for combination 2 than combination 1. However, Maclean is inundated 2 hours earlier in combination 2 than it inundates in combination 1.

5.4. Structural results

As outlined in Chapter 2, the Clarenza Floodplain and the Maclean area contain flood defences which might have an impact on the flood safety of that area. Below, the performance of both the Swan Creek Floodgate in the Clarenza Floodplain and the Maclean RC Levee Walls are presented.

5.4.1. Swan Creek Floodgate

The Swan Creek Floodgate regulates the water flow between the Clarence River and the Clarenza Floodplain. The Pacific Highway runs through the floodplain and connects urban areas. The two functions of the floodgate are irrigation and drainage of the floodplain. The condition and the operation of the floodgate has direct influence on those two functions. The floodgate must work sufficiently in order to drain the floodplain after a flood. In order to keep the downtime of the Pacific Highway as low as possible and minimising the negative economic consequences for Grafton. In this subsection, the results of the condition and the operation of the floodgate are presented.

The problem that is examined is tilting of the outlet walls and the outlet slab at the river side of the floodgate, as is stated in Chapter 2. Based on expertise of the local flood experts, the cause of the displacements of the outlet walls is erosion. The cause, location and magnitude of the erosion is unknown and not investigated yet.

Piping checks have been performed according to the criteria of Bligh and Lane, see Appendix C. The criteria do not coincide with the calculated seepage length and the seepage length of the technical drawings. Erosion of the rock armour and the river bed in front of the rock armour could also be the cause of the occurring problem. However, too many assumptions have to be made to execute the checks for the rock armour. For this reason, no calculations have been performed for this part.

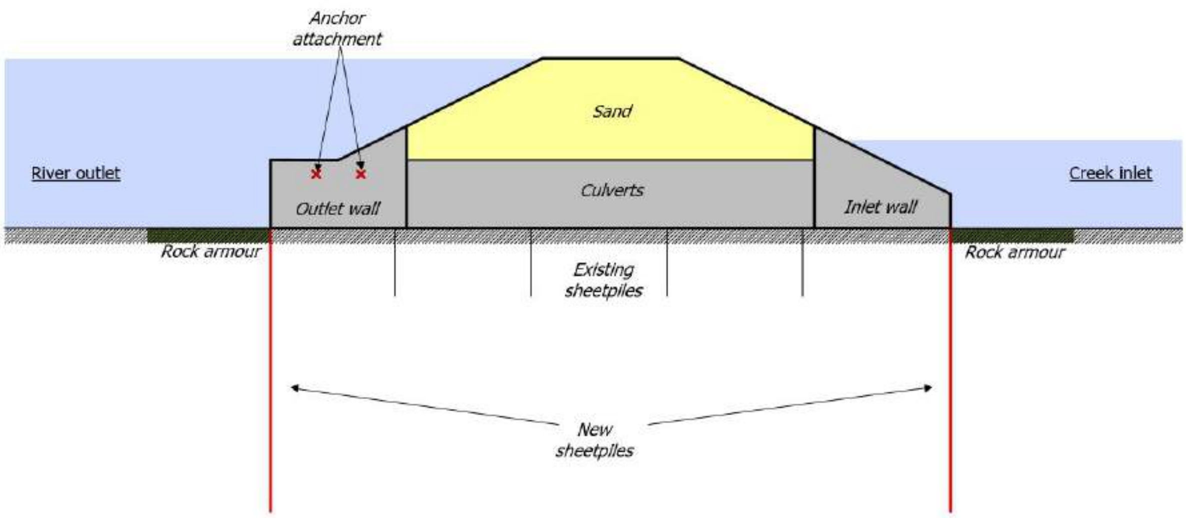


Figure 5.10: Cross section of the Swan Creek Floodgate, including the possible solutions in red.

Two possible solutions have been mentioned, to search for solutions of the problem. These solutions are indicated in red in Figure 5.10. First of all, the seepage length could be increased by adding sheet piles. Secondly, further displacement of the outlet walls could be prevented by applying grout anchors.

The solutions of applying anchors is worked out for the design state. The anchors have been determined according to (Bouw&Infra, 2014). The grout anchors are the most favourable type of anchors, considered the structural aspects. In order to withstand further deflections of the outlet walls, two anchors in each wall are required. One at $1/3^{rd}$ of the length of the wall and the other one at $2/3^{rd}$ of the length of the wall. The anchor near the floodgate requires a minimum length of 13 m and the other anchor needs a minimum length of 9 m.

A recalculation is performed to check the strength and stability of the Swan Creek Floodgate, a complete elaboration can be found in Appendix C. The geometry of the existing floodgate has been used to model in scientific applications (SCIA) Engineer, a software for Finite Element Method (FEM) calculations. The strength and stability of the floodgate meet the requirements, all unity checks suffice, except for piping. A new floodgate design is made in order to prevent tilting of the inlet and outlet walls. In this new design, the inlet and outlet walls are connected to the main structure. Section C.6.4 gives a verification of the new design. The new design results in relatively high bending moments in the top connection of the walls and the floodgate itself, but it does prevent severe displacements of the outlet slab and outlet walls.

5.4.2. Reinforced Concrete Levee Walls Maclean

For a large part of the RC levee walls the criteria for Bligh and Lane do not fulfil. Also the maximum (allowed) hydraulic gradient, is not sufficient, as explained in Appendix D. However, the length of the sheet pile walls and the soil types are estimated for this study as they were not known during this study.

5.5. Discussion

In this chapter the results of hydraulic and structural aspects are elaborated on. The impact of the adjustments at the appointed potential locations are presented. Combinations are made based on positive results of the individual scenarios. The results of the combinations are also presented in this chapter. The results give an insight in the effect of the taken mitigation measures.

The financial aspects have not been taken into account for any proposed simulation or solution in this chapter. For example, information on execution costs, material costs and project costs is unknown. If an budget-objective would have been taken in to account for the multi-criteria analysis, possibly other scenarios would have been assessed in more detail. Due to the limit time of this study and lack of knowledge no financial assessment has been made.

The CVFM is calibrated but, 100% accuracy of the results obtained from the CVFM is not realistic. Non realistic interventions are not taken into account in this study. Some scenarios may be wrongly assessed as 'not promising', which may be due to modelling input errors in TUFLOW.

The structural and hydraulic aspects are dependent on each other. The different scenarios in this chapter form the hydraulic base. Within this base, different structural calculations have been elaborated.

The soil is assumed to be uniform for the ease of the calculations. In reality the soil consists of different soil types. This can have an influence on all the structural calculations made in this report.
For the approximations on piping of the Maclean Levee, an assumption is made for the dimension of the sheet pile walls.
Soil type, dimensions and hydrological information is missing to preform a representative scour check for the floodgate. A detailed erosion mechanism description could have been given if a scour check would have been performed.
If all missing data was available, more realistic or less conservative designs could have been made.
The model made in SCIA Engineer might lack precision in comparison to the real floodgate. The geometry might have small errors, loads are conservative and the supports are line supports instead of soil supports.

6

Conclusion

This multidisciplinary project, on behalf of the Clarence Valley Council, investigated how an increased usage of floodplains can reduce the impact of flooding on the urban areas (Grafton, South Grafton and Maclean) in the Clarence Valley. A preliminary study has been performed, which led to six promising areas (Baker's Swamp, Southampton, South Grafton, Clarenza Floodplain, the Hill Ridge on Woodford Island and Maclean) where flood mitigation measures can be implemented. The applied mitigation measures in this study are based on the Dutch strategy called 'Room for the River'. These six areas have been assessed on hydraulic and structural grounds. The hydraulic assessment has been performed by creating scenarios for the areas in the numerical software program TUFLOW. To check their consequences, three different design flood events (5, 20 and 50 year ARI flood events) have been examined. The structural assessment included an investigation of key flood defences, like Swan Creek Floodgate and the Maclean Levee Walls.

Results indicate that three areas, which are located near the city of Grafton, show promising results. Firstly, the Southampton Floodplain is an area that can store a large volume of water, which reduces the peak of the floodwave during a 20 and 50 year ARI flood event. The stored water will no longer affect downstream urban areas like Grafton and Maclean. Secondly, the storage capacity of the Clarenza Floodplain is increased, thus room for the river is created near a current bottleneck. During any flood event, the flow is no longer restricted by the river banks. Thirdly, the storage capacity of Baker's Swamp can be used more efficiently during 5 and 20 year ARI flood events. By lowering the Westlawn Levee, water can flow in and be retained in Baker's Swamp by a newly constructed levee north of Grafton. The remaining areas had minor impact and showed less promising results. Overtopping of the Maclean Levee can indirectly and partly be mitigated by implementation of upstream measures.

The Swan Creek Floodgate does satisfy horizontal, vertical and rotational stability criteria, as well as all strength tests according to European Standards. However, the Swan Creek Floodgate does not satisfy the criteria for piping. The seepage length is insufficient according to both Bligh's and Lane's formulas. The main problem is the displacement of the outlet walls. To prevent further displacements, the following suggestions are proposed: anchors to stabilise the structure; increasing the seepage length by applying extra sheet piles; improving the rock armour to prevent erosion or attachment of the outlet walls to the construction. A check on the reinforced concrete levee walls of Maclean, leads to a first impression that there might be piping problems. During high water levels the seepage length is insufficient.

One should bare in mind that there are a few discussion points considering the results. Only six very small areas, compared to a total river catchment area of 22,000 km^2, have been assessed. Other measures or areas could also contribute to the reduction of flooding in urban areas. Only basic measures were implemented, like removal, adding and lowering of levees. More advanced modelling changes like lowering of an existing floodplain and including new flood defences have not been taken in to account.

Apart from the general restrictions on the results, a limitation on the usage of the Southampton Floodplain has to be noted. The stored water on this floodplain leads to inundation of South Grafton. Because of the duration of this project, no optimisation of the mitigation measures was performed.

The exact cause of the displacements of the outer walls of Swan Creek Floodgate is unknown. Detailed research on erosion requires more data. Some of the solutions given above might turn out to be of no use. Further research on the cause of the problems is necessary. Piping under the reinforced concrete Maclean Levee Walls depends on the duration of the critical water level difference, no official records of this duration are known in this study. The length of the sheet pile walls and the soil type are estimated in this study and have a large influence on the criteria for piping.

In conclusion, the impact of flooding in urban areas of the Clarence Valley can be reduced by making use of the storage capacity of floodplains. Currently, no urban flooding occurs for the 5 year ARI flood events. The urban flooding during a 20 year ARI flood event, can be mitigated by using only the storage capacity of the Southampton Floodplain. To prevent urban areas from flooding during the 50 year ARI flood event (and higher order flood events), more extensive measures need to be taken. The combination of heightening the levees around Grafton and making use of the Southampton Floodplain, Baker's Swamp and the Clarenza Floodplain should be investigated. Around Maclean, no scenarios were modelled but some upstream measures showed a reduction in the impact of flooding of Maclean as well. For the Swan Creek Floodgate, more research into the cause for the occurring stability problems is required. In order to maintain the floodgate's function in the future, one could apply one of the proposed solutions. For the Maclean Levee Walls, piping problems are identified, which could lead to stability problems.

This report shows the possibility of using floodplains as flood mitigation strategy in the Clarence Valley. Agricultural areas can be inundated in case of high discharges. The most common strategy nowadays is increasing levee heights, which only solves the problem locally. By using the storage capacity of floodplains, one could solve flooding regionally as the storage of water influences downstream areas too. An example is the upstream measures taken near Grafton, which also reduce flooding in Maclean. However, to implement the strategy of creating more 'Room for the River', a shift in mitigation strategies is needed. This shift in mitigation strategy could be a long-term solution to reduce flood impact in urban areas in the Clarence Valley, and possibly other flooding-vulnerable areas in Australia.

7

Recommendations

To conclude this study, some recommendations are presented below. These recommendations are prioritised on their research relevance.

1. This study is limited to six areas, in which flood mitigation measures have been applied. These six areas are just a small part of the entire Clarence catchment. Possibly, more promising areas or combinations of scenarios can be found when more background knowledge is obtained on the Clarence (local experts). Therefore, it is advised to apply the research method to the entire catchment to check for more promising areas.

2. It is suggested to combine the increase in storage capacity of the floodplains around Grafton with the heightening of the levee system (for a 50 year or bigger ARI flood event). This heightening of levees in particular was already investigated by BMT WBM (see (Farr et al., 2014)). The extra storage capacity, created by using floodplains more efficiently, could result in a lower required extra height of the levees.

3. No attention has been paid to the optimisation of the applied measures, because this has only been a feasibility study. Rough estimations have been made when removing, adding or lowering levees. One should conduct further research in optimising the applied measures. For instance, the levee in front of South Grafton should be modelled correctly. Also, the modelling of a lowering of a levee might suggest controlled overtopping if flood defences are used.

4. Local rainfall in the Southampton Floodplain should be taken into consideration in future studies. In the current model, the local rainfall which is supposed to fall in the Southampton Floodplain, is added to the inflow boundary of the Clarence River. This input modification has influence on the storage capacity of the Southampton Floodplain in the current model.

5. For this study, only the 5, 20 and 50 year ARI flood events have been taken into account. To be able to perform a more in-depth flood safety study, additional (higher) ARI flood events need to be taken into account. Hereby, a better view on the system performance during a higher order event can be obtained. As a result, one can quantitatively substantiate the obtained results in more detail.

6. Also, it is necessary to involve the stakeholders on flood mitigation strategies. The effect of measures on the influenced stakeholders in particular. This only has been taken into account indirectly in the multi-criteria analysis.

7. In this study, no in-depth research has been conducted to sort the effect of 'Room for the River' on infrastructure and main evacuation routes. This is only mentioned a few times and taken into account indirectly in the multi-criteria analysis. To really substantiate the option of floodplain usage as flood mitigation strategy, one should take the influence on infrastructure in to account.

8. A cost-benefit analysis of the flood mitigation measures should be performed. The cost and benefits can be included in the multi-criteria analysis if more information is known.
9. A lack of water level and discharge data points in Grafton and South Grafton forms a limitation in the analysis of results. If more data output points are added into the Clarence Valley Flood Model (CVFM), a more intensively study of water flows and water depths can be made in the areas of inundation.
10. Further research is required to find the cause of the occurring problems at the Swan Creek Floodgate. It is recommended to collect more data on the duration of the head differences, the soil characteristics and the rock armour. With the data, more detailed research on piping could be possible and an examination of the rock armour should be enabled. Other causes are not excluded and research on the exact cause of the problems is recommended. The same data as for the Swan Creek Floodgate should be collected to do more detailed research for the possible piping problems of the Maclean Levee.
11. In order to mitigate the problems of the current situation, the following suggestion are made to retrofit to the Swan Creek Floodgate. However, the solutions depend on the exact cause of the occurring problems.
    - Application of anchors to keep the outlet walls from further displacements.
    - Increasing the seepage length by applying extra sheet piles, in order to impede piping.
    - Improving the existing rock armour in order to prevent further erosion.
12. The design of a future floodgate should be executed according to the criteria for piping. For a future design, it might be beneficial to attach the inlet and outlet walls to the main structure of the floodgate. This measure will reduce the chance of tilting of the outlet walls. Large stresses will occur near the new connections, so the new structure must be designed to carry these higher stresses properly.

Bibliography

Australian New Zealand Standard (2002), Structural design actions. part 1: Permanent, imposed and other actions (as1170.1), Technical report, Australian/New Zealand Standard.

Australian New Zealand Standard (2002), Structural design actions. part 0: General principles (as1170.0), Technical report, Australian/New Zealand Standard.

Bouw&Infra (2014), Cur 166 damwandconstructies, Technical report, Bouw en Infra.

Clarence River County Council (1970s*a*), Swan creek floodgate general arrangement. Construction Drawing number: 158/11. Project number: /135.

Clarence River County Council (1970s*b*), Swan creek floodgate reinforcement arrangement. Construction Drawing number: 158/12. Project number: /135.

Clarence River County Council (1970s*c*), Swan creek formwork and joints details. Construction Drawing number: 158/14. Project number: /135.

Clarence River County Council (1973), Maclean levee plan, Technical report, Clarence River County Council.

Clarence Valley Coucil (2018), http://www.clarence.nsw.gov.au/cp_themes/metro/home.asp.

Farr, A. (2014), Grafton, south grafton and maclean emergency management report, Technical report, BMT WBM.

Farr, A. and Huxley, C. (2013), Lower clarence floodmodel update 2013, Technical report, BMT WBM.

Farr, A. and Huxley, C. (2014), Grafton and maclean levee overtopping study - phase 2: Technical report, Technical report, BMT WBM.

Farr, A., Huxley, C. and Rodgers, B. (2014), Grafton and maclean levee overtopping study, Technical report, BMT WBM.

GoogleMaps (2018), http://maps.google.com/.

Molenaar, W. and Voorendt, M. (2018), Manual hydraulic structures, Technical report, TU Delft.

NEN EN 1991-1-1: General Actions - Densities, self-weight, imposed loads for buildings (2011), https://connect.nen.nl/Standard/Detail/38048?compId=10037&collectionId=0.

NEN EN 1992-1-1+C2:2011: ontwerp en berekeningen van betonconstructies (2011), https://connect.nen.nl/Standard/Detail/64758?compId=10037&collectionId=0.

Roads and Maritime Services (2017), Woolgoolga to ballina pacific highway upgrade, Technical report, RMS.

Rogencamp, G. (2004), Lower clarence river flood study review, Technical report, WBM.

Stubbs, B. J. (2007), 'A thematic history of the city of grafton', www.clarence.nsw.gov.au. Not found on website, can be found by Google search.

TUFLOW User Manual (2016).

Western Australia Geodesy Group (2018), http://geodesy.curtin.edu.au/research/resolution/.

List of Figures

List of Tables

A

Preliminary Study

This Appendix contains the entire preliminary study that has been executed. Here one can read the information about the stakeholders, infrastructure, area of interest and hydrological situation in full detail.

A.1. Area Analysis

The Clarence Valley has flooded multiple times in the last decades and the Clarence Valley Council has taken many measures to improve safety against flooding. During the last decades lots of levees, drains, floodgates, floodplains and culverts have been implemented in the area in order to reduce the possibility and the effect of flooding in urban areas. Floods can occur throughout the entire year and are caused by discharge rainfall or local rainfall. This section will cover the existing measures and tries to map the entire lower Clarence Valley.

Land use

The Clarence Valley has multiple types of areas like, urban area, agricultural land, forest, etc. Different land cover type are present by the Clarence Valley Council (Clarence Valley Coucil, 2018). In the Figure A.4 one can see the different types of land cover. These types of land cover are assigned to a Manning coefficient (n), which is used in the Clarence Valley Flood Model. A higher Manning coefficient will result in a higher flow-resistance.

The Clarence Valley Council provided a cadastre map of the valley to give an overview of property ownership. Possible future modifications to the area of interest, are not taken into account in this study.

Elevation data

The elevation data of the area is provided by a LiDAR surveying method (Clarence Valley Coucil, 2018), except for the river bed. The river bed is analysed with an Echo-sounder survey method (Clarence Valley Coucil, 2018). These two surveying methods are combined into one elevation map (see Figure A.1). The Clarence River originates on the east side of the Great Dividing Range. It enters the lower grounds of the Clarence Valley a few kilometers upstream of Grafton.

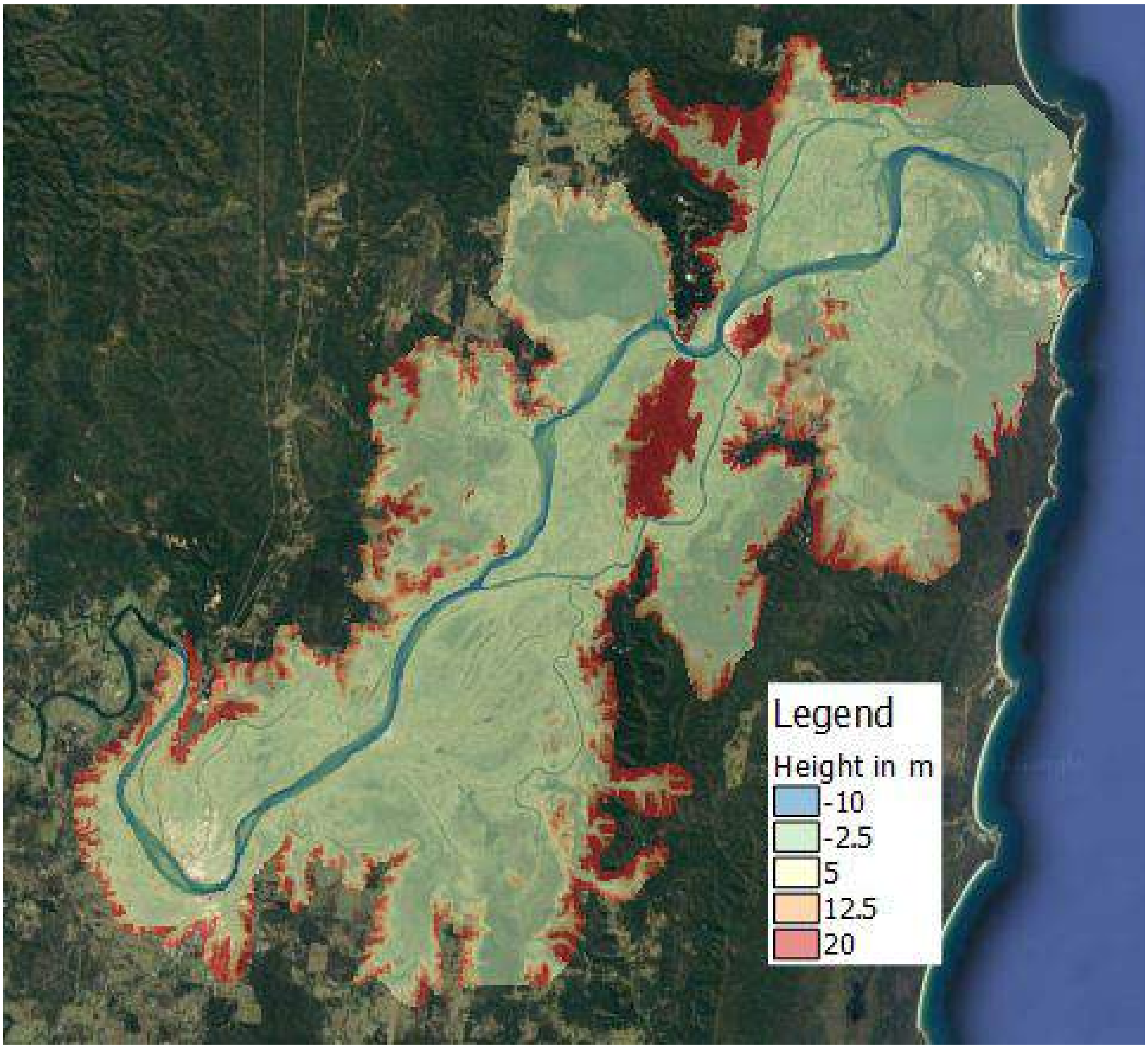


Figure A.1: Elevation of the Clarence Valley. This height data is retrieved from the current flood model of the Clarence Valley (Farr and Huxley, 2013).

In Figure A.1, the red areas indicate grounds above 20 $mAHD$. The blue line represents the river bed, which is below 0 $mAHD$. A large area of the Clarence Valley lies between 0 and 10 $mAHD$. As a result, when higher discharges occur a great area of the Valley is flooded. High grounds are rare in the lower Clarence Valley. However, a hill ridge is located 20 km landward from the coastline. Hills rise up to 200 $mAHD$, dividing the lower grounds of the valley into two parts.

Levees

The Clarence Valley contains many levees to control the water flowing downstream toward the Pacific Ocean. Each levee could be assigned to a different function as: prevention of flooding; control overtopping or prevention of riverbank erosion. Figure 2.6 illustrates the position of the levees near Grafton.

Most of the levees are placed around the river bank near Grafton and South Grafton. Most of the levees have the function of flood prevention. However, the part of the Clarenza Levee that is positioned perpendicular to the river, functions as a Control Levee that intentionally floods the Clarenza Floodplain. Furthermore, the part, the Alipou Levee, parallel to the river prevents erosion of the riverbank, causing a bottleneck in the river on the other hand. This fact diminishes the flow of the river and leads to an area of interest. Grafton has a lower elevation in comparison to the surrounding areas. Due to this fact, the water flows backwards through Alumy Creek into the urban area of Grafton during a flood. In order to prevent the city for this inflow, levees are build landward as well. Figure 2.6 illustrates the locations of the levees around Grafton. The levees further upstream around Grafton are several meters higher in comparison to the levees further downstream. The levee with a height of 9.15 $mAHD$ in between Grafton and Junction Hill prevents flooding of the floodplain Baker's Swamp. This fact diminishes the function of this floodplain and also leads to an area of interest.

Urban areas in the Valley

The most populated areas in the Clarence Valley are Grafton, South Grafton, Maclean and Yamba. Other small populated villages are Ulmarra, Brushgrove, Lawrence, Iluka,Tucabia, Bultitudes, Harwood, Chatsworth and Junction Hill. The villages Lawrence and Junction Hill are located on higher ground and did not flood in the recent years. Brushgrove is placed on the western tip of Woodford Island and is located on low elevated ground. The village is not even protected for moderate floods. The Clarence River splits at Brushgrove into a northern arm and a southern arm. All villages stated above have significantly less inhabitants and due to that, these smaller villages will be neglected in our scope of the project. The main focus will be on Grafton, South Grafton, Maclean and Yamba. An overview of the lower Clarence Valley, with the urban areas is given in Figure A.3.

Inhabitants in the Clarence Valley are familiar with floods and have adjusted their way of building houses to it. Most low elevated houses in the Clarence Valley are built on poles (see Figure B.1). The ground floors of most of the houses are basements or storage rooms, and the living room and bedrooms are higher up. This minimises the losses in case of a flood. Another method is to build a house on top of a levee. Most of the levees in urban areas are private properties, with houses built on top. In Figure B.2 a clear example of this is given. Building upon a levee is nowadays prohibited. The sailing club and the rowing club located in Grafton are examples of buildings, which are positioned at the riverside for practical reasons. These buildings will flood during moderate floods. Important buildings such as hospitals, the jail and schools are placed on higher grounds in the cities, so they are not endangered during a flood. For example, in Grafton the hospital is placed on the higher ground in the southern part of Grafton.

Flood Defences

The Clarence Valley contains many flood defences and they have been changed over the last decades. The flood defences which are commonly used are culverts, floodgates and drains. These structures provide a system for irrigation and drainage . Figure A.2 shows an example of the existing floodgates and culverts in the valley.

In order to drain the valley after local rainfall or high discharges of the river, a system of drains is presented. Existing creeks are connected by man made drains. Completely newly drains there are also constructed, for example a excavated creek or a concrete drain, which discharge into the Clarence River.

Control of flood defences is being done by a wide range of people. For example the Swan Creek Floodgate is being controlled by a member of the Swan Creek Committee. In other regions no volunteers are available and the Council will keep an eye out, for example around Alumy Creek. Sometimes the drains and floodgates are being used by farmers for irrigation and therefore they are in control of the flood defences.

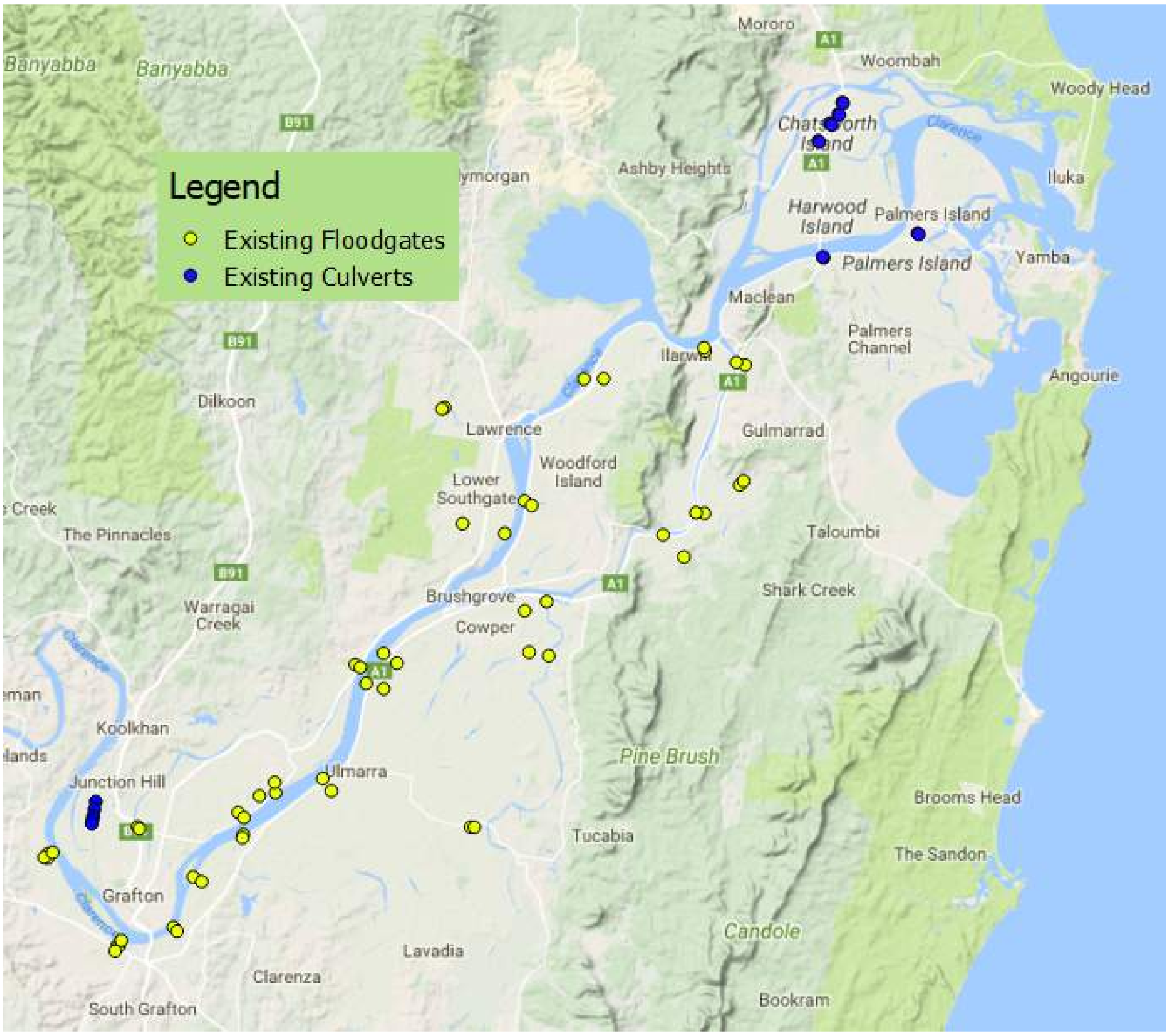


Figure A.2: An overview of the floodgates and culverts in the Clarence Valley.

Existing floodplains

Most of the floodplains are in use for agricultural purposes. The majority of agricultural land was used for cattle farming and growing sugar cane. In case of a flood, these areas can be flooded in times of high water. A big amount of the valley will flood in a major flood. Some floodplains worth mentioning are Baker's Swamp, Southampton, Carrs Peninsula, Clarenza, Lavadia and Collettes Swamp. Baker's Swamp is located between Junction Hill and Grafton. This particular area was marked to be one of the potential areas for mitigation of floods. The area floods by local rainfall and is protected by a levee with a height of 9.15 $mAHD$. Southampton is a huge floodplain at west of South Grafton. Southampton Floodplain is particular interesting area, because at the moment it will only be filled with local rainfall. The levee on the west side of South Grafton prevents the Southampton Floodplain to fill with water from the Clarence River during a flood. Just like Baker's Swamp, Southampton Floodplainis a potential area for mitigation of floods. Carrs Peninsula is at the west side of Grafton and is the first area that will flood of Grafton. The huge areas south of Ulmarra are called Clarenza and Lavadia are part of the floodplains around the Swan Creek. The Alumy creek is surrounded by the Great Marlow on the east part of Grafton. During a major flood, the range of the river from the Great Marlow to Clarenza is up to 20 km. Collettes Swamp is in possession of several farmers. One of the farmers likes his land dry and due to this the entire floodplain is dry. In times of minor floods or less, Collettes Swamp stays dry, since the location is just a bit higher in comparison to other flood plains.

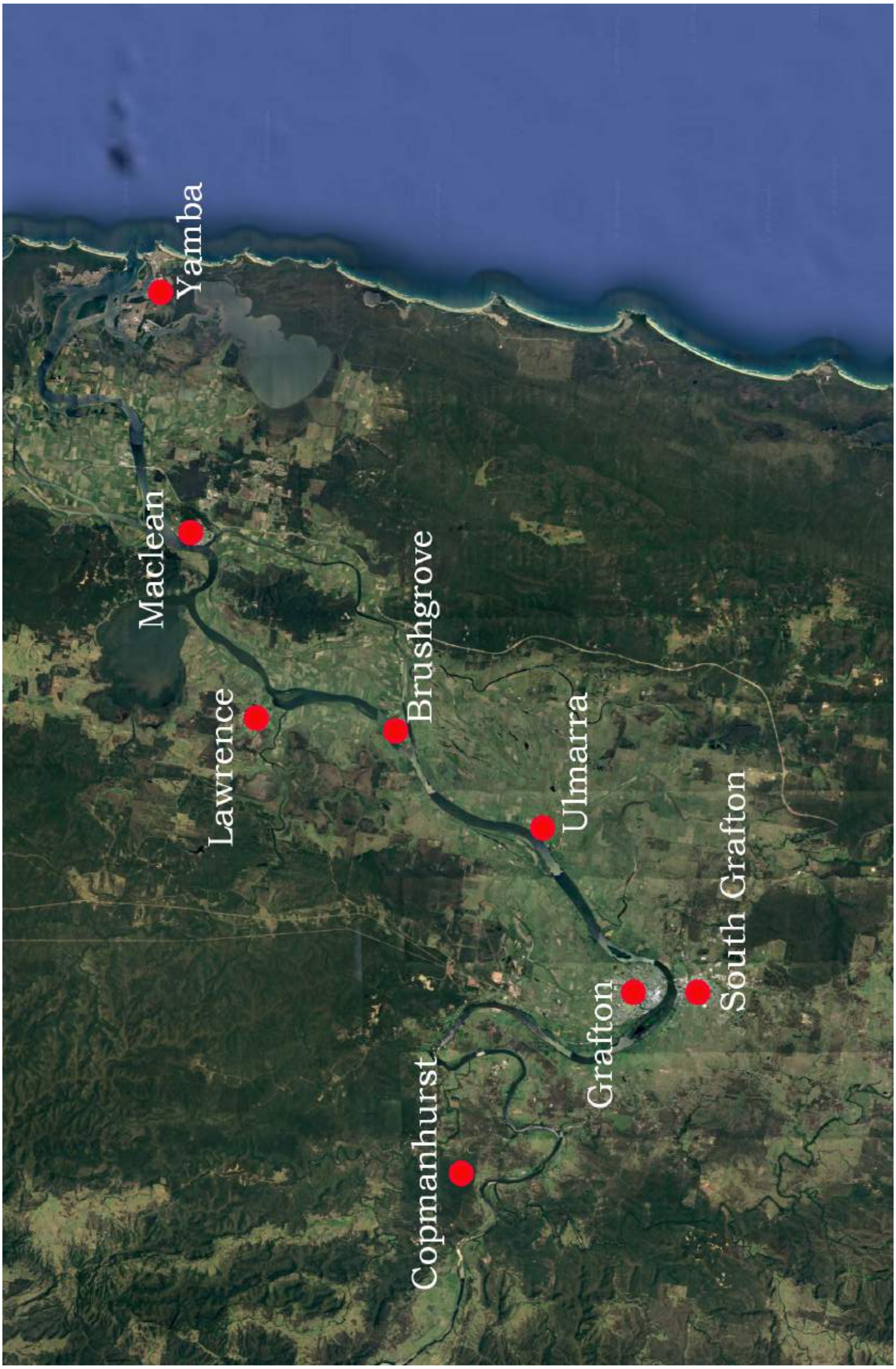


Figure A.3: Full area map with cities and villages in the Clarence Valley.

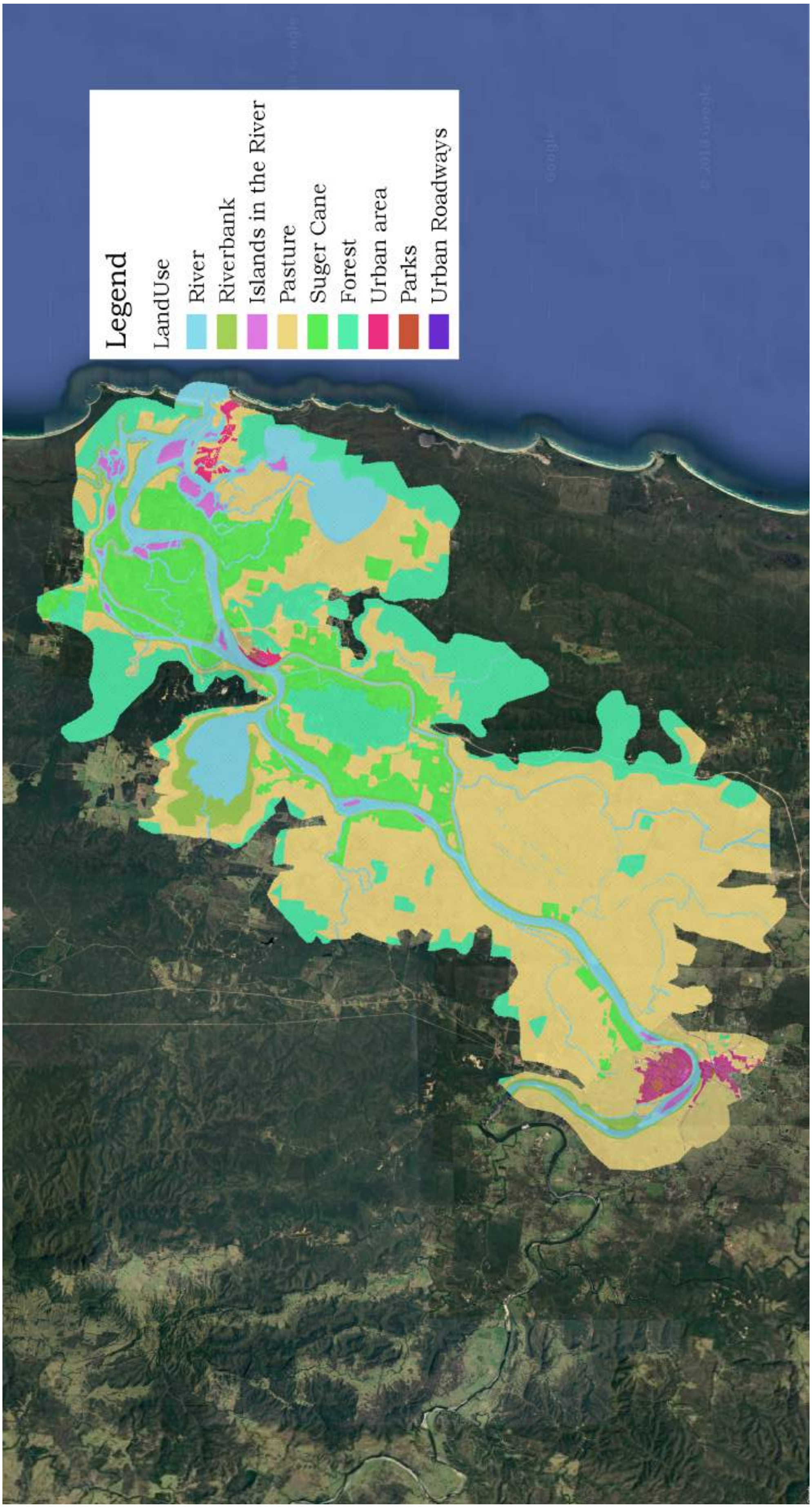


Figure A.4: Overview of the land use in the Lower Clarence Valley.

A.2. Stakeholder Analysis

A description of the stakeholders taken into account can be found in this section. For a quick overview of their power and interest, see Figure 2.1.

A.2.1. Authorities

In Australia, there are three levels of government. The lowest level is the council, which can be compared with the Dutch municipalities. The State Government is a provincial government. Finally, the Federal Government is the nationwide authority in Australia.

Clarence Valley Council

This is the local government in the Clarence Valley. One of the tasks of the Clarence Valley Council is to protect their residents. The Council protects their residents by maintaining and monitoring the flood defence system in the valley. This results in high influence of the Council on the decisions regarding flood safety. Since flooding has been an issue in the Clarence Valley for decades, the impact of the floods on liability of the council is high.

State Government New South Wales

The Clarence Valley Council is located in the state New South Wales. Decisions made by the Clarence Valley Council have to be approved by the State Government New South Wales, which means that they have a final judgement. The Clarence Valley is not the only area in New South Wales that is prone to flooding. This means that the flooding issue is quite known within the State Government. Also the Pacific Highway runs through the Clarence Valley. This highway is an important connection between the cities Brisbane and Sydney. The interest of NSW is to keep the highway accessible during floods.

Federal Government

Australia consists of eight states, which are all ruled from Canberra (Australian Capital Territory). The Federal Government has little influence on decision making, regarding floods in the Clarence Valley. Flood defence is outsourced by the Federal Government to the Local- and State Governments

A.2.2. Citizens

Most of the people in the urban areas are not threatened by the possibility of floods due to the high levees. An insurance against floods is approximately 10,000 Australian dollars per year and people do not see the benefits in investing in a insurance against floods. Due to this, people are in favour of preventing floods. The measures give the people a safe feeling and withholds people from evacuating towards higher grounds in case of danger. The older people have seen many floods before the 60's and are aware of the problem, but they have a strong opinion about raising the levees even further. This would be an expensive measure, since a lot of the levees are property of local people whose houses are located on the levees.

The total population of the Clarence Valley is around 50,000. The great majority lives in villages like Grafton, Maclean and Yamba. For Grafton, people that live on the levees will be discussed separately. A lot of land in the Clarence Valley is used for farming, so farmers will also be reviewed.

Grafton residents

Grafton (including South Grafton) is the largest town situated in the Clarence Valley. Grafton is the first urban area in the Clarence Valley which inundates in case of a flood. Grafton has a ten hour evacuation time after the peak of the flood arrives in Copmanhurst (a small community situated at the upstream part of the Clarence River). During the latest floods in 2011 and 2013, the levee system functioned as it should, so Grafton did not flood. The levee has been overtopped for a brief period at Dovedale st in 2013, this has no further consequences. However, some people were evacuated to higher grounds in South Grafton. During a major flood, the levees will overtop, resulting in inundation of Grafton. The only road to evacuate makes use of the Grafton bridge, which makes a complete evacuation out of Grafton towards South Grafton or even Coffs Harbour difficult in 10 hours. This event will affect all the residents of Grafton, so naturally the impact of the floods in Grafton is quite high. The Grafton residents do not have high influence on the decisions made regarding to flood protection, since the

Clarence Valley Council and the State Government of New South Wales have to decide how much money is spent on flood protection. Of course, the Grafton residents do have some influence, due to the fact that they vote for the Council and the State Government. Also, the Clarence Valley Council takes the opinions of their residents very serious.

Grafton levee residents
The Grafton levees that are located near the river banks, have lots of houses on them. The levees are the highest grounds of the northern part of Grafton. This means that when the water level overtops the levees, the houses located on the levees will inundate. However, if Grafton floods from water ingress from the north, it is possible that the houses within the levee system will flood, but the houses located on the levees will not flood. This results in a slightly lower impact of flooding for the Grafton levee residents. Maintenance work and a possible raise of the levee is difficult to proceed, because the levee is private property of the residents living on top of the levee. Participation of all residents living on the levee is required in order to heighten the levee. Due to this reason, the influence is higher in comparison to regular residents of Grafton.

Maclean residents
Maclean is situated downstream of Grafton. When Grafton floods, it is estimated that Maclean has 18 hours until the levee overtopping will take place. Compared to Grafton, a small portion of Maclean is located on low grounds. Also, higher grounds can be reached easily for the Maclean residents because there are multiple roads leading to the higher grounds of Maclean. So not everyone has to pass one road, which is the case in Grafton with the Grafton Bridge being the main evacuation route. In conclusion, the impact of flooding in Maclean will be slightly lower compared to Grafton. The influence on decisions is the same in comparison to Grafton.

Yamba residents
Yamba is the most downstream located city of the Clarence Valley, close to the ocean. In case of a flood most damage has already be done further upstream. The most severe flood in Yamba occurred in 2009, which was mainly caused by rainfall in the lower catchment of the Clarence River. The impact of the main floods in the Clarence Valley on Yamba is smaller compared to Maclean and Grafton. Again, the influence on decisions remains unchanged, because the residents of Yamba live in the same council.

Farmers in the Clarence Valley
A lot of land in the Clarence Valley is used for farming. Sugarcane is grown and beef cattle grazes in the current floodplains. Sugarcane can withstand floods that last a week. Beef cattle is evacuated in case of a flood, so no animals are lost during a flood of the Clarence Valley. A lot of farmland floods, even during a moderate flood, which means that the impact of floods for the farmers is very high. The floods could lead to financial damage for the farmers, because crops could be lost and cattle has to be moved. The influence on decisions is moderate for the farmers, just like for all the other residents in the Clarence Valley.

A.3. Infrastructure Analysis

The Clarence Valley contains several important infrastructural links. Grafton especially is the main hub in the Clarence Valley for both the road network and the rail network. The Clarence River itself used to be a main waterway as well, but lost its function for transportation nowadays. This section will describe the main infrastructure of the Clarence Valley, subdivided in three types: roads; railways and waterways. Since road traffic is the main type of transportation in Australia, these roads in the Clarence Valley will be discussed the most.

Roads

One of the most important roads on the east coast of Australia is the Pacific Highway, which runs from Brisbane in Queensland to Sydney in New South Wales. This highway enters the Clarence Valley north of Yamba near the coast. It passes the main branch of the Clarence River for a few kilometers downstream of Maclean. The road bends towards Maclean, and continues to run along Woodford island. The Pacific Highway follows the southern bank of the Clarence River, until South Grafton is reached. The highway bends southwards and leaves the Clarence Valley about 40 km further.

In 1996, major construction works started on the Pacific Highway to upgrade the road to a four-lane carriageway. This also included the part that runs through the Clarence Valley. On 24 June 2014, the Pacific Highway upgraded from Woolgoolga to Ballina received approval from the State. The upgrade included duplication of the 155 km motorway to a double carriageway in both directions. Space is created to add a third lane in the future if necessary. Several split-level interchanges are created and a total of hundred bridges are added, including a new bridge over the Clarence River near Harwood.

The new bridge near Harwood will be a concrete girder bridge with thirteen piers placed in the river. These piers will increase the flood levels by 15 mm at the Maclean Levee (Roads and Maritime Services, 2017). Unlike the current Harwood Bridge, the new bridge will not have a movable part. To avoid obstruction for shipping traffic, the new bridge will have a clearance of 30 m. The width of the Clarence River at the location of the bridge is 620 m. Including the ramps over land, the total length will be 1520 m.

The new highway will not pass Grafton any more, because a bypass is created south of Clarenza. This will reduce the travel distance by 13 km and reduce the travel time by 25 minutes. The 4.36 billion dollar project is expected to be completed in 2020.

The Gwydir Highway is another important road in the Clarence Valley. It originates in South Grafton and heads westwards. It is one of the main east-west routes on the east coast of Australia. The Gwydir Highway links to the current Pacific Highway in South Grafton. Due to the new bypass, the old route of the Pacific Highway will still be linked from the new Pacific Highway to the Gwydir Highway.

In South Grafton, the Summerland Way links to the Gwydir Highway, close to the Pacific Highway. This road crosses the Clarence River by the Grafton Bridge. This bridge opened in 1932 and carries the Summerland way, a railway and a footpath. The bascule part of the bridge shut down in 1969 due to a decline in shipping along the Clarence River. Nowadays it not even possible to open the bascule bridge due to the addition of a water pipe. The Summerland Way continues through Grafton and is part of the main street in Grafton (Prince Street). The road reaches the towns of Casino and Kyogle and ends at the border of New South Wales and Queensland, 200 km north of Grafton.

Railways

Grafton lies on the North Coast Railway Line, which is part of the railway between Sydney and Brisbane. The railway was connected to Grafton in 1905. Before the Grafton Bridge was completed in 1932, a train ferry was used to cross the Clarence River. The railway station is located in South Grafton on natural high ground. The railway is not used extensively, with only a few passenger trains and a few freight trains every day. It is a single track railway without a overhead catenary.

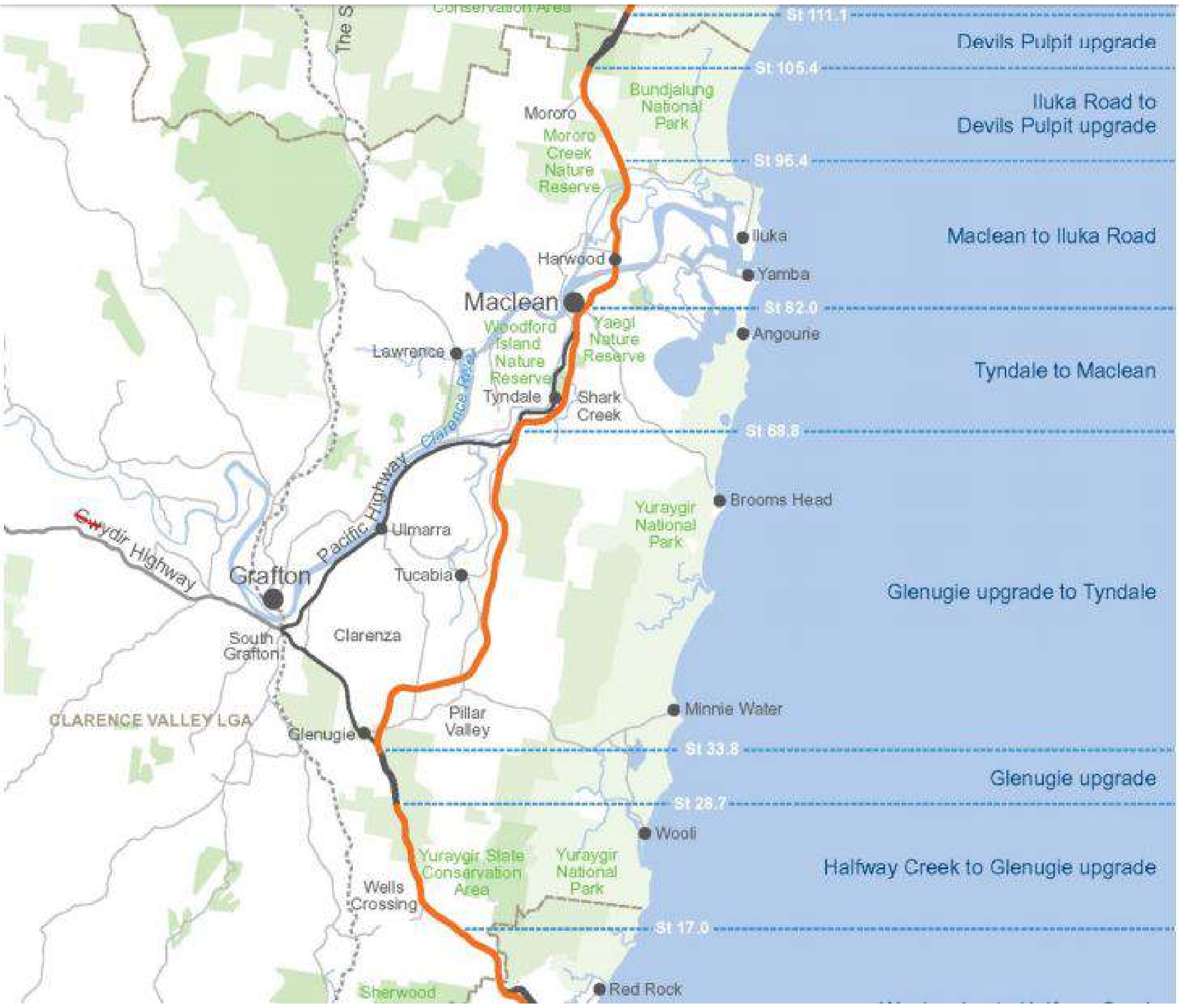


Figure A.5: Horizontal alignment of the new Pacific Highway through the Clarence Valley indicated in orange (Roads and Maritime Services, 2017).

Waterways

The first known settlers in Grafton were cedar-getters. They reached Grafton by a ship named 'Susan'. Susan Island was named after this ship. In the beginning of the history of Grafton, cedar production was important for the environment. The fertile banks of the Clarence River had a lot of cedar trees, which were used for ship building. Also, the Clarence River used to be used to transport the tree trunks further downstream and even to Sydney, the home port of the 'Susan'. The production of cedar stagnated after a few years, but Grafton remained an important port for transportation of goods to Sydney. Until the opening of the railway in Grafton in 1905, shipping was by far the most important type of transportation (Stubbs, 2007).
The riverbed of the Clarence River mainly consist of sand and gravel. The river is dredged to get these raw materials from the riverbed. Next to the old Grafton railway station, a concrete plant is located, named Boral Concrete. This company retrieves sand and gravel from the riverbed with dredging vessels. This process is nowadays the most important traffic on the river.

Near the mouth of the Clarence River, the Wooloweyah Lagoon is located. This shallow lake is connected to the river by three channels: Palmers channel; Micalo channel and Oyster channel. These waters combined with the lower Clarence River estuary are used for fishing.

A.4. Hydrological Analysis

The Clarence Valley (lower catchment area) is a complex system consisting of the main Clarence River, various drains (man-made or connecting creeks), floodgates, levees, lakes and tidal influence. An overview of all the system inlets is given in Figure 2.3. The primary inlet is referred to as the Clarence River. Multiple secondary inlets are referred to as Glenugie Creek, Coldstream River, Shark Creek, Sportsman Creek, Broadwater Lake and Esk River. This section elaborates on the hydrological aspects of the above-mentioned features. The rainfall event in the upper catchment causing the formation of a floodwave (discharge) in the primary inlet is discussed, as well as discharges and local rainfall in various smaller lower catchment areas. The water level variation due to tidal influences is also discussed. Moreover, maximum water levels at two important gauges are presented, along with the river gradient.

In this analysis, the 5, 20, 50, 100 year and extreme Average Recurrence Intervals (ARI) are used. The higher order floods (100 years ARI and extreme flood events) are only included to show the magnitude of these events, but will not be used in simulation of the CVFM. The Probable Maximum Flood (PMF) is the flood which may occur in the catchment due to the Probable Maximum Precipitation (PMP). This PMP-factor is set at 1.53, comparing the 100 year ARI and extreme ARI rainfall event ((Farr and Huxley, 2013)). The PMF-factor is also set at 1.53, comparing the 100 year ARI and extreme ARI flood events. Due to the flood model consisting of only the lower Clarence Catchment, the PMF value is derived from a flood frequency analysis ((Farr and Huxley, 2013)). No return period can be coupled to the extreme ARI flood event, as there is no Australian guidance methodology for this kind of drainage basins.

If a waterlevel of 7.9 $mAHD$ is reached at the Prince Street Gauge, flooding occurs in the surrounding urban areas. The water levels in Maclean can rise up to 3.4 $mAHD$ before flooding occurs (Farr and Huxley, 2014). 80 Km of the Clarence River is included in the Clarence Valley Flood Model (CVFM). The height difference between the upstream and downstream boundaries is around 10 $mAHD$, leading to an average bed level gradient of $1.15 * 10^{-4}$.

Upper catchment

The Australian east coast receives cyclones on regular basis, resulting in major rainfall events. These cyclones are known to bring around 200 mm rain on daily basis, with some exceptions reaching nearly 700 mm a day (based on local judgement). The model only contains a small part of the river and therefore a boundary condition is made including the rainfall of the entire upper Clarence Valley. The excessive water due to local rainfall in this enormous area is drained towards the coast leading to a massive discharge, see Figure 2.2. This discharge forms the primary Clarence River input condition for the flood model.

In Figure 2.3, the light blue transparent areas show the smaller local rainfall catchments within the CVFM. The local rainfall is eventually drained to the Clarence River, after being ponded on the catchments. Only the Grafton area (blue crosses polygon) does not take any local rainfall into account as the purpose of previous studies was not to focus on internal flooding.

Lower catchment

The lower catchment is modelled in the CVFM. For this purpose, the discharges and local rainfall are split. The discharges from all the secondary inlets, obtained from Figure 2.3 are visualised below. Lake Woolooweyah discharges directly to the outlet boundary and is therefore not a secondary inlet. Note, that the vertical axis is kept constant to show the difference in magnitude between the inlet discharges. The total discharged water by the secondary inlets together is around 20% of the Clarence River discharge coming from the upper catchment.

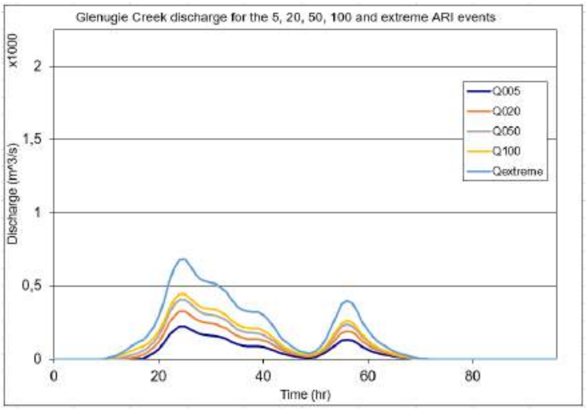


Figure A.6: Discharge floodwave for the 5, 20, 50, 100 year and extreme ARI flood events for the Glenugie Creek (see figure 2.3).

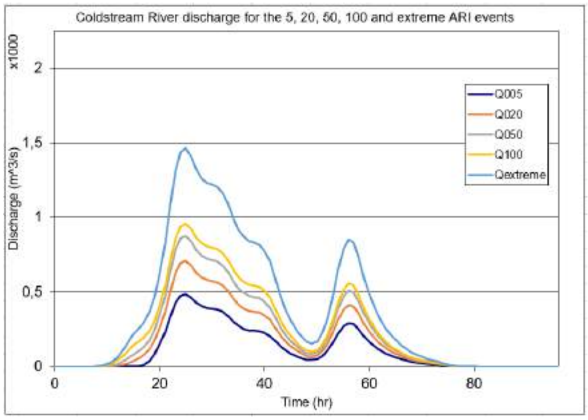


Figure A.7: Discharge floodwave for the 5, 20, 50, 100 year and extreme ARI flood events for the Coldstream River (see figure 2.3).

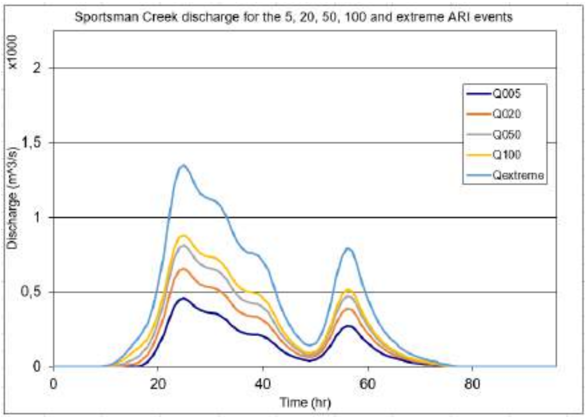


Figure A.8: Discharge floodwave for the 5, 20, 50, 100 year and extreme ARI flood events for the Sportsman Creek (see figure 2.3).

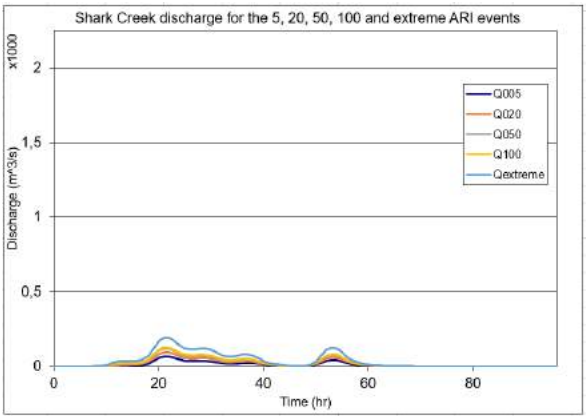


Figure A.9: Discharge floodwave for the 5, 20, 50, 100 year and extreme ARI flood events for the Shark Creek (see figure 2.3).

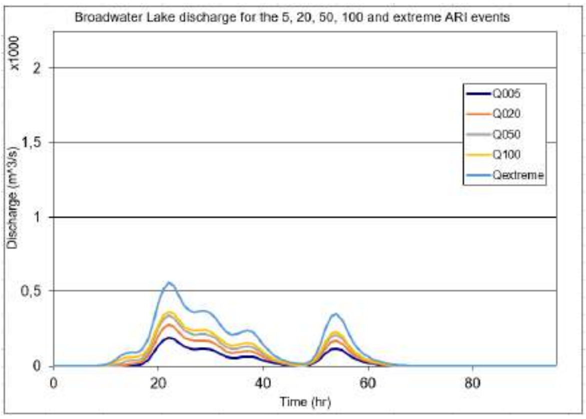


Figure A.10: Discharge floodwave for the 5, 20, 50, 100 year and extreme ARI flood events for the Broadwater Lake (see figure 2.3).

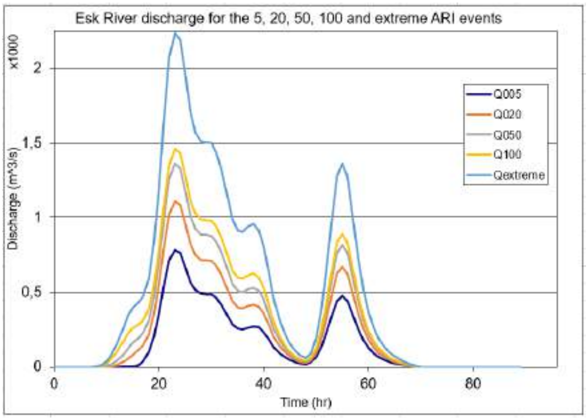


Figure A.11: Discharge floodwave for the 5, 20, 50, 100 year and extreme ARI flood events for the Esk River (see figure 2.3).

Tidal influence

The tide of the Pacific Ocean on the outlet boundary has influence up to Copmanhurst, a town around 40 km upstream of Grafton. As a result, the tide strongly influences the water levels in the downstream areas of the Clarence Valley but has minor influence further upstream (the tide is not visible during a floodwave here). The floodwave influence decreases further downstream, opposite to the observation on the tidal influence (the floodwave is almost not visible here). This can be visualised by Figure A.12 and A.13 below.

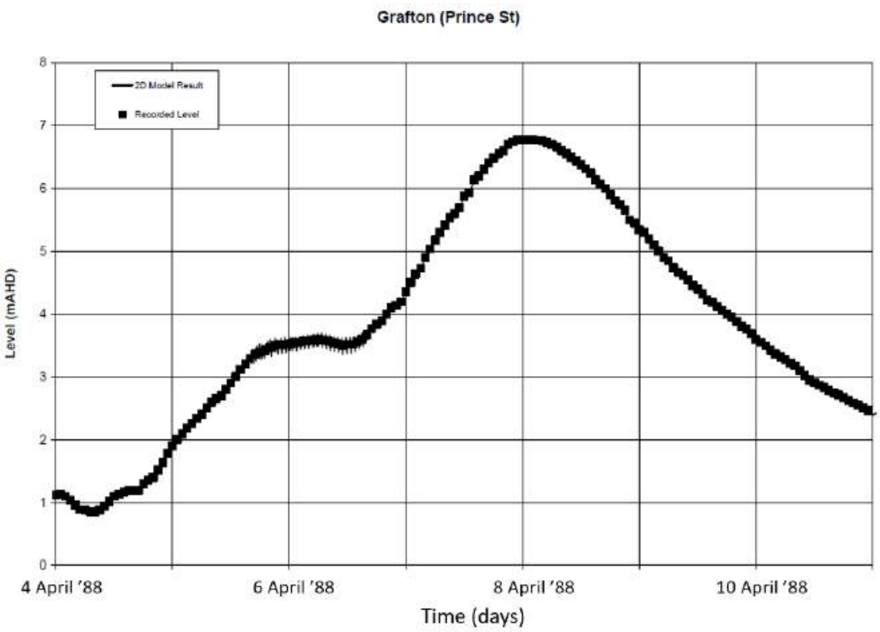


Figure A.12: Tidal influence in the measured water levels for the April 1988 flood at Grafton (Rogencamp, 2004).

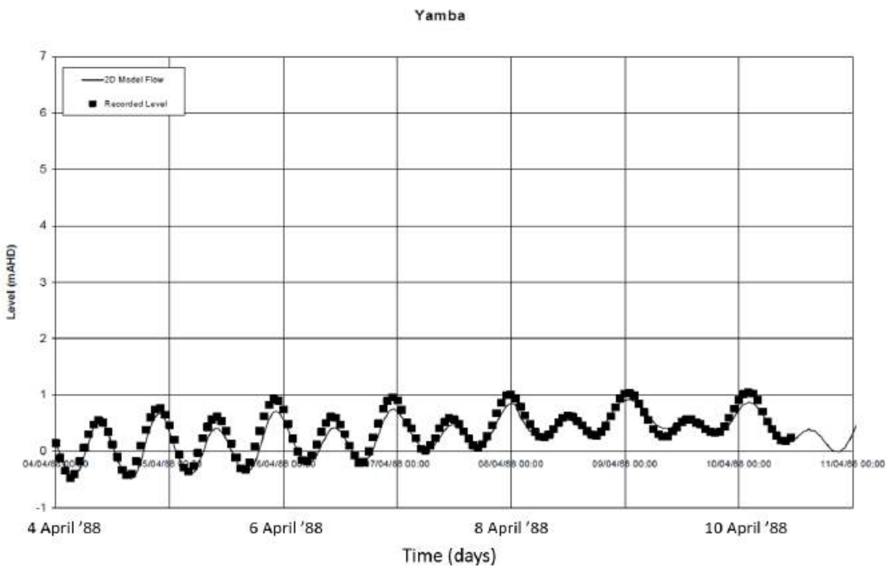


Figure A.13: Tidal influence in the measured water levels for the April 1988 flood at Yamba (Rogencamp, 2004).

Recorded Floods

The table has been published by the Clarence Valley Council. These are the flood peak heights at Grafton since 1839 measured in $mAHD$ at the Prince Street Gauge.

DAY	MONTH	YEAR	PRINCE ST GUAGE(m)
		1839	5.70
31	1	1841	5.95
	5	1845	6.10
20	2	1857	5.49
10	6	1857	6.33
30	4	1861	6.08
15	2	1863	6.95
18	3	1864	6.34
9	8	1864	6.59
29	4	1867	6.23
	2	1875	3.35
17	7	1876	7.48
23	1	1887	7.83
19	7	1889	6.89
18	1	1890	5.14
4	2	1890	5.44
12	3	1890	7.88
28	3	1890	6.08
4	4	1892	6.64
12	2	1893	6.99
18	2	1893	7.73
12	6	1893	7.20
	3	1894	3.66
	1	1895	4.57
	9	1903	3.05
9	11	1917	4.27
17	5	1921	5.55
24	7	1921	6.87
22	6	1925	2.21
28	1	1927	3.96
20	2	1928	6.76
20	4	1928	3.09
16	2	1929	4.27
5	4	1929	2.58
27	7	1933	3.99
21	2	1937	3.96
16	3	1937	4.76
21	1	1938	3.66
18	3	1939	3.35
2	1	1944	3.35
26	8	1944	2.40
12	6	1945	6.45
26	3	1946	7.10
26	1	1947	2.30
2	3	1947	3.32
2	4	1947	2.41
16	6	1948	7.17
20	1	1950	3.04
20	2	1950	3.87
24	6	1950	7.78
21	7	1950	3.61
27	1	1951	3.13
21	3	1951	2.40
29	7	1951	3.86
15	8	1952	2.40
25	2	1953	3.42
22	2	1954	7.72
14	7	1954	5.90
29	3	1955	5.82
1	5	1955	5.34
22	1	1956	3.67
10	2	1956	4.53
19	2	1956	6.97
3	3	1956	2.62
2	5	1956	4.76
23	1	1959	6.74
19	2	1959	6.03
14	11	1959	3.08
8	4	1962	5.60
12	7	1962	5.64
2	1	1963	5.68
29	4	1963	2.50
9	5	1963	7.63
10	3	1964	3.67
22	7	1965	3.33
31	1	1967	2.55
19	3	1967	3.66
14	6	1967	7.60
13	1	1968	6.22
19	2	1971	3.59
29	10	1973	4.98
11	1	1974	4.36
27	1	1974	6.66
13	3	1974	7.35
	4	1974	2.23
22	1	1976	3.01
12	2	1976	7.28
23	2	1976	4.98
29	2	1976	2.69
19	5	1977	2.63
10	5	1980	6.40
7	4	1988	6.78
3	4	1989	6.54
27	4	1989	6.09
5	5	1996	7.07
3	2	2001	6.66
10	3	2001	7.70
6	1	2008	3.66
23	5	2009	7.37
12	1	2011	7.64
21	1	2012	5.55
29	1	2013	8.06
24	2	2013	6.25
4	3	2013	3.62
3	5	2015	2.69
21	3	2017	3.19
1	4	2017	2.75

Figure A.14: Historical Flood Peak Heights at Grafton.

A.5. Prior Council Decisions

The Clarence Valley Council is working on solutions for decades. A popular suggestion for flood defence is the construction of a dam upstream, because this would absorb the peak of the floodwave and withhold the water. This possibility is declined, because of multiple reasons. Building a dam would have an impact on environment which could not be avoided. Likewise the energy that would be generated by the dam could not have been used. The power consumption of the surrounding area is lower than the energy generated by the dam. Building a hydropower dam would therefore be useless and not taken in to account in this study. Another solution that's already proposed is a flood mitigation channel which would pass Grafton on the north side of town. BMT WBM already proposed this solution and ran the model for this flood mitigation measure. This resulted in reduced floodrisk for Grafton but induced higher risk and more trouble for downstream areas. These shortcut channels basically shift the problem to another location so are not taken in to account in this project. Investing in temporary solutions is not recommended. The Council would like to provide a permanent solution, so scenarios like dredging are not taken in to account.

The Clarence Valley Council has already taken many mitigation measures over the last few years. Levees and flood defence structures are the most common solutions for this area. Local residents like farmers have taken decisions based on experience and personal insight. The public opinion (Appendix B) is that rising the levees is the best solution.

The CVC asked BMTWBM to create a floodmodel of the lower Clarence Valley. This floodmodel is assisting the CVC in making decisions about flood mitigation. The concept of applying floodplains is not new in the Valley. During highwaters in the past, the plains used for agricultural activities were flooded first. However, research in this field hasn't been performed yet.

B

Fieldvisit

During the first week of the project a visit was made to the Clarence Valley. In collaboration with the Clarence Valley Council we did a three days field visit to several locations within the Valley. Floodplains, flood defences and of course the river itself where viewed. The field visit provided a more detailed view about the current situation, along with its problems and possible opportunities in the area. In this Appendix one can find the itinerary of the visit, the write-up and interviews taken.

B.1. Itinerary

Wednesday, 14 February 2018

- 10am - Kieran and Frank to pickup from Grafton Train Station
- 10:30am - Induction at council offices - 42 Victoria Street, Grafton
- 11am - free time for you to walk the main street of Grafton and have lunch
- 12:15pm - meet local Grafton businessman and member of the Floodplain Risk Management Committee Des Harvey at Prince Street Gauge
- 12:45pm - commence tour of flood infrastructure around Grafton, levee between Bacon and Powell Streets, ALice St Levee, Pine St Levee, Baker's Swamp Floodgate, Cowans Pond. Possibly visit levee Dr. Hal Leaver's house (time permitting)
- 4:30pm approx. - Meet Clarence Valley Council's Director Troy Anderson and meet your hosts at Victoria Street and travel to their house

Thursday, 15 February 2018

- 8:30am - Meet at council office (42 Victoria Street, Grafton) and depart for Swan Creek
- 9am - Swan Creek Floodgates to meet member of the Swan Creek Committee - Geoff Duckworth
- 10:30am - Briner Bridge Tucabia (via Coldstream Road) or Collettes Island or Avenue Road
- 12pm - Lunch Ulmarra Hotel
- 1pm - Cross river on Ulmarra Ferry
- 1:30pm - Visit Fabridam on Alumy Creek area, then visit other parts of this system including Southgate. Drive back to Grafton via Trenayr/Experimental Farm Road
- 3:00pm - arrive back in Grafton Office
- 3:15pm - presentation and brainstorm with council Staff
- 3:50pm - meet council's media adviser (David Bancroft) for chat and photos.

Friday, 16 February 2018

- 8:30am - Meet at council office and depart for Maclean. Leave luggage in Kieran's office
- 9am - Brief stop over at Brushgrove
- 9:30am - McLachlan Park to meet up with local flood expert and Floodplain Risk Management Committee member Paul O'Halloran
- 10:30am - Visit Maclean Lookout (very good view of floodplain). Council's Peter Wilson to join us.
- 11am - Meet Robin Knight from the Port of Yamba Historical Society at the Whiting Beach breakwater car park, before driving with Robin to the Yamba headland
- 12pm - Lunch Pacific Hotel
- 1:30pm - Visit Lake Wooloweyah area (ring drains). Meet local Vince Castle at his house.
- 2:30pm - Return to Grafton via Lawrence Ferry, stopping at Sportsman Creek Levee and Everlasting Swamp
- 4:30pm - Take luggage to Kieran's house - remain in Grafton until train departs at 9pm - Kieran will drop you at the train station at 8:30pm

B.2. Write-Up

Day 1: 14-02-2018 Tour in and around Grafton.

- Meeting with Des Harvey (local businessman), who has experienced the 1967 flood where the levees overtopped and Grafton was inundated.
  - In 1967 two major floods occured in one year. During this flood the water level was 2.5 m high in Prince Street.
  - The inhabitants of Grafton at that time did not have an insurance against flooding. Back in the days they stored personal belongings and stayed safe in the higher parts of the houses, for example the space between the ceiling and the roof.
  - The water level in Copmanhurst is the reference level for the people in Grafton as a warning for the floods.
  - The runoff of the flood was not very good at that time so it was very common that households ran out of food. There was a water brigade that supplied food for the locals. This brigade mainly consisted of volunteers.
  - Since 1967 there has not been a flood that inundated Grafton. Floods like the one in 2011 and 2013, were close to overtopping the levees but they did not. According to Des Harvey there are a lot of ignorant people in Grafton who are not aware of the danger of the floods, because they have not experienced a flood like the one 1967. These people feel safe behind the constructed levees, but they are not aware that it is always possible that a bigger flood may occur. People are more likely to evacuate than in the 50's, 60's and 70's. A cut off of water and power is one of the main reasons to evacuate. The council provides facilities for evacuation and organises meetings about floods and evacuation.
  - During this meeting with Des Harvey some solutions were discussed. In his opinion the solution is to raise the levees. This is a very common opinion in the Clarence Valley since not a lot is known about other options under the local people. Another option which has been discussed is the option of dredging. At the moment dredging is only applied for commercial ends to produce concrete, because the sandbed consists of sand and rocks. The people were not very fond of the option of dredging the river to make the area more flood prone, since it is a temporarily solution and they are more interested in permanent solutions.
  - In Australia it is very common to buy a large piece of ground and build your own house on it. In case there is a levee on that land, the buyer also becomes the owner of the levee (see Figure B.2). So in case maintenance or other changes are needed for the levee, the owner of the land has to be involved.
- Great Marlow Swamp contains a huge catchment and drains downstream of the river by installed floodgates to release pressure.
- Tidal differences are not in $mAHD$ at the coast. A tidal difference of 2 $mAHD$ at the coast is a change of about 1.1 $mAHD$ in Grafton.
- Some buildings in Grafton are placed on the riverside of the levee because of easy access to the water. Examples of these buildings are the rowing and the sailing club. In case of a flood they move the personal belongs to the top levee. The buildings are prone to damage. The roof of the sailing club collapsed in 2013.
- Grafton is divided by the Clarence River in to Grafton and South Grafton. The southern part of South Grafton is the highest part of Grafton and does not flood in case of a 100 year ARI flood event. Despite this fact a lot of people choose to live in Grafton, since this area is seen as a more valuable part of Grafton. Most of the economic activities take place in Grafton and also the view on the river is better. For these reasons the price of houses is higher in Grafton. Back in the days, before the levees were constructed they even used South Grafton as a floodplain to protect Grafton.

- A bottleneck is present at the bend of the river between Grafton and South Grafton at Dovedale. Both sides of the river are obstructed by levees which influences the flow of the river. The Clarence River narrows due to adding up of sediments at the inner bend while the outer bend is protected by rock armour. The levee South Grafton Rural at the side of South Grafton is 5.95 $mAHD$, rock armoured and needs ongoing maintenance. The levee overtops in case of a major flood, the floodplain behind is the basin for this overtopping water. At the side of South Grafton the Heber Street Levee with a height of 7.80 $mAHD$ has been built to prevent flooding of the urban area. At the opposite of this levee a Control Levee, the Clarenza Levee, is built of 5.4 $mAHD$ to gradually fill the floodplain behind. The council particularly pointed out that in this region improvements can be made towards the use of floodplains.
- Southampton is a huge floodplain at the left hand of South Grafton. The area is low ground and floods according to the council by local rainfall from the catchment uphill and stays dry in the absence of local rainfall. This opinion does not fully agree with Chris Huxley's who claimed the water wharps around the levee upstream and flows in the Southampton Floodplain.The water is blocked by the Gwydir Highway.
- Susan Island located in the Clarence River floods at 3.6 $mAHD$. The Clarence Valley Council made regulations about new building construction. It is not allowed to build in 3 m of the levee toe. Existing buildings are not restricted to this regulation.
- Piping is a problem when water table levels of the river gets higher than the level at the adjacent land. Water flows through the porous sandy soil by pressure. This phenomenon occurs often at basins.
- Baker's Swamp is located between Junction Hill and Grafton. This particular area was marked to be one of the potential areas for mitigation of floods. The area floods by local rainfall and is protected by a levee at 9.15 $mAHD$. The council pointed this area to potentially function as a floodplain.
- Great Marlow Floodgate, four times (2.2*2.2) m^2 culverts with gates to regulate the water flow of Baker's Swamp and Alumy Creek.

Figure B.1: An elevated house in Grafton.

Figure B.2: A house built on a levee in Grafton.

Day 2: 15-02-2018 Tour to Swan Creek, Ulmarra, Alumy Creek and presentation and brainstorm with the council staff.

- Swan Creek: meeting with Geoff Duckworth, farmer and member of the Swan Creek Committee. Personal tasks: Operator of floodgates built in 2006 in the Swan Creek, communication to farmers in the area. He has 12 years of experience in operating the floodgates (see Figure B.3). High tide in the Clarence River is higher than the water level in the Swan Creek. So gates open only for a few hours a day to irrigate the agricultural land. Some farmers complain that when a small rise occurs, the water does not run off quickly enough. However, this is not possible because the water level in the Clarence River is higher during high tide. The floodgates should maintain a water level of minimum 0.2 $mAHD$ and maximum 0.4 $mAHD$. The opening and closing of the floodgates is based on weather forecast which ask for experience. The major floodgate in the Swan Creek is a seven times (2.2 x 2.2) m^2 box culverts gate with automatic penstock gates. When a flood is likely to occur, Geoff opens two or three of the seven gates. During the flood itself, all seven gates are opened. A remarkable feature of the floodgate is the current status of the concrete side walls of the floodgate towards the river. After 11 years of operation time the side walls start tilting in lateral direction and longitudinal direction. The tilt of the west side wall is greater than the tilt of the east side wall. Both sides are filled with rock between the wall and levee. Frank came up with possible explanations for the situation of the side walls: erosion of the river bed due to opening of the floodgates with the consequence of lacking of foundation support of the side walls and the lack of maintenance. The floodgates in that time were constructed by local farmers and the concrete was produced at the construction site and there was no supervision. For these reasons the quality of the concrete is not up to standards. An ongoing discussion between farmers makes the water management in the Swan Creek more difficult. Preferences for drainage and irrigation collide.

- Wilcox bridge where the Four Mile Lane crosses the Swan Creek. On one side of the bridge the riverbank was vegetated and the other side is not. The reason for the non vegetated side is the grazing cattle. At this side a rock protection was applied. On the vegetated side of the river, oak trees were growing. Oak trees are not good as a bank protection since their root system is shallow. They fall down easily and take out large piles of earth, which damages the riverbank. After the 1974 flood, Swan Creek and the bridge had to be widened to release the pressure. At this location the relationship between the landowners and the council is discussed. The attitude of the landowners towards the council is not easy. The council is often criticised by the landowners.

- The Avenue, connection between the Coldstream River and the Swan Creek by culverts. The Avenue is a manually operated penstock gate.The current state of the Creek at the Avenue is blocking the stream. The creek is completely silted and needs maintenance.

- Several minor levees are build in the Swan Creek in the 60's and 70's based on local knowledge of farmers and council members. The function of those levees is preventing the water from Swan Creek and Coldstream River to merge and to stop major flow of water in the floodplain.

- Collettes Island, located above the Avenue is natural higher ground which keeps dry in any flood so far. Behind the Collettes Island is the Collettes Swamp located, a notable dry piece of land, which is even dry in case of a moderate flood. The swamp is prevented from floods by a local landowner and farmer who obstructs water flow into the Collettes Swamp, at a disadvantage of irrigation of surrounding agricultural land.

- Floodgate in the Clarenza Floodplain: three culverts with penstock gates which leak a certain amount of water. The cement of the culverts erodes because of low pH value (about 3.5) due to a high level of phosphate in the groundwater. However, the maintenance of these culverts is not in the budget of the council.

- Fabridam used for drainage of Alumy Creek and Baker's Swamp. The Alumy Creek has a level of 0.6 $mAHD$. If the Clarence River reaches 1.0 $mAHD$ it overtops at the Fabridam. Their used to be a inflatable dam, but this dam was vandalised by farmers in case they needed water for irrigation.

Figure B.3: The Floodgate between Swan Creek and the Clarence River.

Figure B.4: The drains in Clarenza, a floodplain near the Swan Creek.

Day 3: 16-02-2018 Trip along Brushgrove towards Maclean, Yamba and Lake Wooleweyah.

- First stop at Brushgrove. This village is located on the west side of Woodford island. The Clarence River splits at Brushgrove, where the northern river arm is much larger compared to the southern river arm. Woodford island is not protected by a levee system, which means Brushgrove floods, even during a moderate flood. South of Brushgrove, there is a bit natural high ground (near Tyndale). This results in a narrow passage for the Clarence River.
- Visit Maclean meet up with Paul O'Halloran, Flood Risk Management Committee member: good levee system along the river line consisting of concrete walls founded on sheet piles of around 20 m, provided with gates and levees. Since the system was built in 1979, no overtopping of the levees and walls occurred so far. However, the phenomenon 'piping' occurs at the concrete walls. Water was boiling up behind the concrete walls during the event of a flood. In the beginning of the levee system, the openings in the concrete walls were closed during a flood using wooden planks. Sometimes the planks would not fit due to bad shape of the planks. Nowadays large movable gates are used to close the openings in the concrete wall. Adjacent to Maclean a catchment drains towards the village which causes raising water levels behind the levee system. Several pumps are installed at the levees to prevent flooding of the village by local rainfall in the catchment. In case of a flood, water can flow around the levees to cause a controlled and slow water rise against the levee system. Paul's personal opinion is to maintain the levees by applying a rock armour at the top of the levees to prevent erosion by people. Paul worries about the influence of the construction of the new Pacific Highway which might negatively influence the flood levels. Maclean is further downstream than Grafton. When the peak reaches Grafton, the people in Maclean have 18 hours left to evacuate before the peak reaches Maclean. During the flood of 1974, a high tide, a cyclone and a peak flood were combined. So Maclean flooded.
- Meeting with Robin Knight in Yamba, member of the Port of Yamba historical Society. Explanation about the history of modifications of the Clarence River mouth. Robin's personal opinion about floods in Yamba is the 100 year ARI flood event being too conservative. The worst flood that occurred in Yamba was in 2009, when a lot of rainfall fell in the lower catchment of the Clarence Valley. Minor flooding of Yamba was the effect.
- Short meeting with Vince Castle, local sugarcane farmer. The history of the sugarcane industry and the relations of the sugarcane farmers with the local fisherman was explained. There are a hundred islands in the Clarence River. All the creeks near Yamba contain saltwater and have the same water level as the sea. Lake Wooloweyah is very shallow, a maximum depth of approximately 2 m. Last decade a small sized 'building with nature' project was started to protect the shorelines of lake Wooleweyah. The improvements were clearly visible.

B.3. Interviews

Interview Shane and Ronald

Shane and Ronald are both inhabitants of the city of Grafton. Shane lives in the city centre, where Ronald lives on the outskirts of the city

Are you insured against flooding?

Shane is insured against flooding, but Ronald is not. Both of them think that the flood insurance is very expensive. Ronald has a strong belief that the levees will protect his house.

Would you evacuate in case of a flood?

Both Shane and Ronald are not planning on evacuating during a flood, because they have nowhere else to go to. Ronald considers to brings his belongings to a safer location. Shane is not planning on doing this, because everything he owns of any importance is insured. Shane also adds that, in past events, it seemed fairly easy to claim money from the insurance companies even though nothing was damaged during a flood.

Would you move to somewhere else in the Clarence Valley, so your house stays dry, even during a major flood?

Ronald has built his own house where he lives with his girlfriend and two children. From his house he also runs his own business, therefore he is not planning on leaving this place because he likes the location and the house he has built. Shane is also not planning on moving. He used to live in South Grafton, but he did not enjoy it as much as Grafton where he lives now. The reason he moved to Grafton is because Grafton is considered as a nicer place in comparison to South Grafton. The crime rate in South Grafton is much higher than Grafton.

Do you trust the levees?

Both Ronald and Shane trust the levee system in and around Grafton. However, they both are aware of the fact that it may overtop in the future and maintenance of the levees is important. They see it as a risk of living in Grafton.

Interview Erica and Marc

Erica and Marc are a couple living in Grafton close to the Grafton Levee at McHugh Street.

Do you think the Grafton Levee will overtop?

Marc thinks it will happen somewhere in the future. Erica has never seen them overtop so she has more trust the levees. They do worry about the influence of global warming on future floods and the influence of the new Grafton Bridge on the water level.

Would you evacuate in case of a Flood?

They probably would evacuate because this wall has not been breached since it was built. Moreover, in case of a flood, the water cuts them of from first world features like electricity. After a flood, the mud deposit on the floodplains smells awfully. In case of an evacuation they would try to avoid the Grafton Bridge, because it shakes a lot due to the high Clarence River discharge.

Are you insured against flooding?

No, because an insurance is very expensive. Their house is built on poles, so they do not worry about potential water damage done to their property.

What would be the best solution against flooding in your opinion?

They do believe there are other options than just raising the levees and are willing to give these options a chance.

Does the Clarence Valley Council provide enough information about flooding?

The information provided about the flooding problems in the Clarence Valley is widely available, if someone is interested. There is information on the website of the council and they host information evenings.

C

The Swan Creek Floodgate

In this Appendix problems concerning the Swan Creek Floodgate are elaborated. The Appendix concludes with possible solutions to improve the floodgate. Calculations concerning reinforcements, bending moments and force equilibrium have been made with Microsoft Excel.

C.1. Current Situation

The Swan Creek enters the Clarence River five kilometres downstream of Grafton. The Swan Creek is of high importance for farmers, whom use water from the creek to irrigate their land. The Swan Creek is located between the Clarenza Floodplain and Swan Creek Floodplain. The water level in the Swan Creek is maintained by pen-stock floodgates, operated by a local farmer. The gates open a few hours a day to obtain the right water level in the Swan Creek for irrigation. In Appendix B an extensive description of the regulation of the water level is given.

The Swan Creek Floodgate is situated where the creek flows into the Clarence River. The floodgate is part of the Swan Creek Levee, it consists of seven square box culverts of 2.13 m in height and width. To maintain the levee height along the floodgate, the top deck of the structure is covered by 3 m of soil. Both the inlet (towards the Swan Creek) and outlet (towards the Clarence River) of the floodgate consist of concrete walls, these walls have an earth-retaining function, to conserve free flow of water through the floodgate.

Before the current floodgates were installed, a tree trunk was used to open the old floodgates on the outlet side. The trunk was positioned on top of the outlet walls. Cables were attached to the floodgates and were pulled around the tree. People on top of the floodgate could then open the floodgate by pulling the cable. Nowadays, the tree trunk has lost its function, due to automation of the process. However, the tree trunk is still in place, due to the fear (see Appendix B) of collapse of the outlet walls when the tree would be removed.

C.2. Problem Statement

Tilting of the outlet walls is the main problem of the Swan Creek Floodgate. Eleven years after construction, the outlet walls started to tilt. The deflections of the structural walls cause a problem for the floodgate. Failure of the outlet walls would leave the Swan Creek Floodgate out of order. Water can not be drained back to the Clarence River. Structural drawings indicate that the walls are not connected to the main structure by reinforcement (Clarence River County Council, 1970s*b*). However, the walls are connected to a concrete slab, located on the river bed.

In order to examine the problem, the floodgate needs to simulated and modelled. The software tool SCIA Engineer is used to make a structural assessment of the current floodgate.

Figure C.1: Overview of the outlet walls of the Swan Creek Floodgate towards the Clarence River.

Figure C.2: Current state of the outlet wall of the Floodgate. It is clearly visible that the outlet wall tilted.

C.3. SCIA Engineer

SCIA (Scientific Applications) Engineer is a Finite Element Method (FEM) program with a wide variety of applications. All kinds of structural elements, such as reinforced concrete plates, timber frames and steel beams, can be used to create entire structures. A wide variety of results can be obtained to assess the structure's strength, stability and deformations. The software has the ability to assess a structure according to certain building standards. However, the Australian Standards are not included. The Eurocode will be applied where needed. Some material properties might differ in the Eurocode in comparison to the Australian code.

SCIA Engineer makes use of linear calculations with the Mindlin-Reissner plate theory or Kirchhoff plate theory to obtain the results. Non-linear effects, such as concrete cracks, are not taken into account. Boundary conditions, like prescribed displacements and fixed displacements, are applied where needed. FEM programs like SCIA Engineer cut the entire structure in small elements to create a mesh. Each element is connected to neighbouring elements by their nodes. The displacements of shared nodes of adjacent elements are the same. Deformations occur within an element. These deformations depend on the element stiffness matrix, which depends on material properties, and the external forces. Once the deformations are known, a stress-strain relation is used to evaluate the stresses from the strains:

$$\sigma = \mathbf{D}\epsilon \tag{C.1}$$

Here, both σ and ϵ are vectors. **D** is the stiffness matrix, whose elements depend on material parameters (the Young's Modulus E, the Shear Modulus G and the Poisson's ratio ν) and dimensions of the structure (the thickness of the elements). These are all input parameters in SCIA Engineer, so the whole stiffness matrix for each element is known. This means all the stresses can be evaluated.

In this project, an educational version of SCIA Engineer is used. The software tool is used to determine displacements and stresses in the structure. The moment capacity and the shear force capacity must be checked at governing locations. It will also be useful to check whether the maximum stresses exceed the maximum concrete strength. Other checks, like stability and seepage, will not be checked using SCIA Engineer. These phenomena will be checked by hand calculations.

C.4. Structural Model of the Swan Creek Floodgate

C.4.1. Assumptions and Limitations

The original structural drawings with the dimensions will be used to create a model. Loads will be estimated, since these are not given. Strength verification tests will be performed and deformations will be reviewed, in order to draw conclusions regarding the structural integrity.

Stability checks will be performed to assess whether instability could be a cause of the structural failure of the outlet walls of the floodgate. A first impression of piping (internal backward erosion) for the floodgate will also be given. Finally, an assessment is made whether anchors might prevent further tilting of the in- and outlet walls.

Dimensions

The Clarence Valley Council provided technical drawings of the Swan Creek Floodgate (Clarence River County Council, 1970*sa*) (Clarence River County Council, 1970*sb*). Relevant parts of these drawings are given in Section C.10. The dimensions are given in inches and feet, which are transferred to meter. The exact height of the levee on top of the structure is not given, but the width of the levee crest is given. Using the slope of levee (1:2), the exact height of the levee crest can be calculated. The total width is also indicated in the plan of the floodgate (see Figure C.13), and amounts 16.97 m.

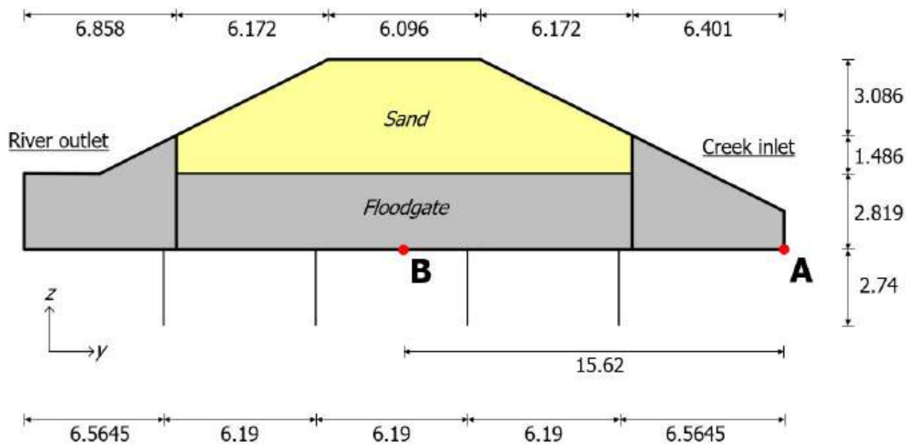


Figure C.3: The dimensions (in m) of a cross section of the Swan Creek Floodgate.

Material

The concrete strength is given in the technical drawing of formwork and joints of the Swan Creek Floodgate (Clarence River County Council, 1970*sc*). The compressive strength is shown as a function of time. After 28 days, the strength is 4,000 lb/ins^2, which corresponds to characteristic compressive strength of 27.57 N/mm^2. It is not stated how this value is determined, and whether is concerns the cylinder or cube compressive strength. According to the Eurocode, C25/30 has a cylinder compressive strength of 25 N/mm^2 and a cube compressive strength of 30 N/mm^2. This concrete strength class will be a good approximation to implement in the SCIA Engineer model of the floodgate. If C25/30 is sufficient, another stronger concrete class will also be sufficient.

Loads

Sand and water are the main dead weights that cause forces on the floodgate. The resulting pressure increases linearly as the depth h increases for both materials:

$$p = \rho * g * h \tag{C.2}$$

The gravitational acceleration is indicated by g. This value depends on the distance between the centres of gravity of two masses. Since the exact distance of the earth's surface to the centre of gravity of the earth differs from place to place, a different value for g must be used in Australia compared to the Netherlands. The Curtin University in Perth performed resolution modelling of earth's gravity field, resulting in a value of 9.79 m/s^2 for Grafton (*Western Australia Geodesy Group*, 2018).

The water level in the Clarence River near the Swan Creek Floodgate is influenced by tidal changes. The sea water is pushed upstream, but the water near the floodgate is not salt. Floods mainly consist of rainwater, which is also fresh water. A value of 1,000 kg/m^3 will be taken for the density of water to evaluate the water pressure resulting in loads on the structure.

The Australian Standards do not provide a value for the density of sand, unlike the Eurocode, which gives a range between 1,400 and 1,900 kg/m^3 (*NEN EN 1991-1-1: General Actions - Densities, self-weight, imposed loads for buildings*, 2011). Judging from the Eurocode, a value of 1,700 kg/m^3 is used to determine the resulting soil loads.

Water level
During a 50 year ARI flood event, the Swan Creek Levee will overtop. This leads to a water level of 0.5 m above the levee crest, according to the Clarence Valley Floodmodel provided by BMT WBM (Farr and Huxley, 2013). This will be the reference water level in the SCIA Engineer model, since it is assumed that a flood occurs in the governing load combination for strength checks. For stability, high water level in the river (without overtopping the levee) and no water in the creek will be the governing case. This load case is not realistic, but it is a worst case scenario which will most likely never occur in reality. For piping, a head difference of 4.88 m will be assumed, according to the technical drawings of the Swan Creek Floodgate (Clarence River County Council, 1970s*a*).

C.4.2. Determination of Loads and Load Combinations

Self weight, soil loads and water loads are the static loads on the floodgate. The floodgate is an element of the Swan Creek Levee system. In order to maintain the water retaining function, the levee continues over the floodgate. This results in a 3 m sand layer on top of the structure. The inlet and outlet walls also serve as earth retaining elements.
The structure is mostly underground, so wind loads are not applicable. Temperature loads, fire loads and collision loads are not taken into account for the same reason. Earthquakes are not present on the east coast of Australia. Temperatures rarely drop below zero degrees Celsius, so frost and snow loads are also neglected. There is no through road on the levee. One or two cars can pass over the floodgate towards the agricultural land, but this is very uncommon. Also the dead weight of a car is way lower than the soil weight on top of the floodgate and therefore neglected.

The normative load combinations are extracted from the Australian code (Australian New Zealand Standard, 2002). For the Swan Creek Floodgate the dead loads of Equation C.3 and the live loads of Equation C.4 can be normative and are implemented in the SCIA Engineer model. Equations C.3 and C.4 are ultimate limits state and are used for the strength verification of the structure such as the bending moments and shear forces. For the anchorage length, the support reactions in ultimate limit state have been used, see Section C.6.1. Verification of the deflections is done in serviceability limit state. For the serviceability limit state, all the load factors are equal to 1.

$$E_{d,stb} = 1.35 * G \quad \text{(If dead loads are normative)} \tag{C.3}$$
$$E_{d,stb} = 1.2 * G + 1.5 * Q \quad \text{(If live loads are normative)} \tag{C.4}$$
$$E_{d,stb} = 1.2 * G + W_u + \gamma_c * Q \quad \text{(If wind is normative)} \tag{C.5}$$
$$E_{d,stb} = G + E_{us} + \gamma_E * Q \quad \text{(If earthquakes are normative)} \tag{C.6}$$
$$E_{d,stb} = 1.2 * G + S_u + \gamma_c * Q \quad \text{(If imposed actions are normative)} \tag{C.7}$$

The load combinations have been implemented in SCIA Engineer, SCIA automatically uses the governing load combination for values.

C.4.3. Model Swan Creek Floodgate (SCIA Engineer)

In the current design, the floodgate and the outer walls are constructed separately from each other and do not interact. In order to check the strength of the current structure, the elements have been created separately from each other. A possible solution might be attaching the outer walls and inner walls to the floodgate to avoid tilting of the walls. By connecting the two members, peak bending moments will occur at the upper part of the connection between the outer walls and the floodgate. If the structure has sufficient reinforcement to withstand the bending moment, this might be a good solution. However, this would mean that the structure should have been poured as a whole at once.

The floodgate is founded on a shallow foundation. The total settlement of the floodgate cannot be determined, because probing figures are not available. The student version of SCIA Engineer does not have the ability to use soil supports, so line supports with a spring constant are used to simulate the same behaviour. The spring constants of the structure are necessary to avoid peak values at the supports. The line supports are not a correct estimation of reality, however the total settlement of the structure is not of importance. SCIA Engineer is used to determine the maximum and minimum horizontal settlements of the walls, bending moments and shear forces in the critical areas of the entire structure. The construction drawn in SCIA is given in Figure C.4.

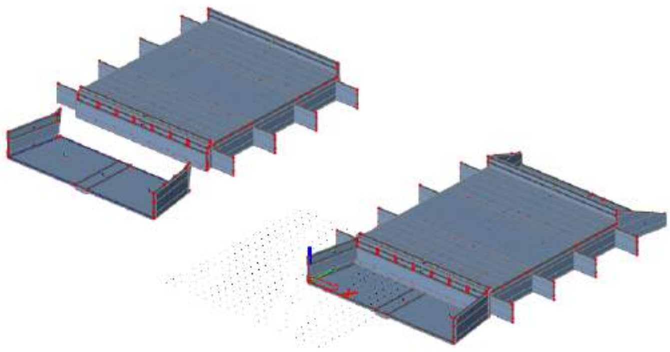


Figure C.4: The structure such as it is drawn in SCIA Engineer. On the left, the outlet slab and walls are shown separately from the floodgate. On the right, the structure is shown as with all element attached to each other.

The model will be calculated with the Mindlin-Reissner plate theory, because the elements are relatively thin and use a linear stress distribution in the elements.

C.5. Results

C.5.1. Results from SCIA Engineer

Interpreting the results from SCIA gives the normative forces $M_{xD}+$ (Bending moment) and q_{maxb} (shear force). The maximum bending moment in the floodgate is approximately $250kNm/m$ and occurs in the top slab of the floodgate. The thickness of the slab is 343 mm at the place of the maximum bending moment. The bending moments are not significant in the outer walls. The maximum Shear force is approximately $900kN/m$ for the floodgate near the edges of the top slab. Both forces are present in the permanent load combination. According to the Eurocodes, the shear force can be mediated over a distance of twice the effective thickness of the slab, the average shear force is in this case approximately $750kN/m$. The shear force in the slab of the outer walls is determined in the same way, this results in a shear force of approximately $750kN/m$. The thickness of the slab underneath the outer walls is 254 mm thick and the top of the floodgate slab is 343 mm thick. The slab of the outer walls will be normative and be tested on shear force. To ease the calculations, the Eurocode is chosen as reference material.

The reinforcement on the critical point can be determined with Figures C.15 and C.16. For the calculations, the minimal amount of reinforcement to withstand the bending moments and shear forces will be calculated. With Microsoft Excel the minimum amount of longitudinal reinforcement per meter and transverse reinforcement per meter has been determined. This resulted in eight longitudinal reinforcement bars with a diameter of 20 mm and eight transverse reinforcement brackets with a diameter of 12 mm.

Bending Moment resistance

For a normative reference of the bending moment resistance, see (*NEN EN 1992-1-1+C2:2011: ontwerp en berekeningen van betonconstructies*, 2011). The bending moment resistance is calculated by using the horizontal equilibrium and the moment equilibrium in the concrete. The calculation has been made with Microsoft Excel. The unity check (UC) with the applied reinforcement is given below.

$$UC_{moments} = 0.97 \tag{C.8}$$

The amount of transverse reinforcement steel is sufficient, because the requirement UC < 1, applies.

Shear resistance

For the normative reference of shear resistance, see (*NEN EN 1992-1-1+C2:2011: ontwerp en berekeningen van betonconstructies*, 2011). The shear resistance of the concrete is determined with the guidelines of the Eurocode. The concrete is not sufficient to carry the shear force, transverse reinforcement is required.

$$UC_{concrete} = 3.17 \tag{C.9}$$

The concrete cannot carry the shear force and transverse steel reinforcement brackets are required.

$$UC_{reinforcedconcrete} = 0.92 \tag{C.10}$$

With these brackets, the shear force resistance of the structure is sufficient.

C.5.2. Stability of the Structure

Three stability checks will be performed for the Swan Creek FLoodgate: horizontal; vertical and rotational stability. High water levels in the river, without overtopping of the levee, will be the governing load case for all these checks. It is assumed that there is no water in the Swan Creek and the floodgates are closed on the river side, so there is no water in the box culverts. These are very conservative assumptions, because water will always be present in the creek in case of high water level, due to the fact that the cause of a flood is severe rainfall. But once again this is considered as the extreme case scenario.

Figure C.5 shows all the vertical and horizontal forces acting on the structure, except the vertical upward force, which will be treated separately. To be complete, it is shown that water is present in the creek. In the case of no water in the creek, h_{creek} will be zero. Horizontal soil forces (in x direction) have no influence on stability and have been neglected.

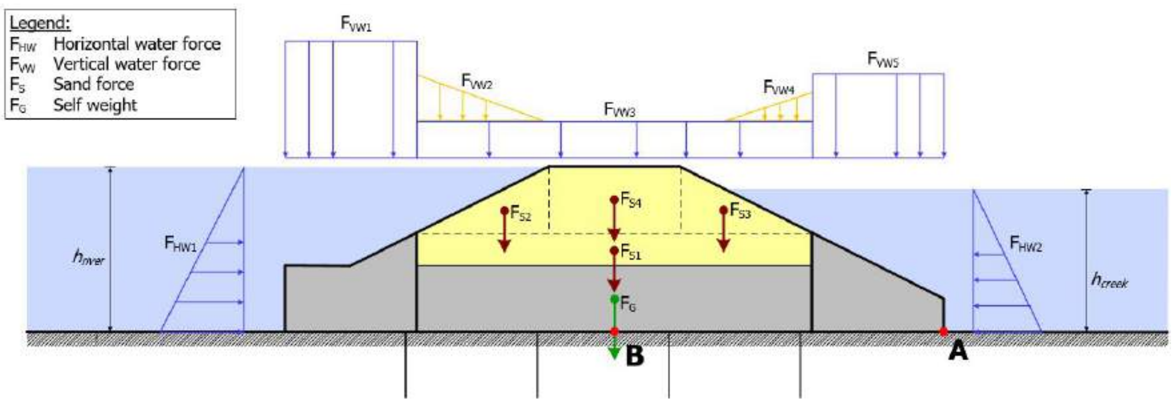


Figure C.5: Horizontal and vertical forces on the Swan Creek Floodgate.

Both F_{VW1} and F_{VW5} represent the vertical downward force due to water on the inlet and outlet slab, where the inlet slab is located on the creek side and the outlet slab is located on the river side. F_{VW3} indicates the force due to water in the floodgates itself. The value of this force is limited by the height of the culverts, which is 2.13 m. The water level in the culverts is determined by the water level in the creek, at the inlet side of the floodgate. F_{VW2} and F_{VW4} are downward forces due to the water on top of the sand of the levee that crosses the floodgate. If the water level is below the height of the face of the floodgate on the river side, which is 4.3 m, the F_{VW2} will be zero. The same holds for F_{VW4} on the creek side of the floodgate.

The sand layer on top of the structure is divided into four parts (F_{S1} to F_{S4}) to determine the total weight. F_G indicates the total weight of the floodgate itself. The total volume of the concrete is indicated in the technical drawing and amounts 487.25 m^3 (Clarence River County Council, 1970sa). The Australian Codes indicate a self weight of concrete of 24 kN/m^3 (Australian New Zealand Standard, 2002).

If it is assumed that h_{creek} is equal to zero, several water forces will be excluded. F_{HW2} is not a relevant horizontal force any more. F_{VW3}, F_{VW4} and F_{VW5} will become zero. There will still be a vertical upward force due to water pressure along the whole structure. This force depends on the seepage length of the floodgate. Four sheet piles are present to increase the seepage length. These sheet piles also influence the upward water pressure, which is shown in Figure C.6.

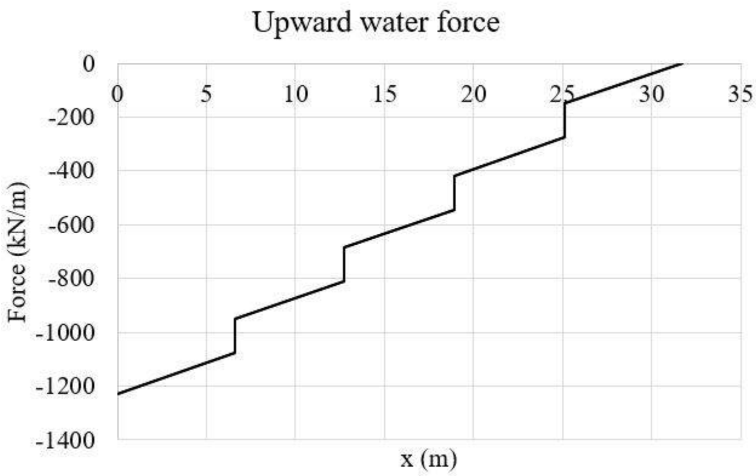


Figure C.6: The upward water force in case of no water in the creek and maximum water in the Clarence River. $x = 0$ indicates the edge of the outlet slab on the river side. The inlet slab ends at $x = 31.7m$.

Sum of forces and moments

To evaluate all the stability checks, the total sum of horizontal and vertical forces must be known, as well as the sum of moments. For these checks, it is assumed that no water is present in the creek ($h_{creek} = 0$) and high water occurs in the river. This water level is assumed to be at the crest of the levee, which means $h_{river} = 7.39m$.

The only horizontal forces acting on the floodgate are the forces due to water pressure. In the case of no water in the creek, only F_{HW1} will be present.

Table C.1: The horizontal forces on the Swan Creek Floodgate, see figure C.5.

Horizontal Forces in kN	
F_{HW1}	4538.12
F_{HW2}	0.00
ΣF_H	4538.12

Three kinds of vertical forces are acting downward: vertical water loads; the dead weight of the sand on top of the deck of the floodgate and self weight of the structure. The total upward forces due to water pressure are denoted as F_{UW}. The values are all listed in Table C.2, including the total sum of vertical forces.

Table C.2: Vertical forces on the Swan Creek Floodgate, see figure C.5.

Vertical Forces in kN	
F_{VW1}	7860.19
F_{VW2}	1582.18
F_{VW3}	0.00
F_{VW4}	0.00
F_{VW5}	0.00
F_{S1}	7739.15
F_{S2}	2689.71
F_{S3}	2689.71
F_{S4}	5313.18
F_G	11 694.00
F_{UW}	−19 462.60
ΣF_V	20 105.52

The total sum of moments is needed to check the vertical and rotational stability of the structure. The sum of moments is taken around point B, which is in the middle of the structure. First, all the distances from the individual forces have been calculated with respect to point A. Next, the distance between A and B (15.62 m, see Figure C.3) has been subtracted from all the distances. This method is used, because distances between the sheet piles define the location of the resultants of the upward water pressure forces. It is much easier to calculate the distances from these nine forces to an edge (point A) than to a random point (point B), since the sheet piles are not evenly distributed along the structures length.

Table C.3 shows all the relevant forces for the total moment calculation. Three resultant forces go through point B (F_{S1}, F_{S4} and F_G), so they have no lever arm and can be excluded. F_{S2} and F_{S3} have the same value and the same lever arm, which means they cancel each other out. F_{VW3}, F_{VW4}, F_{VW5} and F_{HW2} are still zero, so these do not have to be taken into account when the sum of moments is evaluated. The sum of moments of the upward water force is determined for every one of the nine parts. Table C.3 shows the resultant lever arm for the total moment, which has been calculated by dividing the total moment by the total force due to the upward water pressure.

Table C.3: Moments due to relevant forces on the Swan Creek Floodgate.

Name	Force in kN	Lever arm in m	Moments in kNm
F_{HW1}	4538.12	−2.46	−11 180.87
F_{VW1}	7860.19	12.65	99 423.58
F_{VW2}	1582.18	7.16	11 332.64
F_{UW}	−19 462.60	5.97	−116 267.40
ΣM			−16 692.06

Horizontal stability check

The horizontal stability depends on the total sum of the vertical and horizontal forces. The friction coefficient between the soil and concrete relates these forces to each other:

$$\Sigma F_H < f * \Sigma F_V \tag{C.11}$$

The value of f depends on the kind of soil the structure is constructed on. It is assumed that medium sand is present at the location of the floodgate. This results in a value of 0.5 for the friction coefficient

(Molenaar and Voorendt, 2018). Using the calculated values for the sum of forces, the following unity check is obtained for horizontal stability:

$$UC_H = 0.45 \tag{C.12}$$

In conclusion, the structure will not fail on horizontal stability.

Vertical stability check

Two kinds of vertical stability checks must be performed. First of all, the maximum stress must not exceed the bearing capacity of the soil. Again, the bearing capacity depends on the type of soil. For sand, a value of 400 kN/m^2 can be assumed (Molenaar and Voorendt, 2018). The maximum stress depends on the vertical forces and moments:

$$\sigma_{max} = \frac{\Sigma F_V}{A} + \frac{\Sigma M}{W} < p'_{max} \tag{C.13}$$

Here, A indicates the total area of the bottom plate of the floodgate. The total length is 31.699 m and the width is 16.97 m, resulting in a total area of 537.93 m^2. W represents the section modulus for a square plate, which is 1,521.45 m^3 in this case. This results in the following unity check for vertical stability, considering the bearing capacity of the soil:

$$UC_{V,max} = 0.12 \tag{C.14}$$

It must also be validated whether the minimum stress is positive. A negative minimum stress would result in tensile forces in the soil, which is not possible.

$$\sigma_{min} = \frac{\Sigma F_V}{A} - \frac{\Sigma M}{W} > 0 \tag{C.15}$$

All the values are the same that are used for Equation C.13. The following unity check is obtained:

$$UC_{V,min} = 0.29 \tag{C.16}$$

Both the minimum and maximum vertical stress checks result in low unity checks, which means the floodgate will also not fail on vertical stability.

Rotational stability check

The rotational stability depends on whether the resultant force of the structure intersects the core of the structure. The core of the structure is located around the centre, with $\frac{1}{6}^{th}$ of the structure's total width on each side of the middle (which is point B in this case) (Molenaar and Voorendt, 2018). If the resultant does not go through the core, the soil will collapse and the structure will fail. The resultant force depends on the total sum of vertical forces and the sum of moments around the centre of the structure (point B). In conclusion, the following check must be performed:

$$\frac{\Sigma M}{\Sigma F_V} < \frac{1}{6} * b \tag{C.17}$$

Here, b indicates the structure's width. Due to the fact that the bottom plate of the floodgate is a rectangle, this check is actually the same as the vertical stability check for the minimum vertical force. This means the unity check will also be the same:

$$UC_{rot} = 0.29 \tag{C.18}$$

In conclusion, all the unity checks are below 1, so the floodgate will not fail based on stability checks.

Piping

Piping (internal backwater erosion) is examined for the Swan Creek Floodgate as a part of the section stability. Empirical formulas based on research describe the critical situations in which piping can occur. The most famous are the Bligh and Lane formulas. These formulas set a limit state between the differential head and the seepage length (Molenaar and Voorendt, 2018). Both methods, Bligh and

Lane, are being applied at the floodgate to examine the occurrence of piping. It should be realised that the Bligh and Lane formulas are inaccurate, but they show a first estimation of possible piping problems.

Total seepage length:

$$Bligh : L = \Sigma L_{vert} + \Sigma L_{hor} \tag{C.19}$$

$$Lane : L = \Sigma L_{vert} + \Sigma \frac{1}{3} L_{hor} \tag{C.20}$$

The criterion for Bligh and Lane are shown below:

$$Bligh : L \geq \gamma * C_B * \Delta H \tag{C.21}$$

$$Lane : L \geq \gamma * C_L * \Delta H \tag{C.22}$$

Maximum (allowed) hydraulic gradient:

$$i_{max} = \frac{\Delta H}{L} \tag{C.23}$$

Where:

- L [m] = Total seepage distance, which is the distance through the soil where the water flow is impeded by the soil structure
- γ = safety factor
- C_B = Bligh's constant, depends on soil type
- C_L = Lane's constant, depends on soil type
- ΔH [m] = Differential head across the structure

The manual hydraulic structures (Molenaar and Voorendt, 2018) states two basic conditions which have to be met before piping can occur:

- The duration of the water level difference has to be sufficiently long to start this mechanism.
- Sand particles must have a possibility to extrude. So, if the ground level at land side of the structure is relatively high, piping becomes unlikely, just like in case of a closed and sufficiently strong surface. Before erosion can start, rupture of the top layer should occur. In case of low-permeable cohesive soil layer this phenomenon is called uplift; for sand (aquifer material) the term heave is used.

The height difference and the duration of the height difference, between the Clarence River and the Swan Creek depend on many factors: The situation with tidal effects but without a flood, is the most common situation and has the longest duration. The tidal effect at the location of the floodgate causes a head difference of maximum +/- 0.6 mAHD (Farr and Huxley, 2013).

During a flood, including local rainfall in the catchment of the Swan Creek the head difference between river side and land side will be limited. During a flood without local rainfall in the catchment of the Swan Creek, the head difference will be the largest. The water level in the river can rise but the water level of the Swan Creek could stay unchanged.
After flooding of the entire Swan Creek Floodplain and the Clarenza Floodplain, the area is drained by the Swan Creek through the floodgate. A differential head occurs in the opposite direction. The actual value of this differential head is not defined. The duration of this event could take several weeks to several months.
The second condition stated by (Molenaar and Voorendt, 2018) is met, because the ground level at land side is almost equal to the ground level at the river side. The floodgate is constructed horizontally, without an inclination.
The assumption is made that the Swan Creek Floodgate is constructed on a base of fine sand. The seepage length for Bligh and Lane are determined in this study at L = 55.45 m and L = 34.32 m respectively. A boundary value is calculated to see for which differential head across the structure the

criteria do not suffice. The formulas to calculate the boundary values are shown below:

$$\Delta H \leq \frac{L}{\gamma * C_B} \tag{C.24}$$

$$\Delta H \leq \frac{L}{\gamma * C_L} \tag{C.25}$$

The boundary value for the criteria of Bligh is at: ΔH = 2.46 m calculated with Equation C.24 and for Lane at: ΔH = 3.27 m calculated with Equation C.25.

A designed head of 4.88 m is specified for the floodplain according to the Clarence River County Council, see (Clarence River County Council, 1970sa). Both the criteria for Bligh and Lane are not met for the designed head of 4.88 m, with the seepage lengths calculated in this study. The first impression of the formulas, lead to possible piping problems. Further research is needed to get an actual prove for the occurrence of piping. Data about the fluctuating water level of the Swan Creek is important to obtain more specific differential heads.

C.6. Possible Solutions for the Current Situation

C.6.1. Anchorage

The displacements of the walls are inwards in horizontal direction according to the results from SCIA Engineer are given in Figure C.7.

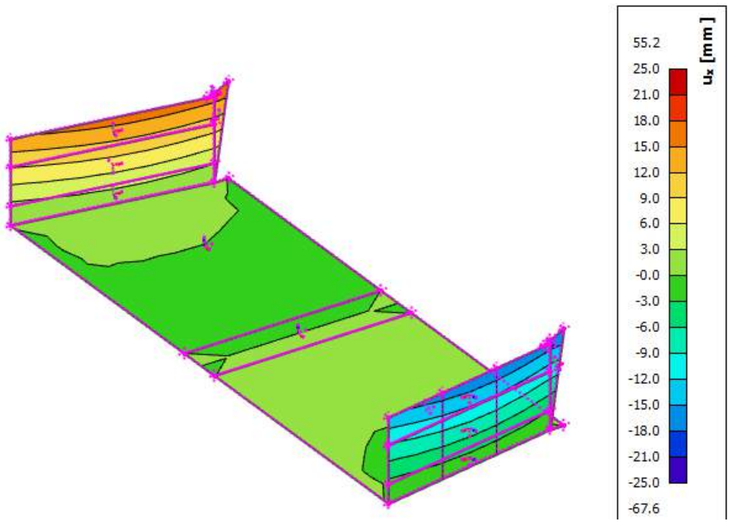


Figure C.7: The displacement of the outlet walls in horizontal (x) direction.

Grout anchors are examined for this problem. This anchor type has the highest bearing strength compared to other anchorage types and is the most common method to use. The required installation space is also low compared to other anchorage methods. Budget wise they are not favourable compared to other anchorage methods. Normally, grout anchors have a diameter between 110 mm and 200 mm and a length between 4 m and 20 m (Molenaar and Voorendt, 2018). The model made in SCIA can be used to verify the anchors. Two supports in horizontal direction have been added in the top part of the outlet wall. The supports are placed 0.5 m below the upper edge of the wall. The supports have been placed on $1/3$ and $2/3$ of the length of the walls, see Figure C.8. The horizontal displacement, see Figure C.7, is prevented in the top part of the outlet wall by the added supports. The resulting support reactions of the supports will be related to the tensile strength in the anchors. The resulting forces are $208.4kN$ for support 1 towards the river and 316.5 kN for the support 2. The anchor will be placed under an angle of 45°. This results in a minimum tensile forces in the anchor of $316.5 * \sqrt{2} = 448kN$ and $216.35 * \sqrt{2} = 295kN$.

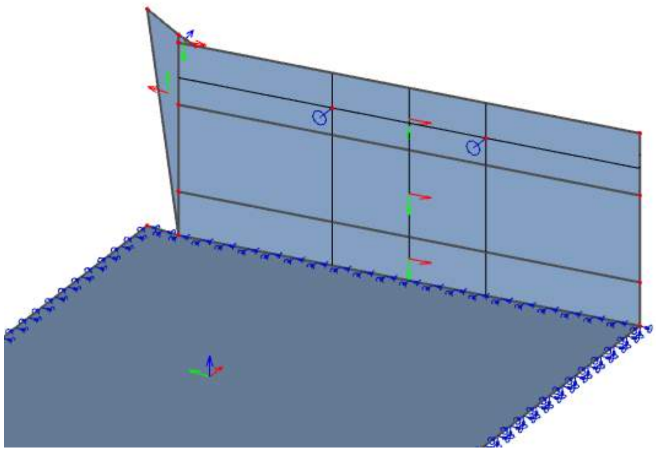


Figure C.8: The supports (in blue), which are used on the wall to verify the anchors.

The strength of the grout anchor depends on the soil type, the resistance of the surrounding soil determines the strength of the anchor. The minimum cone resistance of sand is 5 N/mm^2. The minimum strength of the grout anchors per meter ($f_{k,rep}$) is 65 kN/m in a sandy soil, (Bouw&Infra, 2014). According to the (Bouw&Infra, 2014), failure tests for anchors should be performed which lead to extra safety factors. If one failure test is performed, a reduction value $\epsilon_a = 1.39$ and a partial material factor $\gamma_a = 1.35$ need be applied. The required length of the anchors is determined below:

$$R_{a,d} = \frac{R_{a,k}}{\gamma_a} \rightarrow R_{a,k} = R_{a,d} * \gamma_a \tag{C.26}$$

$$R_{a,k} = \frac{R_{a;min}}{\epsilon_a} \rightarrow R_{a;min} = R_{a,k} * \epsilon_a \tag{C.27}$$

$$R_{a;min} = f_{k,rep} * L_a \rightarrow L_a = \frac{R_{a;min}}{f_{k,rep}} \tag{C.28}$$

Where:

- $R_{a,d}$ is the design value of the strength
- $R_{a,k}$ is the characteristic value of the strength
- $R_{a,min}$ is the minimum value of the strength

Table C.4: Calculation of the required anchor length.

	Minimum tensile force in kN ($R_{a,d}$)	$R_{a,k}$ in kN	$R_{a,min}$ in kN	L_a
Anchor 1	448	604	840	13
Anchor 2	295	398	553	9

The required lengths of the anchors are rounded to 13 m and 9 m in order to avoid further displacements of the wall in horizontal direction. These lengths have been rounded upwards, and are realistic dimensions for grout anchors, according to (Molenaar and Voorendt, 2018).

C.6.2. Seepage Length

According to the criteria of Bleigh and Lane, piping might occur near the Swan Creek Floodgate. One way to decrease the possibility of piping, is to add sheet piles to increase the seepage length. Rewriting Equation C.24 and Equation C.25 leads to the following two equations to determine the required seepage length:

$$L \geq \Delta H * \gamma * C_B \tag{C.29}$$

$$L \geq \Delta H * \gamma * C_L \tag{C.30}$$

Using a design head difference of 4.88 m, the minimum required seepage length for Bleigh's criterion will be 109.8 m . Lane's criterion results in a minimum seepage length of 51.24 m. If Bleigh's criterion is considered as governing, the seepage length has to be increased by 54.35 m. It is not possible to increase the length of the existing sheet piles, because they cannot be reached due to the floodgate above. New sheet piles could be added at the edges of the inlet and outlet slab, so two sheet piles in total. The new sheet piles will require a length of at least 13.59 m to obtain the required seepage length according to Bleigh's criterion.

C.6.3. Connection of the outlet wall to the main structure

In order to prevent further displacement of the outlet slab and outlet walls, the elements could be attached to the main structure. However, several problems arise when the structure would be retrofitted by this attachment. First of all, significant displacements of the outlet walls have already occurred. This means that gaps of around 20 cm are present between the top of the outlet wall and the floodgate (see also Figure C.2). These displacements are neglected in the drawings presented in this section. Secondly, attachment of the new elements will be difficult. This fact will be further elaborated in the next paragraphs with two design proposals. In this section, no calculations are presented. To investigate the feasibility of the proposed solutions, further research must be performed.

Dowels

Dowels could be implemented between the top of the outlet walls and the face of the structure on the outlet side. A basic sketch of this solution is provided in Figure C.9.

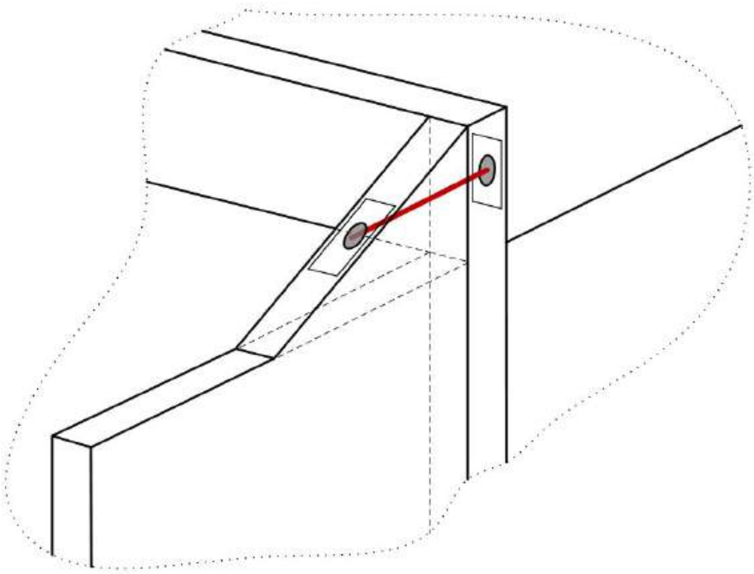


Figure C.9: Retrofitting of the outlet walls using dowels. The new dowel is indicated in red. This figure is not to scale.

Further tilting of the wall could be prevented by the shear resistance of the dowel. Normally, dowels are not used in tension, but if the dowels are fixed in longitudinal direction, further tilting of the whole outlet slab might also be minimised. The attachment might cause high stresses on the concrete, so attachment plates could be used to distribute the stresses. To install the dowels, holes must be drilled through the outlet wall and the face of the main structure. To fully attach the dowel to the back of the face of the floodgate, the sand on top of the culverts must be removed to be able to reach the attachment point.

Angle brackets
The outlet walls could be attached to the outlet slab by angle brackets. These might prevent further tilting of the outlet walls. Angle brackets could also be used to connect the outlet walls to the face of the culverts. Steel connection plates can be implemented on the side of the floodgate, also to connect the outlet walls to the main structure. Figure C.10 gives an overview of locations where the angle brackets and steel plates could be applied.

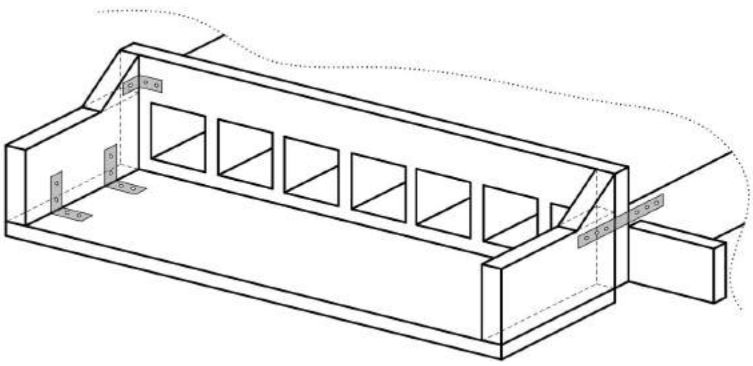


Figure C.10: Retrofitting of the outlet walls and outlet slab using angle brackets. This figure is not to scale.

Attachment of the angle brackets between the outlet wall and outlet slab has to be performed in water, which is not preferable. Since the steel connectors are in a wet environment, corrosion problems might occur. It might be useful to investigate whether other less corrosive sensitive materials, such as Fibre Reinforced Plastics (FRP), could be applied.

The steel connection plates on the side of the structure have to be installed on a section of the floodgate that is covered by sand. The plates need to be placed above the transverse walls that are used for horizontal stability.

C.6.4. Possible solution for a new design of the floodgate
In order to solve the displacement and tilting problems of the outlet walls, a proposed solution is to connect the walls to the floodgate. If the inlet and outlet walls are attached to the floodgate as a whole, the Bending moment will be approximately $1300kNm/m$ and the normative shear force will be approximately $1150kN/m$. Both these forces occur at the top of the connection between the walls and the floodgate. The walls have not been supported in this scenario and want to deflect downwards on the outer point. The floodgate prevents this and causes high moments and shear forces in the connection. These high moments are visible in Figure C.11

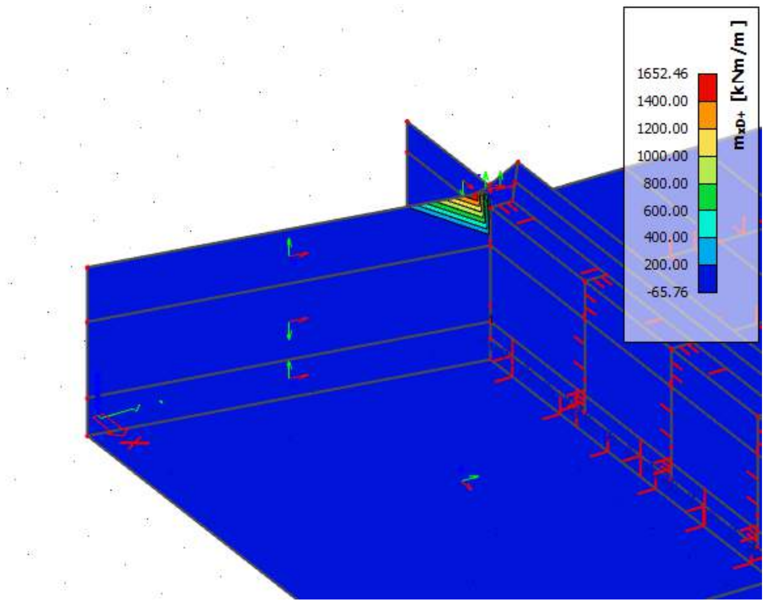


Figure C.11: Peak moments in the upper corner of the walls due to the attachment to the floodgate.

The bending moment in the connected version of the structure is $1300kNm/m$. Since the walls and slabs of the structure are too thin, not enough reinforcement can be implemented in the walls and slabs to carry the load. The idea of attaching the floodgate and the inner and outer walls has been rejected. A possible thicker connection might be a solution for a new floodgate with thicker walls, but has not been investigated.

C.7. Discussion

The Swan Creek Floodgate has been subjected to several stability and strength checks. However, due to a lack of time and resources, there are some limitations to the verification of the floodgate, which will be discussed in this section.

The main restrictions for a solutions of the Swan Creek Floodgate is a lack of research. The exact reason why the walls started tilting is unknown. Since the event started eleven years after construction, the reason is probably based on erosion. However, the kind and the location of the erosion is unknown. Due to these uncertainties erosion haven't been assessed in this report.

Loads

The structure is assumed to be at rest. The movable mechanism of the floodgate has not been taken into consideration in the design and verification of the floodgate. Fatigue, creep and shrinkage have not been taken into account for the concrete calculations, which could lead to deviations in the results.

The loads used for the verification of the structure are not realistic. The loads are surrealistic and would never occur in real life. Even with these ridiculous high loads, the safety checks are sufficient, meaning the entire structure is heavily overdimensioned.

Scia Engineer model

The supports in the SCIA model are line supports on the edges of the elements, because soil supports are not an option for the student version of SCIA Engineer. The spring constant of the line supports is higher than the spring support of soil. Peak values might occur in the field, because the elements are only supported on the edges. Displacements in x direction for anchorage differ as well due to the different way of supporting the structure. Errors from the real situation might occur due to the different way of supporting the structure.

The model in SCIA might lack precision in the geometry, since SCIA draws walls and floors in a 2D plane. The thicknesses might be included or excluded when walls and floors are connected. Due to this, the loads on the structure might differ, since loads cannot be placed on the thicknesses of the walls and floors. However, these precision errors are only small errors and the SCIA model is a reasonable representation of the floodgate in practice. Due to the lack of time, the SCIA model has not been checked on small errors in geometry. The model is verified by looking if the results are reasonable and using common sense.

Since the technical drawings of the reinforcement are difficult to interpret, the minimum required reinforcement in the structure is determined with the results from SCIA. Reasonable amounts for a structure of this size were obtained. In this way, a comparable floodgate can be constructed based on this verification of the old floodgate.

Other restrictions to the solutions

The outlet wall has an earth retaining function. However, about one meter of rocks is located next to the wall, which means the anchor should reach through this first rock layer to get to the sand. For the calculations, it is assumed that the sand layer starts immediately at the wall. Enough safety factors have been applied for a safe calculation of the anchors. The anchors are only a solution for the tilting of the outlet walls towards each other. Since the outlet walls also tilt towards the river, the point of attachment of the anchors will also shift. This could result in shear forces, and anchors are designed to only deal with axial forces. No calculations have been performed to check the possible shear forces on the anchors. Shear force might cause failure of the anchors. This has to be checked in a future study.

If new sheet piles will be used at the edges of the inlet and outlet slab of the floodgate, several measures will have to be taken. First of all, the connection between the top of the sheet pile and the edge of the slab must prevent water ingress. If this is not the case, the new sheet pile will make no difference. Furthermore, it must be checked whether the water will still follow the path along the sheet piles and the bottom plate of the structure. Due to the high length of the sheet piles and the short distance between the new sheet pile and the existing sheet pile, it might be possible that the water will choose a shorter seepage path then expected.

No cost benefit analysis has been performed. Cost wise it might be unfavourable to use such long sheet piles to increase the seepage length. It is also not known what kind of soil layers are present underneath the floodgate. If rock layers are present above a depth of 13.59 m (which is the sheet pile length), it is not possible to implement sheet piles of that length. Of course, if sheet piles will reach to possible rock layers, piping will also be prevented, since these layers are virtually impermeable.

No solutions have been provided for tilting of the outlet slab. If piping is the cause of washout of soil underneath the slab, further tilting could be prevented by the new sheet piles, because piping would be prevented. If other causes are the case, different solutions might be needed. For example, foundation piles could be retrofitted to the structure, to prevent further displacements of the outlet slab. Further research will be needed to asses this solution.

An assessment on scour of the rock armour has not been done for the floodgate. Crucial information is missing to preform a representative scour check. Field measurements have to be executed to obtain the right data.

C.8. Conclusion

The current Swan Creek Floodgate has been subjected to strength and stability calculations. All other verification checks have a unity check between 0 and 1. Piping might occur according to the approximation formulas of Bleigh and Lane, causing severe displacements to the structure. However, the duration of the critical water level difference is unknown in this study, so therefore no detailed piping analysis can be made.

C.9. Recommendations

Further research must be performed to be able to provide a full solution to the Swan Creek Floodgate problem. First of all, it is not known what kind of soil is present near the floodgate. Assumptions have been made regarding soil parameters, such as the bearing capacity and the density. It is also not known whether rock layers are present and if so, at what depth they occur. This could highly influence the new design of the sheet piles.

Piping could be prevented by implementing two new sheet piles The inwards tilting of the outlet walls might be solved by attaching grout anchors to the outlet walls. To reduce the anchor length, two anchors can be used on each of the outlet walls.

No calculations have been performed regarding scour protection. The technical drawings (Clarence River County Council, 1970s*a*) provide the location and size of the rock armour near the inlet and outlet slab. Research has to be done to investigate what kind of rocks (grain size, composition) are present, in order to perform scour protection calculations. Also, no calculations have been performed regarding the possible connections between the outlet walls and main structure. These design calculations need to be done in order to present a appropriate solution.

For a better representation of the reality, it might be useful to use soil supports in the SCIA Engineer model. This function is not supported by the student version used for this verification of the structure.

C.10. Technical Drawings

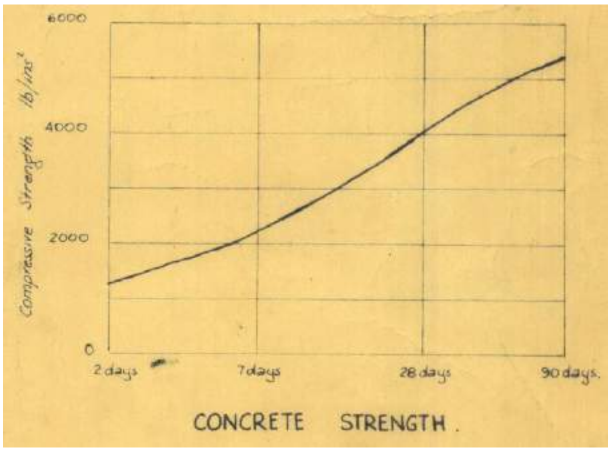


Figure C.12: Concrete strength of the Swan Creek Floodgate (Clarence River County Council, 1970sc).

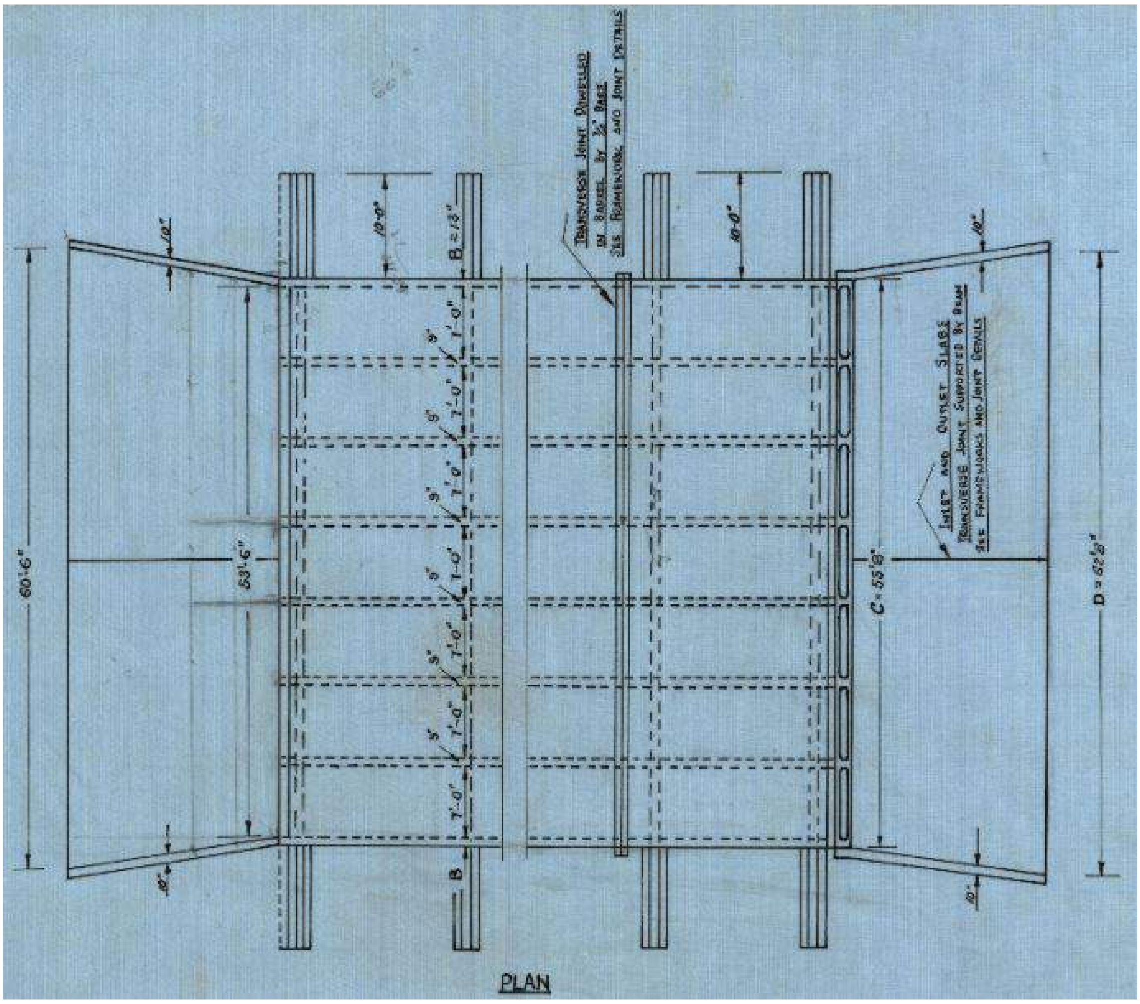


Figure C.13: Top view of the Swan Creek Floodgate, including dimensions (Clarence River County Council, 1970sb).

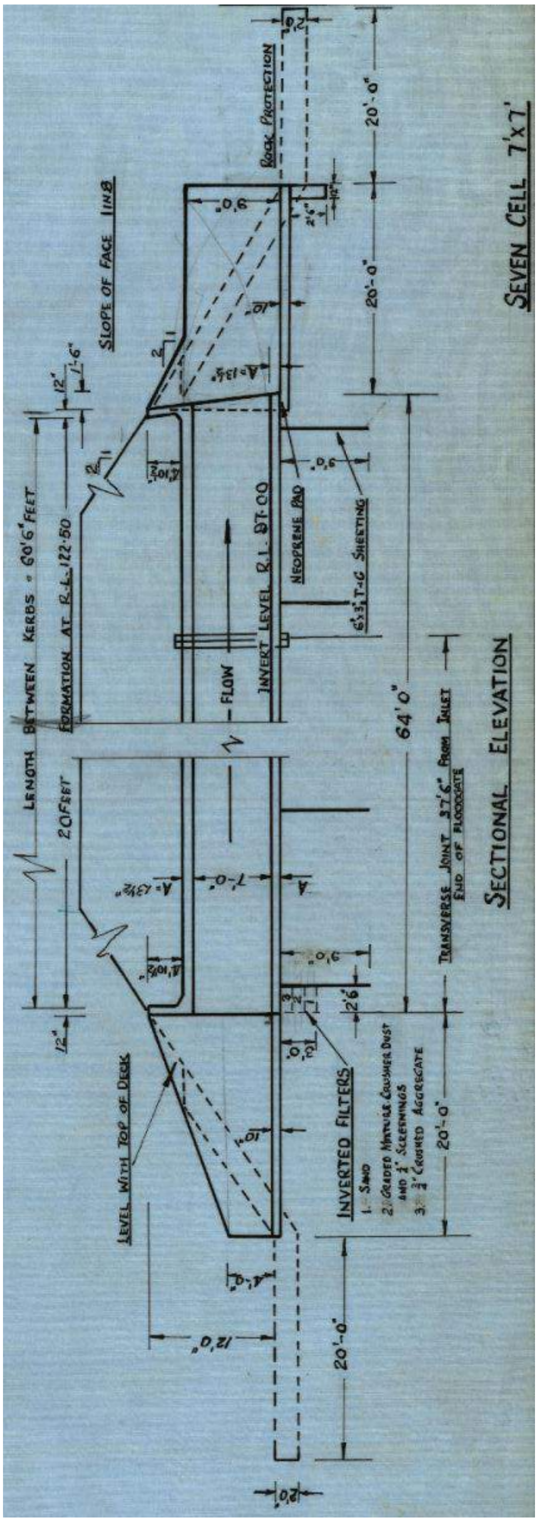


Figure C.14: Cross-sectional dimensions of the Swan Creek Floodgate (Clarence River County Council, 1970sb).

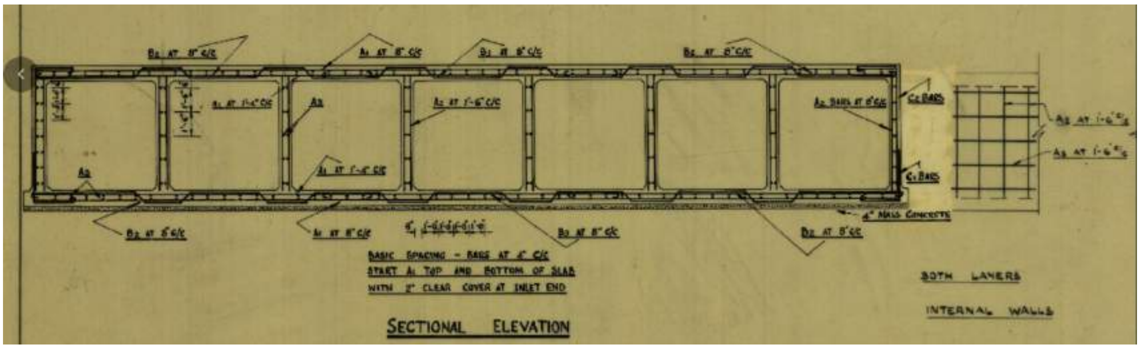


Figure C.15: The reinforcement in the floodgate (Clarence River County Council, 1970sb).

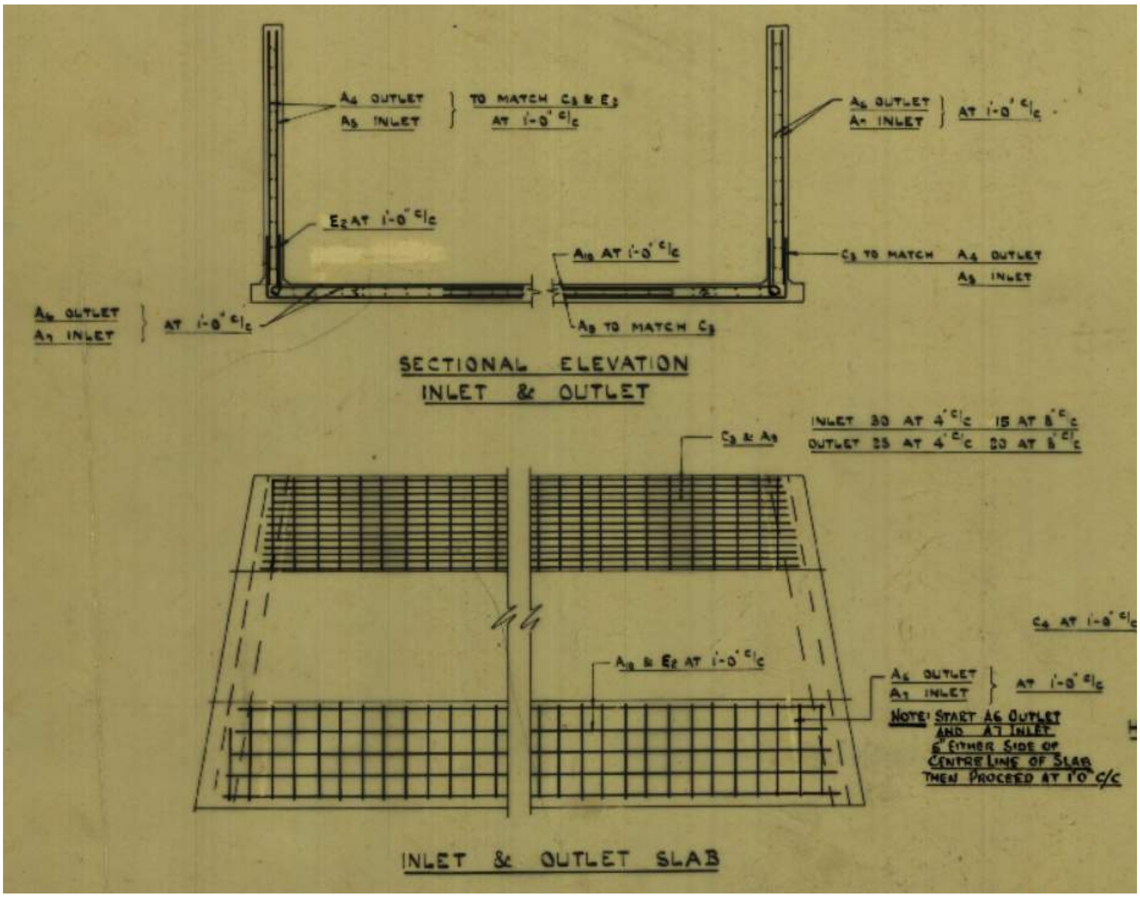


Figure C.16: Reinforcement in the outer and inner walls and slabs (Clarence River County Council, 1970sb).

D

Maclean Levee

The Maclean Levee consists of earth walls and reinforced concrete (RC) levee walls. Section D.2 focuses only on the RC levee walls. Section D.3 focuses on the event of overtopping the Maclean Levee.

D.1. Problem Statement

The RC part of the Maclean Levee is placed on top of a sheetpile wall. The levee is also connected to a earth levee. During a flood, water seems to boil up from the ground on the land side of the river wall, according to Appendix B. This phenomena can be seen as piping. For this Appendix the piping problem is going to be assessed. Chapter 2 also showed overtopping problems during a 50 year ARI flood event. This overtopping is minor and has a minor change of occurrence, so therefore it is no subject to large research. Other reasons are, higher grounds are nearby and warning time is sufficient for evacuation. Influences of upstream mitigation adjustments are the main source of solutions for the minor overtopping.

D.2. Piping (internal backwater erosion)

During a flood the potential difference across the structures is the largest, because of the large water depth at the river side and no water at the land side of the levee wall.The problem of piping could occur during a flood, according to Appendix B behind the RC levee wall. The RC levee walls differ in height and structural dimensions. The levee walls are designed in dimensions of feet and inches but are converted to meters. The dimensions are obtained from the technical drawings (Clarence River County Council, 1973). The concrete levee walls have a height of 1.63-2.64 meters and have a sheetpile wall underneath. The earth levees have a height of 1.02-1.63 meters, but do not include a sheetpile wall.

Groundwater flow under wall

The approximation formulas of Bligh and Lane are applied at the Maclean RC levee walls (see Equations C.21 and C.22). The assumption is made that the Maclean Levee Walls are constructed on a base of fine sand, but the exact data is unknown. The sheet pile walls have an assumed length of 20 m. Groundwater flow besides the walls is not examined.

Results

Both methods, Bligh and Lane, are applied at the nine RC levee walls with different heights to examine the occurrence of piping. An overview of the critical head difference for both methods is shown in the Tables D.1 and D.2. The boundary point at ΔH is the desired height of the levee wall to prevent piping:

Table D.1: Difference table, values obtained by Bligh's method.

RC levee wall	With sheet pile wall					Without sheet pile wall			
Height RC levee wall (m)	2.64	2.44	2.24	2.03	1.83	1.63	1.42	1.22	1.02
Boundary point at ΔH (m)	1.89	1.88	1.88	1.88	1.87	0.18	0.16	0.16	0.12

Table D.2: Difference table, values obtained by Lane's method.

RC levee wall	With sheet pile wall					Without sheet pile wall			
Height RC levee wall (m)	2.64	2.44	2.24	2.03	1.83	1.63	1.42	1.22	1.02
Boundary value at ΔH (m)	3.93	3.92	3.92	3.92	3.91	0.31	0.27	0.27	0.20

Discussion
One can see the difference between the height of the levee walls and boundary values for the criteria. For the criteria of Lane, the boundary values lie below the height of the levee walls, for the four lowest levee walls. For the criteria of Bligh all the boundary values of the levee walls lie below the height of the levee walls expect for the 1.83 m levee wall. In case of a flood, the head difference can exceed the boundary value which might induce piping. But for piping to occur the head difference has to be maintained over a certain time-interval. If not, piping will not occur. This time-interval of high waterlevels is unknown, it is depended on the total duration of the floodwave. However, the assumed length of the sheet pile walls and the assumed soil type have a large influence on the criteria.

D.3. Overtopping Maclean Levee

Overtopping of the Maclean Levee occurs in a 50 year ARI flood event and has minor impact on the city of Maclean. Due to the small impact no scenario is being constructed for Maclean (see Chapter 4). However, influences of upstream scenarios on the event of overtopping the Maclean Levee in a 50 year ARI flood event are considered.
In scenario 2.3 (see Figure 5.4) can be seen that for a 50 year ARI flood event, the amount of area inundated is reduced compared to the reference situation. For complete mitigation of overtopping in a 50 year ARI flood event more research is needed.

D.4. Conclusion

For a large part of the RC levee walls the criteria for Bligh and Lane do not fulfil, as well for the maximum (allowed) hydraulic gradient. Large part of the calculated boundary points have a value below the height of the corresponding levee wall. The duration of the critical water level difference is unknown in this study, so no explicit conclusion can be drawn. The first impression of the formulas, lead to possible piping problems. Further research is needed to quantify the location where piping occurs, the qualification of the consequences of piping and the duration of the critical hydraulic gradient.
Overtopping of the Maclean Levee can indirectly be mitigated by upstream measures as is shown in scenario 2.3.

E

Evaluation Clarence Valley Model

In this Appendix the TUFLOW program is being explained. Tips & tricks are given in order to work with TUFLOW.

E.1. Theoretical Background TUFLOW Classic

Both the 1D and 2D configurations are outlined below.

Solving 1D configuration

For solving the 1D SWE the following equations are considered. Equation E.1 is the one-dimensional continuity equation and Equation E.2 is the momentum equation. Both are first order differential equations. The equations include the wave propagation terms, inertia terms and bed friction term.

$$\frac{\partial(uA)}{\partial x} + B\frac{\partial \zeta}{\partial t} = 0 \tag{E.1}$$

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + g\frac{\partial \zeta}{\partial x} + k|u|u = 0 \tag{E.2}$$

Where:

- u = depth and width averaged velocity $[m/s]$
- ζ = water level $[m]$
- t = time $[s]$
- x = distance in x-direction $[m]$
- A = cross-sectional area $[m^2]$
- B = width of flow $[m]$
- k = energy loss coefficient [-]
- n = Manning coefficient [-]
- f_l = form (energy) loss coefficient [-]
- R = hydraulic radius $[m]$
- g = gravitational acceleration $[m/s^2]$

This setup has two unknowns (u & ζ) and two equations, therefore it can be solved numerically.

Solving 2D configuration

The equations below are the two-dimensional continuity equation (E.3) and momentum equation in x-direction (E.4) and y-direction (E.5).

$$\frac{\partial \zeta}{\partial t} + \frac{\partial(Hu)}{\partial x} + \frac{\partial(Hv)}{\partial y} = 0 \tag{E.3}$$

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial t} + v\frac{\partial u}{\partial y} - c_f v + g\frac{\partial \zeta}{\partial x} + gu(\frac{n^2}{H^{4/3}} + \frac{f_1}{2g\Delta x})\sqrt{u^2+v^2} - \mu(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}) + \frac{1}{\rho}\frac{\partial p}{\partial x} = F_x \tag{E.4}$$

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial t} + v\frac{\partial v}{\partial y} - c_f u + g\frac{\partial \zeta}{\partial y} + gv(\frac{n^2}{H^{4/3}} + \frac{f_1}{2g\Delta y})\sqrt{u^2+v^2} - \mu(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2}) + \frac{1}{\rho}\frac{\partial p}{\partial y} = F_y \tag{E.5}$$

Where:

- ζ = water surface elevation $[m]$
- u = depth averaged velocity in x-direction $[m]$
- v = depth averaged velocity in y-direction $[m]$
- H = depth of water $[m]$
- t = time $[s]$
- x = distance in x-direction $[m]$
- y = distance in y-direction $[m]$
- Δx = cell size in x-direction $[m]$
- Δy = cell size in y-direction $[m]$
- c_f = Coriolis force coefficient [-]
- n = Manning coefficient [-]
- f_1 = form (energy) loss coefficient [-]
- μ = horizontal diffusion of momentum coefficient [-]
- p = pressure $[kN/m^2]$
- ρ = density of water $[kg/m^3]$
- F_x = sum of components of external forces in x-direction $[kN]$
- F_y = sum of components of external forces in y-direction $[kN]$

This setup has three equations and three unknowns (u, v and ζ), which can be solved numerically. The equations are second order differential equations and therefore need two boundary conditions. These need to be imposed at the upstream and downstream side of the grid.

E.2. Other Background

Timestep

Computations with a numerical model can become unstable and useless if a critical timestep value is exceeded. This critical timestep criterion is known as the Courant stability criterion. For a 2D-Scheme the Courant number can be calculated by the following formula:

$$C_r = \frac{\Delta t \sqrt{(2gH)}}{\Delta x} \tag{E.6}$$

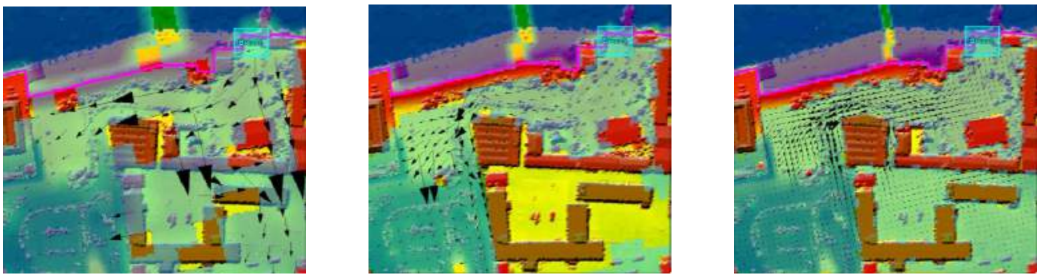
A general assumption for TUFLOW software is that the timestep (seconds) is in the range of proportions of the cellsize (meter):

$$\Delta x/5 \leq \Delta t \leq \Delta x/2 \tag{E.7}$$

This general rule mostly applies for 2D domains. However, in a case with high Froude values and supercritical flow the required timestep might be smaller. One should be aware of the fact that if the above assumption doesn't apply, one should not simply reduce the timestep to get stability. When the timestep has to be decreased significantly there might be an error in the topography, initial conditions or boundary conditions.

Grid resolution

The effect of different cell sizes can be seen in Figure E.1. Here one can see that the hydraulic behaviour is different for large cells compared to smaller cells. The best estimation of the flood behaviour is obviously found with the smallest grid size.



(a) 20 m (b) 10 m (c) 5 m

Figure E.1: Impact of different cell sizes on model results.

Minor flow paths can be presented more coarsely than they really are, or in an extreme case not even be represented. Due to the fact that the flow paths are minor, they play no significant role in the hydraulic behaviour of the model. An example are small drains across a floodplain which may not affect the peak flood level. Those drains may not be necessary in the model. Another issue that should be addressed is the fact that a narrow channel will be presented poorly in a coarse grid, see Figure E.2. To evade this issue the narrow flow path can be modelled as a 1D branch cut through the two-dimensional domain. Multiple methods to do so are presented in (*TUFLOW User Manual*, 2016), section 8.2. The simulation time is not increased as the cell sizes don't have to be reduced.

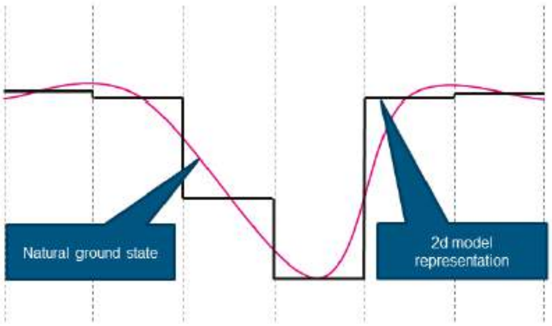


Figure E.2: Poor modelling of a channel in a 2D-model in TUFLOW (*TUFLOW User Manual*, 2016). The 2D model representation does not correspond with the natural ground state, because the model uses a square mesh.

TUFLOW Performance

One way to check the correctness of the TUFLOW simulation results is to look at the model performance indicators. A total of ten indicators are given at the bottom of every .tlf file:

- Total negative depths
- Warnings & checks
- Peak flow in & out
- Volume start & end
- Volume in & out
- Volume error (Final Cumulative ME%)
- Peak +ve and -ve dV
- Peak ddV
- Peak cumulative ME
- Values under Qi+Qo > 5%

More information on these indicators is given in (*TUFLOW User Manual*, 2016). Note, these indicators are not decisive on the correctness of the simulation results. Common sense on the credibility of results is never to be lost in the modelling exercise.

E.3. TUFLOW Input

TUFLOW Classic - Fixed Grid Modelling

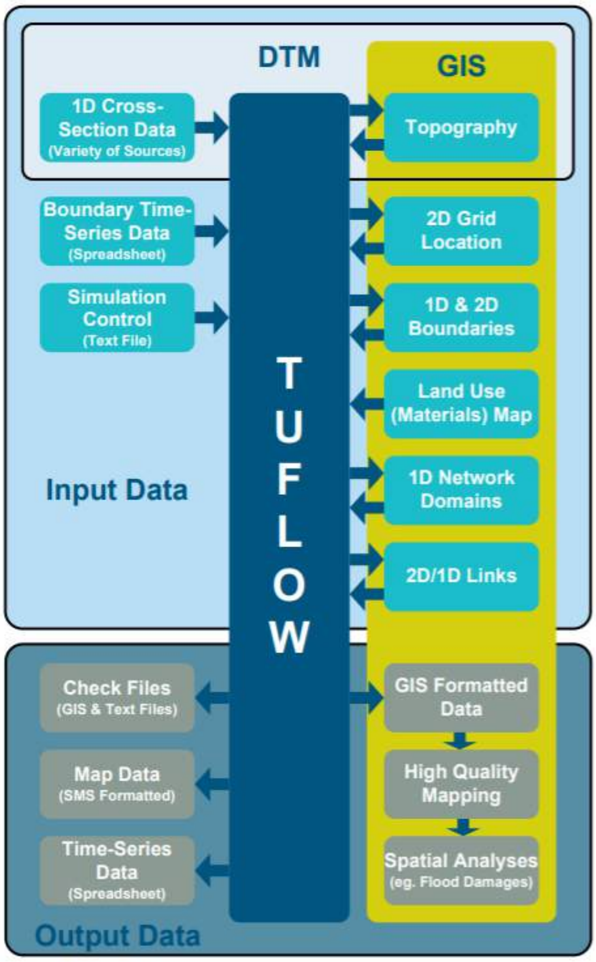


Figure E.3: Data input and output structure of TUFLOW (*TUFLOW User Manual*, 2016).

TUFLOW is only a computational engine and therefore no shell is available. Different software is used for in- and output generation. So therfore there are multiple input files sources and types. In Figure E.3 one can see the main outline of the data in- and output for a TUFLOW Model. All elements are explained below:

- **DTM:** Digital Terrain or elevation Model.
- **1D Cross-section Data:** This data file contains the cross section of the river over the entire length of the river.
- **Boundary Time-Series Data:** This file contains the data at the boundaries. In this case it concerns the discharge upstream and the water levels downstream.
- **Simulation Control:** This is the .TXT file that controls the simulation. In this file all the input files are linked together and run into TUFLOW, this can be considered as batch file.
- **Topography:** This file contains the bed elevation over the entire model area.
- **2D Grid Location:** This contains the grid in which the hydraulic computations are performed. Note, the 2D grid locations do not have to match the topographical data.
- **1D & 2D Boundaries:** This file controls the 1D & 2D boundary condition data input.
- **Land Use Map:** This file contains the land uses over the entire area. In this way a Manning coefficient can be linked to the area, which alters the flowresistance.
- **1D Network Domains:** this file establishes the 1D domain connection to the grid location.
- **2D & 1D Links:** This file controls the link between the 1D Network Domain to the 2D domain.

File extensions

Table E.1: Important file extensions in the TUFLOW model.

File name	Extension	Function
TUFLOW Control File	.TCF	Connects all elements of the model
1D Input File	.ECF	Contains 1D elements
Geometry Control File	.TGC	Contains the geometry of the domain
Boundary Control File	.TBC	Contains boundary conditions
TUFLOW Log File	.TLF	File to check and debug
DEM Topography File	.FLT	Contains topography and used to apply topographic changes
SMS mesh file	.2DM	To check topographic changes (in SMS or Crayfish Plugin)
Output Result File	.DAT	Check results in Crayfish plugin
ASCII grid File	.ASC	Contains elevation data

General .TCF File SetUp

The .TCF files of the CVFM are set up following a recognizable format like shown below

- Model Initialisation Commands
- Model Inputs
- Simulation time control commands
- Bed resistance format commands
- Output commands
- Viscosity Formulation
- Clarence River Model Grid Sizes 2D
  - Clarence Valley - 60 m grid
  - North & South Grafton - 30 m grid
  - North & South Grafton - 10 m grid
  - Maclean - 30 m grid
  - Maclean - 10 m grid
- 2D & 1D links
- Adjustments by TU Delft Students

E.4. TUFLOW Tips

Paths

to easily copy the path of a certain file. Hold Shift when right-clicking the file. The option 'Copy as Path' will be presented.

First Run

For the test run cancel the 'Write Check Files'-command, when executing you should enable this command. A command can be disabled by adding a ! in front.

Simulation in series

If multiple simulations have to be run it could be usefull to run them in series. This can be done in this way:

```

start  /low         ..\exe\\2012-05\w64\2012-05-AD\TUFLOW_iSP_w64.exe -b -x CLA_multi_Q005_267.tcf
start  /low /wait   ..\exe\\2012-05\w64\2012-05-AD\TUFLOW_iSP_w64.exe -b -x CLA_multi_Q020_267.tcf
start  /low         ..\exe\\2012-05\w64\2012-05-AD\TUFLOW_iSP_w64.exe -b -x CLA_multi_Q050_267.tcf



```

Figure E.4: Batch file command to let simulation run in series.

The wait command let the third simulation begin if the other two are finished. Note, this is depending on the license, because licenses exist where more then 2 simulation can be run at once.

E.5. TUFLOW Utilities

ASC to ASC

The ASC to ASC Utility is used to rewrite a .FLT file into an .ASC format.

- Doubleclick the `ASC_to_ASC_w64.exe`
- In the commandprompt write: `asc_to_asc_w64.exe -cove *.flt`. Where the asterix stands for the path including the filename
- To use the difference function write the following in the commandprompt: `asc_to_asc_w64.exe -out difference.asc -dif after_h.asc before_h.asc`. Where difference.asc is the name of the output file.

TUFLOW to GIS

The TUFLOW to GIS Utility is used to rewrite a .2DM file into a .ASC format

- Doubleclick the `TUFLOW_to_GIS.exe`
- In the commandprompt write: `TUFLOW_to_GIS_w64.exe -asc *.2dm`. Where the asterix stands for the path including the file name
- to rewrite an output result file: `TUFLOW_to_GIS_w64.exe -asc -b -max *.dat`.

Add PO points and lines

If you want to check certain specific points in the model and create h,t-graphs you should add point output (PO) points. If a Q,t-graph need to be formed you create a PO line. The method is presented below:

- Take the `2d_po` files in the Model Directory
- Open the `2d_po_x_x.MIF` file in QGIS
- Save as copy with a .SHP extension.
- Rename it as `2d_po_x_x_P.SHP` for points, `2d_po_x_x_L.SHP` for lines
- Toggle editing of the .SHP layer
- Add points with the 'add-feature'-function
- Specify type as: `H_`
- Specify a label to recognise your point
- Save the layer
- Add the file just made to the .TCF file
- Replace the 'Read MI PO ==' line by 'READ GIS PO ==' and add the files just made including the path like shown in Figure E.5

```
!______________________________________________________________________________
! Clarence River Model 60m Grid size 2D domain
Start 2D Domain == 60mGRID
    Geometry Control File == ..\Model\CLA\CLA_NG_SG_MC_60m_267.tgc
    BC Control File == ..\Model\CLA\CLA_NG_SG_MC_266.tbc
    BC DATABASE == ..\bc_dbase\Design_Floods_Dec2002\Q005\bc_dbase_Q005_043.csv
    Read MI FC == ..\Model\CLA\MapInfo\2d_fc_CLA_60m_252.mif
    Read File == tcf_commands_common_240.trd
    Read GIS PO == ..\Model\CLA\MapInfo\2d_po_CLA_60m_WL_output_268_P.shp
    Read GIS PO == ..\Model\CLA\MapInfo\2d_po_CLA_60m_WL_output_268_L.shp
    Timestep (s) == 12.
End 2D Domain
```

Figure E.5: Example of the added GIS PO line.

E.6. TUFLOW Output

Read PO points and lines (water depths and discharge over time)

After the points and lines being added a simulation has been run. This gives output on the specified locations. These elements can be checked by the following methods:

- Open Microsoft Excel
- Select import data from text
- Open a file with the following format `CLA_Multi_Q005_267_PO.csv`
- Excel will create a column based workmap

- Check if the seperation-symbol is correct for the language Excel is using
- Select Excel Utility TUFLOW Tool
- Click 'Show PO Results'

You can compare the point in the QGIS Software. This can be done by adding the `2d_po_x_x_P.SHP` layer and select the desired point in the Attribute table. Now you can compare the location of the point with the graph made in Excel. If you add the Crayfish-result (.DAT) layer you can also immediately see which points flood. The same holds for cross sections, these files produce a Q,t-graph.

As many PO points and lines make it difficult to interpret and correctly visualise the results, a Python script is constructed to generate the output in terms of graphs and tables.

Difference plots between two scenarios (water depths)
The creation of a map which shows the difference between two outputs is elaborated on in this section. This can be done by converting the .DAT files to .ASC files by applying the technique showed in the section 'TUFLOW to GIS'. After this, a maximum difference calculation according to the 'ASC to ASC' section is performed. Output files have a `_wd.ASC` and a `.ASC` extension. Here, the `_wd.ASC` contains information about grid cells that 'were wet now dry' and 'were dry now wet'. The `.ASC` file contains the maximum water depths in the different cells. QGIS is used to construct standard .TXT colour map formats ,wd.TXT and classification.TXT, for both the `_wd.ASC` and `.ASC` files, respectively. This will visualise a standard layout for every comparison between two scenarios, in terms of colour map and values.

Times
The command Times needs to be specified in the TUFLOW .tcf file according to this line: `Map Output Data Types == h V d z0 Times z1`. Together with this line: `Time Output Cutoff Depths == 0.0,0.1`. With this command, six types of data are saved.

Table E.2: Data types resulting from Times command.

Type of Data	Output Time in QGIS
Maximum Water Level	900001
Maximum velocity	900002
First time to be inundated above 0,0 m water depth	100000
First time to be inundated above 0,1 m water depth	100001
Inundation duration from 0,0 m	200000
Inundation duration from 0,1 m	200001

This data is a valuable source to visualise the times before a certain area is inundated (in other words, evacuation time) for instance.

Velocity
The `_V.DAT` file can be reviewed in QGIS. This is a vector file and can be visualised on top of any (when unlocked) other crayfish QGIS layer.

Topography
The `.2dm` files together with the written `.tin` (when included the `WRITE TIN` command) can be shown in 3D in SMS. In this way, a three-dimensional visualisation of any topography adjustment can be constructed.

F
Scenarios

The different scenarios runned with TUFLOW are elaborated on in this Appendix. Figures are added to visualise the scenarios.

F.1. Scenarios Baker's Swamp

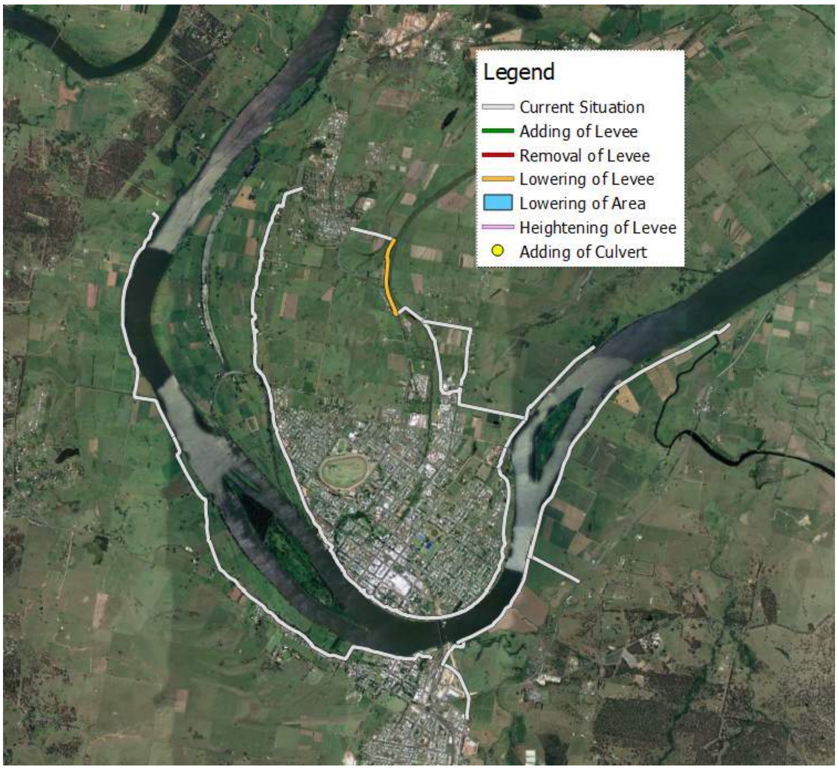


Figure F.1: Scenario 1.1: Lowering Westlawn Levee.

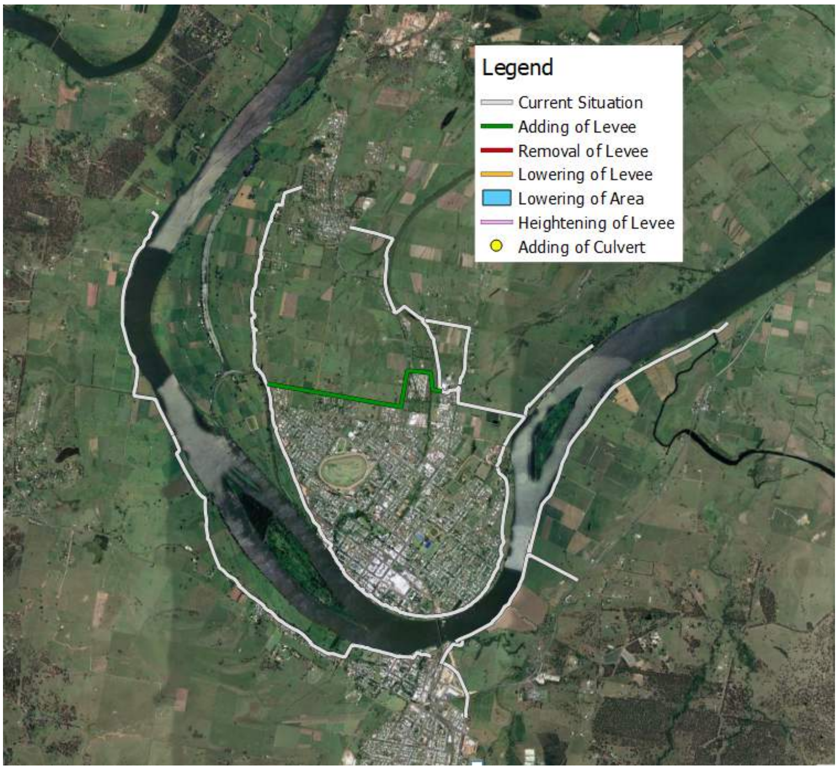


Figure F.2: Scenario 1.2: Construct levee north of Grafton (Carrs Street).

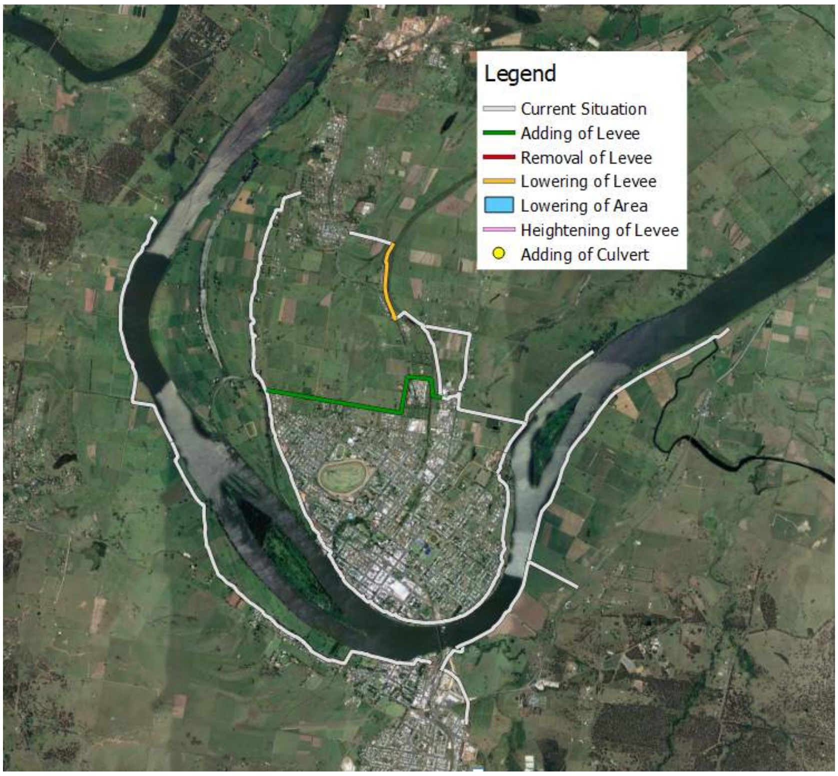


Figure F.3: Scenario 1.3: Combination of both scenario 1.1 & 1.2.

F.2. Scenarios Southampton Floodplain

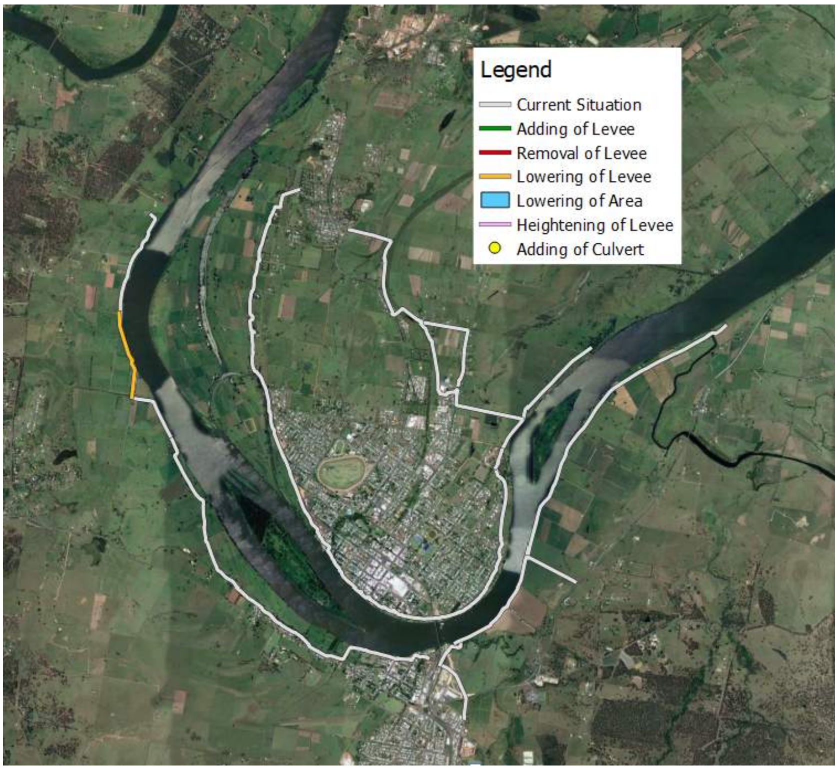


Figure F.4: Scenario 2.1: Lowering Waterview Levee (upstream).

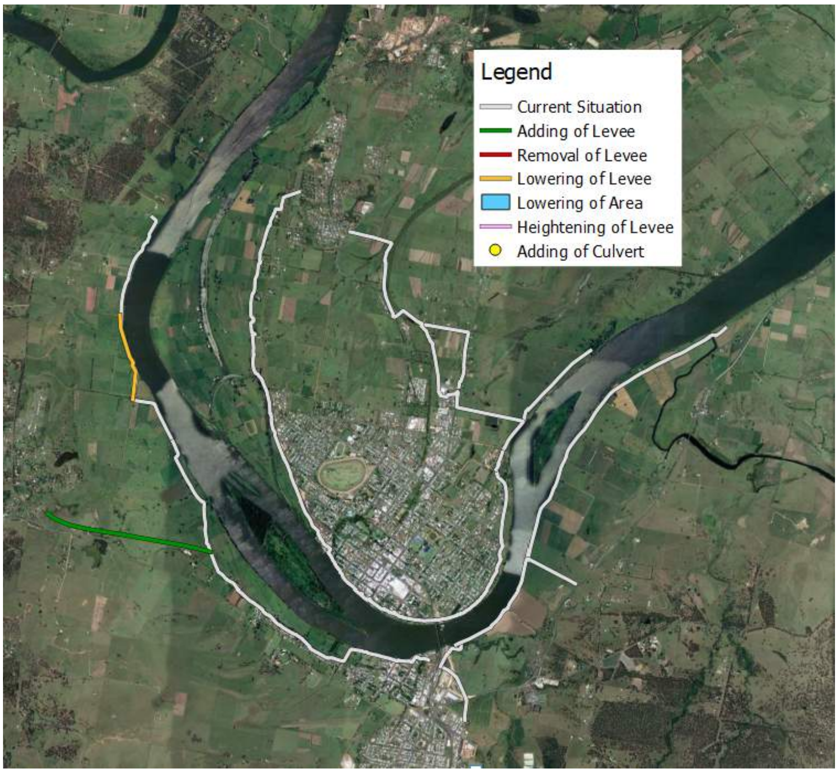


Figure F.5: Scenario 2.2: Scenario 2.1 + construct levee in front of Gwydir Highway.

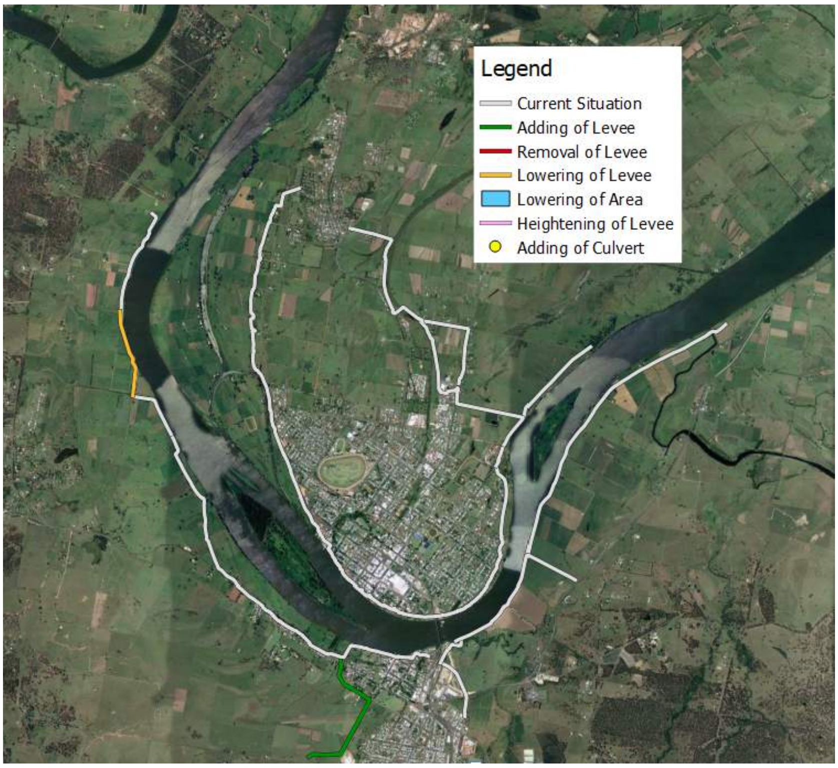


Figure F.6: Scenario 2.3: Scenario 2.1 + construct levee in South Grafton.

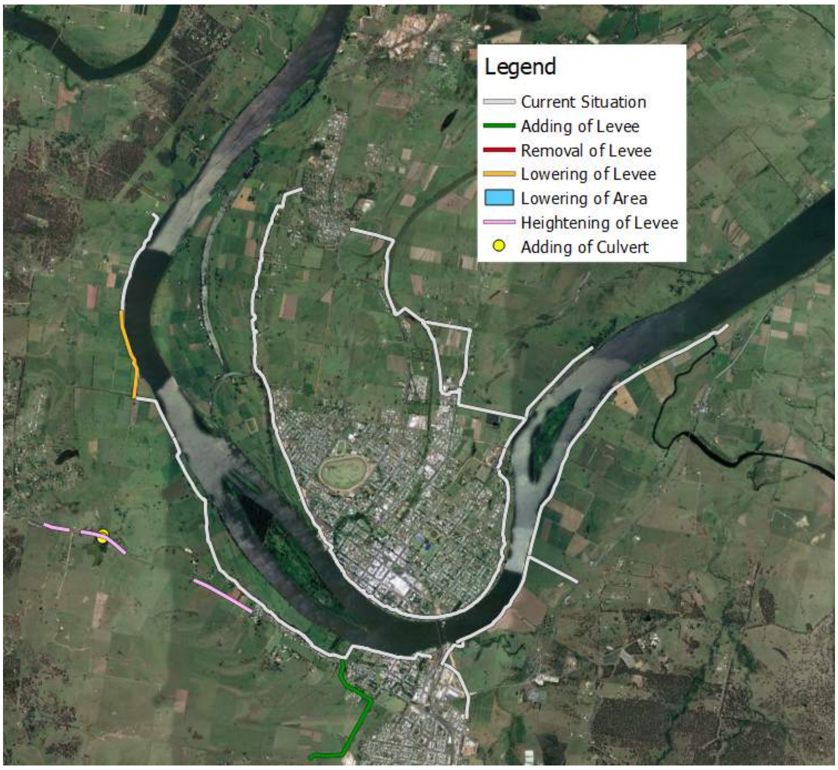


Figure F.7: Scenario 2.4: Scenario 2.3 + construct Gwydir Highway embankment and culverts.

F.3. Scenarios South Grafton

Figure F.8: Scenario 3.1: Streamline South Grafton Levee.

F.4. Scenarios Clarenza Floodplain

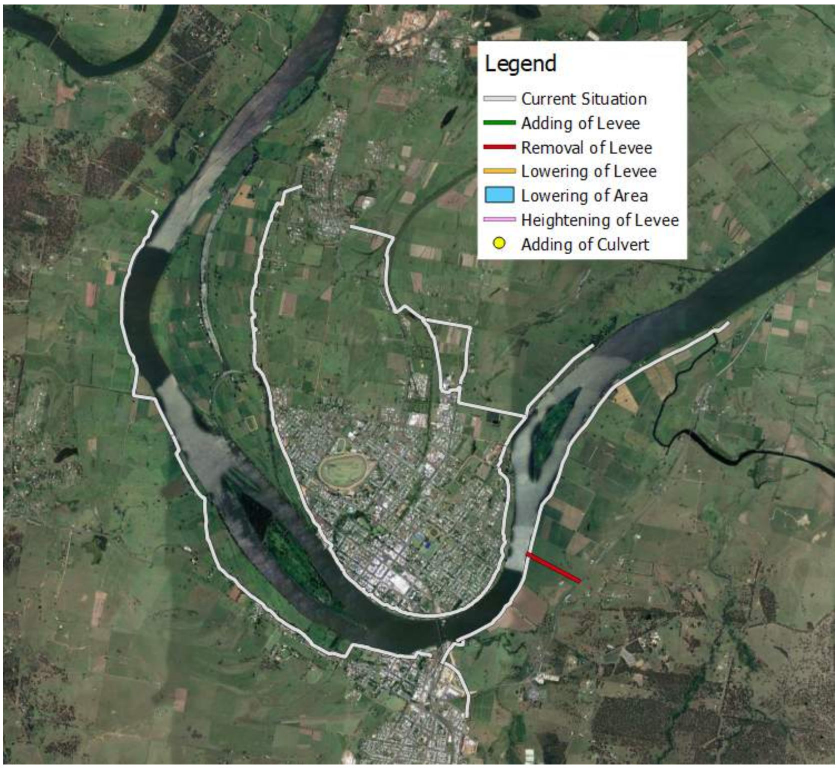


Figure F.9: Scenario 4.1: Remove Control Levee.

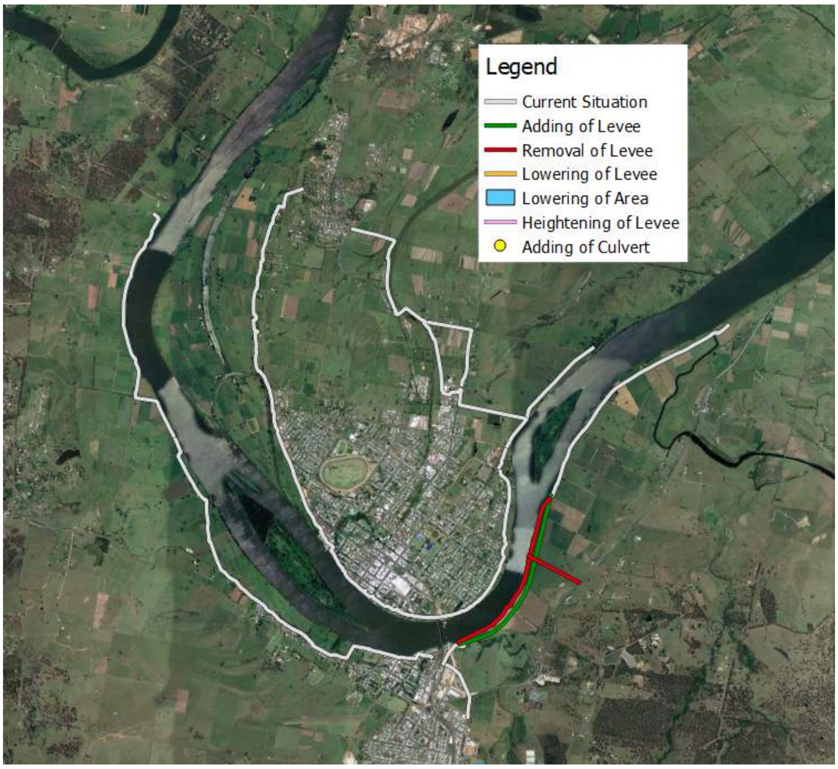


Figure F.10: Scenario 4.2: Scenario 4.1 + widen river at the narrowing section.

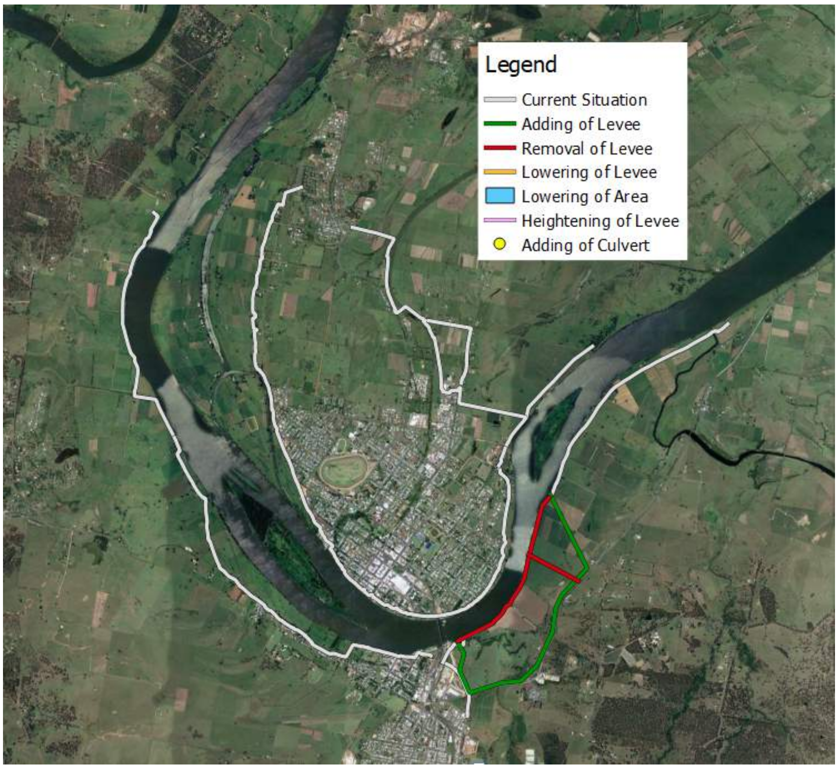


Figure F.11: Scenario 4.3: Scenario 4.1 + move Swan Creek Levee to Pacific Highway.

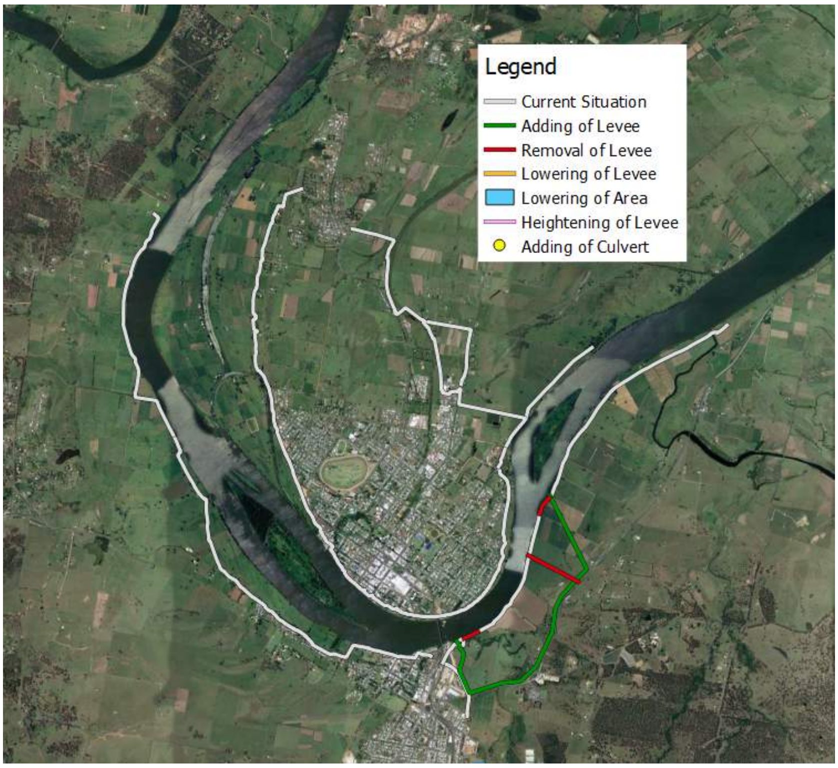


Figure F.12: Scenario 4.4: Scenario 4.1 + construct secondary channel.

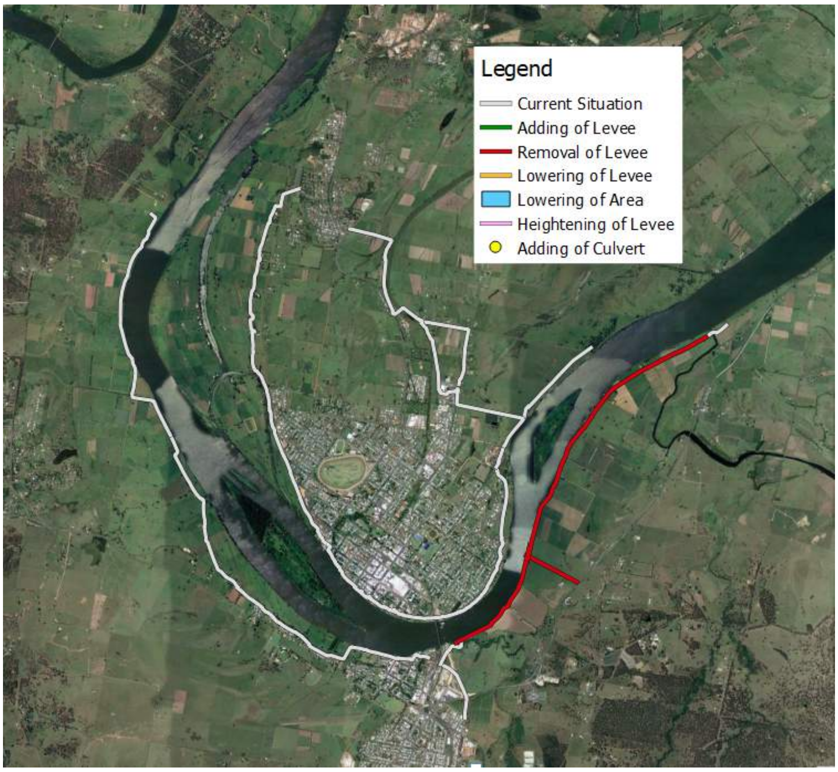


Figure F.13: Scenario 4.5: Scenario 4.1 + remove Alipou Levee and Swan Creek Levee.

F.5. Scenarios Hill Ridge

Figure F.14: Scenario 5.1: Remove part of northern ridge North of the river.

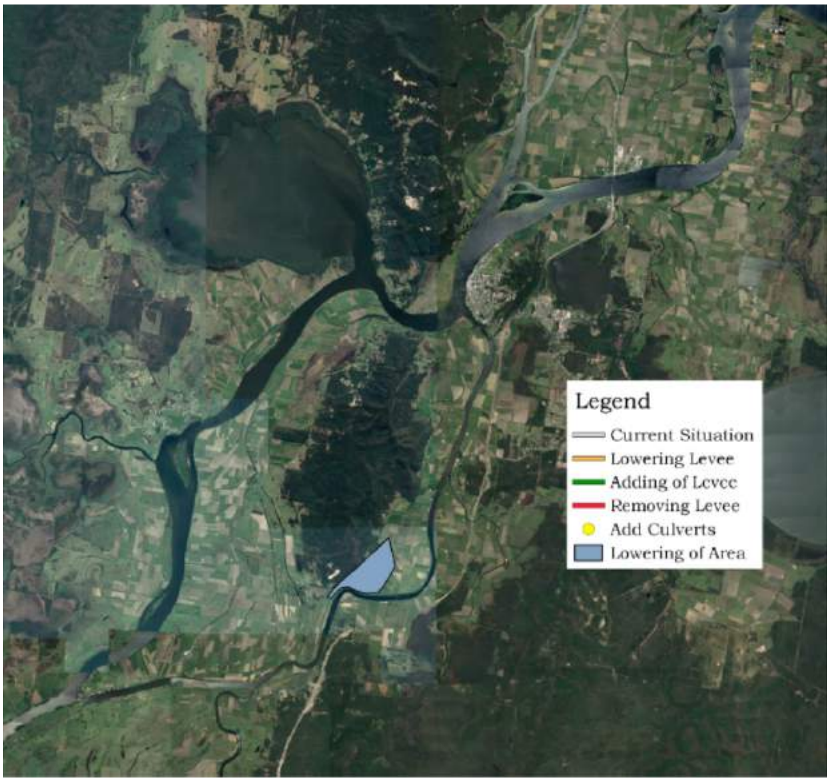


Figure F.15: Scenario 5.2: Remove part of southern ridge North of the river.

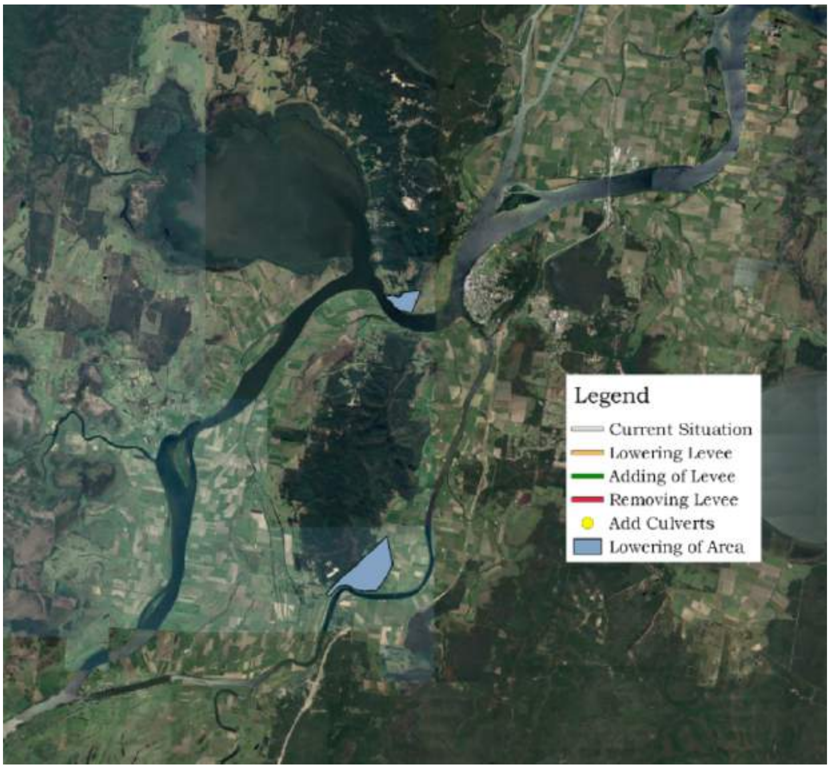


Figure F.16: Scenario 5.3: Combination of both Scenario 5.1 & 5.2.

F.6. Modelling log

In order to construct the scenarios using TUFLOW, a modelling log is used to keep track on the changes made to the reference simulation. In Figure F.17 and F.18 below, the changed TUFLOW setup documents are mentioned per scenario.

Project Name:	Project Clarence Valley
Job Date:	09-02-2018 - 31-03-2018
Modellers:	Etienne Kras & Daan Bader
Developed from earlier models?	B19054
Tuflow build:	2012-05-AD
Base model:	CLA_multi_Qxxx_267 (from BMT WBM), after adding adjustments CLA_multi_Qxxx_271 (TUDelft students)
Location of results:	..\TUFLOW\Results_Des

Run ID	Run characteristic	Date	Event/Duration	tcf/ecf	tgc	tbc	Bc_dbase	Changes
TRIAL SIMULATIONS								
Q005-268		27-2-2018	trial run	CLA_multi_Q005_268, CLA_NG_SG_Exg_268.ecf	x	x	x	added PO and LO to the 267, change location log (.tcf), check (.tcf .ecf) and output (.tcf .ecf)
Q020-268		27-2-2018	trial run	CLA_multi_Q020_268, CLA_NG_SG_Exg_268.ecf	x	x	x	added PO and LO to the 267, change location log (.tcf), check (.tcf .ecf) and output (.tcf .ecf)
Q050-268		27-2-2018	trial run	CLA_multi_Q050_268, CLA_NG_SG_Exg_268.ecf	x	x	x	added PO and LO to the 267, change location log (.tcf), check (.tcf .ecf) and output (.tcf .ecf)
Q005-269		28-2-2018	trial run	CLA_multi_Q005_269	NG_SG_30m_PMF_269.tgc	x	x	removed clarenza control levee
Q005-269b		01-03-2018	trial run	CLA_multi_Q005_269b	CLA_NG_SG_MC_60m_269b.tgc	x	x	added a levee
REFERENCE SIMULATIONS								
Q005-270		01-03-2018	base (150hr) run	CLA_multi_Q005_270, CLA_NG_SG_Exg_270.ecf	x	x	x	added PO and LO
Q020-270		01-03-2018	base (150hr) run	CLA_multi_Q020_270, CLA_NG_SG_Exg_270.ecf	x	x	x	added PO and LO
Q050-270		01-03-2018	base (150hr) run	CLA_multi_Q050_270, CLA_NG_SG_Exg_270.ecf	x	x	x	added PO and LO
Q005-271	Reference	05-03-2018	base (150hr) run	CLA_multi_Q005_271	x	x	x	added LP lines, creating reference
Q020-271	Reference	05-03-2018	base (150hr) run	CLA_multi_Q020_271	x	x	x	added LP lines, creating reference
Q050-271	Reference	05-03-2018	base (150hr) run	CLA_multi_Q050_271	x	x	x	added LP lines, creating reference
AREA 1 - BAKERSWAMP								
Q005-280	Scenario 1.1	06-03-2018	adjustment	CLA_multi_Q005_280	NG_SG_30m_PMF_280.tgc	x	x	lowered part of Wastlawn levee to 5.0m
Q020-280	Scenario 1.1	06-03-2018	adjustment	CLA_multi_Q020_280	NG_SG_30m_PMF_280.tgc	x	x	lowered part of Wastlawn levee to 5.0m
Q050-280	Scenario 1.1	06-03-2018	adjustment	CLA_multi_Q050_280	NG_SG_30m_PMF_280.tgc	x	x	lowered part of Wastlawn levee to 5.0m
Q005-281	Scenario 1.2	06-03-2018	adjustment	CLA_multi_Q005_281	NG_SG_30m_PMF_281.tgc	x	x	implemented levee north of Grafton (7.0m)
Q020-281	Scenario 1.2	06-03-2018	adjustment	CLA_multi_Q020_281	NG_SG_30m_PMF_281.tgc	x	x	implemented levee north of Grafton (7.0m)
Q050-281	Scenario 1.2	06-03-2018	adjustment	CLA_multi_Q050_281	NG_SG_30m_PMF_281.tgc	x	x	implemented levee north of Grafton (7.0m)
Q005-282	Scenario 1.3	06-03-2018	adjustment	CLA_multi_Q005_282	NG_SG_30m_PMF_282.tgc	x	x	lowered Westlawn, implement NorthGrafton_Levee
Q020-282	Scenario 1.3	06-03-2018	adjustment	CLA_multi_Q020_282	NG_SG_30m_PMF_282.tgc	x	x	lowered Westlawn, implement NorthGrafton_Levee
Q050-282	Scenario 1.3	06-03-2018	adjustment	CLA_multi_Q050_282	NG_SG_30m_PMF_282.tgc	x	x	lowered Westlawn, implement NorthGrafton_Levee
AREA 2 - SOUTHAMPTON								
Q005-290	Scenario 2.1	06-03-2018	adjustment	CLA_multi_Q005_290	NG_SG_30m_PMF_290.tgc	x	x	lowered section of Waterview levee of approx 1.5 km to 7 m
Q020-290	Scenario 2.1	06-03-2018	adjustment	CLA_multi_Q020_290	NG_SG_30m_PMF_290.tgc	x	x	lowered section of Waterview levee of approx 1.5 km to 7 m
Q050-290	Scenario 2.1	06-03-2018	adjustment	CLA_multi_Q050_290	NG_SG_30m_PMF_290.tgc	x	x	lowered section of Waterview levee of approx 1.5 km to 7 m
Q005-291	Scenario 2.2	06-03-2018	adjustment	CLA_multi_Q005_291	NG_SG_30m_PMF_291.tgc	x	x	added Gwydir highway levee based on Q005_290 simulation to 5 m
Q020-291	Scenario 2.2	06-03-2018	adjustment	CLA_multi_Q020_291	NG_SG_30m_PMF_291.tgc	x	x	added Gwydir highway levee based on Q020_290 simulation to 5 m
Q050-291	Scenario 2.2	06-03-2018	adjustment	CLA_multi_Q050_291	NG_SG_30m_PMF_291.tgc	x	x	added Gwydir highway levee based on Q050_290 simulation to 5 m

Figure F.17: Scenario modelling log TUFLOW simulations.

Q005-292	Scenario 2.3	06-03-2018	adjustment	CLA_multi_Q005_292	NG_SG_10m_PMF_292.tgc, NG_SG_30m_PMF_292.tgc	x	x	added south grafton levee based on Q005_290 simulation to 9.5 m
Q020-292	Scenario 2.3	06-03-2018	adjustment	CLA_multi_Q020_292	NG_SG_10m_PMF_292.tgc, NG_SG_30m_PMF_292.tgc	x	x	added south grafton levee based on Q020_290 simulation to 9.5 m
Q050-292	Scenario 2.3	06-03-2018	adjustment	CLA_multi_Q050_292	NG_SG_10m_PMF_292.tgc, NG_SG_30m_PMF_292.tgc	x	x	added south grafton levee based on Q050_290 simulation to 9.5 m
Q005-293	Scenario 2.4	07-03-2018	adjustment	CLA_multi_Q005_293, CLA_NG_SG_Exg_293.ecf	NG_SG_10m_PMF_293.tgc, NG_SG_30m_PMF_293.tgc	NG_SG_30m_PMF_293.tbc	x	heightened Gwydir hwy (6.5 and 6 m) and inserted a culvert based on Q005_292 simulation
Q020-293	Scenario 2.4	07-03-2018	adjustment	CLA_multi_Q020_293, CLA_NG_SG_Exg_293.ecf	NG_SG_10m_PMF_293.tgc, NG_SG_30m_PMF_293.tgc	NG_SG_30m_PMF_293.tbc	x	heightened Gwydir hwy (6.5 and 6 m) and inserted a culvert based on Q020_292 simulation
Q050-293	Scenario 2.4	07-03-2018	adjustment	CLA_multi_Q050_293, CLA_NG_SG_Exg_293.ecf	NG_SG_10m_PMF_293.tgc, NG_SG_30m_PMF_293.tgc	NG_SG_30m_PMF_293.tbc	x	heightened Gwydir hwy (6.5 and 6 m) and inserted a culvert based on Q050_292 simulation
AREA 3 - SOUTH GRAFTON URBAN								
Q005-300	Scenario 3.1	07-03-2018	adjustment	CLA_multi_Q005_300	NG_SG_10m_PMF_300.tgc	x	x	added smooth levee South Grafton (8.5m)
Q020-300	Scenario 3.1	07-03-2018	adjustment	CLA_multi_Q020_300	NG_SG_10m_PMF_300.tgc	x	x	added smooth levee South Grafton (8.5m)
Q050-300	Scenario 3.1	07-03-2018	adjustment	CLA_multi_Q050_300	NG_SG_10m_PMF_300.tgc	x	x	added smooth levee South Grafton (8.5m)
AREA 4 - CLARENZA FLOODPLAIN								
Q005-310	Scenario 4.1	07-03-2018	adjustment	CLA_multi_Q005_310	NG_SG_30m_PMF_310.tgc	x	x	removed Clarenza Control Levee
Q020-310	Scenario 4.1	08-03-2018	adjustment	CLA_multi_Q020_310	NG_SG_30m_PMF_310.tgc	x	x	removed Clarenza Control Levee
Q050-310	Scenario 4.1	08-03-2018	adjustment	CLA_multi_Q050_310	NG_SG_30m_PMF_310.tgc	x	x	removed Clarenza Control Levee
Q005-311	Scenario 4.2	07-03-2018	adjustment	CLA_multi_Q005_311	NG_SG_30m_PMF_311.tgc	x	x	moved Clarenza River Levee (6m) to width bridge, lowered river bed (TIN NEEDED)
Q020-311	Scenario 4.2	08-03-2018	adjustment	CLA_multi_Q020_311	NG_SG_30m_PMF_311.tgc	x	x	moved Clarenza River Levee (6m) to width bridge, lowered river bed (TIN NEEDED)
Q050-311	Scenario 4.2	08-03-2018	adjustment	CLA_multi_Q050_311	NG_SG_30m_PMF_311.tgc	x	x	moved Clarenza River Levee (6m) to width bridge, lowered river bed (TIN NEEDED)
Q005-312	Scenario 4.3	07-03-2018	adjustment	CLA_multi_Q005_312	NG_SG_30m_PMF_312.tgc	x	x	moved Clarenza River Levee (6m) to highway, NOT lowered river bed.
Q020-312	Scenario 4.3	08-03-2018	adjustment	CLA_multi_Q020_312	NG_SG_30m_PMF_312.tgc	x	x	moved Clarenza River Levee (6m) to highway, NOT lowered river bed.
Q050-312	Scenario 4.3	08-03-2018	adjustment	CLA_multi_Q050_312	NG_SG_30m_PMF_312.tgc	x	x	moved Clarenza River Levee (6m) to highway, NOT lowered river bed.
Q005-313	Scenario 4.4	07-03-2018	adjustment	CLA_multi_Q005_313	NG_SG_30m_PMF_313.tgc	x	x	created secondary channel in outer bend
Q020-313	Scenario 4.4	08-03-2018	adjustment	CLA_multi_Q020_313	NG_SG_30m_PMF_313.tgc	x	x	created secondary channel in outer bend
Q050-313	Scenario 4.4	08-03-2018	adjustment	CLA_multi_Q050_313	NG_SG_30m_PMF_313.tgc	x	x	created secondary channel in outer bend
Q005-314	Scenario 4.5	12-03-2018	adjustment	CLA_multi_Q005_314	NG_SG_30m_PMF_314.tgc, CLA_NG_SG_MC_60m_314.tgc	x	x	removed all levees downstream of South Grafton
Q020-314	Scenario 4.5	12-03-2018	adjustment	CLA_multi_Q020_314	NG_SG_30m_PMF_314.tgc, CLA_NG_SG_MC_60m_314.tgc	x	x	removed all levees downstream of South Grafton
Q050-314	Scenario 4.5	12-03-2018	adjustment	CLA_multi_Q050_314	NG_SG_30m_PMF_314.tgc, CLA_NG_SG_MC_60m_314.tgc	x	x	removed all levees downstream of South Grafton
AREA 5 - HILL RIDGE								
Q005-320	Scenario 5.1	08-03-2018	adjustment	CLA_multi_Q005_320	CLA_NG_SG_MC_60m_320.tgc	x	x	lowered Northern Hill Ridge North of the Clarence river
Q020-320	Scenario 5.1	08-03-2018	adjustment	CLA_multi_Q020_320	CLA_NG_SG_MC_60m_320.tgc	x	x	lowered Northern Hill Ridge North of the Clarence river
Q050-320	Scenario 5.1	08-03-2018	adjustment	CLA_multi_Q050_320	CLA_NG_SG_MC_60m_320.tgc	x	x	lowered Northern Hill Ridge North of the Clarence river
Q005-321	Scenario 5.2	08-03-2018	adjustment	CLA_multi_Q005_321	CLA_NG_SG_MC_60m_321.tgc	x	x	lowered Southern Hill Ridge North of the Clarence river
Q020-321	Scenario 5.2	08-03-2018	adjustment	CLA_multi_Q020_321	CLA_NG_SG_MC_60m_321.tgc	x	x	lowered Southern Hill Ridge North of the Clarence river
Q050-321	Scenario 5.2	08-03-2018	adjustment	CLA_multi_Q050_321	CLA_NG_SG_MC_60m_321.tgc	x	x	lowered Southern Hill Ridge North of the Clarence river
Q005-322	Scenario 5.3	08-03-2018	adjustment	CLA_multi_Q005_322	CLA_NG_SG_MC_60m_322.tgc	x	x	combination of both presented scenarios above
Q020-322	Scenario 5.3	08-03-2018	adjustment	CLA_multi_Q020_322	CLA_NG_SG_MC_60m_322.tgc	x	x	combination of both presented scenarios above
Q050-322	Scenario 5.3	08-03-2018	adjustment	CLA_multi_Q050_322	CLA_NG_SG_MC_60m_322.tgc	x	x	combination of both presented scenarios above
COMBINATIONS								
Q005-330	Combination 1	15-03-2018	combination	CLA_multi_Q005_330	NG_SG_30m_PMF_330.tgc, NG_SG_10m_PMF_292.tgc	x	x	combined scenario 282, 292 and 310
Q020-330	Combination 1	15-03-2018	combination	CLA_multi_Q020_330	NG_SG_30m_PMF_330.tgc, NG_SG_10m_PMF_292.tgc	x	x	combined scenario 282, 292 and 310
Q050-330	Combination 1	15-03-2018	combination	CLA_multi_Q050_330	NG_SG_30m_PMF_330.tgc, NG_SG_10m_PMF_292.tgc	x	x	combined scenario 282, 292 and 310
Q005-331	Combination 2	15-03-2018	combination	CLA_multi_Q005_331	NG_SG_30m_PMF_331.tgc, NG_SG_10m_PMF_292.tgc, CLA_NG_SG_MC_60m_314.tgc	x	x	combined scenario 282, 292 and 314
Q020-331	Combination 2	15-03-2018	combination	CLA_multi_Q020_331	NG_SG_30m_PMF_331.tgc, NG_SG_10m_PMF_292.tgc, CLA_NG_SG_MC_60m_314.tgc	x	x	combined scenario 282, 292 and 314
Q050-331	Combination 2	15-03-2018	combination	CLA_multi_Q050_331	NG_SG_30m_PMF_331.tgc, NG_SG_10m_PMF_292.tgc, CLA_NG_SG_MC_60m_314.tgc	x	x	combined scenario 282, 292 and 314

Figure F.18: Scenario modelling log TUFLOW simulations.

G
Results

In this Appendix, one can find all results from the scenario simulations. Some relevant scenarios can be found in Chapter 5. Other output is stored here with an explanatory text on why this was not seen as a relevant scenario. The output has been arranged per area of interest and subsequently per simulated scenario. In the figures presented, one can also see the applied adjustments.

G.1. Area results

Presentation of the scenario simulations for area 1 to 6.

G.1.1. Area 1: Baker's Swamp

In the Baker's Swamp area three scenarios are applied. Scenario 1.1 considered the lowering of the Westlawn Levee. Scenario 1.2 added a levee north of the city of Grafton and scenario 1.3 was a combination of both.

Scenario 1.1

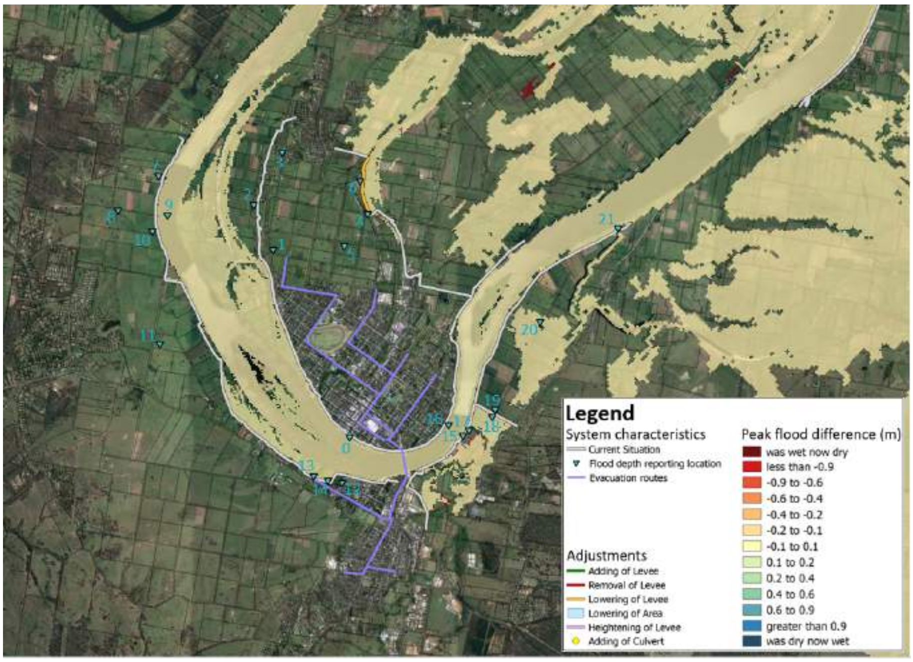


Figure G.1: Peak Flood Difference for scenario 1.1 for the 5 year ARI flood event.

As one can see in Figure G.1, the lowering of the Westlawn Levee did not make any difference. The magnitude of the lowering was not of sufficient size to let the levee overtop during a 5 year ARI flood event. This intervention did not result in a different outcome for the 5 year ARI flood event compared to the reference situation.

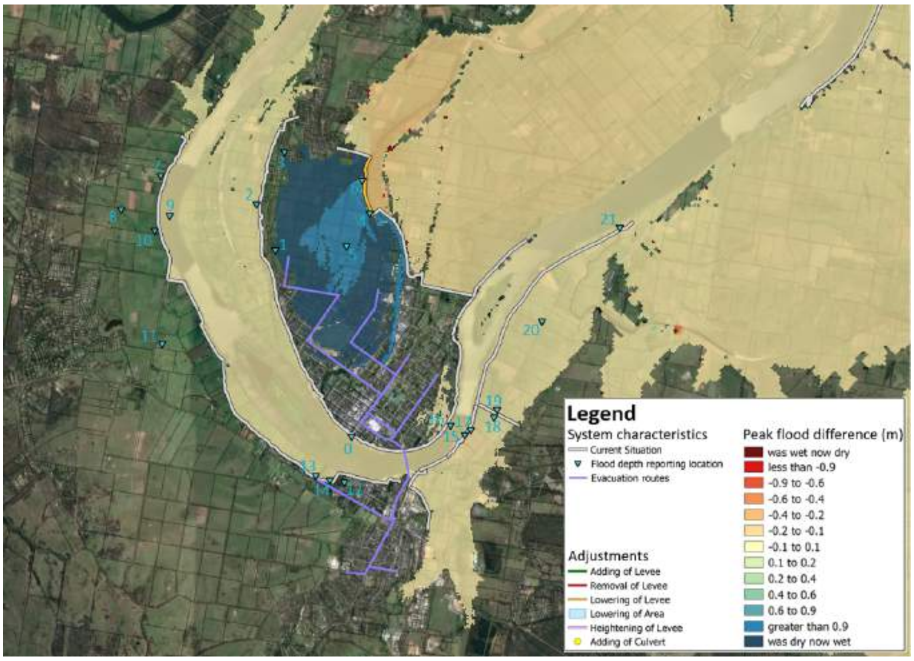


Figure G.2: Peak Flood Difference for scenario 1.1 for the 20 year ARI flood event.

During a 20 year ARI flood event one can see that the lowered Westlawn Levee overtops for both the reference situation and the created scenario. Figure G.2 shows the dark blue colour in Baker's Swamp which indicates that there is an area in Baker's Swamp which was dry and is now wet. Baker's Swamp is inundated entirely now, which solves the problem stated in Section 2.4.1. The downside of this scenario is that the volume of water cannot be contained by the Swamp, so water flows into Grafton from the north.

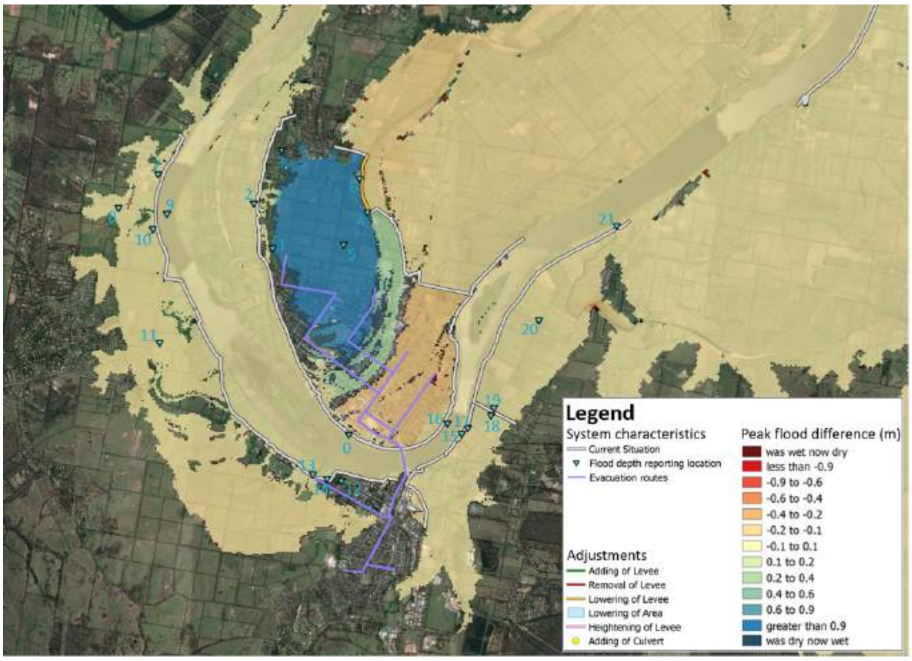


Figure G.3: Peak Flood Difference for scenario 1.1 for the 50 year ARI flood event.

In case of a 50 year ARI flood event, one can see that lowering of the Westlawn Levee will result in flooding of Baker's Swamp, but also contributes to the fact that the Alumy Creek will contain more water and therefore floods Grafton (green). By storing water in Baker's Swamp the inundation depth of the southern part of Grafton reduces with roughly 0.3 meters.

Scenario 1.2

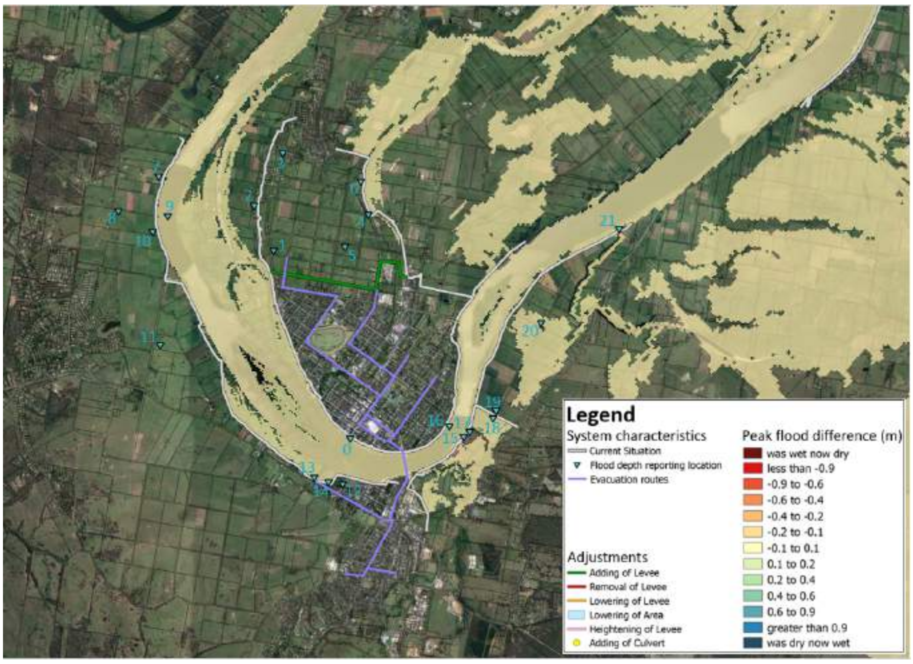


Figure G.4: Peak Flood Difference for scenario 1.2 for the 5 year ARI flood event.

In Figure G.4 one can see the consequences of adding a levee north of Grafton during a 5 year ARI flood event. The reference situation (see Figure 2.4) showed that Baker's Swamp does not flood during a 5 year ARI flood event. Therefore one can say that adding a levee north of Grafton is useless in this design flood event, no water needs to be retained.

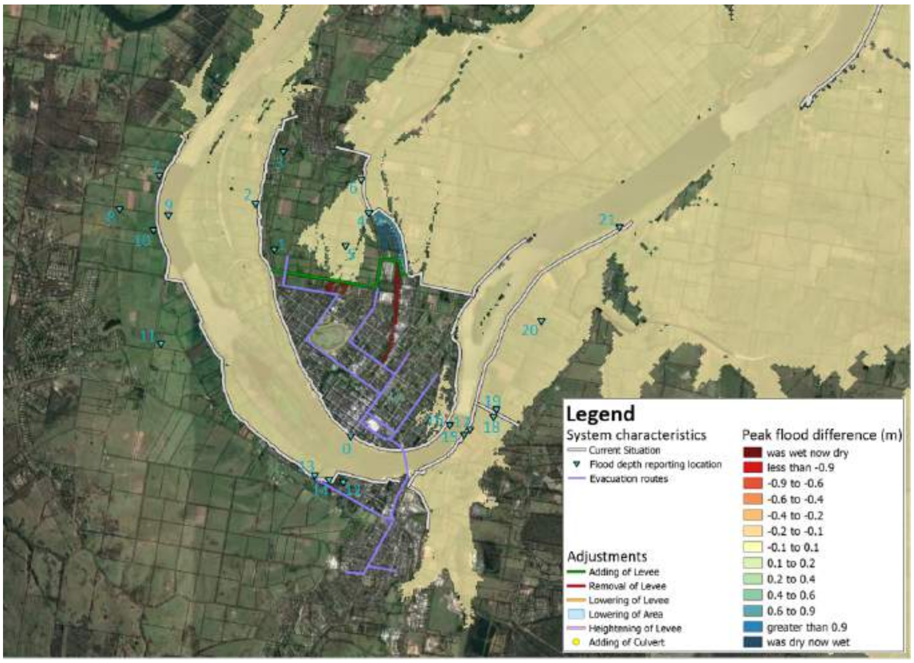


Figure G.5: Peak Flood Difference for scenario 1.2 for the 20 year ARI flood event.

By only adding the levee north of Grafton, one can see (Figure G.5) that the area south of this levee stays dry during a 20 year ARI flood event, where in the reference situation a part of the city flooded (dark red: was wet now dry). The amount of water in Baker's Swamp does not necessarily differ. The levee contains a floodgate at the Alumy Creek which will be closed in case of a flood event, this will result in prevention of overtopping of the creek inside Grafton.

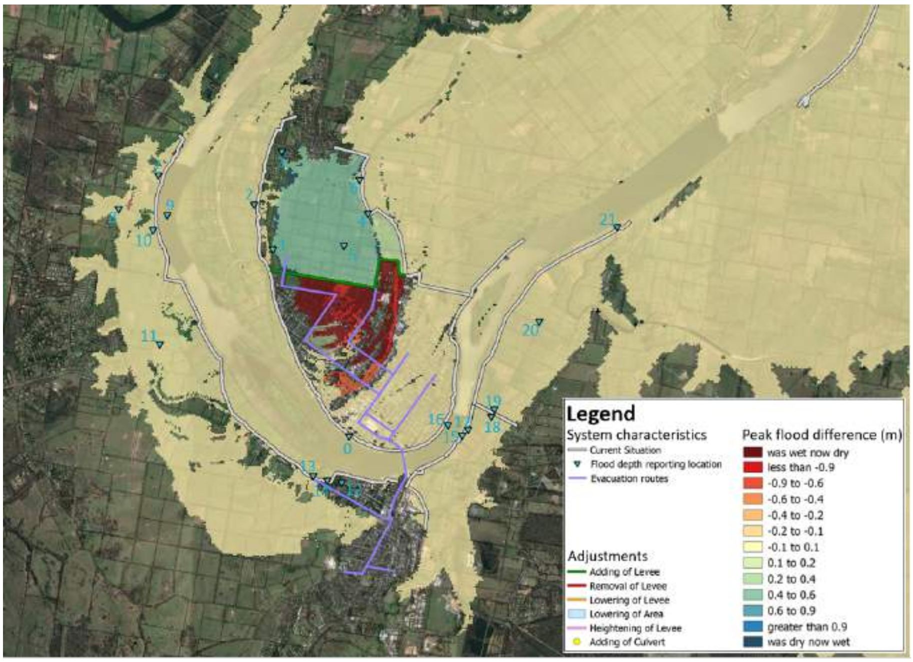


Figure G.6: Peak Flood Difference for scenario 1.2 for the 50 year ARI flood event.

Only building the levee north of Grafton will increase the inundation depth in Baker's Swamp with 0.5 meters (green in Figure G.6), and the northern part of Grafton will stay dry (dark red in Figure G.6). Just like during a 20 year ARI flood event, overtopping of the Alumy Creek will be prevented by the closed floodgate at the location where the levee crosses the creek.

Scenario 1.3

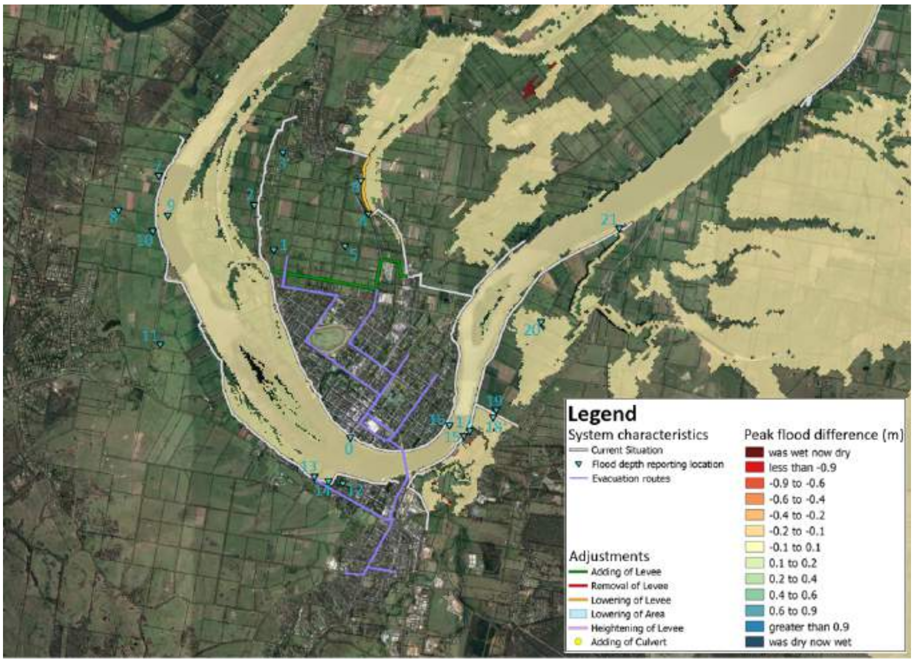


Figure G.7: Peak Flood Difference for scenario 1.3 for the 5 year ARI flood event.

For the third scenario, the combination between scenario 1.1 and scenario 1.2, the same reasoning as for the above-mentioned scenarios is applied. As one can see in Figure G.7, no water comes into Baker's Swamp during a 5 year ARI flood event. Therefore, also the combination of scenarios will not differ in outcome.

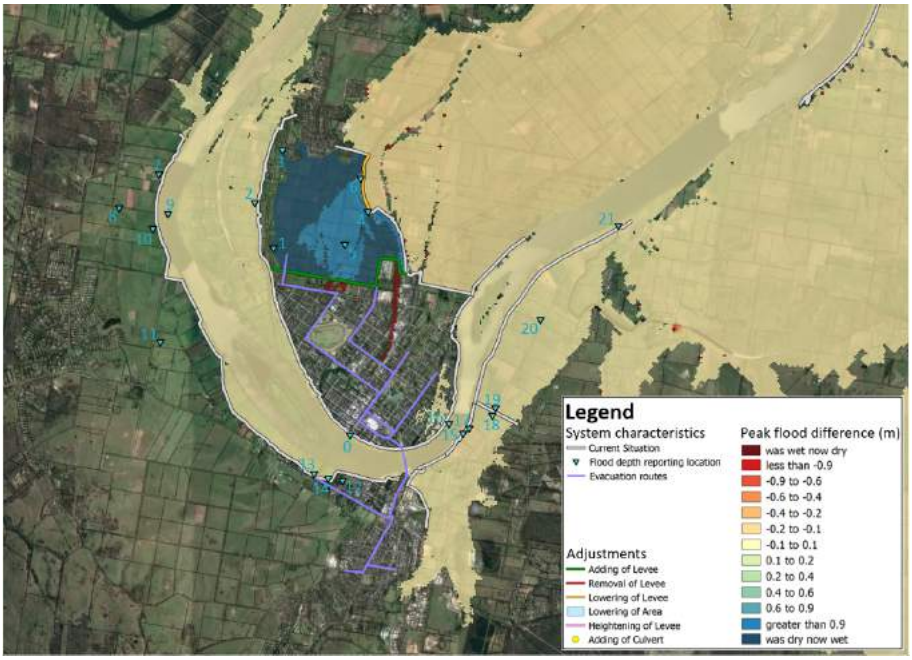


Figure G.8: Peak Flood Difference for scenario 1.3 for the 20 year ARI flood event.

For the combination of both scenarios one can see that Baker's Swamp will be flooded. By storing water in Baker's Swamp a new situation is created where surrounding areas will show a peak flood difference of -0.15 meters (northeast in Figure G.8) compared to the reference situation. But, by checking flooding of urban areas, the southern part of Grafton still floods during a 20 year ARI flood event. The measures taken north of Grafton did not alter the water levels south of Grafton.

The combination of both scenarios is considered as a significant change in the situation during a 50 year ARI flood event. Therefore this result is presented in Section 5.1.1 in the main report.

G.1.2. Area 2: Southampton Floodplain

In Southampton Floodplain, four scenarios are examined. Scenario 2.1 considers the lowering of the Waterview Levee (upstream), this forms the basis for all scenarios. In scenario 2.2, a levee in front of the Gwydir Highway is added. In scenario 2.3, a levee in South Grafton is added. Scenario 2.4 contains a levee near South Grafton and the construction of an embankment and culverts at the Gwydir Highway.

Scenario 2.1

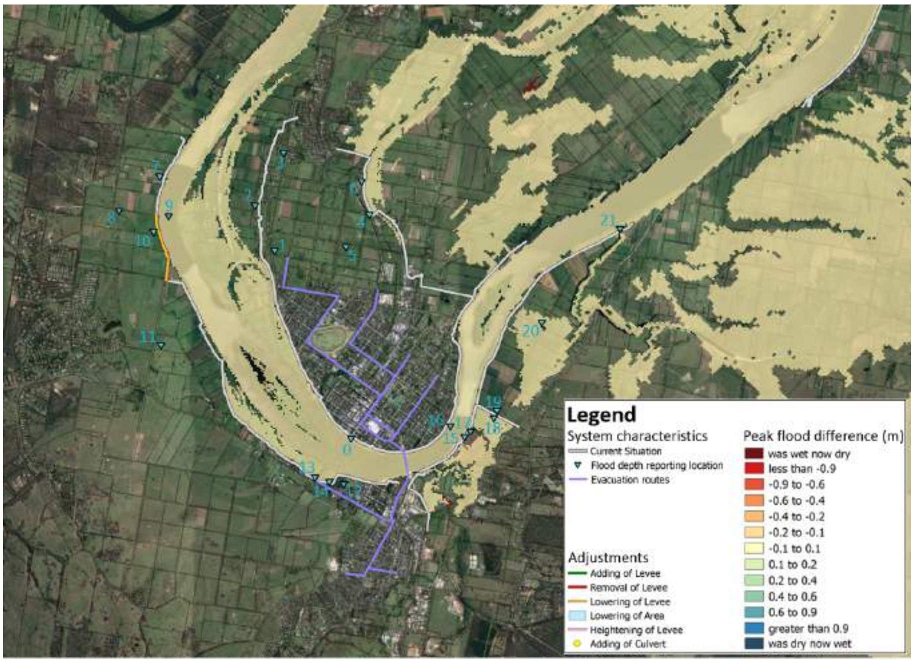


Figure G.9: Peak Flood Difference for scenario 2.1 for the 5 year ARI flood event.

The 5 year ARI flood event shows no difference for scenario 2.1 compared to the reference case. The water level does not rise enough to overtop the lowered Waterview Levee.

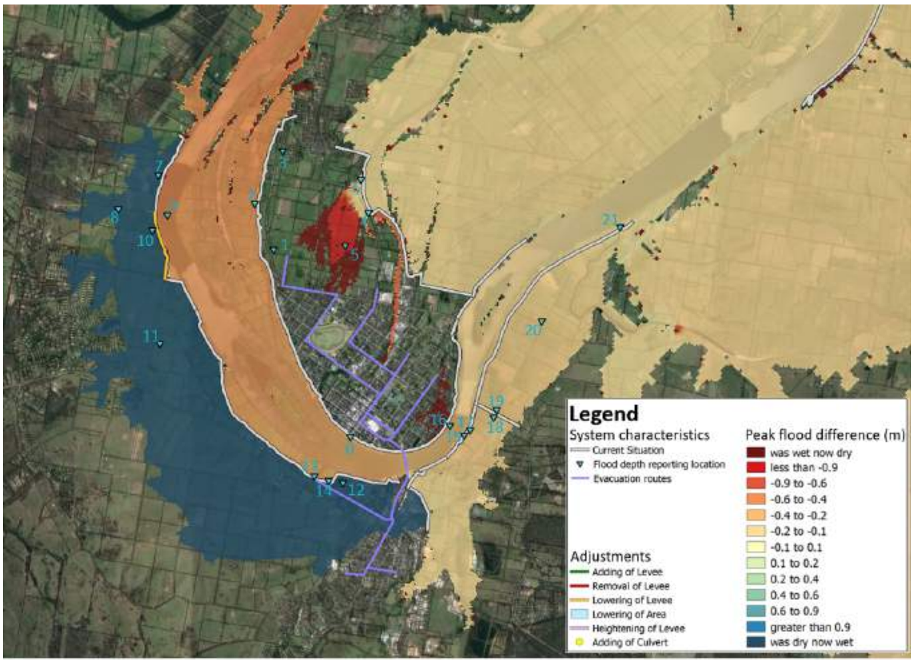


Figure G.10: Peak Flood Difference for scenario 2.1 for the 20 year ARI flood event.

In case of a 20 year ARI flood event one can see that the Southampton Floodplain inundates, due to the lowering of the Waterview Levee. As a result one can see that Grafton no longer floods, because the water is stored in the floodplain. The downside of this scenario is the inundation of South Grafton.

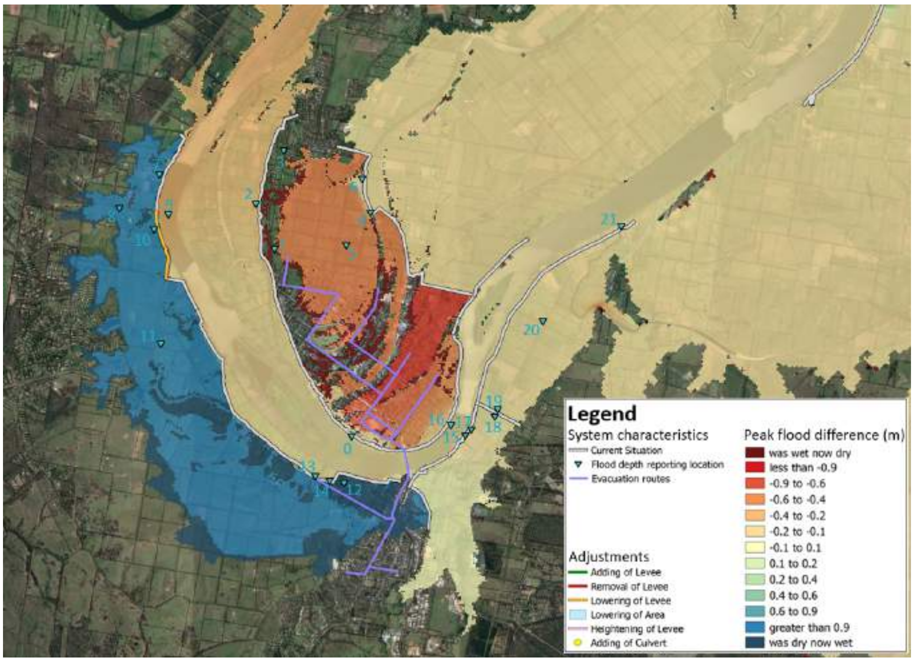


Figure G.11: Peak Flood Difference for scenario 2.1 for the 50 year ARI flood event.

Lowering the Waterview Levee (upstream) in scenario 2.1 results in a further increase of the water level in the Southampton Floodplain and parts of South Grafton. A positive effect of this scenario is the fact that the inundation depth in Grafton is lowered with 0.3 to 1.0 m. This does not evade the problem, but it significantly reduces the impact of the flood. Downside of this scenario is the fact South Grafton floods.

Scenario 2.2

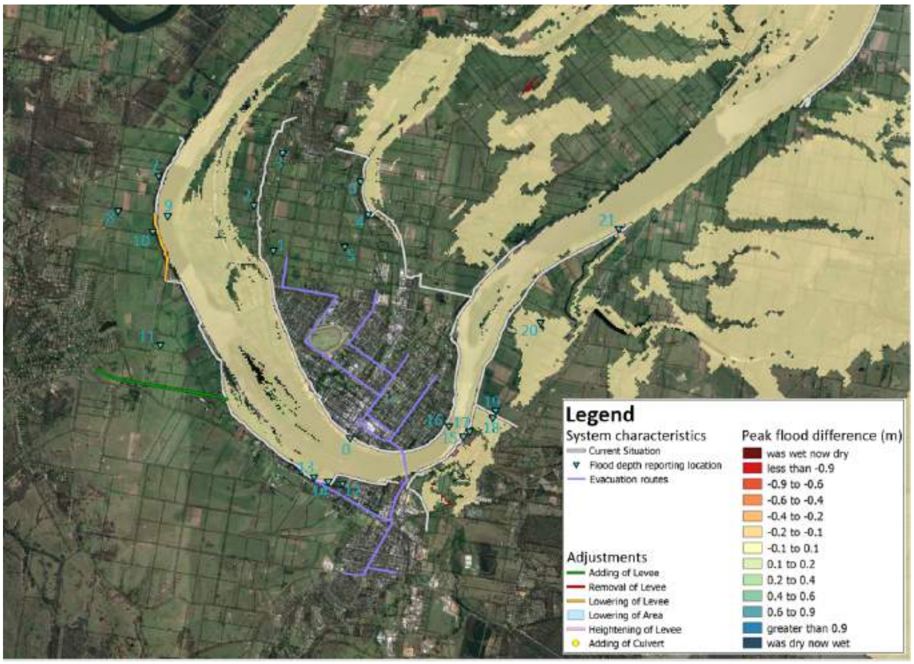


Figure G.12: Peak Flood Difference for scenario 2.2 for the 5 year ARI flood event.

The 5 year ARI flood event shows no difference for scenario 2.2 compared to the reference case. The water level does not rise enough to overtop the lowered Waterview Levee.

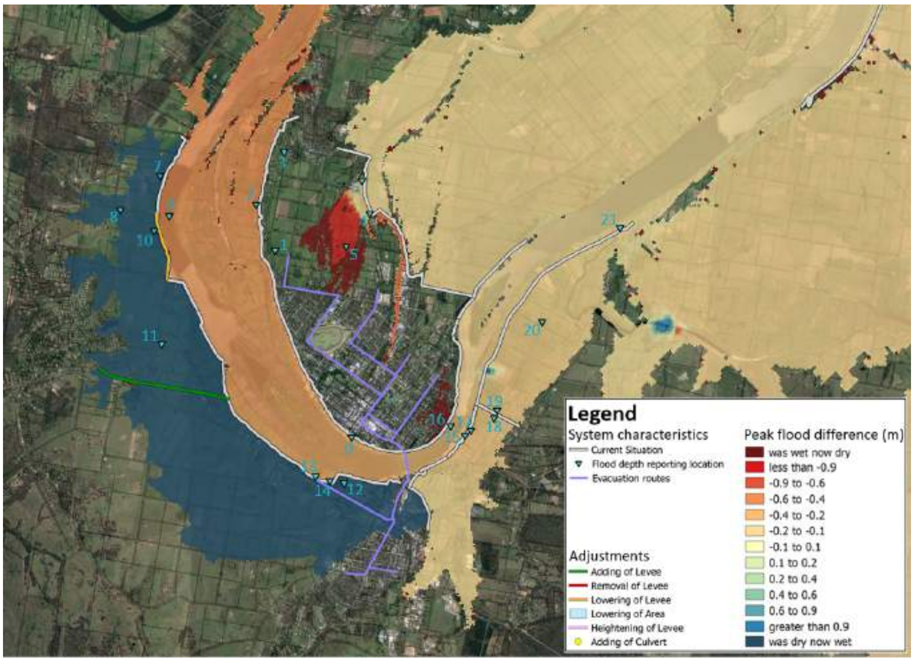


Figure G.13: Peak Flood Difference for scenario 2.2 for the 20 year ARI flood event.

In scenario 2.2 the Southampton Floodplain is inundated. The added levee to protect the Gwydir Highway is overtopped and a part of South Grafton is inundated. Key element of this scenario is the protection of infrastructure. The levee that was built to protect the Gwydir Highway overtops, because the incoming volume of water is to large.

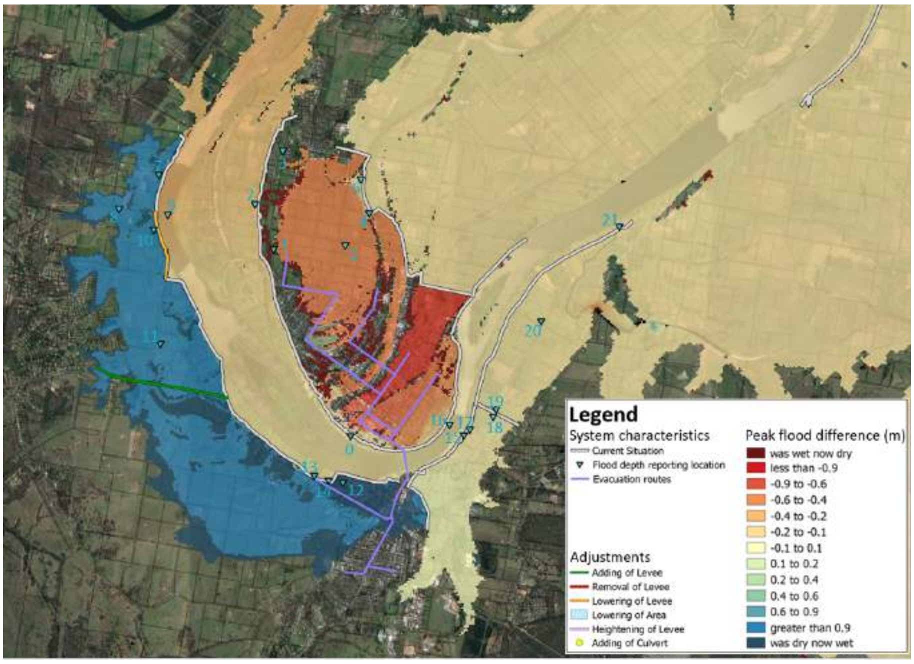


Figure G.14: Peak Flood Difference for scenario 2.2 for the 50 year ARI flood event.

The added levee in scenario 2.2 is not able to protect the Gwydir Highway and South Grafton. Using the same reasoning as for scenario 2.2 the 20 year ARI flood event, this result is not desirable.

Scenario 2.3

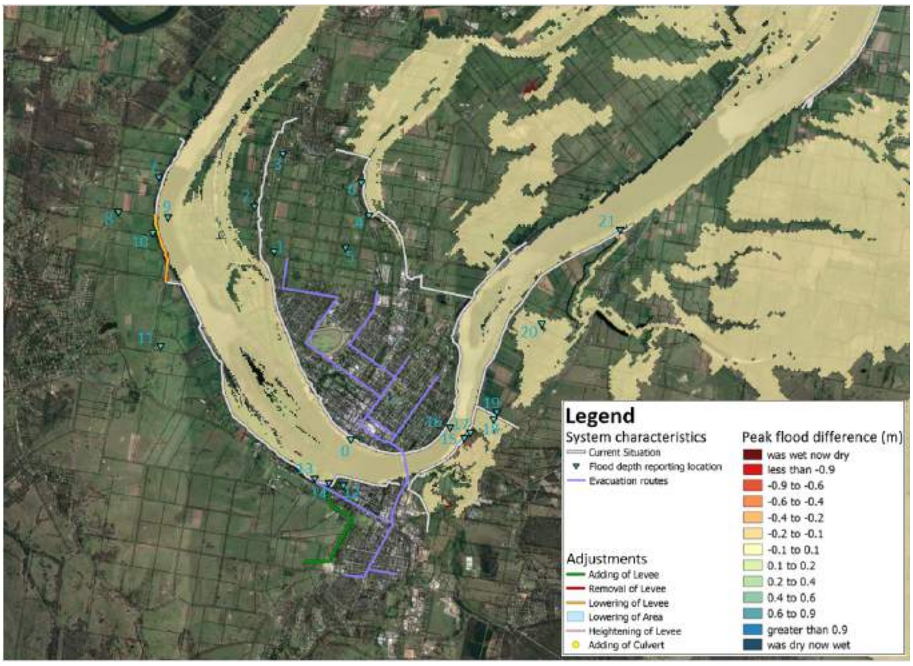


Figure G.15: Peak Flood Difference for scenario 2.3 for the 5 year ARI flood event.

The 5 year ARI flood event shows no difference for scenario 2.3 compared to the reference case. The water level does not rise enough to overtop the lowered Waterview Levee.

Scenario 2.3, lowering the Waterview Levee and adding a levee near South Grafton, is presented in the report (see section 5.1.2 in the main report). The consequences of this scenario showed a significant change in the situation during a 20 year ARI flood event.

Scenario 2.3, lowering the Waterview Levee and adding a levee near South Grafton, is presented in the report. The consequences of this scenario showed a significant change in the situation during a 50 year ARI flood event. Therefore this result is presented in Section 5.1.3 in the main report.

Scenario 2.4

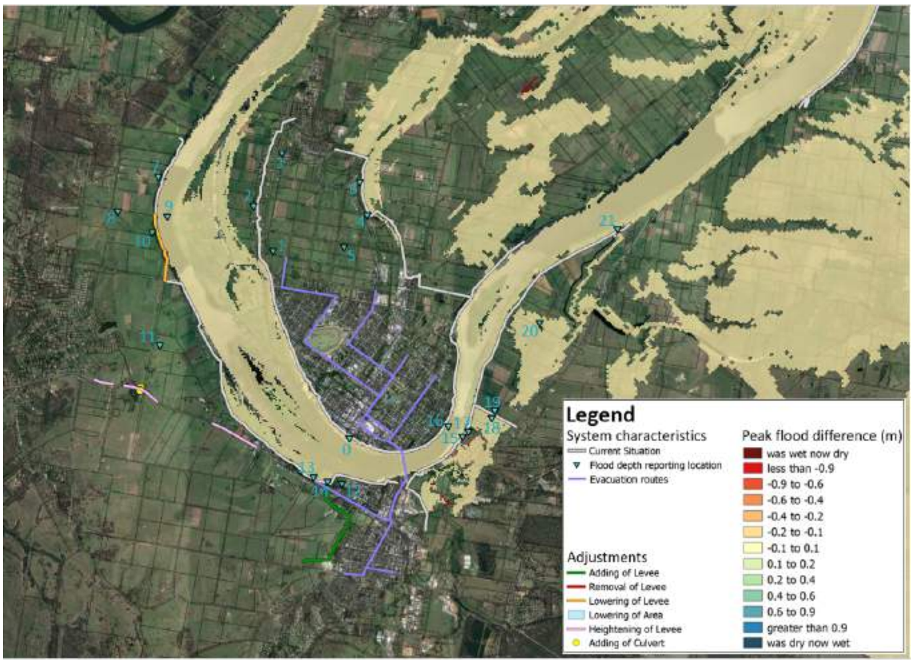


Figure G.16: Peak Flood Difference for scenario 2.4 for the 5 year ARI flood event.

The 5 year ARI flood event shows no difference for scenario 2.4 compared to the reference case. The water level does not rise enough to overtop the lowered Waterview Levee.

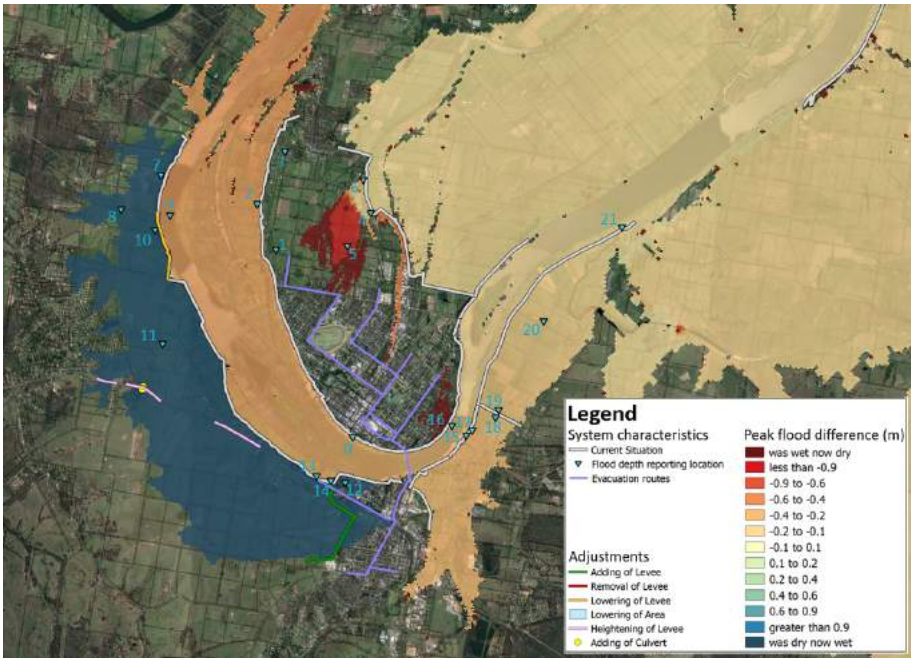


Figure G.17: Peak Flood Difference for scenario 2.4 for the 20 year ARI flood event.

In scenario 2.4, the Gwydir Highway was raised on locations of low elevations. By applying these raises, culverts were constructed underneath it. The raising of those embankments were accompanied by a levee in front of South Grafton. As one can see, the elevation of the South Grafton levee was not sufficient to withhold water from not inundating the Gwydir Highway. The Gwydir Highway still floods during a 20 year ARI flood event. Just like for scenario 2.2 and 2.3, Grafton however experiences less flooding.

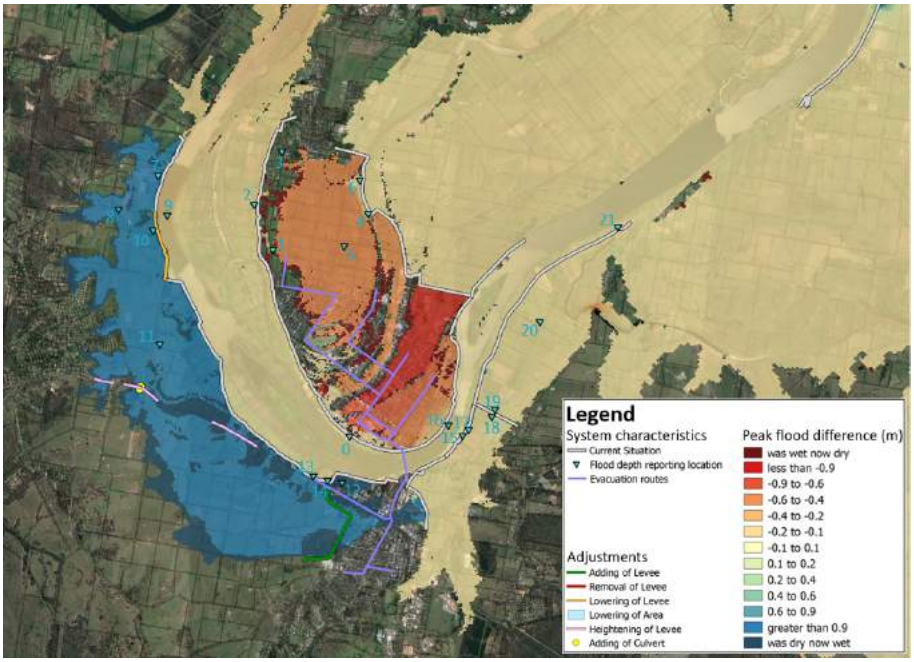


Figure G.18: Peak Flood Difference for scenario 2.4 for the 50 year ARI flood event.

The 50 year ARI flood event in combination with scenario 2.4 has the same effect as the 20 year ARI flood event. The Gwydir Highway still overtops, just like the South Grafton levee. Despite this one can see that Grafton experiences a smaller inundation depth in this scenario.

G.1.3. Area 3: South Grafton

In South Grafton, only one scenario was proposed. This scenario (3.1) included a streamlining of the river bank near South Grafton.

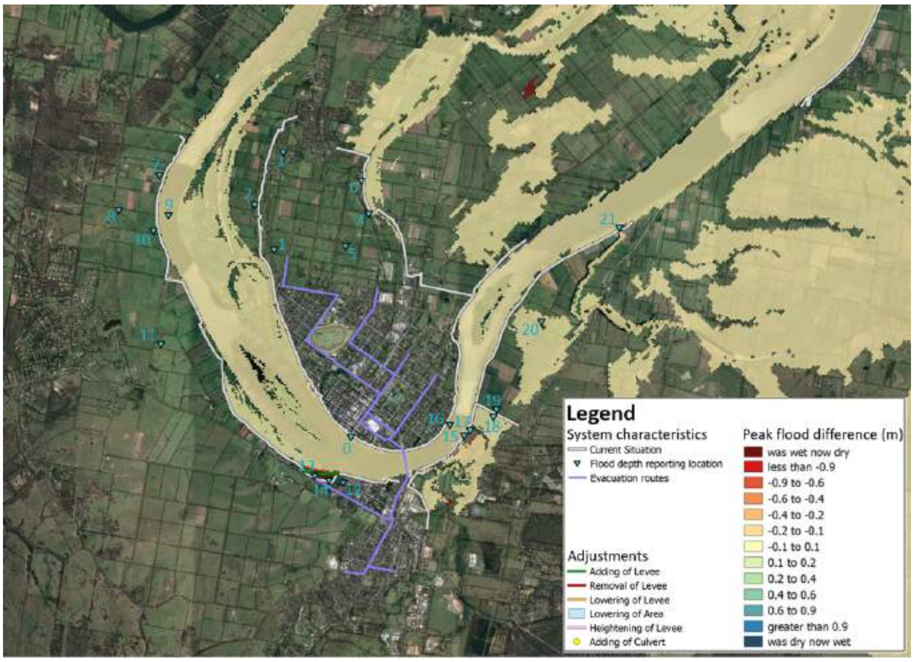


Figure G.19: Peak Flood Difference for scenario 3.1 for the 5 year ARI flood event.

During a 5 year ARI flood event the measurement taken near South Grafton does not change the situation. The only noticeable difference is that the part behind the new built levee was wet and stays dry now. But, this is due to the fact that the levee has moved forward. So, no desirable result is obtained from this scenario during a 5 year ARI flood event.

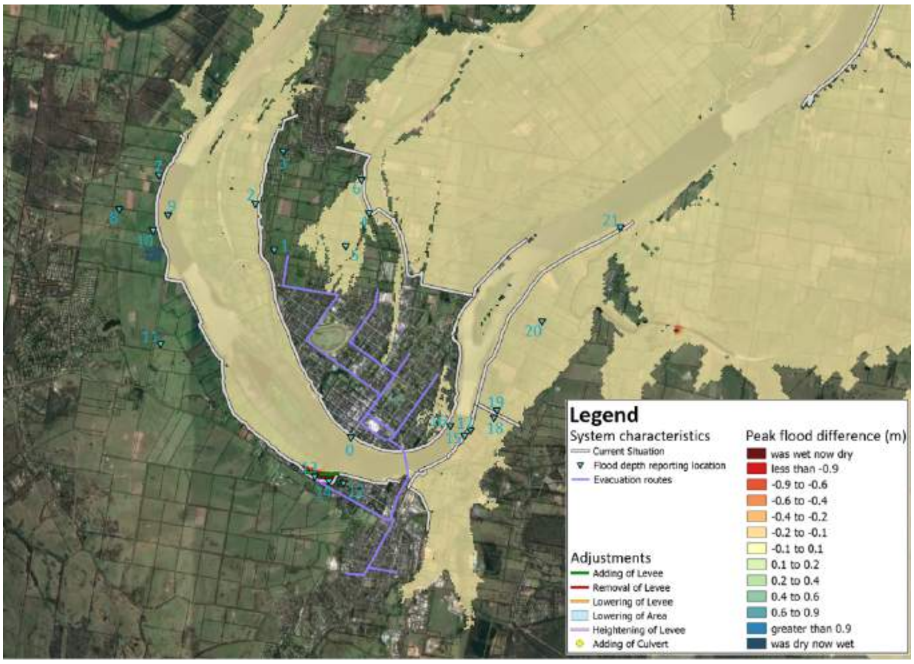


Figure G.20: Peak Flood Difference for scenario 3.1 for the 20 year ARI flood event.

During a 20 year ARI flood event, just like the 5 year ARI flood event, there is no noticeable difference (see Figure G.20). The size of the measurement is too small to have an impact on the total area.

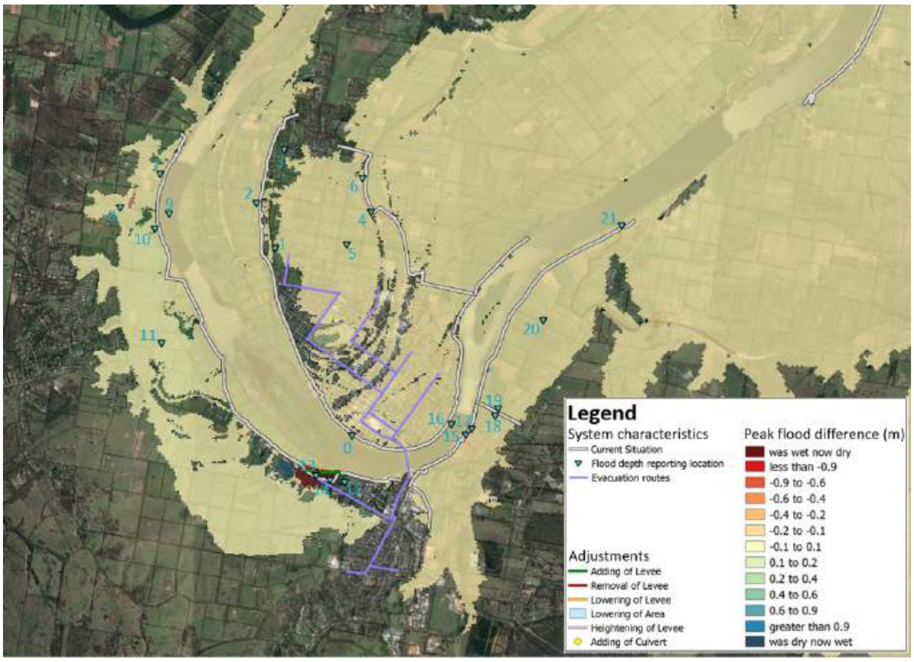


Figure G.21: Peak Flood Difference for scenario 3.1 for the 50 year ARI flood event.

In the 50 year ARI flood event, one can see a minor difference compared to the lower ARI flood events, but this doesn't influence flooding of urban areas. The location where the levee overtops shifts a few meters upstream, but the impact is negligible.

G.1.4. Area 4: Clarenza Floodplain

The scenarios considered for the Clarenza Floodplain area are all based on the removal of the control levee. Scenario 4.1 is the removal of the control levee. In scenario 4.2, the river has been widened to 460 meters, this is the width at the bridge crossing. In scenario 4.3, the Swan Creek Levee and Alipou Levee are moved towards the Pacific Highway to create a floodplain. In scenario 4.4, a secondary channel has been created by removal of small parts of the Swan Creek Levee and the Alipou Levee. The last scenario, 4.5, considers the complete removal of the dike system downstream of South Grafton.

Scenario 4.1

Scenario 4.1, removal of the control levee is presented in the report (see Section 5.1.4). The consequences of this scenario showed a significant change in the situation during a 5 year ARI flood event.

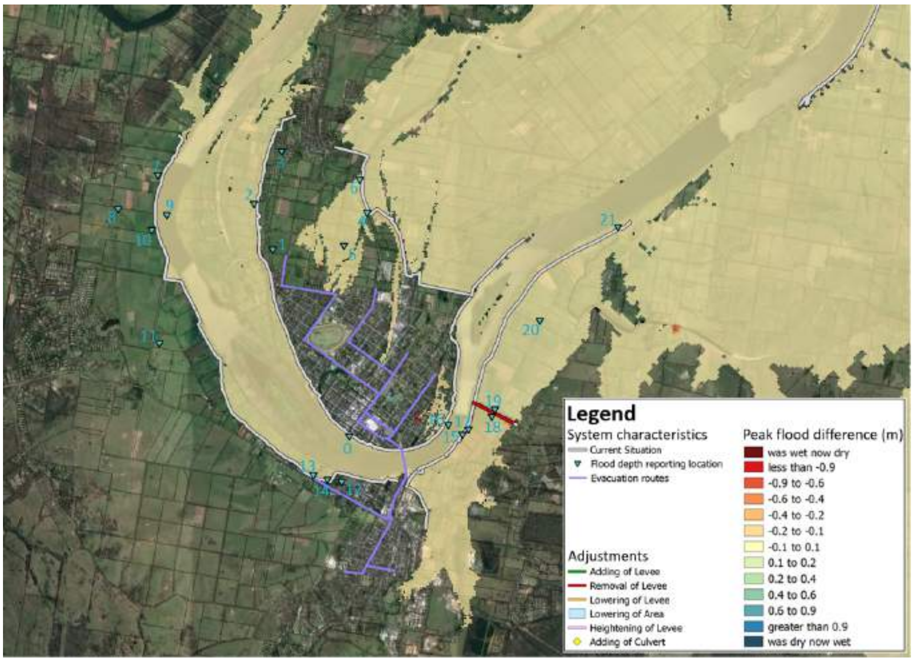


Figure G.22: Peak Flood Difference for scenario 4.1 for the 20 year ARI flood event.

Scenario 4.1 does not show any changes compared to the reference case for the 20 year ARI flood event.

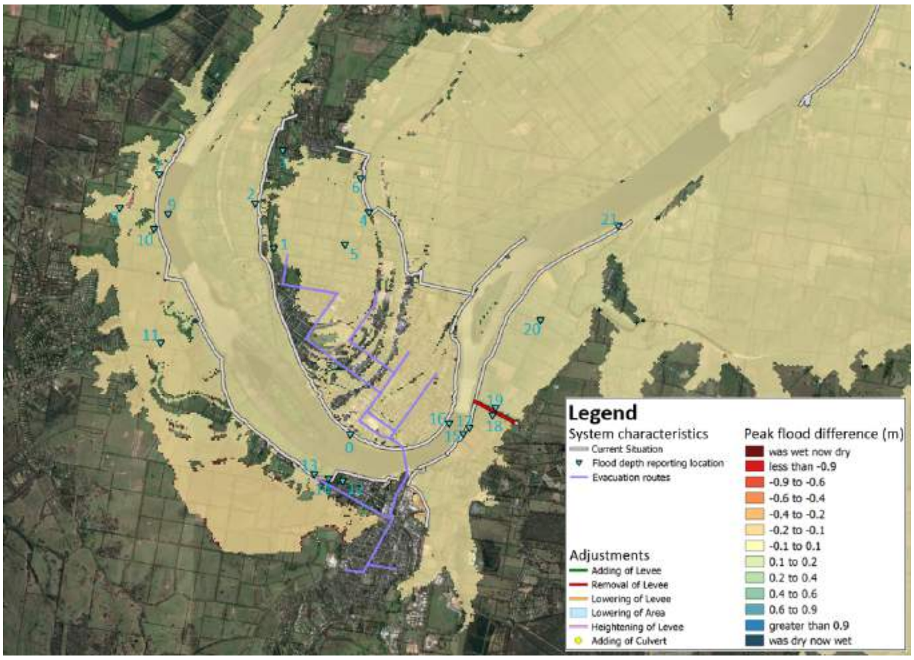


Figure G.23: Peak Flood Difference for scenario 4.1 for the 50 year ARI flood event.

The effect of the control levee in the Clarenza Floodplain can completely be neglected. As can be seen in Figure G.23 the situation does not change in comparison to the reference situation.

Scenario 4.2

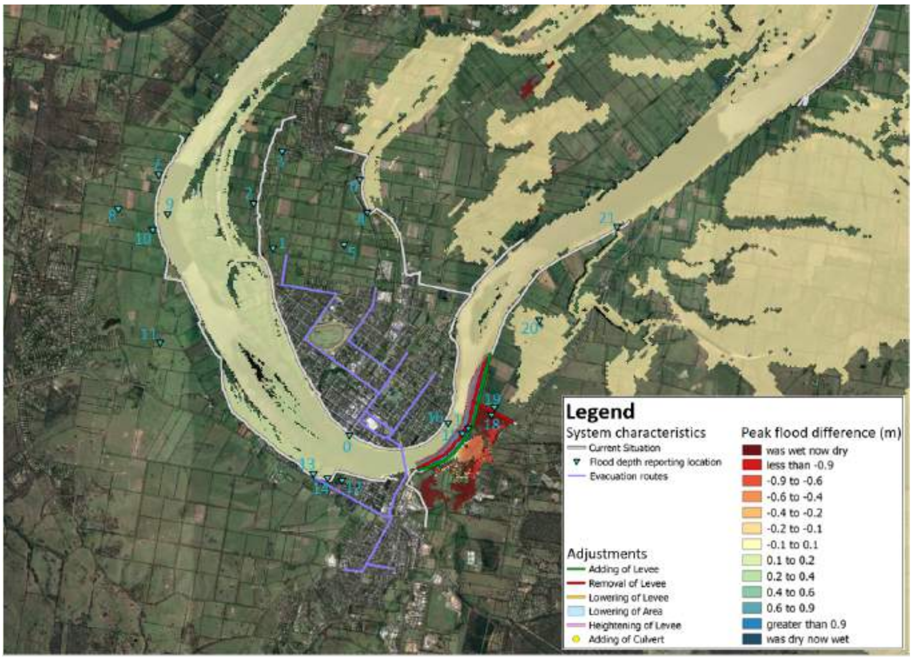


Figure G.24: Peak Flood Difference for scenario 4.2 for the 5 year ARI flood event.

By widening the river in this scenario the volume of water overtopping the Alipou Levee decreases. Some parts of the Clarenza Floodplain stay dry, where they were inundated in the reference situation. The effect of removing the control levee can be neglected, because as one can see, the water does not reach the location of the Clarenza control levee (red line perpendicular on river in Figure G.24).

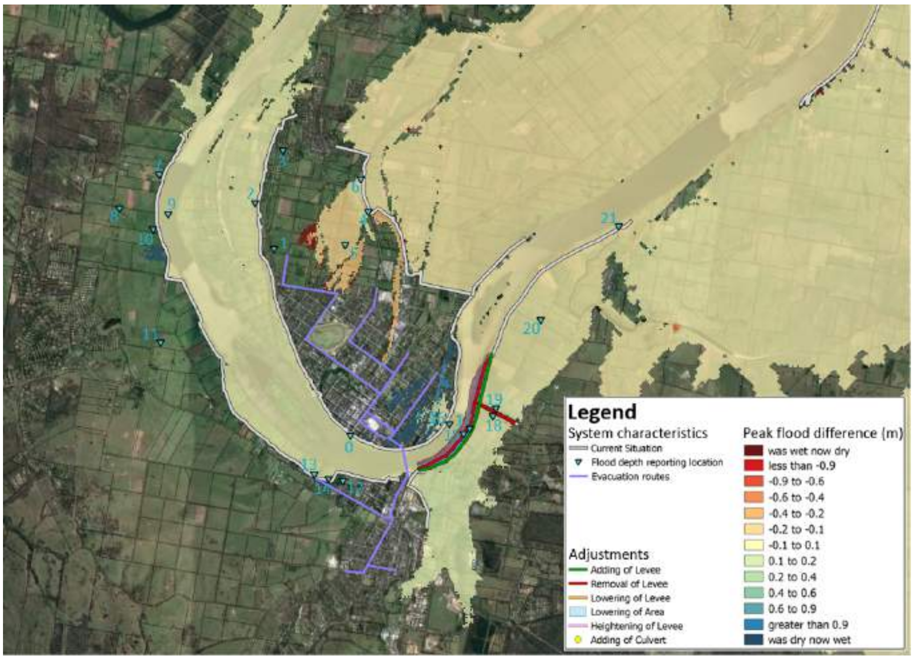


Figure G.25: Peak Flood Difference for scenario 4.2 for the 20 year ARI flood event.

The measures taken in scenario 4.2 do not have the expected outcome compared to the reference situation. Due to widening of the river water seems to overtop the Westlawn Levee (north of Grafton) and increases the inundation depth. Subsequently, the water level in the Clarenza Floodplain raises.

Expectation is that this is due to a modelling input error. Further research is advised.

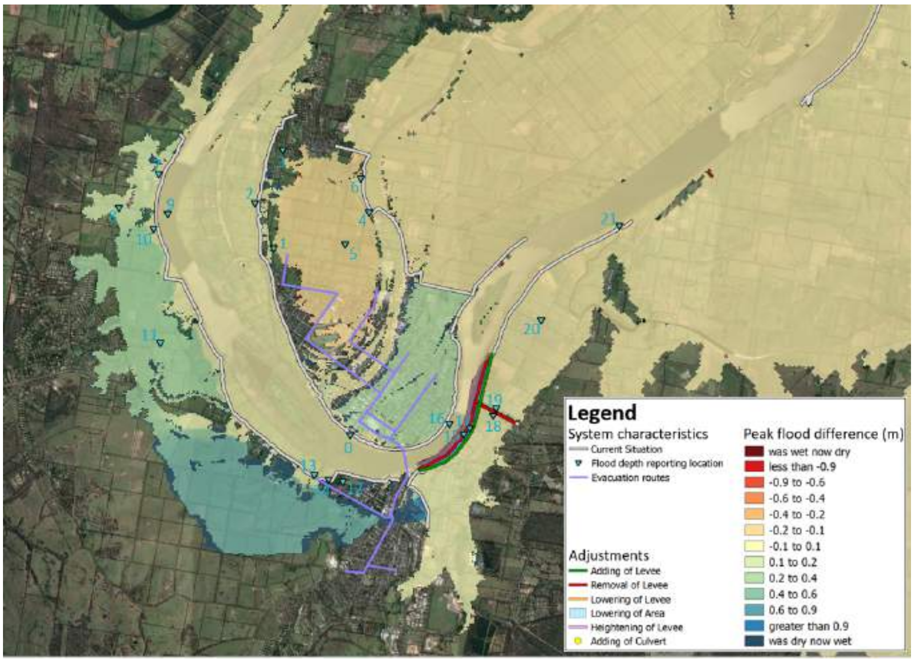


Figure G.26: Peak Flood Difference for scenario 4.2 for the 50 year ARI flood event.

The measures taken in scenario 4.2 have a negative influence on the water level, inundating the Southampton Floodplain and a part of Grafton (Figure G.26). The effect on the Southampton Floodplain is largest. The accessibility of the Gwydir Highway becomes a problem. Also, a small part of South Grafton that remained dry in the reference situation is now inundated. The results of scenario 4.2 are unrealistic. Because space is created for the river, the magnitude of flooding should have been decreased (where it increased now). This could also be a modelling input error, further research is advised.

Scenario 4.3

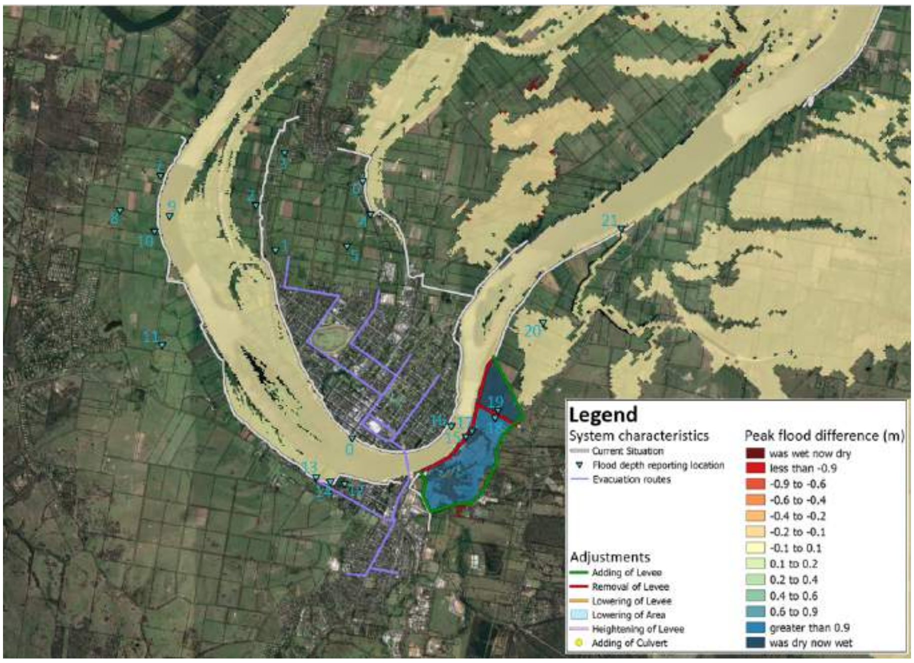


Figure G.27: Peak Flood Difference for scenario 4.3 for the 5 year ARI flood event.

In scenario 4.3, a floodplain was created by shifting the Alipou Levee and Swan Creek Levee to the Pacific Highway. The created floodplain does its job during a 5 year ARI flood event, water is getting stored there. As one can see in Figure G.27, the new situation is not leading to any difference regarding flooding of urban areas.

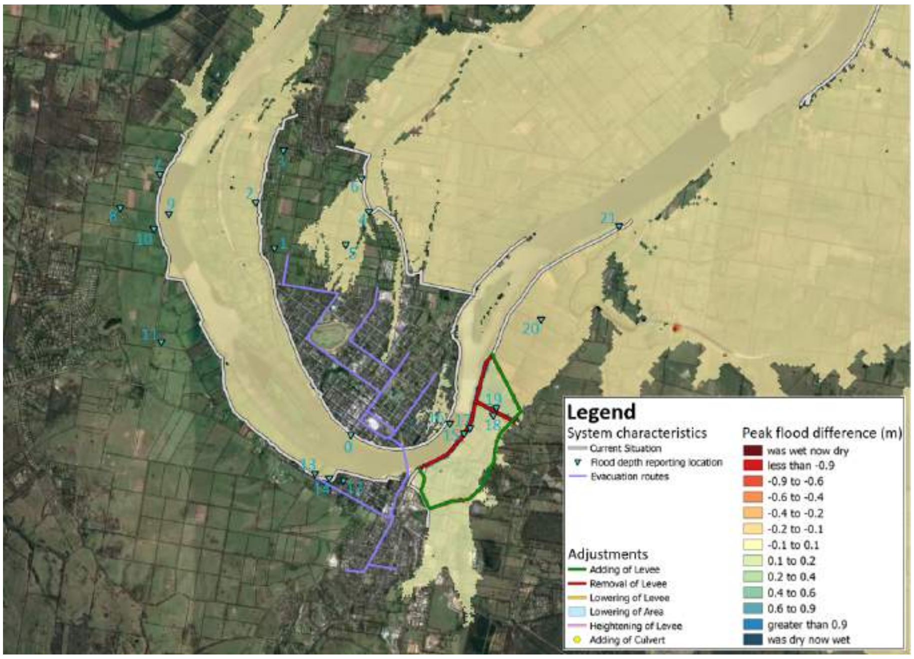


Figure G.28: Peak Flood Difference for scenario 4.3 for the 20 year ARI flood event.

No differences are observed for scenario 4.3 for the 20 year ARI flood event.

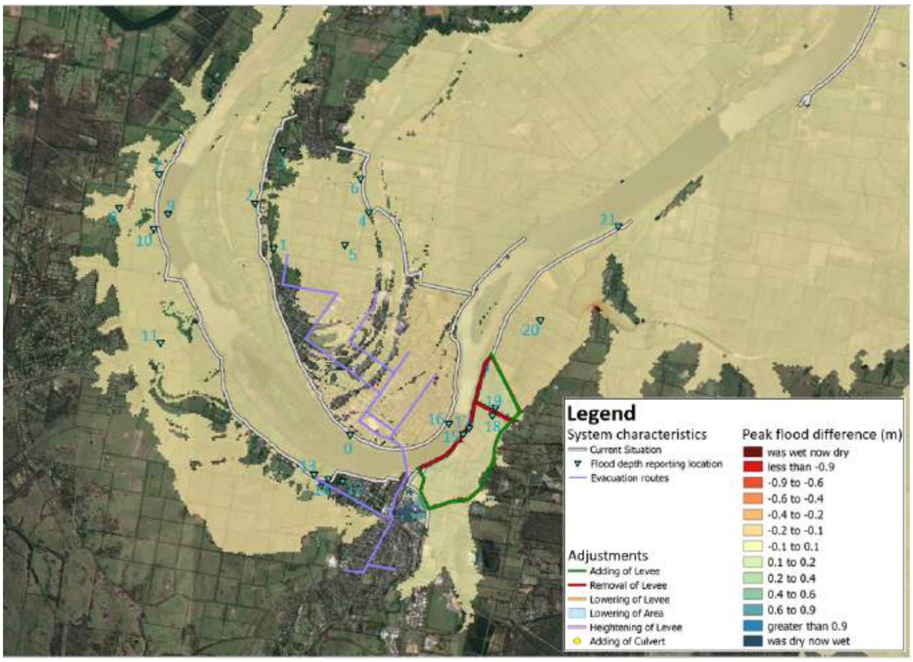


Figure G.29: Peak Flood Difference for scenario 4.3 for the 50 year ARI flood event.

According to Figure G.29, the measures taken in scenario 4.3 have no influence on the reference situation. However, as can be seen in Figure G.26 the action of moving the levee more land inwards does have a negative influence compared to the reference situation. In both scenarios, more room for the river in the river bend is created, nevertheless the results do not match, so further research is advised.

Scenario 4.4

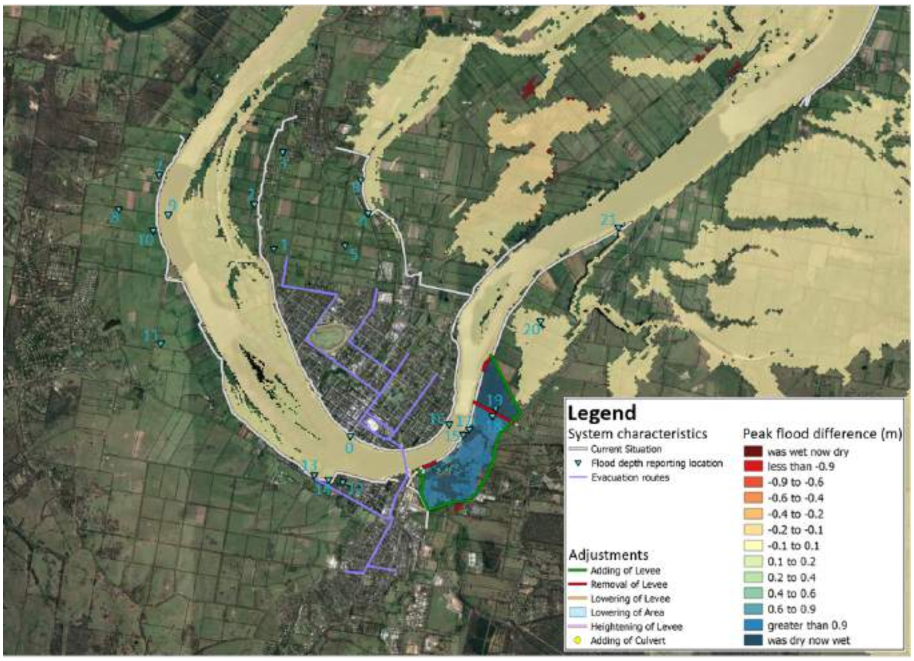


Figure G.30: Peak Flood Difference for scenario 4.4 for the 5 year ARI flood event.

A bypass was included in scenario 4.4. The consequences of this measurement are the same as for scenario 4.3. The new situation does not lead to differences compared to the reference situation (see Figure G.30.

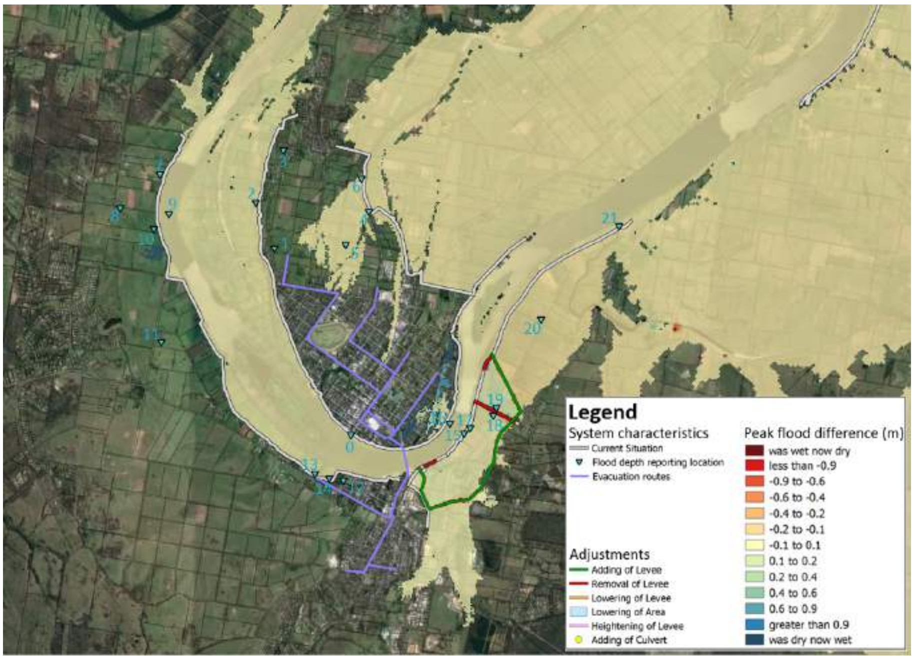


Figure G.31: Peak Flood Difference for scenario 4.4 for the 20 year ARI flood event.

No differences are observed for scenario 4.4 for the 20 year ARI flood event.

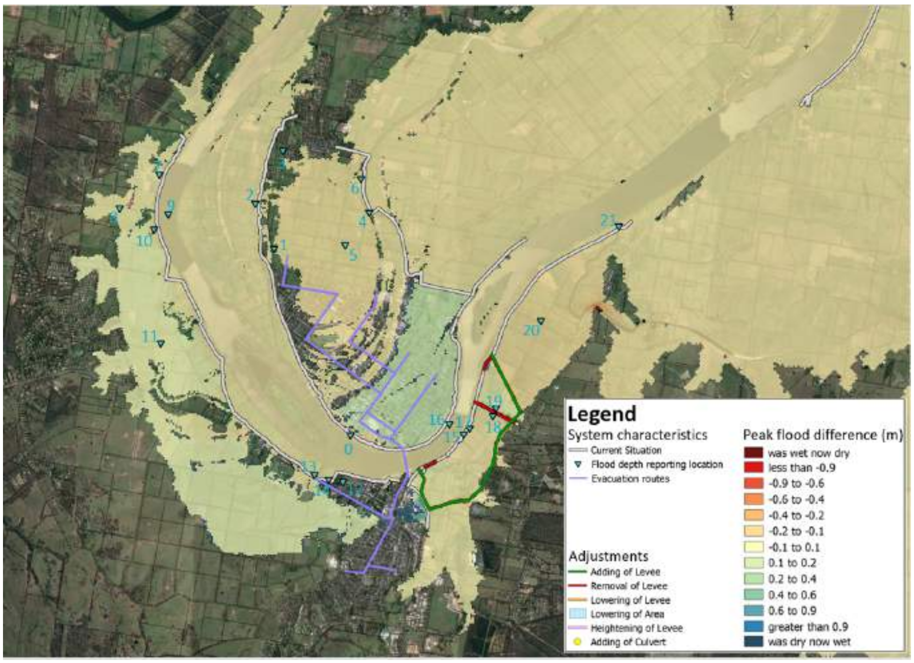


Figure G.32: Peak Flood Difference for scenario 4.4 for the 50 year ARI flood event.

The measures taken in scenario 4.4 do not result in keeping the urban area dry. The water level in Grafton does increase in this case. This can be the result of a return flow towards Grafton.

Scenario 4.5

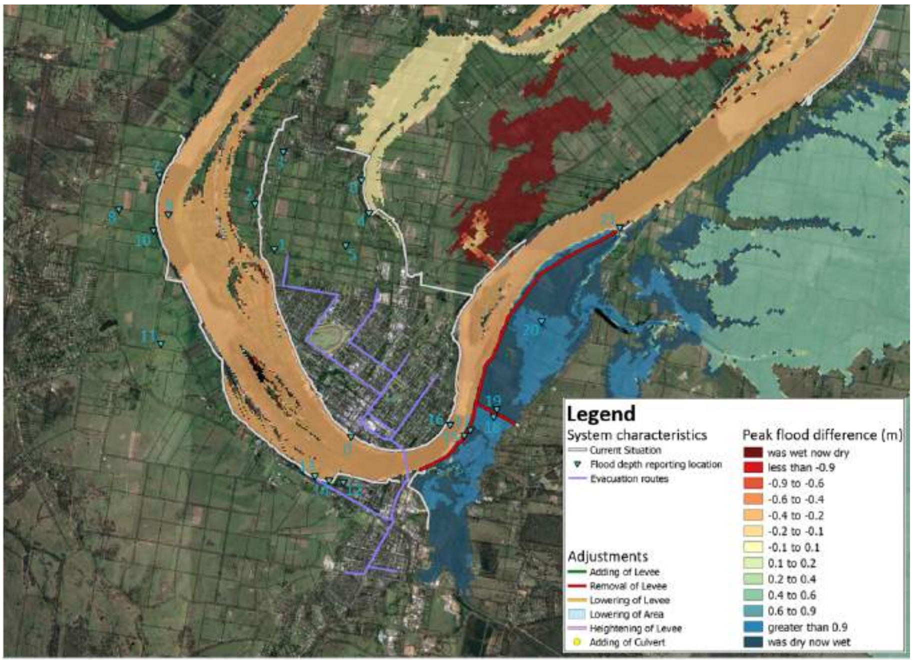


Figure G.33: Peak Flood Difference for scenario 4.5 for the 5 year ARI flood event.

The removal of the Alipou Basin Levee, Swan Creek Levee and the Clarenza Levee result in flooding of a large area downstream of Grafton. The areas inundated are not considered as urban areas and therefore these consequences are positive. However, South Grafton gets partly inundated during a 5 year ARI flood event. This is not desirable. Another notion that can be made is the fact that the entire river system is experiencing a 0.5 meter drop in water level, due to the removal of the constriction in the narrow bend.

Scenario 4.5, removal of all levees downstream of South Grafton, is presented in the report for both the 20 and 50 year ARI flood event (see Sections 5.1.5 and 5.1.6). The consequences of this scenario showed a significant change in the situation during a 20 and a 50 year ARI flood event.

G.1.5. Area 5: Hill Ridge

Higher grounds are present on Woodford Island. These higher grounds create narrow passages for the Clarence River. The scenarios created for these bottlenecks are the following. In scenario 5.1, the higher grounds north of Woodford Island are removed. In scenario 5.2, the higher grounds south of Woodford Island are removed. Scenario 5.3 is a combination of both scenarios.

Scenario 5.1

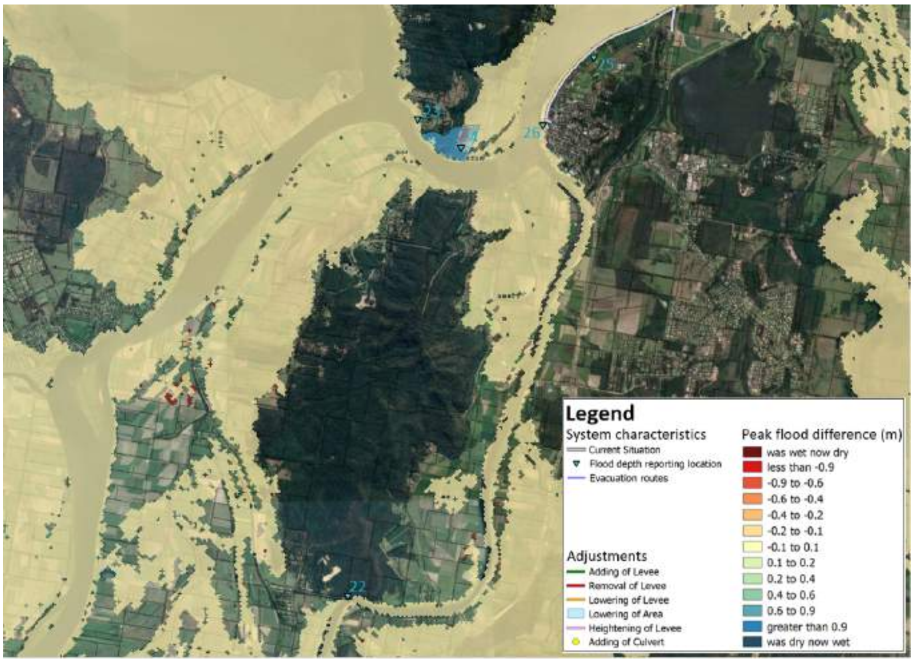


Figure G.34: Peak Flood Difference for scenario 5.1 for the 5 year ARI flood event.

In the case of a 5 year ARI flood event the removed area north of Woodford Island gets inundated and a minor decrease in inundation level can be noted upstream (red in Figure G.34). Due to the small area these results are negligible.

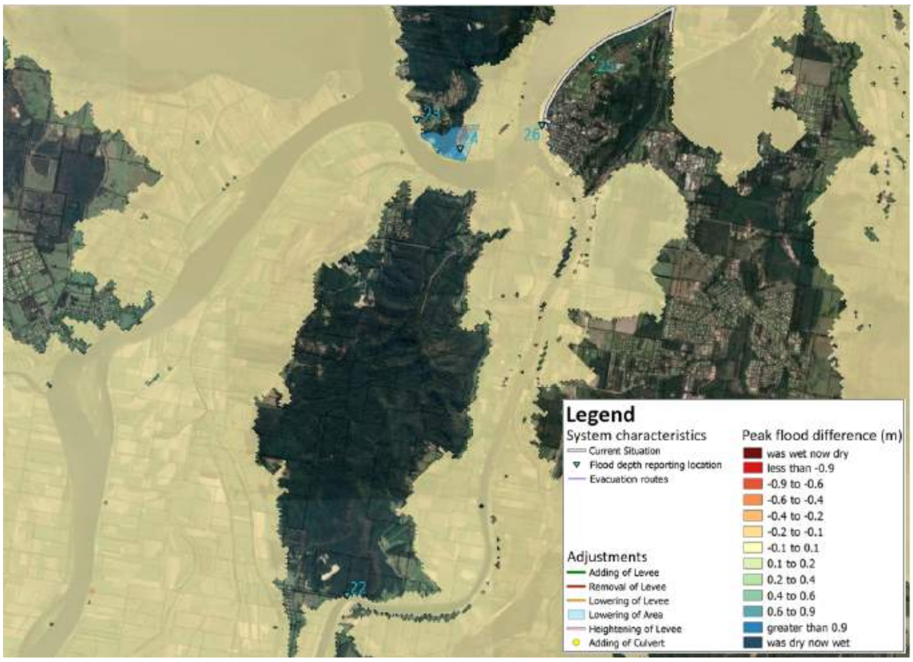


Figure G.35: Peak Flood Difference for scenario 5.1 for the 20 year ARI flood event.

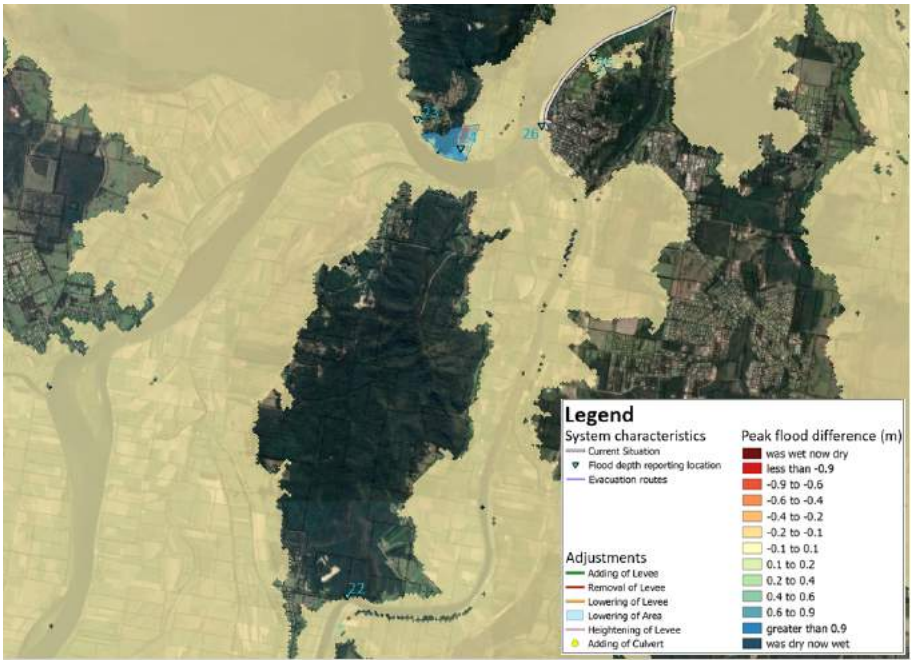


Figure G.36: Peak Flood Difference for scenario 5.1 for the 50 year ARI flood event.

No difference are observed for both the 20 and 50 year ARI flood events for scenario 5.1 compared to the reference scenario. Some minor changes can be extinguished, but these are negligible. It is safe to state that the measures taken around Woodford Island for scenario 5.1 will not make a difference in flood mitigation in the Clarence Valley. At the same time, the measures applied would be very difficult in execution and at the same time expensive.

Scenario 5.2

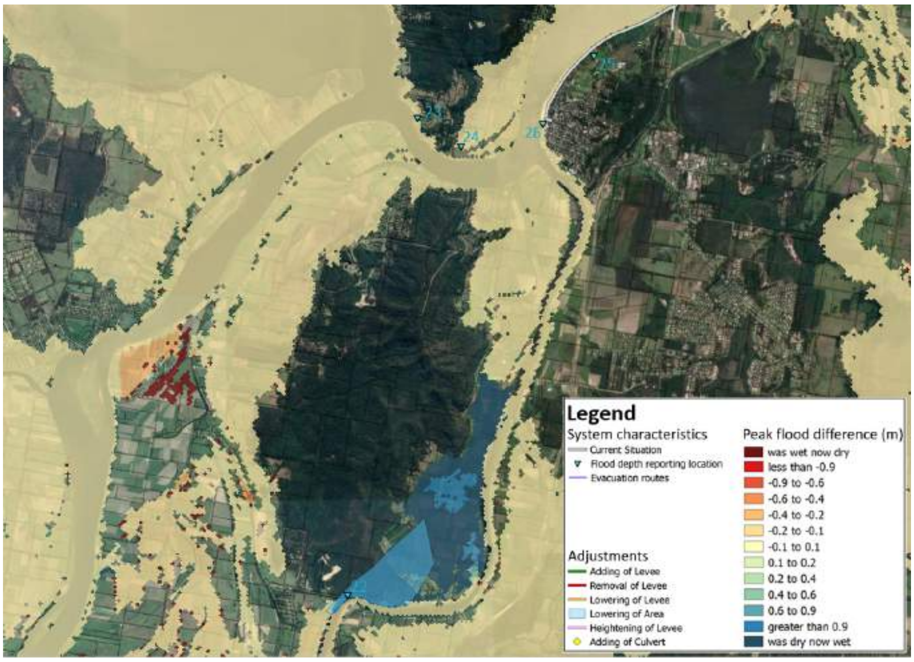


Figure G.37: Peak Flood Difference for scenario 5.2 for the 5 year ARI flood event.

By the same reasoning as for scenario 5.1, the impact of removing the higher grounds south of Woodford Island is negligible. Once again, the removed area gets flooded, but the impact upstream is to small to be considered a positive effect.

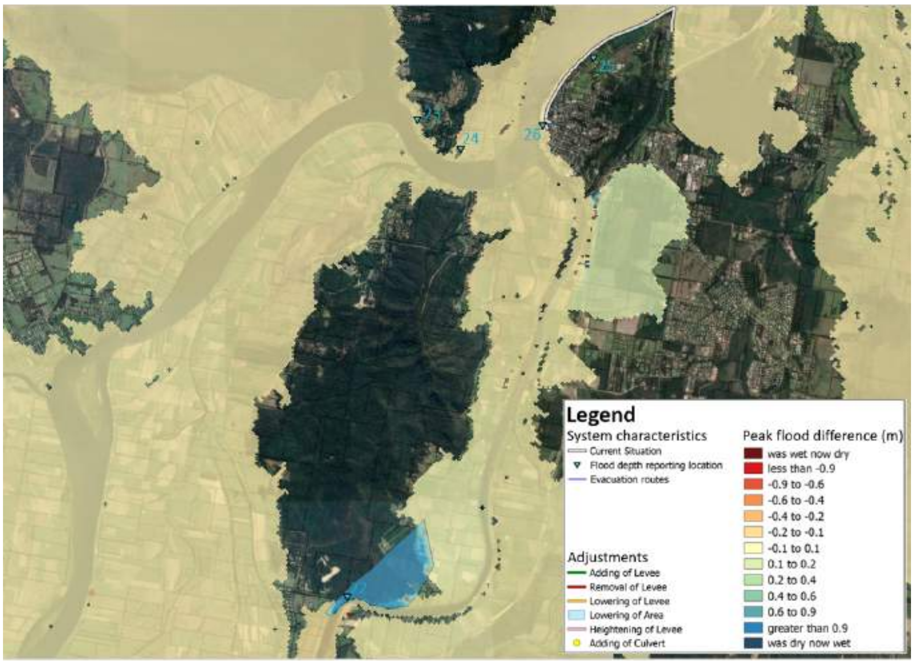


Figure G.38: Peak Flood Difference for scenario 5.2 for the 20 year ARI flood event.

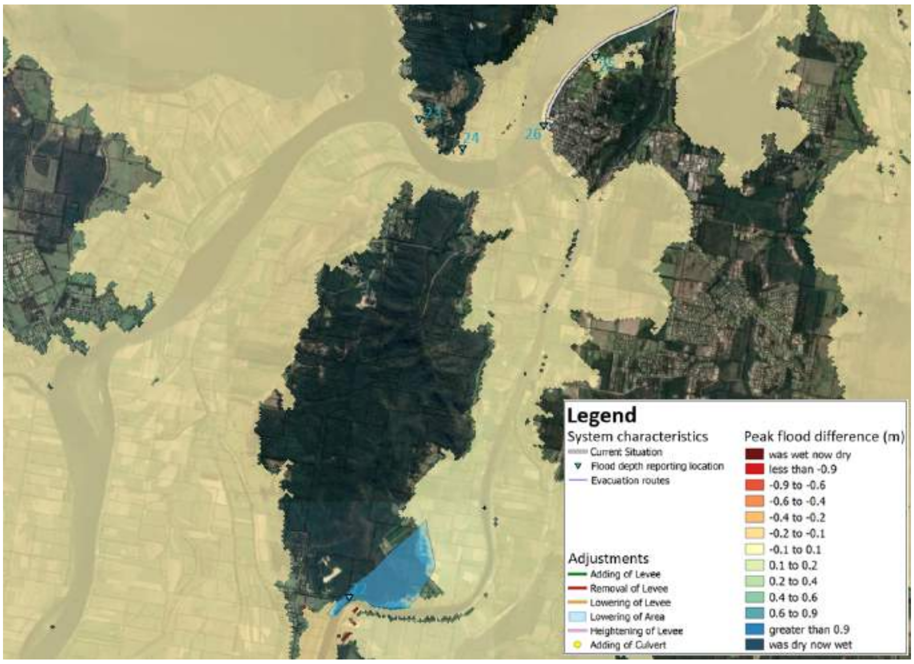


Figure G.39: Peak Flood Difference for scenario 5.2 for the 50 year ARI flood event.

No difference are observed for both the 20 and 50 year ARI flood events for scenario 5.2 compared to the reference scenario. Some minor changes can be extinguished, but these are negligible. It is safe to state that the measures taken around Woodford Island for scenario 5.2 will not make a difference in flood mitigation in the Clarence Valley. At the same time, the measures applied would be very difficult in execution and at the same time expensive.

Scenario 5.3

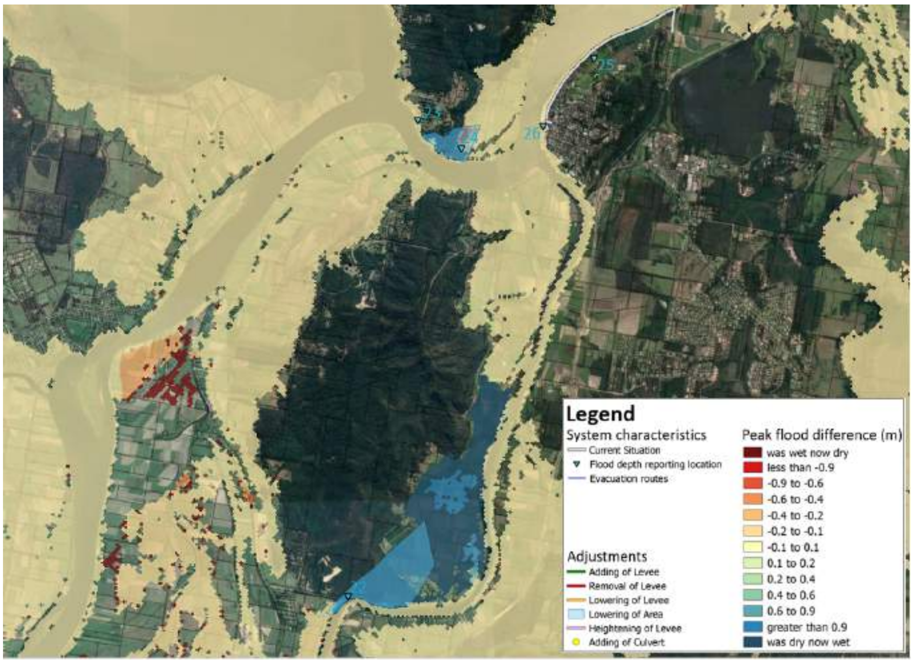


Figure G.40: Peak Flood Difference for scenario 5.3 for the 5 year ARI flood event.

A combination of both areas provides the same results as the single scenarios.

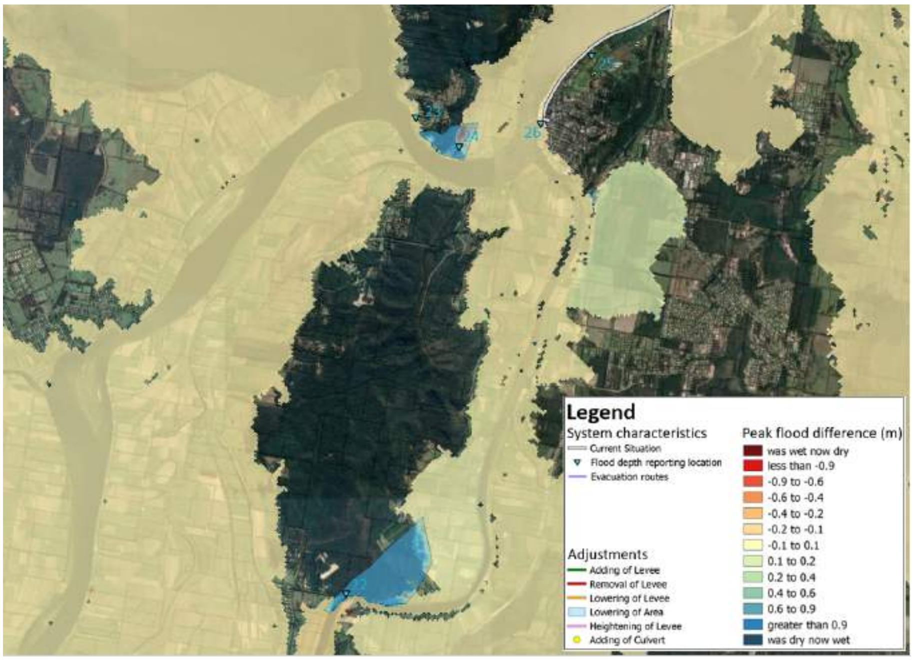


Figure G.41: Peak Flood Difference for scenario 5.3 for the 20 year ARI flood event.

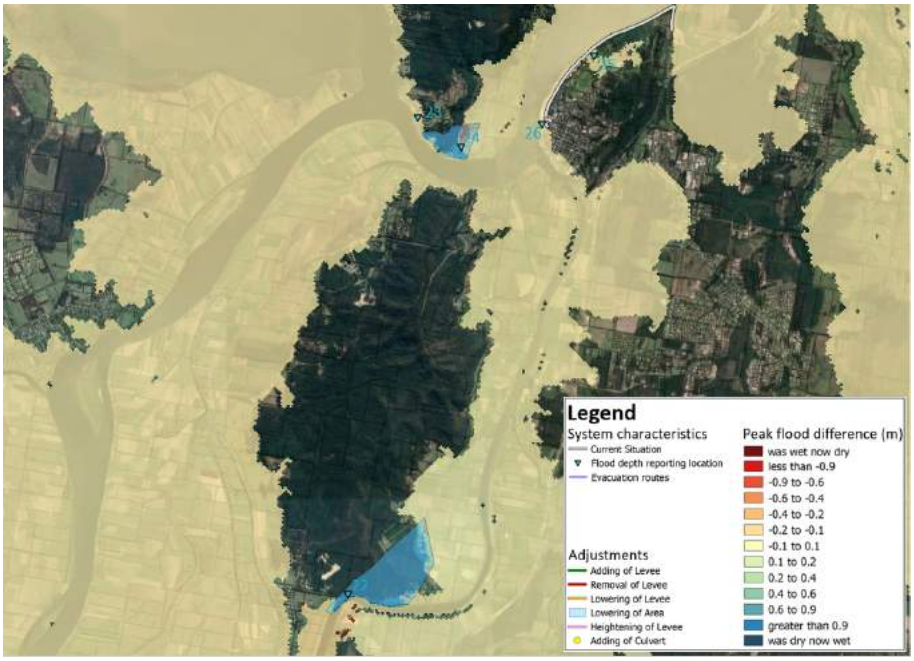


Figure G.42: Peak Flood Difference for scenario 5.3 for the 50 year ARI flood event.

No difference are observed for both the 20 and 50 year ARI flood events for scenario 5.3 compared to the reference scenario, since scenario 5.3 is a combination of scenario 5.1 and 5.2. Some minor changes can be extinguished, but these are negligible. It is safe to state that the measures taken around Woodford Island for scenario 5.3 will not make a difference in flood mitigation in the Clarence Valley.

G.1.6. Area 6: Maclean

Scenario 2.3, has a positive effect on the impact of flooding in Maclean (see Section 5.1.3). The consequences of this scenario showed a significant change in the situation during a 50 year ARI flood event.

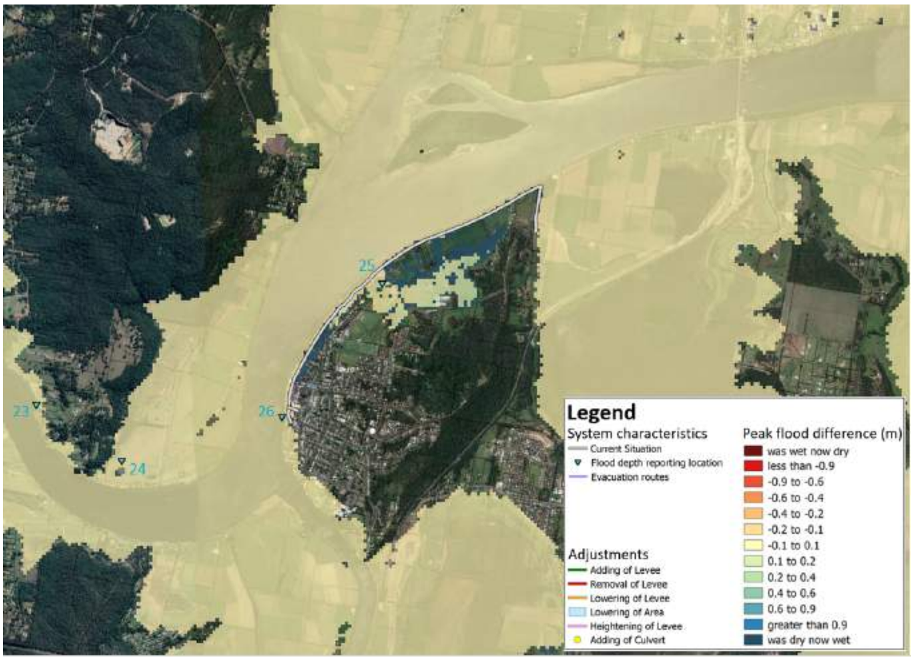


Figure G.43: Peak Flood Difference for scenario 4.5 for the 50 year ARI flood event for Maclean

As one can see in Figure G.43 the upstream measures of scenario 4.5 do negatively impact flooding in Maclean during a 50 year ARI flood event. A larger area of Maclean will be inundated (dark blue).

G.2. Multi-Criteria Analysis

To determine the usability of scenario measures to construct combinations, a multi-criteria analysis is performed. Per scenario, the 5, 20 and 50 year ARI flood events are shown as 5, 20 and 50 respectively. These scenarios and events are scored, using the following signs.

The first objective is assessed using the following indicators (negative signs are not used as a big change is not necessarily negative):

- ++ (minor change);
- + (average change);
- 0 (big change).

The second objective is assessed using the following indicators (negative signs are not used as a minor change is not necessarily negative):

- ++ (big change);
- + (average change);
- 0 (minor change).

The third and fourth objective are assessed using the following indicators:

- ++ (very positive);
- + (positive);
- 0 (no or nearly no effect);
- - (negative);
- - - (very negative).

The last column, total score, is a horizontal summation of all the signs (where a '-' cancels a '+'). Section 5.2 shows a vertical summation of all the signs for one single scenario as it is interesting how the scenario scores on all the simulated events.

Table G.1: Multi-criteria analysis on the modelled scenario events constructed for area 1 to 5.

		Scale of the implemented mitigation measure	Overall effect on water level changes	Effect on urban areas locally	Effect on urban areas downstream	Total score
Scenario 1.1	5	++	0	0	0	++
	20	++	+	- -	0	+
	50	++	+	- -	0	+
Scenario 1.2	5	+	0	0	0	+
	20	+	0	0	0	+
	50	+	0	+	0	++
Scenario 1.3	5	0	0	0	0	0
	20	0	+	+	0	++
	50	0	+	++	0	+++
Scenario 2.1	5	++	0	0	0	++
	20	++	++	- -	0	++
	50	++	++	- -	++	++++
Scenario 2.2	5	0	0	0	0	0
	20	0	++	- -	0	0
	50	0	++	- -	++	++
Scenario 2.3	5	+	0	0	0	+
	20	+	++	++	0	+++++
	50	+	++	++	++	+++++++
Scenario 2.4	5	0	0	0	0	0
	20	0	++	++	0	++++
	50	0	++	++	++	++++++
Scenario 3.1	5	+	0	0	0	+
	20	+	0	0	0	+
	50	+	0	0	0	+
Scenario 4.1	5	++	+	0	0	+++
	20	++	0	0	0	++
	50	++	0	0	0	++
Scenario 4.2	5	0	+	0	0	+
	20	0	0	- -	0	- -
	50	0	+	- -	0	-
Scenario 4.3	5	0	+	0	0	+
	20	0	0	-	0	-
	50	0	0	0	0	0
Scenario 4.4	5	0	+	0	0	+
	20	0	0	-	0	-
	50	0	0	-	0	-
Scenario 4.5	5	0	++	0	0	++
	20	0	+	++	0	+++
	50	0	++	++	-	+++
Scenario 5.1	5	0	0	0	0	0
	20	0	0	0	0	0
	50	0	0	0	0	0
Scenario 5.2	5	0	0	0	0	0
	20	0	0	0	0	0
	50	0	0	0	0	0
Scenario 5.3	5	0	0	0	0	0
	20	0	0	0	0	0
	50	0	0	0	0	0

G.3. Combinations

As a result of the multi-criteria analysis, two combinations have been formed. The best scoring scenarios have been combined into new simulation runs. The first combination consists of scenario 1.3, scenario 2.3 and scenario 4.1. For this combination, the levee north of Grafton is constructed, the Westlawn levee is lowered, the Waterview levee is lowered, a levee near South Grafton is constructed and the control levee in the Clarenza Floodplain is removed. For the second combination, scenario 1.3, scenario 2.3 and scenario 4.5 are used. This combination contains the construction of a levee north of Grafton together with a lowering of the Westlawn levee, a lowering of the Waterview levee and a construction of a levee near South Grafton. Together with this, all levees downstream of South Grafton are removed. These combinations are simulated for the 5, 20 and 50 year ARI flood events.

5 year ARI flood event

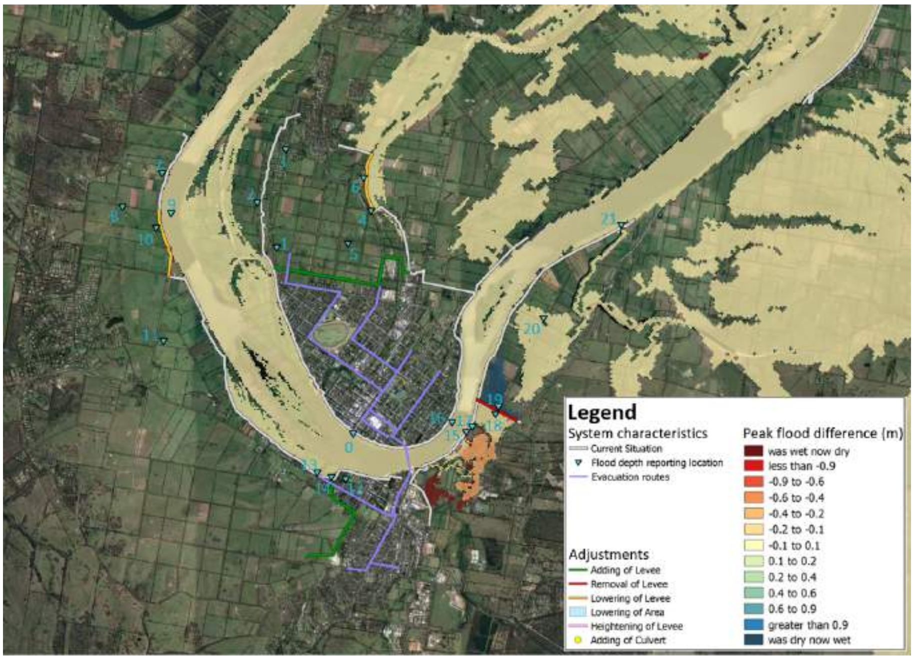


Figure G.44: Peak Flood Difference for combination 1 for the 5 year ARI flood event.

In case of a 5 year ARI flood event one can see the same result as for scenario 4.1. Only the part near the Clarenza control levee gets inundated.

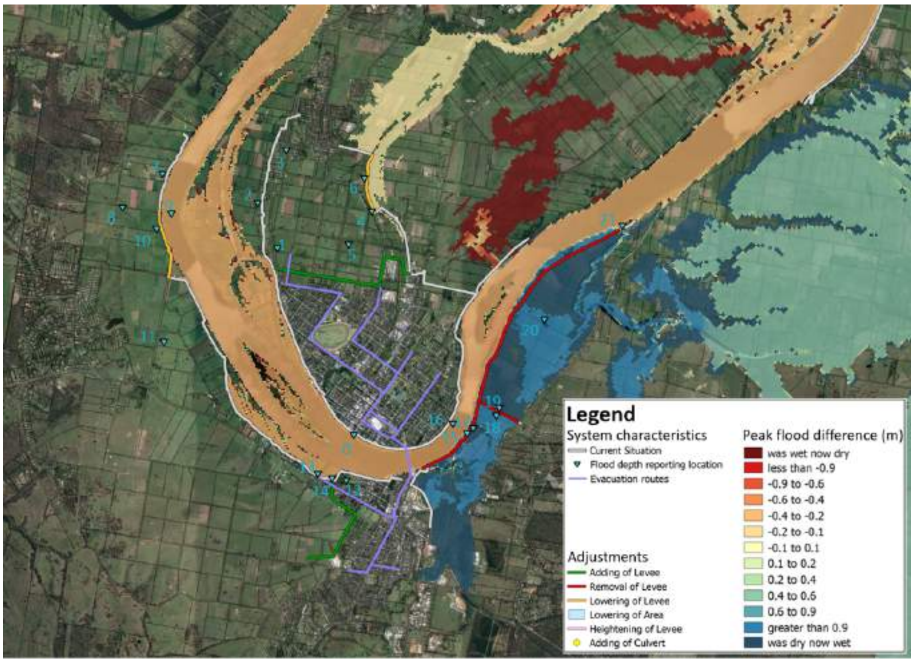


Figure G.45: Peak Flood Difference for combination 2 for the 5 year ARI flood event.

In case of a 5 year ARI flood event one can see the same result as for scenario 4.5. No urban areas will be inundated.

20 year ARI flood event

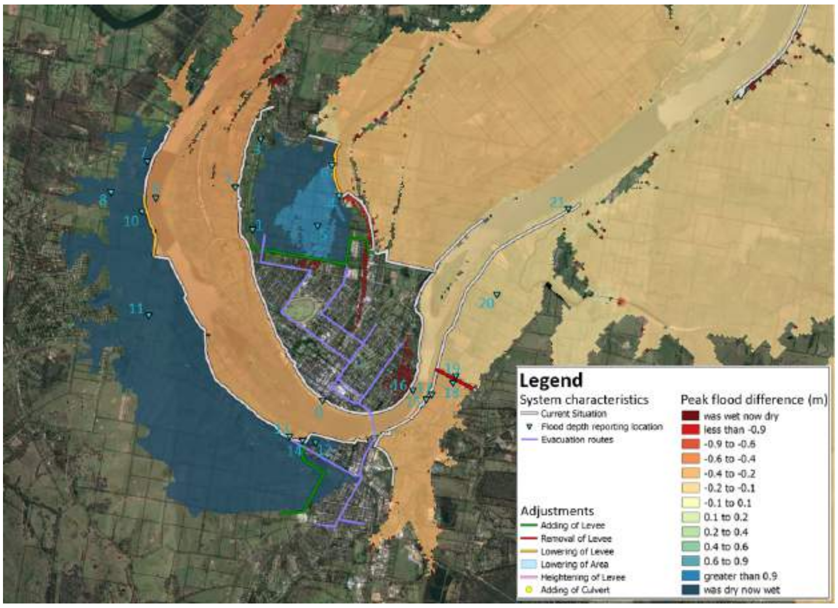


Figure G.46: Peak Flood Difference for combination 1 for the 20 year ARI flood event.

The first combination shows a positive result compared to the reference situation for the 20 year ARI flood event. As one can see Baker's Swamp and the Southampton Floodplain fill with water. As a result, the general water level in the area drops up to 0.5 meters. This makes sure that Grafton keeps dry. It should be noted that the levee at South Grafton should be of higher elevation or better connected to the surrounding levees to contain the water. This requires further research.

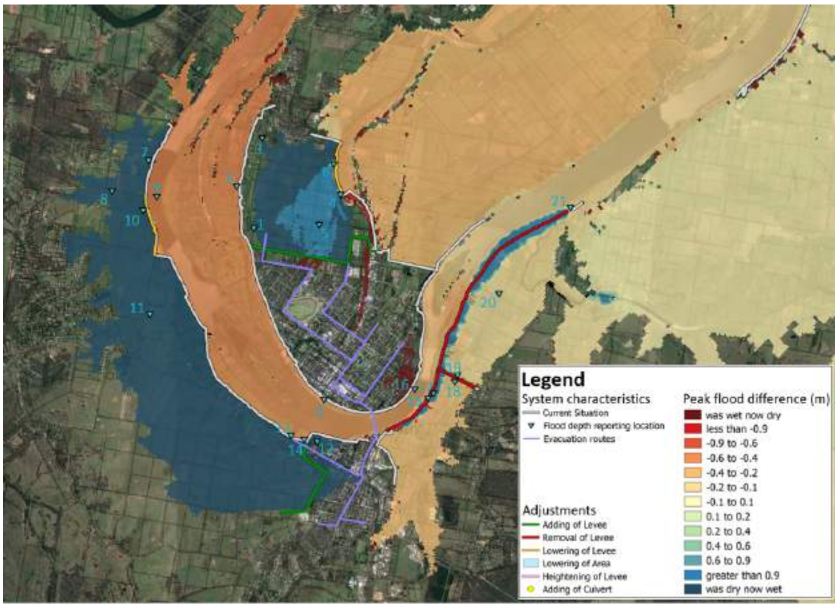


Figure G.47: Peak Flood Difference for combination 2 for the 20 year ARI flood event.

For the second combination one can see that acquired results are roughly equal to combination 1. The water level in the Clarence River drops, Baker's Swamp and Southampton Floodplain fill up with water. These results are not presented as urban areas are already protected if scenario 2.3 is applied for the 20 year ARI event.

50 year ARI flood event

These results are shown in Section 5.3.1 and 5.3.2 in the main report, as scenario combinations are particularly interesting for this kind of flooding event.

G.4. Water Levels and Discharges

Each TUFLOW simulation includes 150 hours of data. To assess the usability of the selected promising flood mitigation measures (based upon the multi-criteria analysis) and the combinations to a greater extend, tables with water levels and discharges are created. In these tables, the maximum water levels and discharges and their differences compared to the reference simulation are shown. Note, the maximums do not necessarily represent the same timestep. For the following scenarios tables are presented:

- Scenario 1.3;
- Scenario 2.3;
- Scenario 4.1;
- Scenario 4.5;
- Combination 1;
- Combination 2.

The tables show the consequences of the 5, 20 and 50 year ARI flood events. However, only the 50 year ARI flood event was considered promising for scenario 1.3, for instance. Therefore, the promising scenario columns are marked green. From these tables, water levels or discharges with remarkable or interesting features are visualised in graphs separately. Note, in these graphs oscillation could be the result of instability of the numerical model. These water levels or discharges are represented by green marked lines in the tables.

Figure G.48 and G.49 contain the points and lines along which respectively the water levels and discharges over time are measured. The points of water level measurements can be seen in every peak flood difference plot in the main report and appendices. For clarity, the discharge lines are left out in every other figure.

Figure G.48: Output locations for water level (blue) and discharge (yellow) around Grafton.

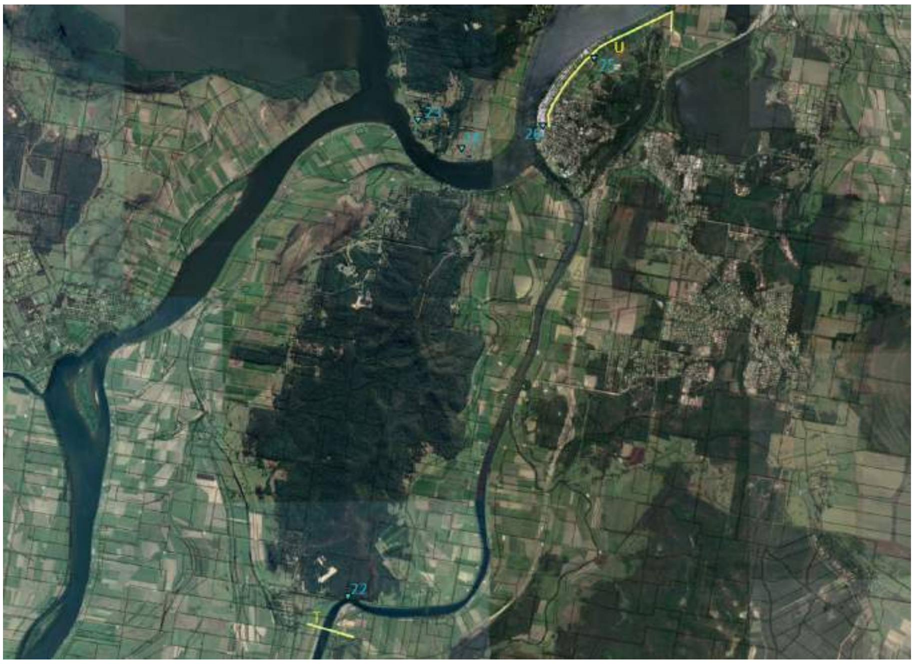


Figure G.49: Output locations for water level (blue) and discharge (yellow) around Maclean.

Water level output
For the differences in peak water levels, a positive value indicates a water level rise and a negative value represents a decrease in water level.

A selection of water level measurements has been included. If the water level difference is insignificant, the location has been left out of the table. The significance of the water level differences has been assessed by applying boundaries. The CVFM has been calibrated to 30 cm (Farr and Huxley, 2013). It is assumed that all the relevant differences are taken into account when the boundaries are below the calibration level. Consequently, if the water level difference falls within the range of -20 to +20 cm for all the ARI flood events, the location has been removed from the table.

Note, the water levels presented in the graphs are not the same as the water depth for that location, as a result of the default TUFLOW settings. The graphs indicate the total water height in $mAHD$. The ground level is plotted if a graph shows a fully horizontal line. If some difference is observed, it means that this location is being inundated.

Discharge output
In the table, a positive value for the discharge differences indicate an increase in discharge. A decrease in discharge is indicated by a negative value.

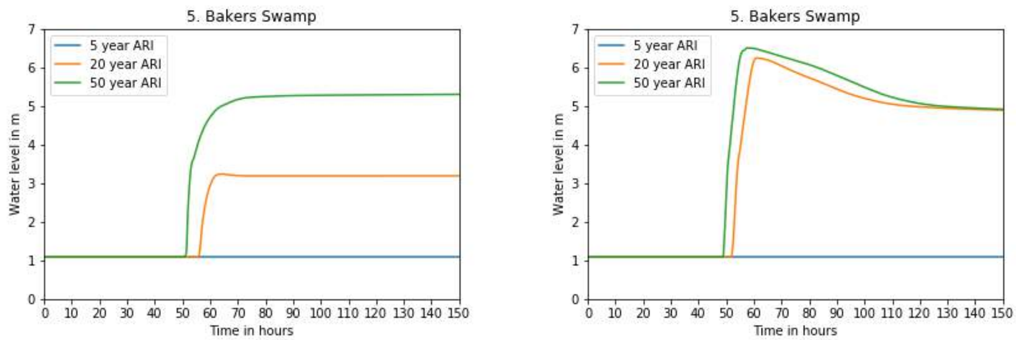
Not all the differences have been included. The values of the discharge differ in magnitude, ranging from only 1 m^3/s to 20,000 m^3/s. For this reason, a percentage based boundary has been included, ranging from -10 to +10 %. If the discharge differences for all three ARI flood events fall within this range, the location of the line has been deleted from the table. The CVFM has not been calibrated on discharges, so the ten percent range is based on expert judgement.

G.4.1. Scenario 1.3

Table G.2: Water level results for scenario 1.3.

		Maximum water level ($mAHD$)			Water level difference ($mAHD$)		
Location		5	20	50	5	20	50
1. Lower Railway	Area 1	5.85	6.24	6.50	0	0.39	0.65
3. Upper Railway	Area 1	6.28	6.28	6.50	0	0	0.22
4. Alumy Creek Floodgate	Area 1	3.95	6.24	6.50	0	0.55	0.67
5. Baker's Swamp	Area 1	1.10	6.24	6.50	0	3.00	1.20
6. Great Marlow Floodplain	Area 1	5.26	6.24	6.50	0	0.69	0.69

Table G.2 indicates the five locations where significant changes were observed for scenario 1.3. Especially location 5, Baker's Swamp, shows remarkable results. During a 50 year ARI flood event, the difference in maximum water level shows an increase of 1.2 $mAHD$. This is confirmed by Figure G.50.



(a) Reference scenario (b) Scenario 1.3

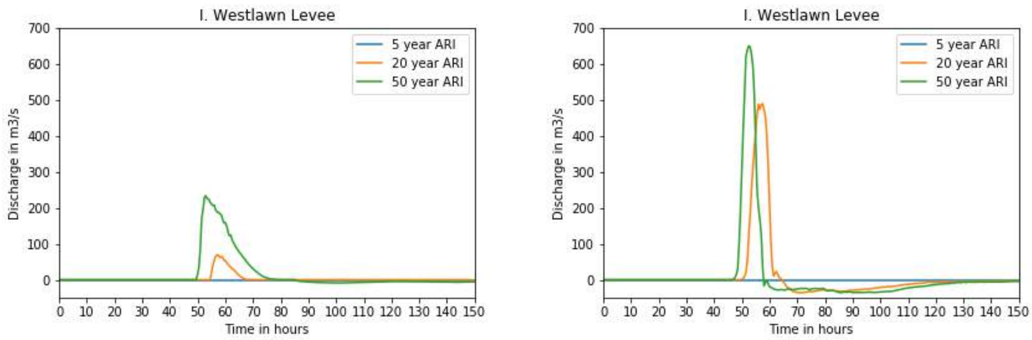
Figure G.50: Water level in Baker's Swamp.

The reference scenario indicates a lower water level in Baker's Swamp compared to scenario 1.3. At this location, a high water level is preferable as Baker's Swamp is designated as a floodplain in scenario 1.3.

Table G.3: Discharge results for scenario 1.3.

	Maximum discharge (m^3/s)			Discharge difference (m^3/s)		
Location	5	20	50	5	20	50
C. Pine Street Levee	0	0.35	1.88	0	0.35	0
D. Alice Street Levee	0	0	12.00	0	0	1.13
H. Butterfactory Lane Levee	0	52.86	74.64	0	0	−10.64
I. Westlawn Levee	0	488.16	648.78	0	418.91	415.93
L. Heber Street Levee	0.03	2.50	1.52	0	0.72	−0.18
O. Alumy Creek	24.42	567.80	733.49	0	452.26	427.79
P. Baker's Swamp	0	83.45	103.29	0	83.45	103.26
Q. North Grafton Levee	0	0	4.66	0	−0.87	−4.39

Figure G.51 shows the difference between the maximum discharge over the Westlawn Levee for scenario 1.3, as is also indicated in Table G.3. The 50 year ARI flood event shows a significant increase in maximum discharge, which means much more water is able to flow into Baker's Swamp. The functionality of Baker's Swamp as a floodplain is increased.



(a) Reference scenario

(b) Scenario 1.3

Figure G.51: Discharges over the Westlawn Levee.

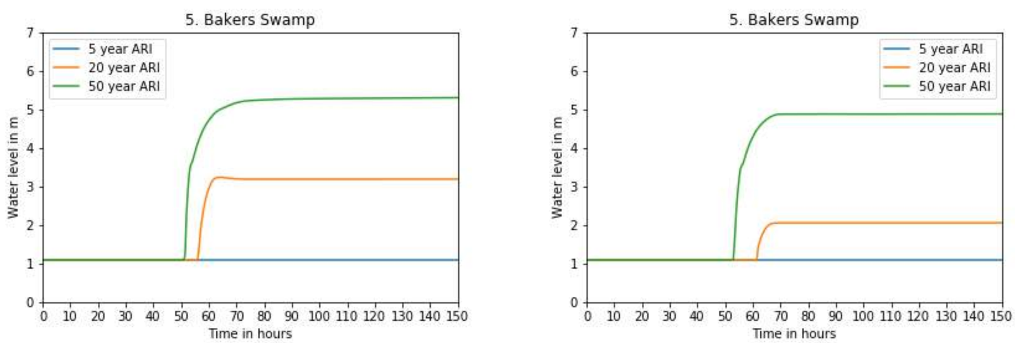
G.4.2. Scenario 2.3

Table G.4 shows the water level differences for scenario 2.3. For the 20 year ARI flood event, significant water level rises are observed for the Saltwater Creek Floodplain (location 8) and the Southampton Floodplain (location 11, is shown in the graph). These points used to be located behind a levee, which has been lowered for scenario 2.3.

Table G.4: Water level results for scenario 2.3.

		Maximum water level ($mAHD$)			Water level difference ($mAHD$)		
Location		*5*	*20*	*50*	*5*	*20*	*50*
0. PRINCE ST. GAUGE		6.09	7.69	8.19	0	−0.21	0
2. Middle Railway	Area 1	3.50	8.36	9.00	0	−0.31	0
5. Baker's Swamp	Area 1	1.10	2.06	4.88	0	−1.18	−0.42
7. Upper Waterview Levee	Area 2	6.01	8.54	9.11	0	2.52	2.85
8. Saltwater Creek Floodplain	Area 2	1.34	8.54	9.11	0	7.20	3.14
9. Clarence River	Area 2	6.60	8.53	9.11	0	−0.31	0
10. Lower Waterview Levee	Area 2	3.33	8.54	9.11	−3.27	1.93	2.02
11. Southampton Floodplain	Area 2	2.40	8.53	9.05	0	6.13	3.36
12. South Grafton	Area 2	2.72	6.13	6.25	0	3.40	3.52
13. Upper South Grafton Levee	Area 3	6.17	7.87	8.44	0	−0.23	0
14. Lower South Grafton Levee	Area 3	6.17	7.85	8.39	0	−0.24	0

The water level in Baker's Swamp (location 5) decreases for scenario 2.3. Unlike scenario 1.3, the Westlawn Levee has not been removed. This means that the decrease in water level is a positive change, since less water is likely to reach the north part of Grafton. Figure G.52 shows the decrease in water level, which is especially noticable for the 20 year ARI flood event.

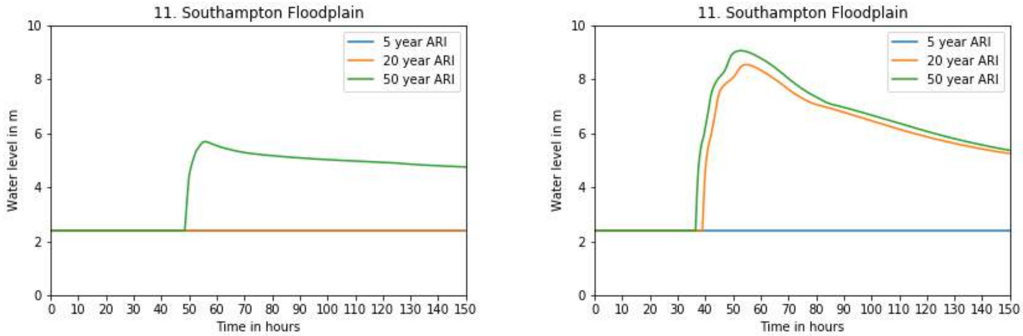


(a) Reference scenario

(b) Scenario 2.3

Figure G.52: Water level in Baker's Swamp.

Figure G.53 confirms the table results for the Southampton Floodplain. The 20 year ARI flood event shows a significant water level rise over the full simulation. For the 50 year ARI flood event, the water levels also increases by approximately 3 m.

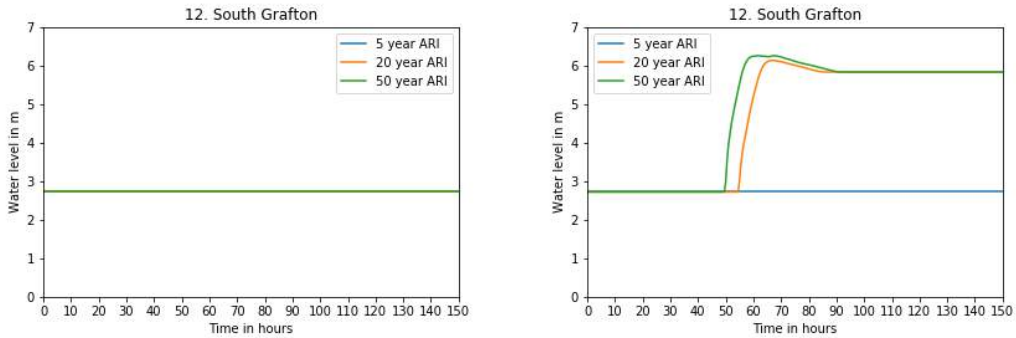


(a) Reference scenario

(b) Scenario 2.3

Figure G.53: Water level in the Southampton Floodplain.

With the removal of the Waterview Levee, the Southampton Floodplain gets inundated. An extra levee of 9.5 $mAHD$ has been constructed in order to protect South Grafton. However, South Grafton still inundates during the 20 and 50 year ARI flood events, due to a misfit in the new levee. Figure G.54a shows that there used to be no water at this location in South Grafton, even during a 50 year ARI flood event. So, if South Grafton is considered, scenario 2.3 has a negative influence. This could be prevented by refining the levee design, as this is a promising scenario for the other urban areas.



(a) Reference scenario

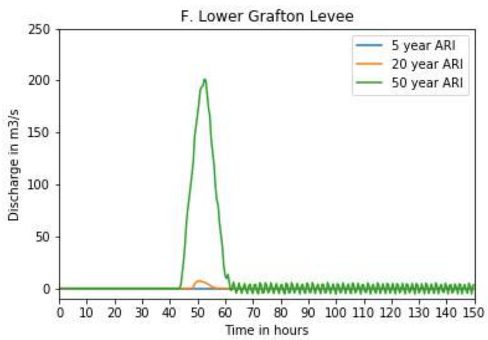
(b) Scenario 2.3

Figure G.54: Water level in South Grafton.

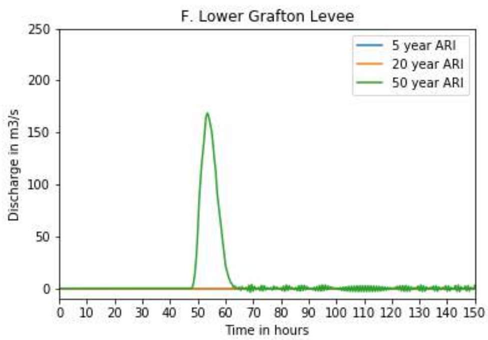
Table G.5: Discharge results for scenario 2.3.

	Maximum discharge (m^3/s)			Discharge difference (m^3/s)		
Location	*5*	*20*	*50*	*5*	*20*	*50*
C. Pine Street Levee	0	0	0	0	0	−1.97
D. Alice Street Levee	0	0	9.69	0	0	−1.18
E. Upper Grafton Levee	0	0	44.01	0	0	−13.85
F. Lower Grafton Levee	0	0	168.71	0	−7.24	−32.54
G. North Grafton Levee	0	0	9.77	0	0	−1.18
H. Butterfactory Lane Levee	0	41.95	94.85	0	−13.99	9.58
I. Westlawn Levee	0	23.33	186.64	0	−45.92	−46.21
J. Waterview Levee	0	1603.68	1972.68	0	1603.68	1475.45
K. South Grafton Urban Levee	0	1.36	5.68	0	1.36	−5.75
L. Heber Street Levee	0.03	2.33	2.16	0	0.55	0.46
M. Alipou Basin Levee	46.70	1440.89	1898.70	0	−226.73	0
N. Control Levee	0	1239.06	1716.84	0	−205.23	0
O. Alumy Creek	25.48	61.34	248.95	0	−54.20	−56.75
P. Baker's Swamp	0	0	0.06	0	0	0.03
Q. North Grafton Levee	0	0.72	4.44	0	−0.15	−4.61
R. Gwydir Highway	0	1108.15	1243.84	0	1108.15	1165.94
U. Maclean Levee	0	0	0.26	0	0	−0.96

Due to the improvement of the Southampton Floodplain in scenario 2.3, the Lower Grafton Levee does not overtop during a 20 year ARI flood event. Table G.5 shows a decrease of 7.24 m^3/s of the maximum discharge, which indicates that the Lower Grafton Levee did overtop in the reference scenario. For the 50 year ARI flood event, the maximum discharge is only lowered. This results in less water in the urban area of Grafton. Figure G.55 shows the difference between the reference scenario and scenario 2.3. The wiggles are due to modelling input errors.



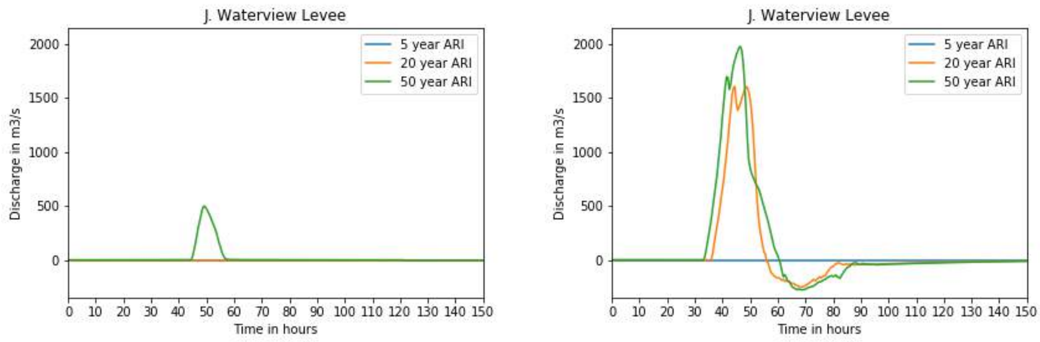
(a) Reference scenario



(b) Scenario 2.3

Figure G.55: Discharge over the Lower Grafton Levee.

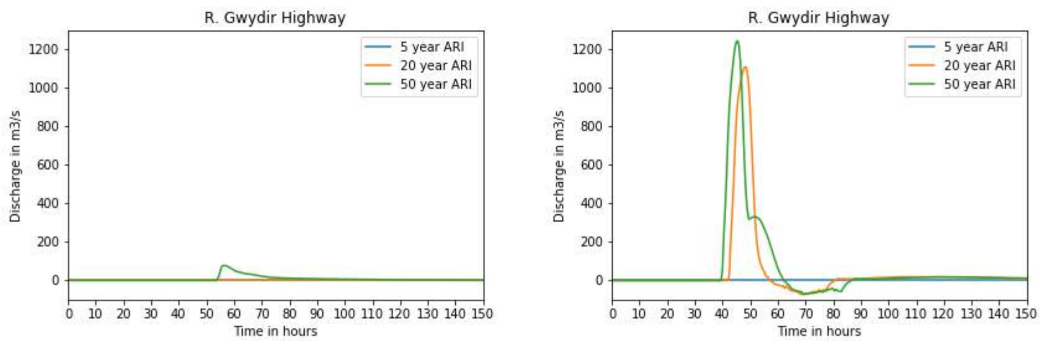
Figure G.56a shows that the Waterview Levee only overtops during a 50 year ARI flood event. For scenario 2.3, this levee has been lowered, resulting during a much higher maximum discharge for the 50 year ARI flood event (see Figure G.56b). For the 20 year ARI flood event, there was zero discharge in the reference simulation and a massive increase in discharge for scenario 2.3. This means, the Waterview Levee has been lowered properly and water can flow into the Southampton Floodplain.



(a) Reference scenario (b) Scenario 2.3

Figure G.56: Discharge over the Waterview Levee.

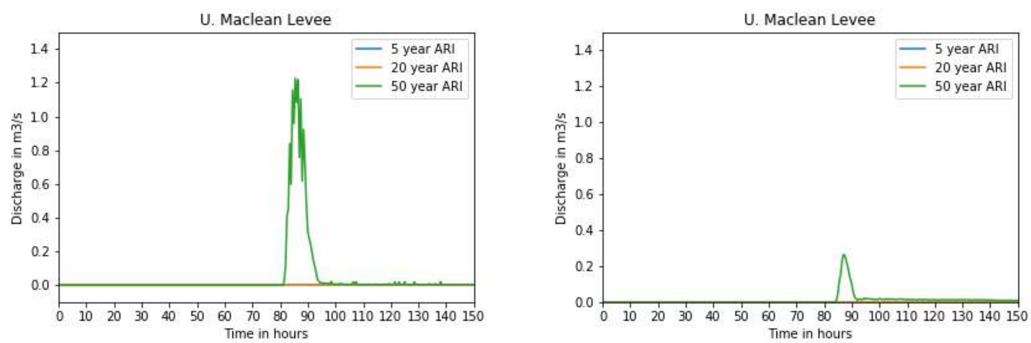
Due to the water ingress in the Southampton Floodplain, the Gwydir Highway is also affected. During the 20 year ARI flood event, the highway is still accessible for the reference scenario. However, a significant value of maximum discharge is recorded for scenario 2.3. Table G.5 indicates an increase of 1108.15 m^3/s. Figure G.57 confirms this great difference, which can clearly be seen if the vertical axes are viewed. As a result, scenario 2.3 has a negative effect on the accessibility of the Gwydir Highway.



(a) Reference scenario (b) Scenario 2.3

Figure G.57: Discharge over the Gwydir Highway.

Scenario 2.3 also affects parts downstream in the Clarence River. Figure G.58 shows a decrease in maximum discharge over the Maclean Levee for the 50 year ARI flood event. Figure 5.4 (main report) illustrates the reduction of inundation of Maclean, due to the discharge decrease. This is a postive effect.



(a) Reference scenario (b) Scenario 2.3

Figure G.58: Discharge over the Maclean Levee.

G.4.3. Scenario 4.1

Table G.6 shows the relevant water level difference for scenario 4.1. The Lower Control Levee is the only point with a difference, which occurs only during the 5 year ARI flood event. This point is located downstream of the Clarenza Levee and has been removed for scenario 4.1. For completeness, Table G.7 has been added with the maximum discharge differences. This scenario has a positive effect on the Pacific Highway, as water flows over the floodplains.

Table G.6: Water level results for scenario 4.1.

		Maximum water level ($mAHD$)			Water level difference ($mAHD$)		
Location		*5*	*20*	*50*	*5*	*20*	*50*
19. Lower Control Levee	Area 4	4.37	7.19	7.50	1.86	0	0

Table G.7: Discharge results for scenario 4.1.

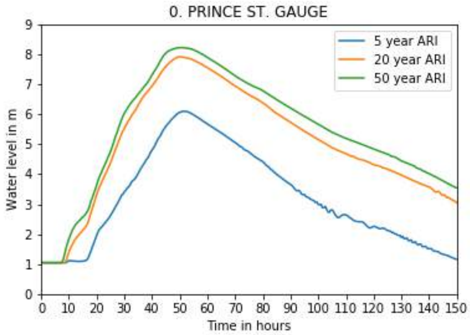
	Maximum discharge (m^3/s)			Discharge difference (m^3/s)		
Location	*5*	*20*	*50*	*5*	*20*	*50*
C. Pine Street Levee	0	0	1.26	0	0	−0.71
F. Lower Grafton Levee	0	6.27	206.18	0	−0.97	0
H. Butterfactory Lane Levee	0	64.85	84.04	0	8.91	0
K. South Grafton Urban Levee	0	0	9.88	0	0	−1.56
L. Heber Street Levee	0	1.47	2.81	−0.03	−0.31	1.11
M. Alipou Basin Levee	50.43	1759.10	2064.21	6.32	0	0
N. Control Levee	30.10	1581.65	1893.74	30.10	0	0
P. Baker's Swamp	0	0	0.03	0	0	−0.00
R. Gwydir Highway	0	0	66.54	0	0	−11.36
S. Pacific Highway	0	35.18	28.36	−0.17	0	0

G.4.4. Scenario 4.5

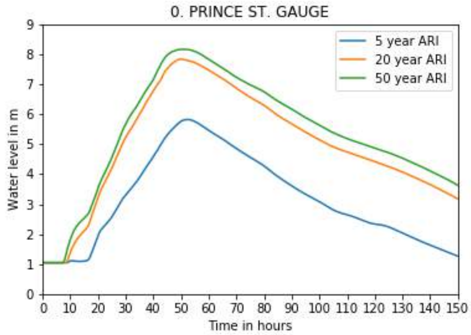
Table G.8: Water level results for scenario 4.5.

		Maximum water level ($mAHD$)			Water level difference ($mAHD$)		
Location		*5*	*20*	*50*	*5*	*20*	*50*
0. PRINCE ST. GAUGE		5.82	7.83	8.16	−0.27	0	0
5. Baker's Swamp	Area 1	1.10	2.92	5.28	0	−0.32	0
8. Saltwater Creek Floodplain	Area 2	1.34	1.34	5.76	0	0	−0.21
9. Clarence River	Area 2	6.36	8.78	9.20	−0.25	0	0
11. Southampton Floodplain	Area 2	2.40	2.40	5.03	0	0	−0.66
13. Upper South Grafton Levee	Area 3	5.90	8.03	8.38	−0.27	0	0
14. Lower South Grafton Levee	Area 3	5.90	8.01	8.36	−0.27	0	0
15. Alipou Levee	Area 4	5.63	7.44	7.75	−0.67	0	0
16. Grafton Levee	Area 4	5.58	7.31	7.61	−0.27	0	0
17. Upper Clarenza Floodplain	Area 4	5.63	7.44	7.75	0.76	0	0
18. Upper Control Levee	Area 4	5.56	7.30	7.62	1.15	0	0
19. Lower Control Levee	Area 4	5.53	7.25	7.56	3.02	0	0
20. Lower Clarenza Floodplain	Area 4	5.26	6.83	7.14	2.50	0	0
21. Swan Creek Floodgate	Area 4	5.12	6.53	6.78	−0.27	0	0

Table G.8 shows that a lot of locations are influenced by the removal of the levees downstream of South Grafton in scenario 4.5. Although the adjustments are done in area 4, the first three areas also show water level differences. It should be mentioned that during a 5 year ARI flood event, the water level near the Prince Street Gauge is lowered by 0.27 m. The Prince Street Gauge is taken as a reference point for warnings and evacuations. The maximum water level for the Prince Street Gauge is still above the major flood level of 5.4 $mAHD$. However, no urban areas are inundated. No significant differences in water levels are to be seen for the 20 and 50 year ARI flood events.



(a) Reference scenario



(b) Scenario 4.5

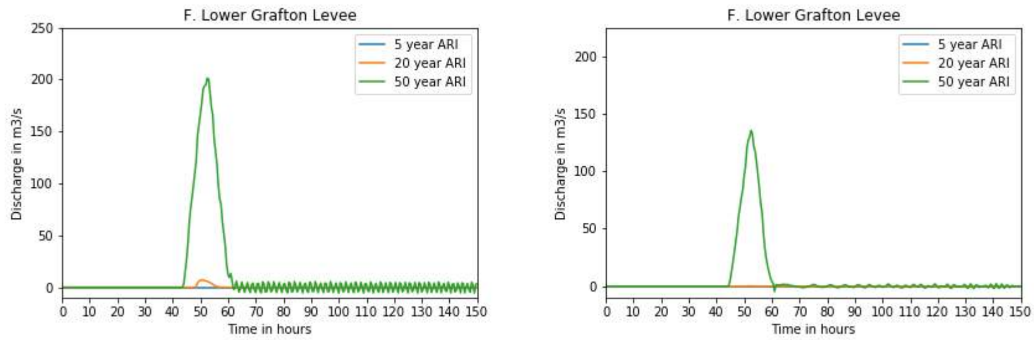
Figure G.59: Water level Prince Street Gauge.

Table G.9 shows the discharge differences for scenario 4.5.

Table G.9: Discharge results for scenario 4.5.

	Maximum discharge (m^3/s)			Discharge difference (m^3/s)		
Location	*5*	*20*	*50*	*5*	*20*	*50*
C. Pine Street Levee	0	0	0.22	0	0	−1.75
D. Alice Street Levee	0	0	4.31	0	0	−6.56
E. Upper Grafton Levee	0	0	30.39	0	0	−27.47
F. Lower Grafton Levee	0	0.25	135.54	0	−6.99	−65.71
H. Butterfactory Lane Levee	0	49.53	89.32	0	−6.41	0
I. Westlawn Levee	0	50.45	228.63	0	−18.80	0
J. Waterview Levee	0	0.06	391.33	0	0.06	−105.91
K. South Grafton Urban Levee	0	0	3.60	0	0	−7.83
L. Heber Street Levee	0.78	1.23	2.41	0.75	−0.54	0.71
M. Alipou Basin Levee	468.72	1922.11	2248.22	424.61	254.49	312.82
N. Control Levee	599.36	2078.68	2419.35	599.36	634.39	669.24
O. Alumy Creek	31.79	90.56	299.60	6.64	−24.97	0
P. Baker's Swamp	0	0	0.05	0	0	0.02
Q. North Grafton Levee	0	0.03	5.22	0	−0.84	−3.83
R. Gwydir Highway	0	0	6.11	0	0	−71.80
S. Pacific Highway	16.99	28.47	27.48	16.82	−7.98	0
U. Maclean Levee	0	0	2.94	0	0	1.71

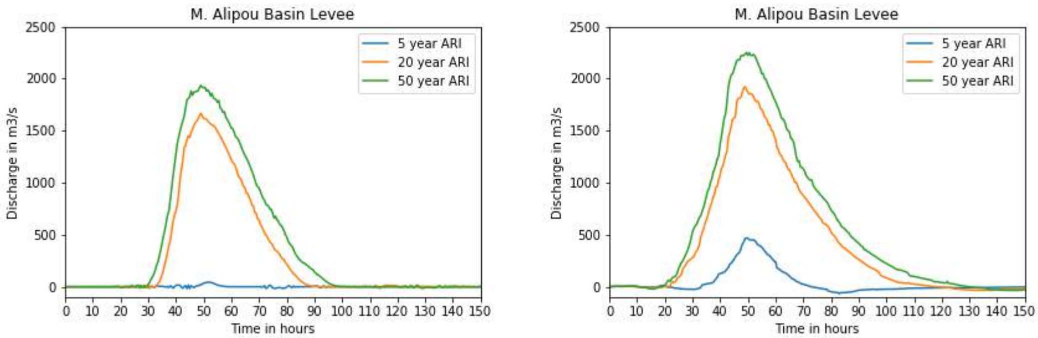
Scenario 4.5 influences the overtopping rates of the Lower and Upper Grafton Levee. Figure G.60 indicates that during a 20 year ARI flood event, almost no overtopping of the Lower Grafton Levee occurs. The maximum discharge is lowered during a 50 year ARI flood event. The latter also holds for the Upper Grafton Levee.



(a) Reference scenario (b) Scenario 4.5

Figure G.60: Discharge over the Lower Grafton Levee.

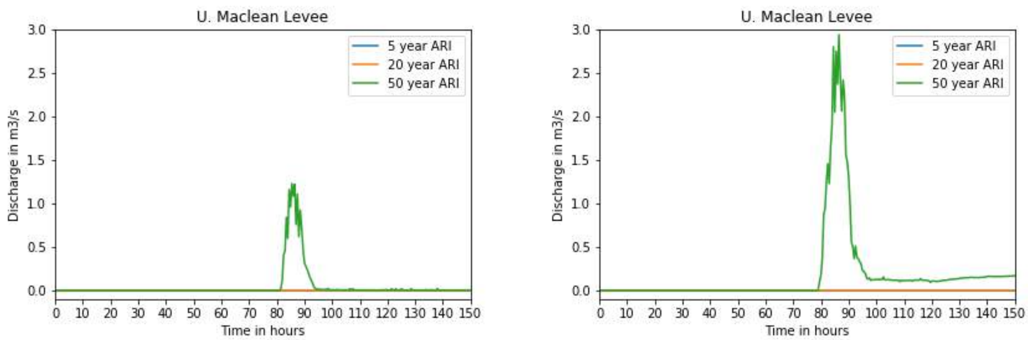
The Alipou Basin Levee has been removed for scenario 4.5. It is expected that the discharge increases. Figure G.61 confirms this. Especially for the 5 year ARI flood event, the discharge will increase significantly, which means more water can be stored in the Clarenza Floodplain. This directly explains the water level difference found at Prince Street Gauge for the 5 year ARI flood event. Water can more easily access the floodplain (leading to water level differences), where it already had access to the floodplain in the 20 and 50 year ARI flood events.



(a) Reference scenario (b) Scenario 4.5

Figure G.61: Discharge over the Alipou Basin Levee.

Again, this scenario affects parts downstream of its own area. The removal of the Alipou Basin Levee and the Swan Creek Levee result in an increase of discharge over the Maclean Levee for the 50 year ARI flood event, which can be seen in Figure G.62.



(a) Reference scenario (b) Scenario 4.5

Figure G.62: Discharge over the Maclean Levee.

G.4.5. Combination 1 & 2

Out of all the promising scenarios, two combination scenarios have been created. Only the 50 year ARI flood event columns are marked green as the combinations are particularly interesting for these events. The two combinations will be discussed and compared in further detail in this section. First of all, the tables with the water level differences are presented (see Tables G.10 and G.11). The first thing that is noticeable, is the fact that combination 1, unlike combination 2, has little influence on the maximum water levels of the 5 year ARI flood event. This is, of course, related to the fact that scenario 4.5 also resulted in great changes for the 5 year ARI flood event.

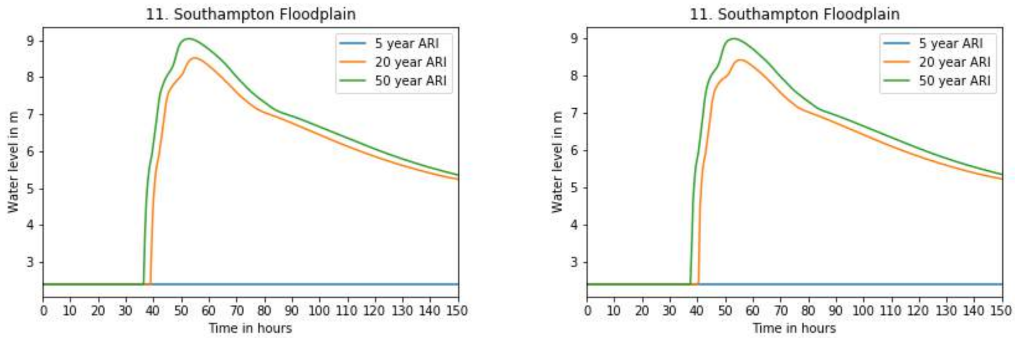
Table G.10: Water level results for combination 1.

		Maximum water level ($mAHD$)			Water level difference ($mAHD$)		
Location		*5*	*20*	*50*	*5*	*20*	*50*
0. PRINCE ST. GAUGE		6.09	7.68	8.18	0	−0.23	0
1. Lower Railway	Area 1	5.85	6.12	6.46	0	0.28	0.62
2. Middle Railway	Area 1	3.50	8.34	8.99	0	−0.32	0
4. Alumy Creek Floodgate	Area 1	3.95	6.12	6.46	0	0.43	0.63
5. Baker's Swamp	Area 1	1.10	6.13	6.46	0	2.89	1.17
6. Great Marlow Floodplain	Area 1	5.26	6.12	6.46	0	0.58	0.66
7. Upper Waterview Levee	Area 2	6.01	8.52	9.10	0	2.51	2.84
8. Saltwater Creek Floodplain	Area 2	1.34	8.52	9.10	0	7.18	3.13
9. Clarence River	Area 2	6.60	8.52	9.10	0	−0.33	0
10. Lower Waterview Levee	Area 2	3.33	8.53	9.10	−3.27	1.92	2.01
11. Southampton Floodplain	Area 2	2.40	8.52	9.04	0	6.12	3.36
12. South Grafton	Area 2	2.72	6.11	6.25	0	3.39	3.52
13. Upper South Grafton Levee	Area 3	6.17	7.85	8.43	0	−0.25	0
14. Lower South Grafton Levee	Area 3	6.17	7.83	8.37	0	−0.25	0
17. Upper Clarenza Floodplain	Area 4	4.87	7.22	7.70	0	−0.22	0
18. Upper Control Levee	Area 4	4.37	7.06	7.51	0	−0.23	0
19. Lower Control Levee	Area 4	4.37	7.03	7.46	1.86	0	0

Table G.11: Water level results for combination 2.

		Maximum water level ($mAHD$)			Water level difference ($mAHD$)		
Location		*5*	*20*	*50*	*5*	*20*	*50*
0. PRINCE ST. GAUGE		5.82	7.57	8.10	−0.27	−0.33	0
1. Lower Railway	Area 1	5.85	6.06	6.45	0	0.21	0.60
2. Middle Railway	Area 1	3.50	8.23	8.92	0	−0.43	0
4. Alumy Creek Floodgate	Area 1	3.95	6.05	6.45	0	0.36	0.62
5. Baker's Swamp	Area 1	1.10	6.06	6.45	0	2.82	1.15
6. Great Marlow Floodplain	Area 1	5.26	6.05	6.45	0	0.51	0.65
7. Upper Waterview Levee	Area 2	6.01	8.42	9.04	0	2.41	2.78
8. Saltwater Creek Floodplain	Area 2	1.34	8.42	9.04	0	7.08	3.07
9. Clarence River	Area 2	6.36	8.42	9.04	−0.25	−0.43	0
10. Lower Waterview Levee	Area 2	3.33	8.43	9.04	−3.27	1.82	1.95
11. Southampton Floodplain	Area 2	2.40	8.42	8.99	0	6.02	3.31
12. South Grafton	Area 2	2.72	6.02	6.34	0	3.30	3.62
13. Upper South Grafton Levee	Area 3	5.90	7.74	8.35	−0.27	−0.36	0
14. Lower South Grafton Levee	Area 3	5.90	7.74	8.30	−0.27	−0.35	0
15. Alipou Levee	Area 4	5.63	7.24	7.69	−0.67	−0.29	0
16. Grafton Levee	Area 4	5.58	7.13	7.55	−0.27	−0.27	0
17. Upper Clarenza Floodplain	Area 4	5.63	7.24	7.69	0.76	0	0
18. Upper Control Levee	Area 4	5.55	7.12	7.55	1.15	0	0
19. Lower Control Levee	Area 4	5.53	7.08	7.50	3.02	0	0
20. Lower Clarenza Floodplain	Area 4	5.26	6.71	7.06	2.50	0	0
21. Swan Creek Floodgate	Area 4	5.12	6.44	6.70	−0.27	0	0

The water level for the Southampton Floodplain is increased for both combinations during both the 20 and 50 year ARI flood event. For combination 1, the increase is slightly higher than for combination 2 (about 0.1 m for the 20 year ARI flood event). Although this difference is insignificant, it is stored in a very large area, namely the whole Southampton Floodplain. Figure G.63 shows both the water levels for location 11. No difference is clearly visible, but the difference is present, as explained above.



(a) Combination 1 (b) Combination 2

Figure G.63: Water level in the Southampton Floodplain for both combinations.

The discharges of both combinations are also considered. The result tables are presented below.

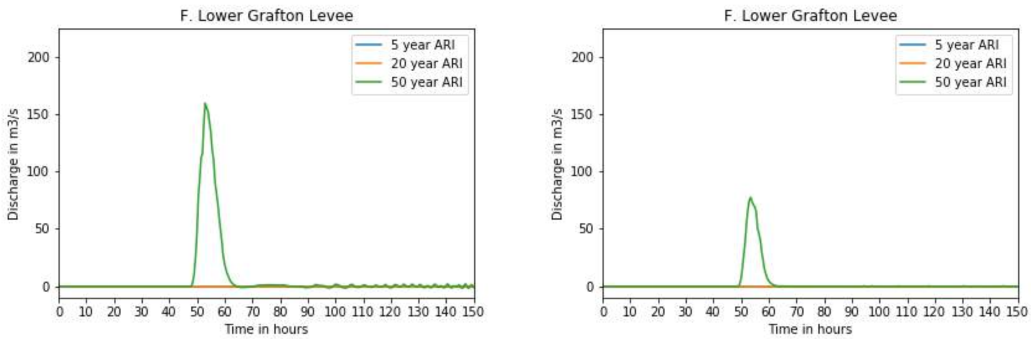
Table G.12: Discharge results for combination 1.

	Maximum discharge (m^3/s)			Discharge difference (m^3/s)		
Location	*5*	*20*	*50*	*5*	*20*	*50*
C. Pine Street Levee	0	0.27	0.43	0	0.27	−1.54
D. Alice Street Levee	0	0	7.04	0	0	−3.83
E. Upper Grafton Levee	0	0	35.12	0	0	−22.73
F. Lower Grafton Levee	0	0	159.42	0	−7.24	−41.82
H. Butterfactory Lane Levee	0	29.16	96.51	0	−26.78	11.23
I. Westlawn Levee	0	417.59	605.01	0	348.34	372.16
J. Waterview Levee	0	1588.67	1969.77	0	1588.67	1472.54
K. South Grafton Urban Levee	0	3.21	6.27	0	3.21	−5.16
L. Heber Street Levee	0	2.99	2.61	−0.03	1.22	0.91
N. Control Levee	30.34	1358.10	1856.49	30.34	0	0
O. Alumy Creek	25.00	468.09	693.25	0	352.55	387.55
P. Baker's Swamp	0	66.12	95.82	0	66.12	95.79
Q. North Grafton Levee	0	0	0.70	0	−0.87	−8.35
R. Gwydir Highway	0	1104.14	1241.48	0	1104.14	1163.57
S. Pacific Highway	0	37.14	38.97	−0.17	0	10.48
U. Maclean Levee	0	0	0.23	0	0	−1.00

Table G.13: Discharge results for combination 2.

	Maximum discharge (m^3/s)			Discharge difference (m^3/s)		
Location	*5*	*20*	*50*	*5*	*20*	*50*
C. Pine Street Levee	0	0.14	0.50	0	0.14	−1.47
D. Alice Street Levee	0	0	0.35	0	0	−10.51
E. Upper Grafton Levee	0	0	11.44	0	0	−46.42
F. Lower Grafton Levee	0	0	77.42	0	−7.24	−123.82
G. North Street Levee	0	0	0.12	0	0	−10.83
H. Butterfactory Lane Levee	0	29.95	79.59	0	−25.99	0
I. Westlawn Levee	0	395.15	590.52	0	325.90	357.67
J. Waterview Levee	0	1616.47	1962.69	0	1616.47	1465.46
K. South Grafton Urban Levee	0	6.02	5.29	0	6.02	−6.14
L. Heber Street Levee	0.72	1.26	1.80	0.69	−0.52	0
M. Alipou Basin Levee	468.77	1642.49	2174.85	424.66	0	239.45
N. Control Levee	599.05	1800.24	2328.43	599.05	355.95	578.32
O. Alumy Creek	32.02	440.35	678.33	6.87	324.82	372.64
P. Baker's Swamp	0	47.35	95.82	0	47.35	95.79
Q. North Grafton Levee	0	0	0	0	−0.87	−9.05
R. Gwydir Highway	0	1115.46	1256.31	0	1115.46	1178.41
S. Pacific Highway	17.31	35.27	26.10	17.14	0	0
U. Maclean Levee	0	0	0.92	0	0	−0.31

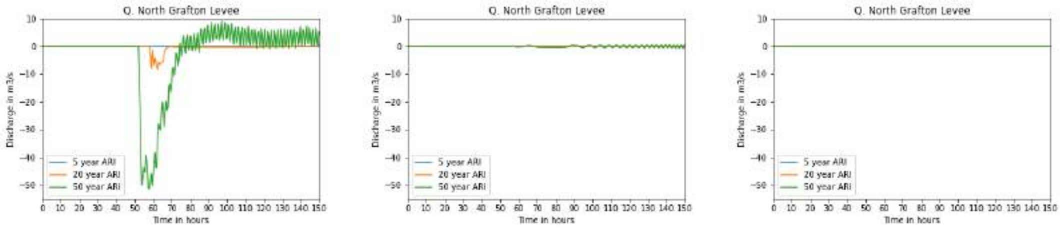
Here, some interesting features are seen. In the reference simulation, the maximum discharge for the 50 year ARI flood event at the Lower Grafton Levee is 201.25 m^3/s. The discharge for scenario 2.3 (168.71 m^3/s) and 4.5 (135.54 m^3/s) already showed a significant decrease compared to the reference simulation. The discharge over the Lower Grafton Levee decreased to 159.42 m^3/s for combination 1, which is greater than the discharge for scenario 4.5 but lower than the discharge of scenario 2.3. Since combination 2 combines the scenarios 2.3 and 4.5, the maximum discharge over the Lower Grafton Levee decreases even further to 77.42 m^3/s for the 50 year ARI flood event. So, when comparing combination 2 with combination 1 for the 50 year ARI flood event, much less water will flow over the Lower Grafton Levee for combination 2 (see Figure G.64). The same holds for the Upper Grafton Levee.



(a) Combination 1 (b) Combination 2

Figure G.64: Discharge over the Lower Grafton Levee.

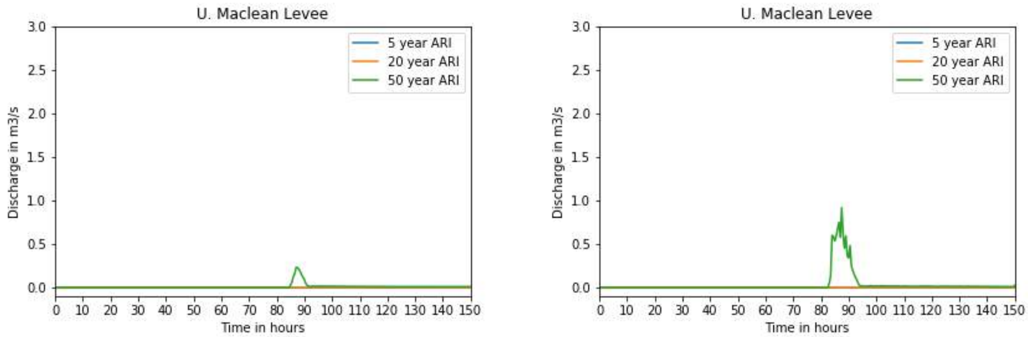
Figure G.65a shows results for the discharge over the North Grafton Levee for the reference scenario. Negative discharges are plotted, possibly due to instability of the model. Both new combinations include heightening of the North Grafton Levee. Figure G.65c shows that no water will pass this levee anymore for every ARI flood event. Figure G.65b shows some oscillation, also possibly due to instability of the model. This figure also shows a discharge of zero if the oscillation is neglected. To conclude, water can not enter Grafton anymore for both combinations.



(a) Reference scenario (b) Combination 1 (c) Combination 2

Figure G.65: Discharge over the North Grafton Levee.

Scenario 2.3 indicated a decrease in discharge over the Maclean Levee for the 50 year ARI flood event where scenario 4.5 resulted in an increase in discharge over the Maclean levee. These scenarios are combined in combination 2, which apparently still leads to a decrease in discharge (see Table G.13). From this fact, it can be concluded that scenario 2.3 has more influence on the discharge over the Maclean Levee than scenario 4.5. Figure G.66 shows the discharge over the Maclean Levee for both combinations.



(a) Combination 1 (b) Combination 2

Figure G.66: Discharge over the Maclean Levee.

G.5. Velocity

Area 2: Southampton Floodplain

The main measurement taken in the Southampton Floodplain is the lowering of the Waterview Levee. As a consequence, more water overtops the Waterview Levee during a 20 & 50 year ARI flood event. Water overtopping the levee can cause damage on the inner slope of the levee due to increasing flow velocities. This mechanism can lead to a total levee failure. Therefore, it is important to apply a proper slope protection. Further research regarding this failure mechanism is required to see the consequences of the measure taken in Southampton Floodplain.

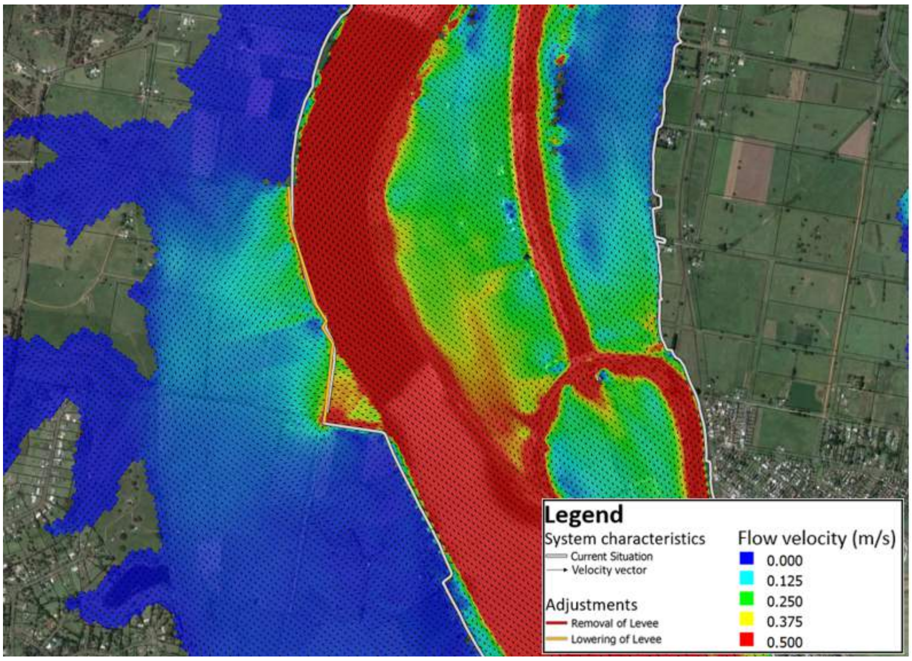


Figure G.67: Flow velocity in m/s for the Southampton Floodplain during a 20 year ARI flood event.

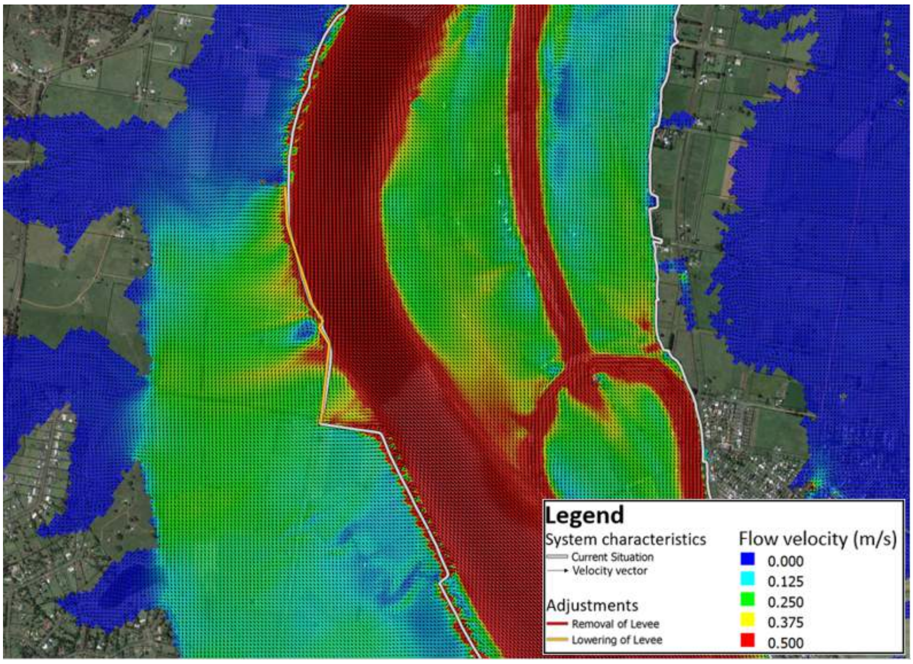


Figure G.68: Flow velocity in m/s for the Southampton Floodplain during a 50 year ARI flood event.

Area 4: Clarenza Floodplain
One of the scenarios applied in the Clarenza Floodplain is the removal of the Alipou Basin Levee, Clarenza Levee and a part of the Swan Creek Levee to create a new floodplain. As can be seen in Figure G.69 & G.70 the flow velocity in the newly created floodplain will increase. To prevent erosion a bed protection may be needed. This falls outside the scope of this project.

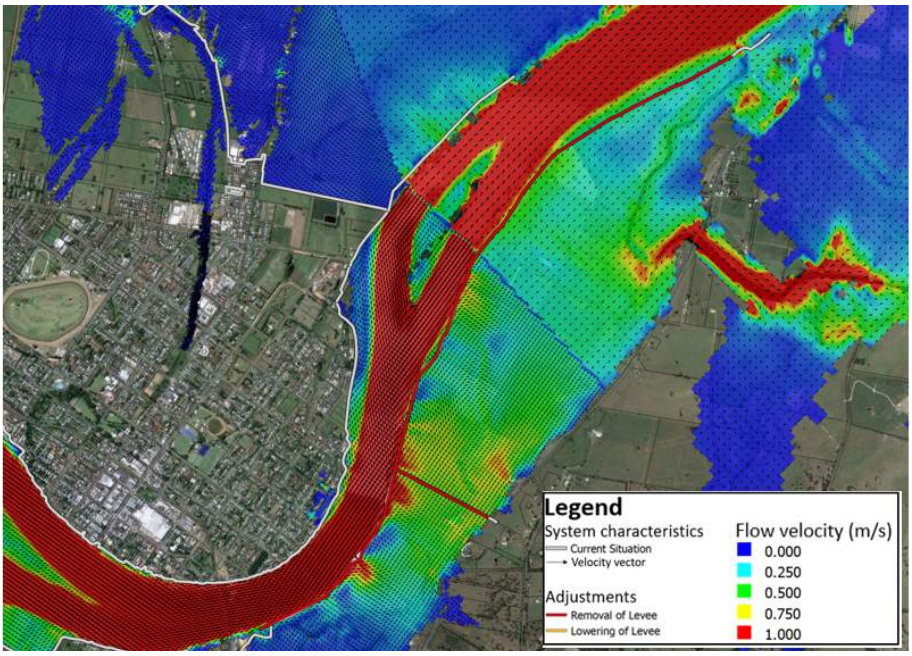


Figure G.69: Flow velocity in m/s for Clarenza Floodplain during a 20 year ARI flood event.

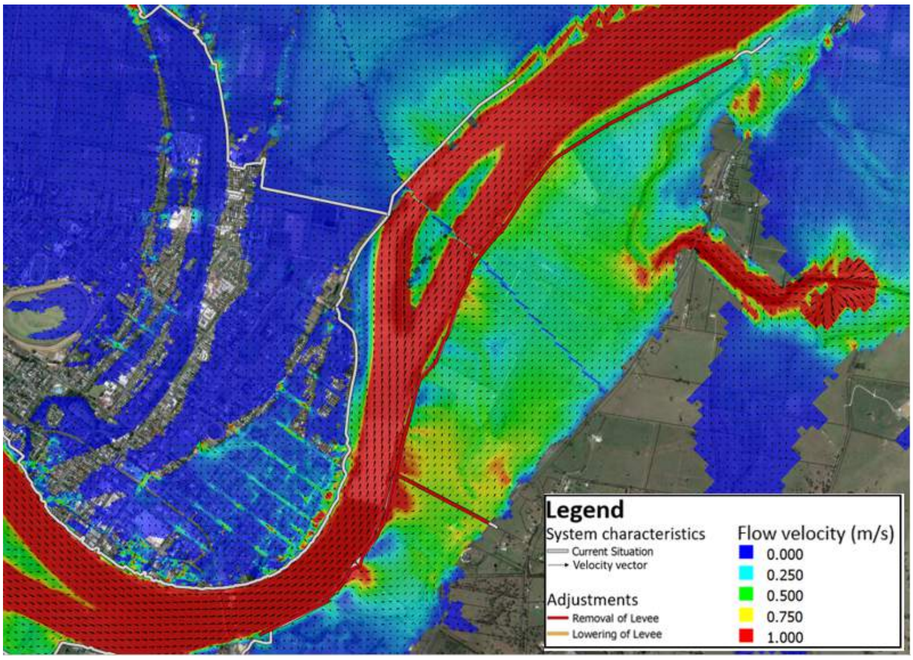


Figure G.70: Flow velocity in m/s for Clarenza Floodplain during a 50 year ARI flood event.

G.6. Location & Time of First Inundation

Due to the applied adjustments to the system, the moment of first inundation may occur on another moment in time. In this section, the new inundation times will be compared to the reference situation. Relevant scenario 4.1 is not considered. This scenario was interesting in a 5 year ARI event where no urban levees are overtopped. Relevant scenario 1.3 is only interesting in a 50 year ARI flood event. Relevant scenarios 2.3 and 4.5 and both combinations are interesting for the 20 and 50 year ARI flood events. In Tables G.14 and G.15 the difference with the reference situation can be seen. Figures representing the location of first overtopping are linked to the tables below and given on the next few pages.

Table G.14: Time (in hours) after initiation of floodwave, of first inundation on different locations for a 20 year ARI flood event.

20 year ARI flood event		
Location	Grafton	South Grafton
Reference	48 hr	dry
Scenario 2.3	dry	55 hr (Figure G.72)
Scenario 4.5	50 hr (Figure G.76)	dry
Combination 1	dry	55 hr (Figure G.79)
Combination 2	dry	56 hr (Figure G.83)

Table G.15: Time (in hours) after initiation of floodwave, of first inundation on different locations for a 50 year ARI flood event.

50 year ARI flood event				
Location	Grafton (south)	Grafton (north)	South Grafton	Maclean
Reference	44 hr	55 hr	96 hr	79 hr
Scenario 1.3	44 hr	dry	102 hr (Figure G.71)	79 hr
Scenario 2.3	48 hr (Figure G.73)	55 hr	50 hr (Figure G.74)	83 hr (Figure G.75)
Scenario 4.5	45 hr (Figure G.77)	55 hr	dry	76 hr (Figure G.78)
Combination 1	48 hr (Figure G.80)	dry	50 hr (Figure G.81)	83 hr (Figure G.75)
Combination 2	50 hr (Figure G.84)	dry	51 hr (Figure G.85)	81 hr (Figure G.86)

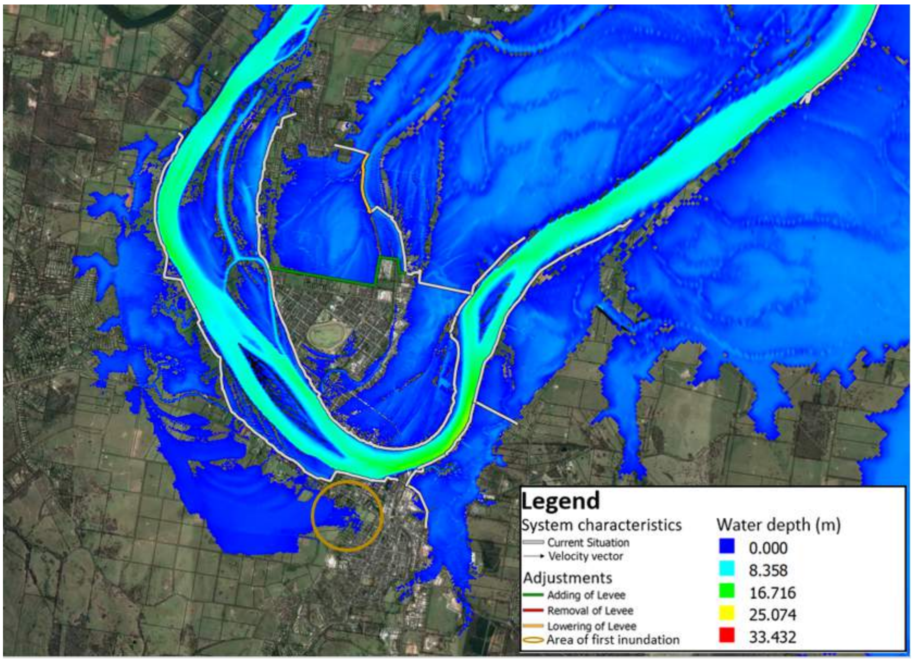


Figure G.71: Location of first overtopping during a 50 year ARI flood event near South Grafton, see yellow circle.

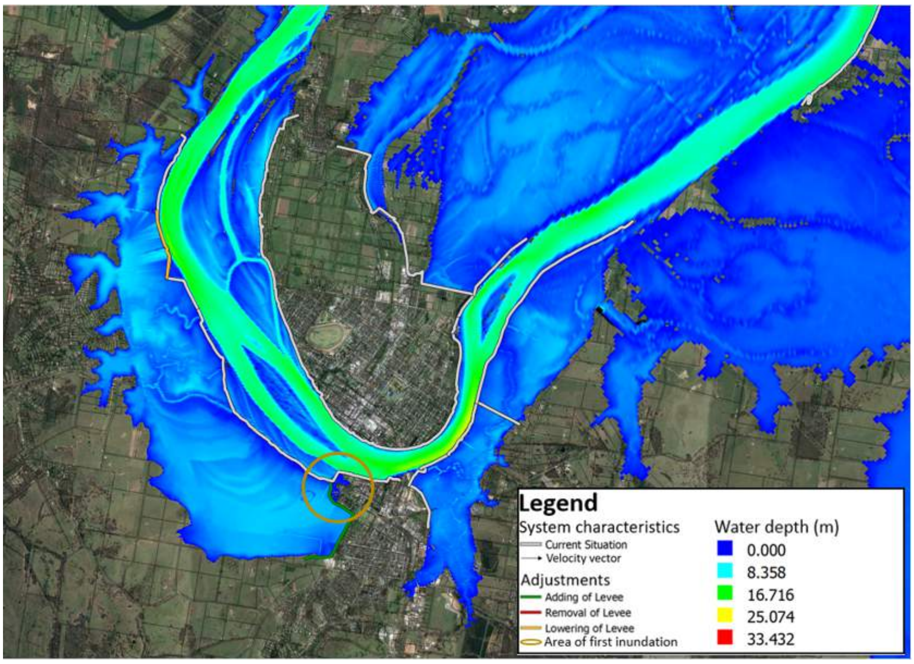


Figure G.72: Location of first overtopping during a 20 year ARI flood event in the Southampton Floodplain, see yellow circle.

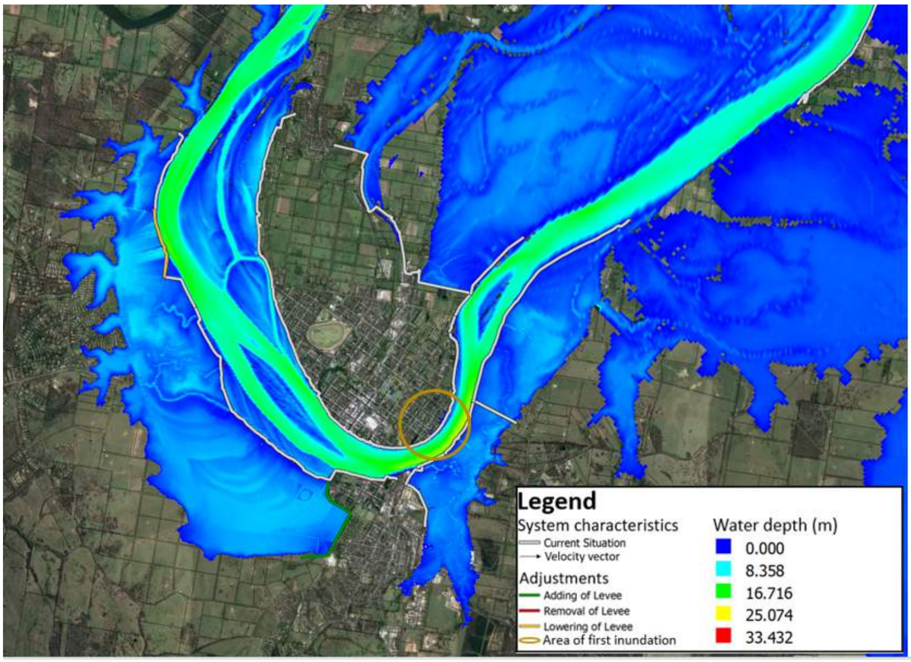


Figure G.73: Location of first overtopping during a 50 year ARI flood event near Grafton (south), see yellow circle.

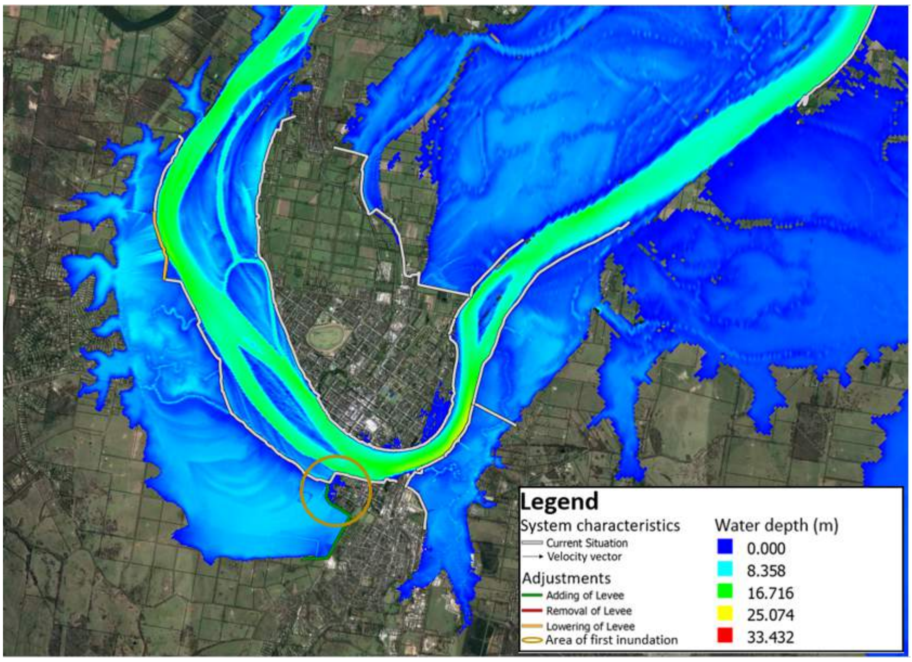


Figure G.74: Location of first overtopping during a 50 year ARI flood event near South Grafton, see yellow circle.

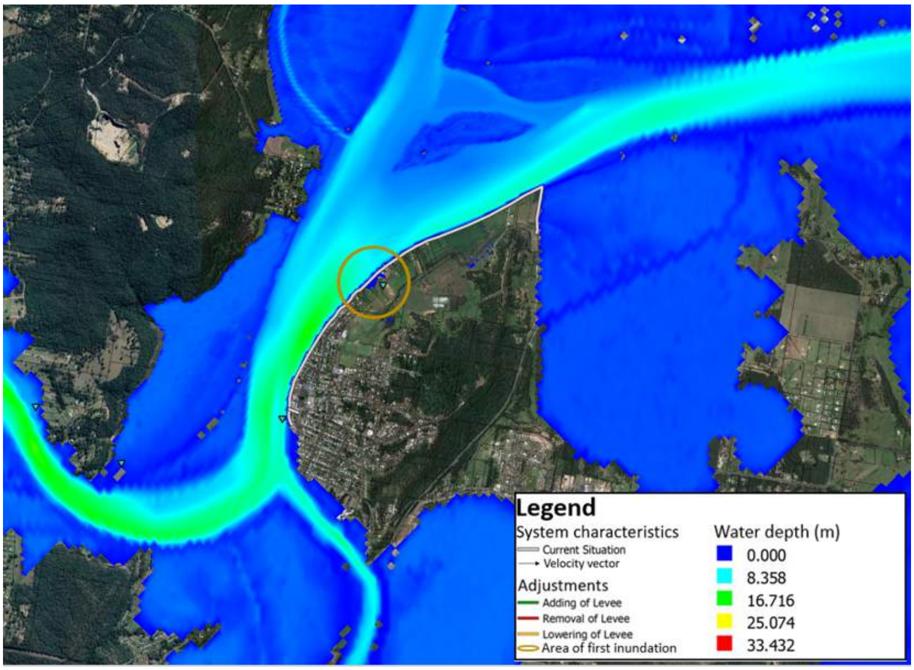


Figure G.75: Location of first overtopping during a 50 year ARI flood event in Maclean, see yellow circle.

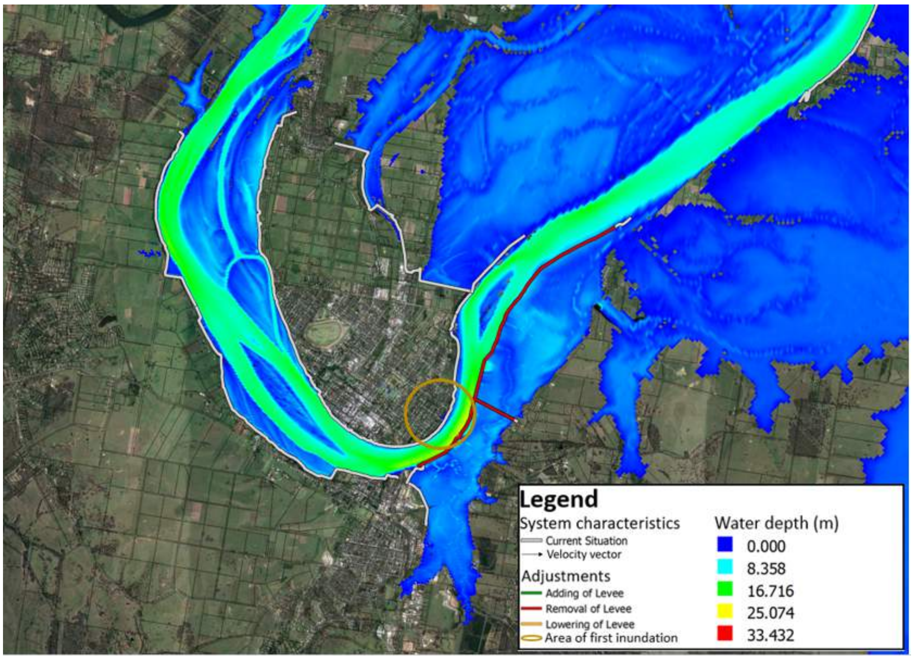


Figure G.76: Location of first overtopping during a 20 year ARI flood event near Grafton (south), see yellow circle.

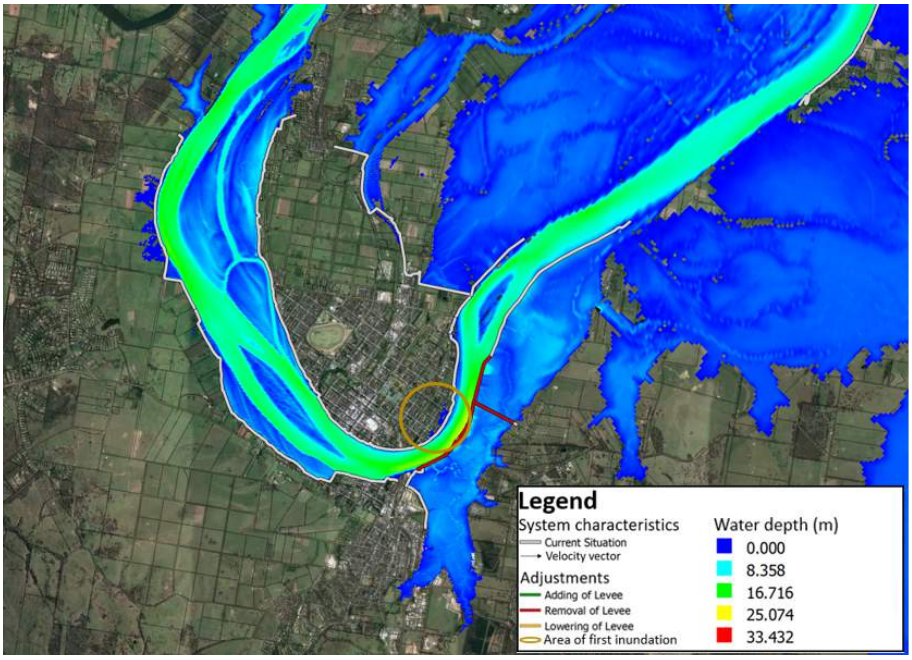


Figure G.77: Location of first overtopping during a 50 year ARI flood event near Grafton (south), see yellow circle.

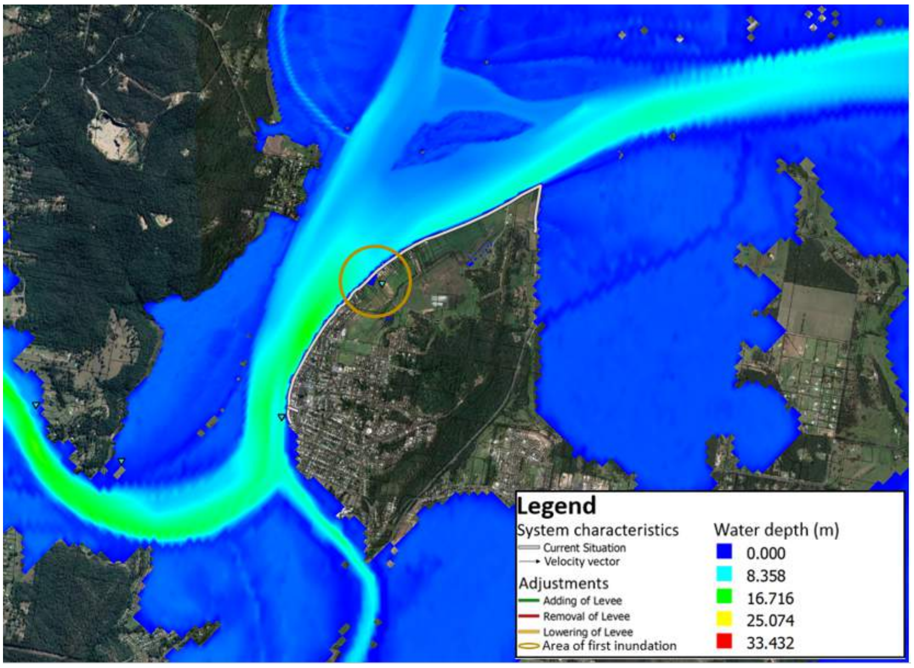


Figure G.78: Location of first overtopping during a 50 year ARI flood event in Maclean, see yellow circle.

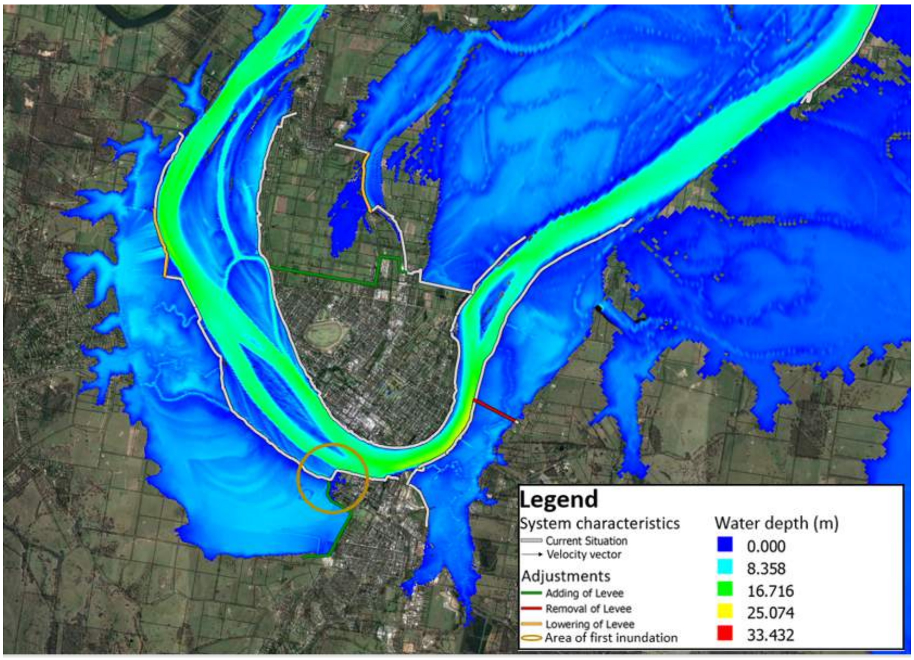


Figure G.79: Location of first overtopping during a 20 year ARI flood event near South Grafton, see yellow circle.

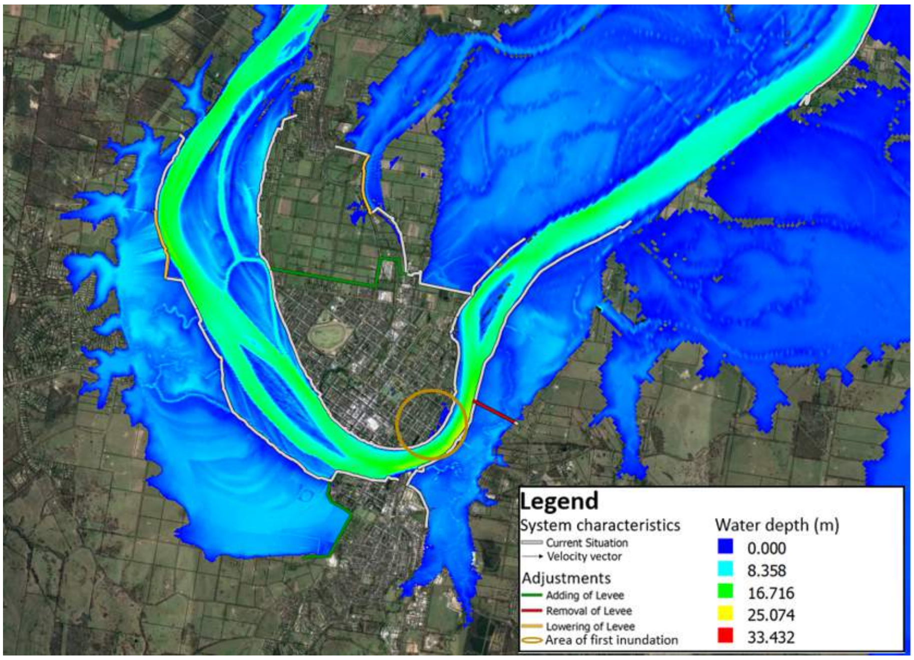


Figure G.80: Location of first overtopping during a 50 year ARI flood event near Grafton (south), see yellow circle.

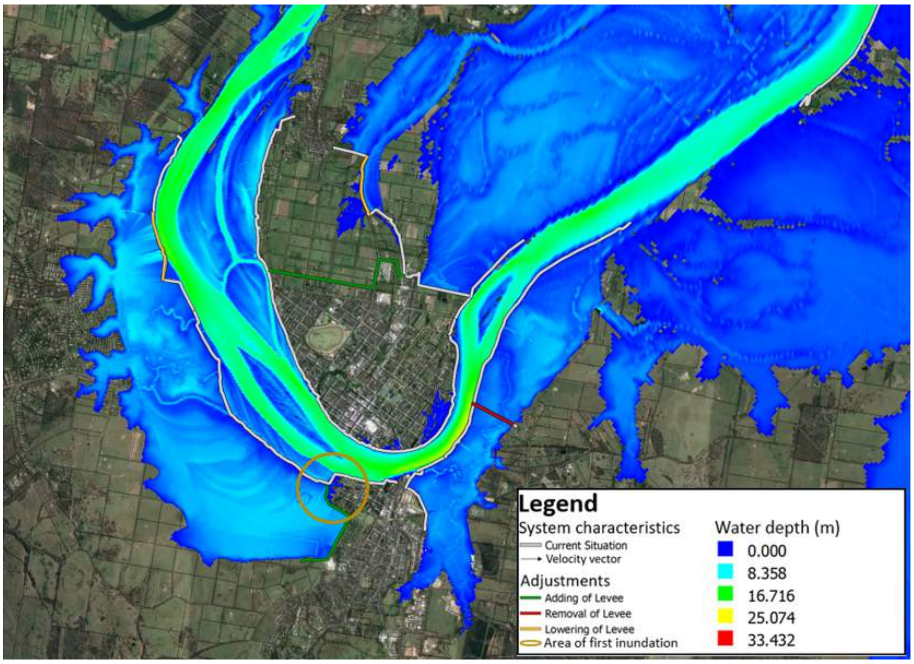


Figure G.81: Location of first overtopping during a 50 year ARI flood event near South Grafton, see yellow circle.

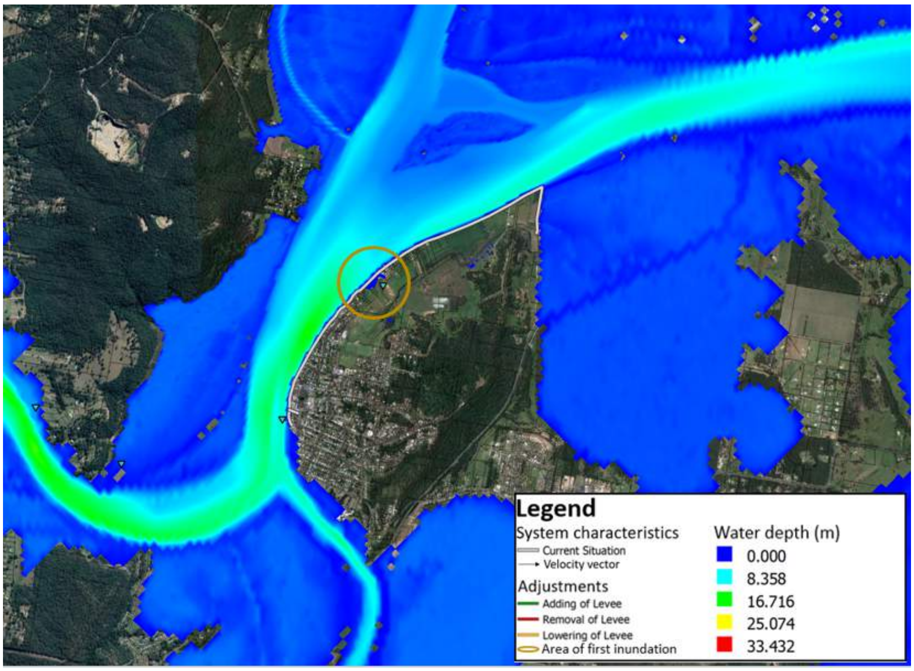


Figure G.82: Location of first overtopping during a 50 year ARI flood event in Maclean, see yellow circle.

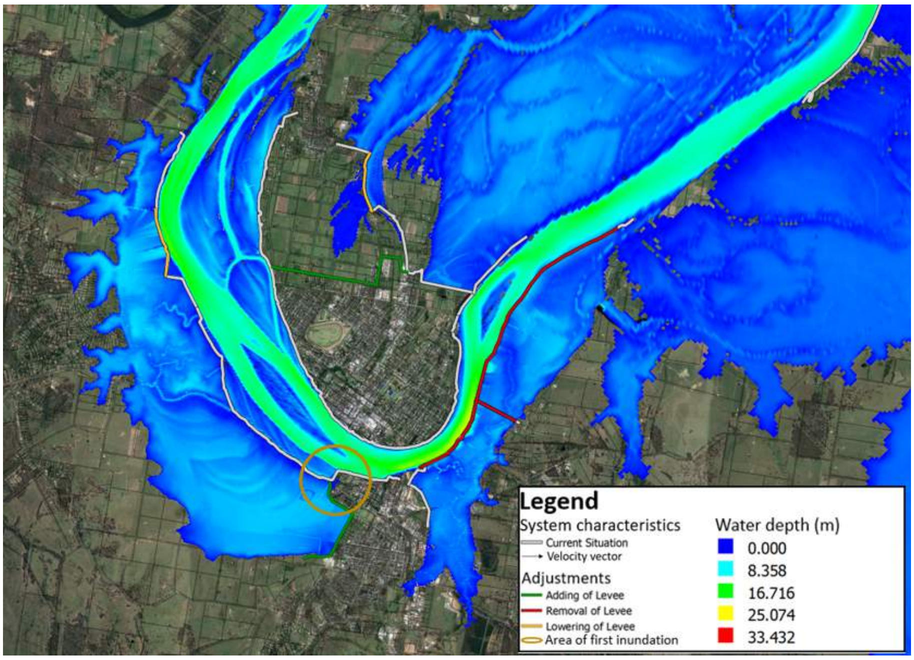


Figure G.83: Location of first overtopping during a 20 year ARI year flood event near South Grafton, see yellow circle.

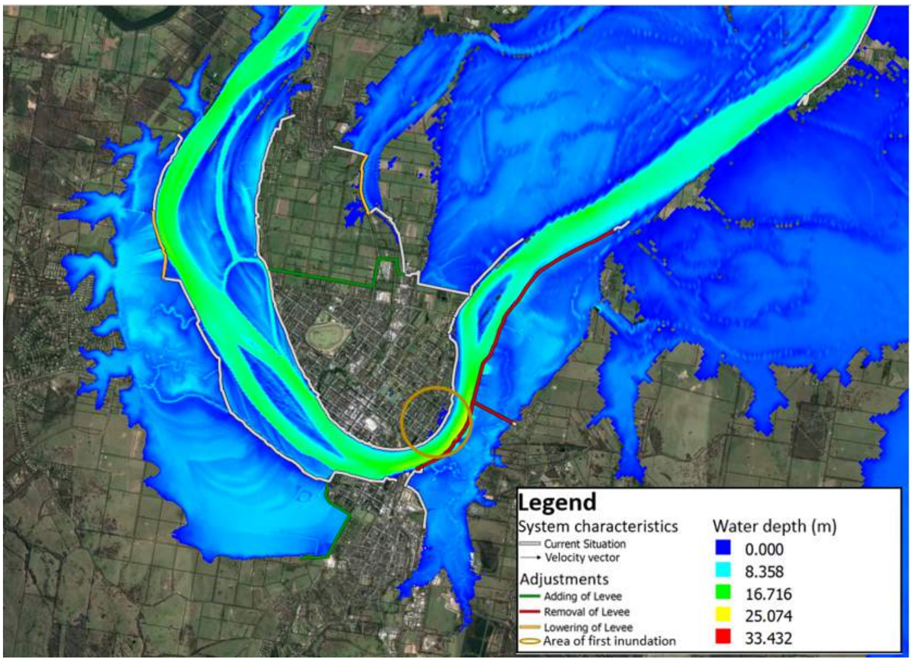


Figure G.84: Location of first overtopping during a 50 year ARI flood event near Grafton (south), see yellow circle.

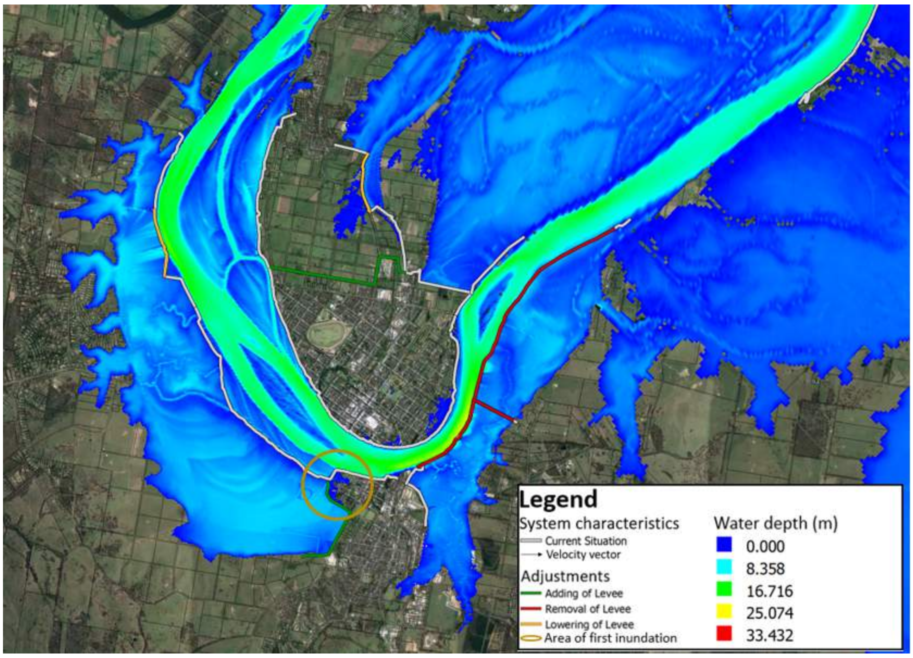


Figure G.85: Location of first overtopping during a 50 year ARI flood event near South Grafton, see yellow circle.

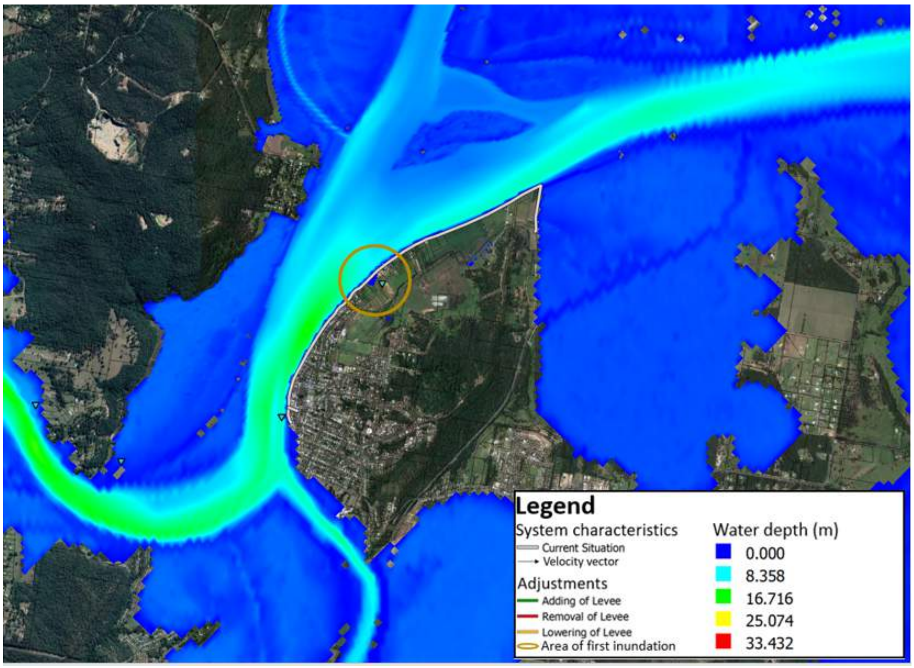


Figure G.86: Location of first overtopping during a 50 year ARI flood event in Maclean, see yellow circle.