TWO RIVERS URBAN PARK SPECIALIST STUDY:

MODELLING OF FLOOD MITIGATION OPTIONS ON THE SALT RIVER

TASK 2 FINAL REPORT

Prepared for:

Western Cape Government in partnership with The City of Cape Town

22 March 2017

This report was compiled by:

Royal HaskoningDHV
163 Uys Krige Drive
RHDHV House
Tygerberg Office Park
Plattekloof
Cape Town
7500
Email: Tasneem.steenkamp@rhdhv.com

With contributions from the NM&A TRUP

Project Team

Melanie Attwell and Associates

PROJECT NAME: TWO RIVERS URBAN PARK (TRUP)		
PROJECT NUMBER: S174/14	**DATE:** 22 March 2017	**REPORT STATUS:** Draft Report
CARRIED OUT BY: Royal HaskoningDHV (Pty) Ltd (RHDHV)		**COMMISSIONED BY:** Western Cape Government (WCG): Department of Transport and Public Works (DTPW) Regeneration Programme in partnership with the City of Cape Town (CCT) Development and Facilitation Unit
AUTHOR(S): Peter Hirschowitz		**CLIENT CONTACT PERSON:** WCG Project Manager: Gerhard Gerber 021 483 2787 Gerhard.Gerber@westerncape.gov.za CCT Stormwater Representative: Ben de Wet 021 4005036 Ben.dewet@capetown.gov.za

SYNOPSIS:

A hydraulic model of the river and bulk stormwater network within TRUP and in the flood-prone area downstream was set up. Flood extents were plotted for the current situation with climate change, as well as for the proposed development and various flood mitigation options. The model showed that proposed development scenario 7, as well as infill of the River Club island would have little effect on flood levels and extents. Construction of a small stormwater detention facility at the M5 Office Park adjacent to Berkley Road can reduce the frequency of flooding of Berkley Road at the M5 interchange.

COPYRIGHT: Western Cape Government				
QUALITY VERIFICATION:				
Verification	**Capacity**	**Name**	**Signature**	**Date**
Checked by:	Specialist water management	Marieke de Groen		20 March 2017
Approved by:	Project Managers (in as far as meeting the TOR are concerned)	Nisa Mammon and Jody Paterson		22 March 2017
Authorised by:	WCG: Project Manager / Director	Gerhard Gerber		

Table of Contents

Appendices

Executive Summary

Purpose of Study

Two components of the terms of reference of the currently appointed TRUP Project Team[1] are to (1) prepare a 2D model of the flooding in the Two Rivers Urban Park (TRUP) project area and the flood-prone area downstream including Paarden Eiland and Maitland and (2) to propose and model various mitigation interventions aimed at reducing flooding extents. This specialist study "Modelling of Flood Mitigation Options on the Salt River" addresses these two components.

A separate report on preparation of the model (Task 1 report) was submitted in October 2016. This report (the Task 2 report) covers both the preparation of the model and the modelling of development scenarios and mitigation options. Substantial revisions to the base models described in Task 1 have since been undertaken. The relevant models, maps and report sections have therefore been revised in this report, which supersedes the Task 1 report.

The scope of this specialist study did not include the design of the local stormwater system in TRUP, to avoid flooding by local rainfall events. Local rainfall events do hardly contribute to the river flooding, as the rain that falls locally is only a small amount in comparison to the peak inflows from upstream during extreme flood events.

Methodology

A model that simulates the flow of water through TRUP over time in a two-dimensional way (a 2D hydrodynamic model with 1D inflows) was set up based on the current river and floodplain geometry. This model was then modified to test the effects of the latest development scenario of the urban planners (scenario 6 at the start of the modelling and later scenario 7). Possible mitigation options were identified and evaluated qualitatively on their likely effect on flood levels and extents. Promising options were modelled and finally evaluated based on the model results.

[1] Appointed to provide Professional Services to undertake Urban Planning, Landscape Architecture, Engineering, Environmental and Heritage Studies for the Two Rivers Urban Park, Cape Town.

Initial model setup

The software specified in the terms of reference is Personal Computer Stormwater Management Model (PCSWMM), which is designed primarily for the modelling of stormwater networks. Its numerical set up is a quasi 2D model, representing flows between cells as if they would flow through canals. For the urban design challenge and river 2D modelling in general, the consultant recommended other software, but PCSWMM was nonetheless preferred by the City of Cape Town, as this is the software which they currently use.

As agreed with the Client, for the upstream inflows, the flows determined in the Stormwater Infrastructure Asset Management Plan (Phase 2A) - Rainfall Analysis and High Level Master planning, as developed by SRK in 2012 were used. As per the SRK study, to account for climate change the design rainfall events were increased by 15%.

The one-dimensional model of the rivers and bulk stormwater network by SRK (2012) was improved. This was in particular done for the area downstream of TRUP for the stormwater network and within TRUP for the bridges and stormwater network inflows. The geometry of the river channel and structures was refined using a topographic survey undertaken on all bridges (25) within TRUP and in the Salt canal, as well as many other rivers cross-sections (83) and weirs and stormwater outlets. A two-dimensional representation of overland floodplain flow was added based on airborne surveys of the City of Cape Town (LiDAR-method).

The bulk stormwater network was included in the model, including areas within the floodplain of the Salt River Canal that are outside of the area that drains to the Salt River catchment in non-flood circumstances. Although the consultant did not consider this of high importance for the urban design questions of the overall project, it was a requirement in the terms of reference.

The sea level modelled is a combination of the high tides that are less high 90% of the time, and the low tides that are less low 90% of the time, combined with a sea level rise to 2060 which is regarded by the City of Cape Town as the best available estimate for this kind of flooding study. For the flood that has an expected frequency of exceedance of 1:100 years, the storm surge and wave setup that has a probability of exceedance of 1:50 years was added to this tidal range. For all other floods, the 1:20 year storm surge and wave setup was added to the tidal range above. The timing of the tide had a marginal effect on flood extents.

Status Quo Findings

A long-section along the lowest point of the river / canal bed produced from the survey shows that the bottom of the Salt River canal is more than a meter higher than the bottom of the wetland at the junction of the Liesbeek and Black Rivers adjacent to the PRASA yard, which is below sea level.

Modelled flood extents are presented in Appendix A for different estimated probabilities that a certain flood will be exceeded in any year (1:100 year, 1:50 year, 1:20 year, 1:10 year and 1:5 year). This implies that design floods were used, not historic floods, for return periods of 100 years, 50 years, 20 years, 10 years and 5 years.

It was assumed that the flood for a certain return period in TRUP had the same return periods in the Liesbeek and Black Rivers, as well as local rainfall and stormwater runoff of the same return period. Thus the model only investigated extreme storms over both catchments at the same time. In reality, an extreme storm over the Liesbeek catchment would usually occur together with more moderate rainfall over the Black River catchment and an extreme storm over the Black River catchment would likely occur together with more moderate rainfall in the Liesbeek catchment.

Exacerbating circumstances such as the blockage of downstream bridges or the failure of upstream dams were not considered. It is pointed out here that generally these have large impacts on floods on the ground. As was mentioned in the workshop of 2 November, also in Cape Town there are experiences with floods being locally exacerbated due to the system not functioning as designed.

Main conclusions for the purposes of advising the TRUP professional design team from this report with the adjusted Task 1 findings are:

Attenuation of the 1:100 year inflows within TRUP is predicted by the model to be 13%, from a total inflow peak of 303 m^3/s to approximately 264m^3/s.

Local flooding due to capacity of the bulk stormwater network being exceeded by the 1:10 year flood is shown between Berkley Road and Frere Road and at the soccer stadium and by the 1:20 year flood in northern parts of Maitland Garden Village, in the area around Eastman Road.

The modelled flood extents for the current situation for the 1:100 year and 1:50 year floods are very similar to the SRK models of 2012. However, flood extents for the 1:20 year flood differ significantly in the area west of Liesbeek Parkway, where the TRUP analysis predicts only minimal flooding for this return period. According to the two-dimensional model, the river water only leaves the TRUP area through the Salt River Canal for floods up to the 1:20 year flood, and the water flooding the PRASA depot is basically coming from the bulk stormwater network further to the West, except for direct flooding of a narrow strip along the bank of the Liesbeek. However, flow begins to leave TRUP via the PRASA yard for the 1:50 year flood, and for the 1:100 year flood, approximately 0.5% of the flood volume or 1.7% of the peak flow is predicted to leave TRUP through the PRASA yard.

Urban development scenarios modelled

The latest urban planning scenario at the start of flood modelling Task 2 was identified as scenario 6. During the modelling, an updated scenario 7 was made

available, and this was included in the later models. Both scenarios are shown in Appendix I. The proposed development includes buildable areas, green areas and parking areas. As green areas and parking areas do not form an obstruction to overland flow, only the buildable areas were incorporated into the model. The buildable areas were added as areas that could not be flooded.

There has been some discussion regarding development on the River Club Island, which is likely to be one of the first areas to be re-developed within TRUP. While scenario 6 considers development of only a narrow strip of floodplain along the western edge of the River Club Island and scenario 7 considers development of only the northern and southern tips of the River Club Island, initial proposals by the property owner consider more extensive development. In order to investigate the effect of maximum development on the River Club Island, it was decided to also model the entire property as raised above all flood levels.

Effect of urban development scenarios on flooding

The proposed development scenario 7 has little impact on flood levels. The maximum effect is along the Liesbeek River, where the obstruction presented by the proposed development causes backing up. Expected changes in the 1:100 year flood levels are up to 0.05m. The proposed development leads to an increase in the flood extents at the PRASA yard and between the PRASA yard and Voortrekker Road due to the loss of upstream attenuation within TRUP. The development also blocks off some of the flow towards the western side of Liesbeek Parkway, resulting in a decrease in flood extents to the west of Liesbeek Parkway. There is an increase in flooding along the stormwater main through Maitland Garden Village where this enters the Black River floodplain, due to a slight increase in water levels along the Black River.

The effect of the proposed development on the volume stored in TRUP is 94 000 m^3 for the 1:100 year flood. This is an indication for a compensation needed elsewhere, in case a "builder compensates" principle will be applied.

The additional effect of infill of the River Club Island for the building areas in scenario 6 is up to 0.1m for the 1:100 year flood levels, but the effect on flood extents is marginal.

Mitigation measures considered

After an initial consultation during a workshop of key role players conducted in May 2016, the decision on which mitigation options to be considered for modelling was made. The mitigation option concepts considered at the workshop in May 2016 are illustrated in Appendix B.

While in this executive summary the effects of mitigation measures on TRUP are described, as this is relevant for the urban planning in this project, it needs to be realised that most benefits of some mitigation measures are downstream of the TRUP

area, as is shown in the maps in the report. The following are the main mitigation measures considered.

Mitigation: flood protection berms, infill and construction

Wherever development is proposed within the floodplain, the buildings themselves will either be designed to be resilient to, or they will need to be protected by constructing these areas of the development on fill or by constructing berms (dykes, flood protection walls) around them for flood protection. Hydraulically, all these options are similar, since they all keep the flood water out. The modelling of the proposed development scenario 7 has already taken this into account by considering the proposed development as an 'obstruction' to flow, in the sense that flow cannot go through the boundaries of the building areas.

Mitigation: Reducing catchment inflows through catchment measures

It was agreed to consider the effect of upstream catchment flow reduction measures (such as rainwater and stormwater harvesting or widening of floodplains) by modelling a reduction in the 1:100 year peak flow in both the Liesbeek and Black rivers by 15%. The reduction in modelled flood extents within and downstream of TRUP is very marginal. The only area in which a more noticeable reduction in flood extent is predicted is along the Liesbeek upstream of the N2.

An indication of the effect of other reductions in flow can be gained by comparing the flood extents for other different return periods in the modelling of the current situation. While each lower flow corresponds to a slight reduction in flood extent, the largest differences in flood extent are between the 1:50 year and 1:20 year floods and between the 1:20 year and 1:10 year floods. The 1:20 year flood extent corresponds to a flow reduction of 26% on the Liesbeek River and 32% on the Black River when compared to the 1:100 year flood. This magnitude of reduction is highly unlikely to be achievable.

Mitigation: Reducing catchment inflows through high flow diversion of Elsieskraal River

The Elsieskraal River is one of the main tributaries contributing to flood flows in the Black River. Diversion of flood flows from the Elsieskraal River from a point at Thornton / Wingfield directly to the ocean was considered. This intervention would have the potential to reduce the 1:100 year flood peak to the range of the current 1:10 year or 1:20 year flood peak. However, this option is likely to have prohibitive costs and extensive legal implications.

Mitigation: Raising of bridges within and downstream of TRUP

This option was briefly considered, but not further recommended because of the high costs involved with the raising of bridges. In particular rail bridges are very difficult to raise due to the vertical alignment requirements for railways. The effect of

most individual structures on water levels is in any case relatively small, as was investigated by studying the model results of the longitudinal profile of the river (along the river axis) for differences in water levels up- and downstream of bridges for the current situation. Cumulatively, however, their effect is significant.

Mitigation: Increasing the flow area in the river channel

More flow area means increasing the cross-sections of rivers, to allow more flow through and therefore a lower flood level. A variety of different channel cross-sections could be used to increase the channel area. The Liesbeek Dead Arm could also potentially function as an additional parallel channel for the Liesbeek if it were connected to the Liesbeek at the upstream end. However, this would have limited benefit in terms of flood reduction, since the water surface profile along the Liesbeek River below Observatory Road is very flat.

In order to ascertain the potential maximum effect of increasing the channel area, it was decided to model an additional rectangular channel in parallel with the existing Salt River and Black River channels, together with widening the bridges crossing these channels. A widening by means of an additional parallel rectangular channel of 25m wide was agreed and modelled. Immediately downstream of the N2, there is a predicted reduction of 0.83m in the water level due to the widening, based on preliminary models. It is clear from both the derived water levels and the flood extents that the main effect is on the Black River towards the upstream end of TRUP, in the floodplain near Oude Molen.

The main reduction in flood extents was towards the upstream end of the Black River. Therefore, widening or otherwise modifying for increased cross-sectional area for only the Black River channel and bridges and not the Salt River Canal was also modelled. This model achieved slightly less reduction in flood extents than with widening the Salt River Canal as well. Both mitigation options did not significantly reduce the flood risk because most of the development footprints within the floodplain are along the Liesbeek River and not the Black River.

In view of the ecological sensitivity of the western side of the Black River, channel widening would most likely be through the wetland in front of Oude Molen, and further mainly along the eastern side of the existing channel. However, the enlargement would need to extend westwards into the Raapenberg Bird Sanctuary adjacent to the South African Astronomical Observatory, where space on the eastern side is limited by the M5 freeway. The ecological implications would have to be looked into further.

Channel enlargement would involve significant capital costs. These would include widening of the existing bridges across the Black River, as well as excavation, landscaping and vegetation of the channel. The largest cost would be the widening of the M5 Bridge, for which a cost of the order of R 50 to R 60 million seems likely, but the total costs of widening would be far higher.

The Liesbeek Dead Arm could potentially function as an additional parallel channel for the Liesbeek if it were connected to the Liesbeek at the upstream end. However, this would have limited benefit in terms of flood reduction in view of the very small change in water level along this reach.

Mitigation: Stormwater attenuation ponds

Within TRUP, the base model indicated potential flooding from the two bulk stormwater lines entering the Black River from the east (at Maitland Garden Village and adjacent to Berkley Road). Attenuation ponds were therefore modelled in open areas along or adjacent to these two culverts.

Routing the stormwater flow adjacent to Berkley Road through a small detention pond would help to reduce flooding downstream of the off-take to the proposed detention ponds. Although there is no proposed development downstream of this pond, flooding along Berkley Road at the M5 interchange could potentially be reduced. The alternative of larger stormwater conduits in this area might be preferable in terms of land requirements and cost.

For the bulk stormwater main pipeline at Maitland Garden Village, model results indicate that the proposed detention pond would have a large effect on flooding in this area during major storm events, but is probably not necessary since flooding is predicted to occur only rarely and provision of a flood water escape route will still be required.

Stormwater detention ponds are common, and their cost is generally not prohibitive. A capital cost of several million rand would be expected for the two ponds.

Mitigation: Extension of the existing wetlands within the M5 / N2 interchange and west of Oude Molen

The existing wetlands within the M5/N2 interchange and west of Maitland Garden village provide some flood storage and their extension could provide additional storage. The extension of these wetlands was modelled, but has a negligible effect on peak flows in the Black River.

Mitigation: Improvement of the Zoarvlei outlet

Water from the Salt River spills over into the Zoarvlei. The Zoarvlei discharges to the Mouth of the Diep River through a stormwater culvert. It was considered that improvement of this outlet could potentially reduce flood levels in the Salt River and even further upstream along the Black River. However, it would appear that the high ground between the Salt River Canal and the Zoarvlei has more influence on flow into the Zoarvlei than the water levels in the Zoarvlei. It was therefore concluded that this is unlikely to have a significant effect on flooding within TRUP and was not modelled.

Mitigation: Converting the River Club to a flood detention area

This is the main potential detention area along the Liesbeek, and was modelled. Flooding and the effect of storage will depend on the relative magnitudes and timing of the flood peaks in the Black and Liesbeek Rivers. There is little change in predicted flood extents along the Liesbeek and Black Rivers within TRUP, but storage within TRUP reduces downstream flows (Figure 23 on page 66), resulting in a decrease in flood extents in the PRASA yard and in the Woodstock or Foreshore area.

The conversion of the River Club Island to a stormwater detention pond would involve large capital costs. The main capital costs would be land acquisition, design, procurement and contract administration and inspection, excavation, construction of an inlet structure. If the natural ground between the storage area and the Liesbeek River and that between the storage area and the Liesbeek Dead Arm are structurally unsuitable, a retaining wall may be required, which would be a major additional cost.

Mitigation: Flood storage upstream of TRUP

Storage could also be considered upstream of TRUP, in the open area of the Rondebosch and King David Mowbray golf clubs. Flood storage ponds covering the area of these two golf clubs could potentially provide a significant reduction in flood peaks along the Black River. This effect was not modelled, as it would require an extension of the hydraulic model and it is recommended for future modelling in case it would be a consideration to implement this flood mitigation measure. Capital costs would be high.

Mitigation: Combination of increased Black River channel cross-sectional area and flood storage

The combination of widening the Black River channel and providing flood storage both on the River Club Island and through widening the existing wetlands along the Black River in the floodplain was also modelled. The effect is similar to the effect of the Black River widening alone, implying that only the flood extent in front of Oude Molen in the 1:100 year flood is decreased. However, in the 1:5 year flood, there is also a noticeable decrease in flooding along the Liesbeek, as well as in Maitland and Paarden Eiland.

Recommendations

The management of both the river channels and flood storage areas would need to be integrated with the broader Green Corridor Management Plan being developed as part of this project.

Construction of a small stormwater detention facility in M5 office park adjacent to Berkley Road would be recommended to reduce the frequency of flooding of

Berkley Road at the M5 interchange. Negotiations with the owners of the M5 Office Park could be initiated concerning the construction of a regional stormwater detention facility, in case the need is expressed to decrease flooding by local stormwater. The City of Cape Town (2002) states that minor stormwater systems should accommodate the 1:5 year flood in general commercial and industrial areas. However, Berkley Road is already predicted to flood during the 1:5 year rainfall event.

In the previous Scenario 6 development plan, the buildable area close to Maitland Garden Village (building L1 on the map in Appendix I) lies in the overland flow path of the major stormwater drainage system. The development plan has been revised so that in Scenario 7, although the buildable areas are still within the floodplain, space has now been left for a flood water escape route. Allowance for a flood escape route should be sufficient to address flooding in this area, and the flood escape route should be shown on the development plans.

Where "nature-friendly" river banks are envisaged, the cross-sectional area of the channel will need to be increased to compensate for the fact that plants along the banks will slow the water, technically termed higher channel hydraulic roughness; therefore a bigger cross-section is needed. However, general widening, and particularly widening of the M5 bridge is unlikely to have sufficient cost benefit, and would damage sensitive habitats.

Two alternatives have been considered for the River Club Island: either flood storage or infill and development. Flood storage should be considered only in combination with storage at the Rondebosch and King David Mowbray golf courses or other upstream measures, as on its own it does not have a significant effect on flooding within TRUP. The effect of the combined flood storage above could be evaluated using a hydraulic model extending further upstream, with additional surveys of the Black River cross-sections upstream. If the results are promising, then a cost-benefit analysis should be undertaken in conjunction with a geotechnical investigation. If this storage combination is not envisaged, then development of the River Club in line with the TRUP proposals would increase flood levels by only 0.05m compared to the current situation where the buildable areas on the River Club Island are available for flood attenuation. There is also little effect on flood extents. When comparing the effect of Scenario 6 and Scenario 7 development footprints on flood levels, the Scenario 7 development footprint is predicted to slightly increase flood levels along the Liesbeek River whereas the Scenario 6 footprint is predicted to slightly increase flood levels along the Salt River.

Permitting requirements for all flood mitigation measures that are accepted in the Green Corridor Plan will need to be integrated into the TRUP Environmental Impact Assessment and Water Use License Application processes. Interventions in the floodplain that might mitigate floods can have landscape or ecological advantages or disadvantages.

We started the project with the recommendation to use another 2D Hydraulic model than quasi 2D PCSWMM (See section 4.1). Many model runs and problems with the software later, we still recommend CCT to not use PCSWMM for any further 2D Hydraulic Models to be made for other catchments, and even for more detailed design of this project area.

Last but not least, while useful flow data were not available for calibration of the models in this specialist study, they will be important for further consideration of interventions in the watercourses in TRUP and in other areas in the same catchment (such as decision on effluent reclamation), for the full season, not just for high flows. It is therefore recommended to CCT to improve on gauges, rating curves, data collection and/or database management, whichever is needed to have a good continuous set of flow gauging data.

Glossary of Terms:

CCT	City of Cape Town
D	Dimensional
EGL	Energy Grade Level
GIS	Geographic Information System
HEC-RAS	Hydrological Research Centre River Analysis System
HGL	Hydraulic Grade Line
LiDAR	Light Detection and Ranging
NM&A	NM & Associates Planners and Designers
PCSWMM	Personal Computer Stormwater Management Model
PRASA	Passenger Rail Agency South Africa
QEGL	Quasi energy grade level
SCS	Soil Conservation Service
SCS-SA	Soil Conservation Service – South Africa
TIN	Triangulated Irregular Network
TOR	Terms of Reference
TRUP	Two Rivers Urban Park

1 Introduction

The Two Rivers Urban Park (TRUP) is an area along the Liesbeek and Black Rivers between the N2 freeway up to and including the junction where these rivers join to form the Salt River. Development of TRUP is being planned using the Package of Plans approach as part of the Western Cape Government Regeneration Programme. Nisa Mammon & Associates (NM&A) was appointed by the Western Cape Government who is working in partnership with the City of Cape Town to provide professional services as part of this project. Royal HaskoningDHV were appointed by NM & Associates to provide various specialist services, one of which is the modelling of flood mitigation options. Most of the report under review was prepared by Peter Hirschowitz and Saieshni Govender and reviewed by Marieke de Groen in 2016, when they were both employed by Royal HaskoningDHV. As they both left Royal HaskoningDHV end of December 2016, Peter Hirschowitz and Marieke de Groen (AquaLinks) were appointed by Royal HaskoningDHV to make final corrections to the modelling and prepare the final report.

As per the terms of reference (TOR), the "aim of this specialist study is to prepare a two-dimensional (2D) model of the flooding of the area; and to propose and model various interventions aimed at reducing flooding." The specialist study was divided into two tasks. Task 1 includes the model setup and calibration and proposing flood mitigation options. Task 2 was to use the model set up in Task 1 to "model creative flood reduction interventions". While the Client of NM & Associates is the Western Cape Government, who is working in partnership with City of Cape Town, the technical guidance of this Task was delegated to Mr Ben de Wet, from the Stormwater and Sustainability Branch of the City of Cape Town. This report covers both Task 1 and Task 2.

After a draft Task 1 report was submitted in July 2016, a final report on Task 1 with accompanying appendices, including maps was submitted in October 2016. However, substantial revisions to the base models described in Task 1 have since been undertaken. The relevant models, maps and report sections have therefore been revised in this report. This report, including the accompanying models, maps and appendices, supersedes the Task 1 report. The relevant revisions are described in sections 4.1, 4.6, 4.7, 5.5 and Appendix H.

The modelling of flood mitigation measures was undertaken in parallel with the separate watercourse management specialist study, the final report (de Groen et al., 2017) for which has been submitted in February 2017. The watercourse management specialist study deals more broadly with the management of the watercourse, and mainly focuses on interventions that can improve the water quality and ecological functioning. It includes consideration of some of the same interventions where these occur within TRUP and are relevant for flood mitigation, as far as they interlink with the objective of improving on the functioning of the river and the considered interventions to improve this functioning.

2 Report Structure

The base models set up (part of Task 1) is described in Part A. The input data are described in section 3. The model development and checking will be described in sections 4 and 5 respectively. Results of the base model runs are presented in section 6 and in the maps in Appendix A.

The latest development scenario available at the time of modelling is scenario 6, and the modelling of this scenario is presented in Part B and in the maps in Appendix J.

Flood mitigation options and refinements modelled in Task 2 will be described in Part C. Maps showing the effect of these options on flood extents are included in Appendix K. Recommendations are included in Chapter 16 at the end of Part C.

PART A: BASE MODEL

3 Input Data

3.1 Existing models

Existing catchment and river models were made available by the City of Cape Town. These models were prepared by the SRK consulting (2012) as part of a "*high level stormwater master plan for the Salt River catchment*".

These included:

- Models of the bulk stormwater network (including rivers) in each primary sub-catchment of the Salt River using Personal Computer Stormwater Management Model (PCSWMM) software. Figure 1 shows primary sub-catchments for which PCSWMM models were available. The Jakkalsvlei and Kalksteenfontein catchments were combined in a single model, but all other primary sub-catchments were modelled separately. The models included a rainfall-runoff analysis to determine the runoff hydrograph from smaller secondary sub-catchments and a hydraulic component for routing the flow along the network. Predicted hydrographs at the downstream end of each upstream primary sub-catchment were used as inflow hydrographs to downstream sub-catchments. Overland or street flow was represented by rectangular channels. "

  Key model parameters were determined as follows:

  - "The impermeable areas were determined using a combination of land use and erf size," accroding to a table of typical impermeable area for various land uses and stand sizes.
  - "The depression storage depth was determined based on land use and slope."
  - "The SCS Curve Number (CN) for the permeable areas was determined from land-use and soil data."
  - Catchment widths were determined by dividing the catchment area by a selected representative length for each catchment.
- A single steady hydrodynamic model of all the main rivers in the catchment, including the Salt River, Black River and Liesbeek, using Hydrologic Engineering Centre – River Analysis System (HEC-RAS) software. This model was used to assess flood extents along each river for 10, 20, 50 and 100 year

return periods[2]. Flows used in the modelling were steady, meaning that they did not vary with time in the computation. Use was made of the peak flows predicted by the PCSWMM bulk stormwater model as described above. The model is based on cross-sections representing the channel and bridge geometry, which were developed using a combination of Light Detection and Ranging (LiDAR) data, site measurements and estimates. No survey was undertaken. The HEC-RAS one-dimensional version was used and later on imported into Geographical Information System (GIS) software for display purposes.

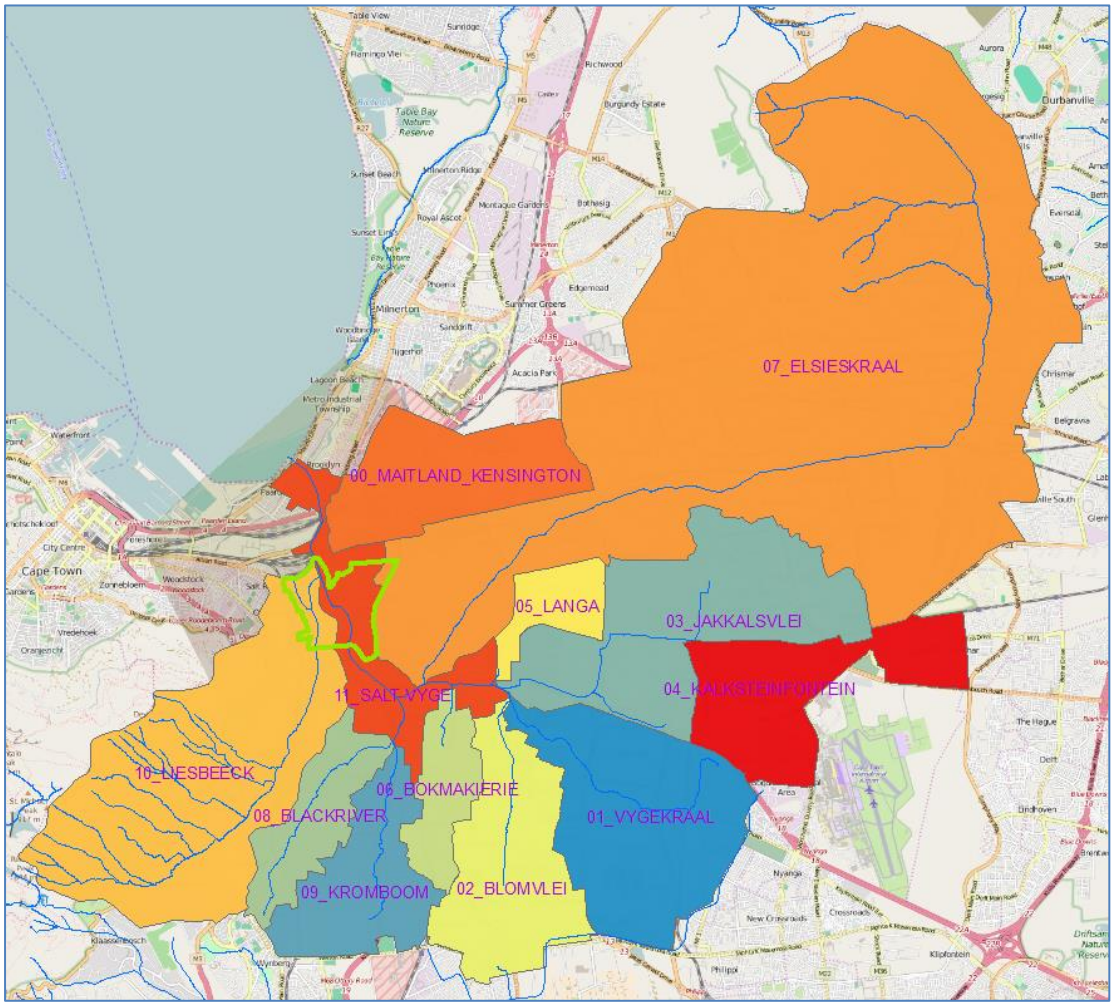


Figure 1: Salt River primary sub-catchments with TRUP boundary in Green (Background: Open Street Map and contributors)

[2] A design flood for a certain return period theoretically has a chance of being exceeded in any year with a probability of 1 divided by that return period, meaning that the chance in any year of having a flood larger than the design flood for a 100 year return period will theoretically be just 1%.

3.2 Data provided by the City of Cape Town

Rainfall and stream flow data

Extracts of the rainfall and stream flow records for gauges in and around the Salt River catchment were provided. The relevant stations are shown in Figure 2 and Figure 3 respectively. The following sets of data were provided:

- 15 minute interval flow data for the stations and over the period listed in Table 1. It will be shown later on in section 5.2 that these data could not be used for model calibration, partly because they do not all cover the flood of August 2004, partly because they do not seem to make sense in combination with the daily rainfall data. As noted in Table 1, flows in the Black River at Sybrand Park are available for two days in October 2004 only. This introduces an uncertainty which is compounded by the unknown inflows between the gauges on the Liesbeek and Elsieskraal rivers and TRUP.

Table 1: Flow data available from before 2011

River	Gauging Station	From	To
Black	Sybrand Park	20/10/2004	21/10/2004
Salt	Glamis Close	18/03/2004	12/04/2005
Liesbeek	Paradise	20/10/2003	16/09/2005
Liesbeek	Durban Road, Mowbray	19/08/2003	16/09/2005
Elsieskraal	Howard Centre, Pinelands	19/08/2003	16/09/2005

- Design rainfall developed by the University of KwaZulu-Natal for the City of Cape Town on a 1' x 1' grid for various return periods and durations ranging from five minutes to seven days, which included an additional factor of 15% to account for increases in peak rainfall due to climate change, and which was used by SRK consulting (2012) in their stormwater models.

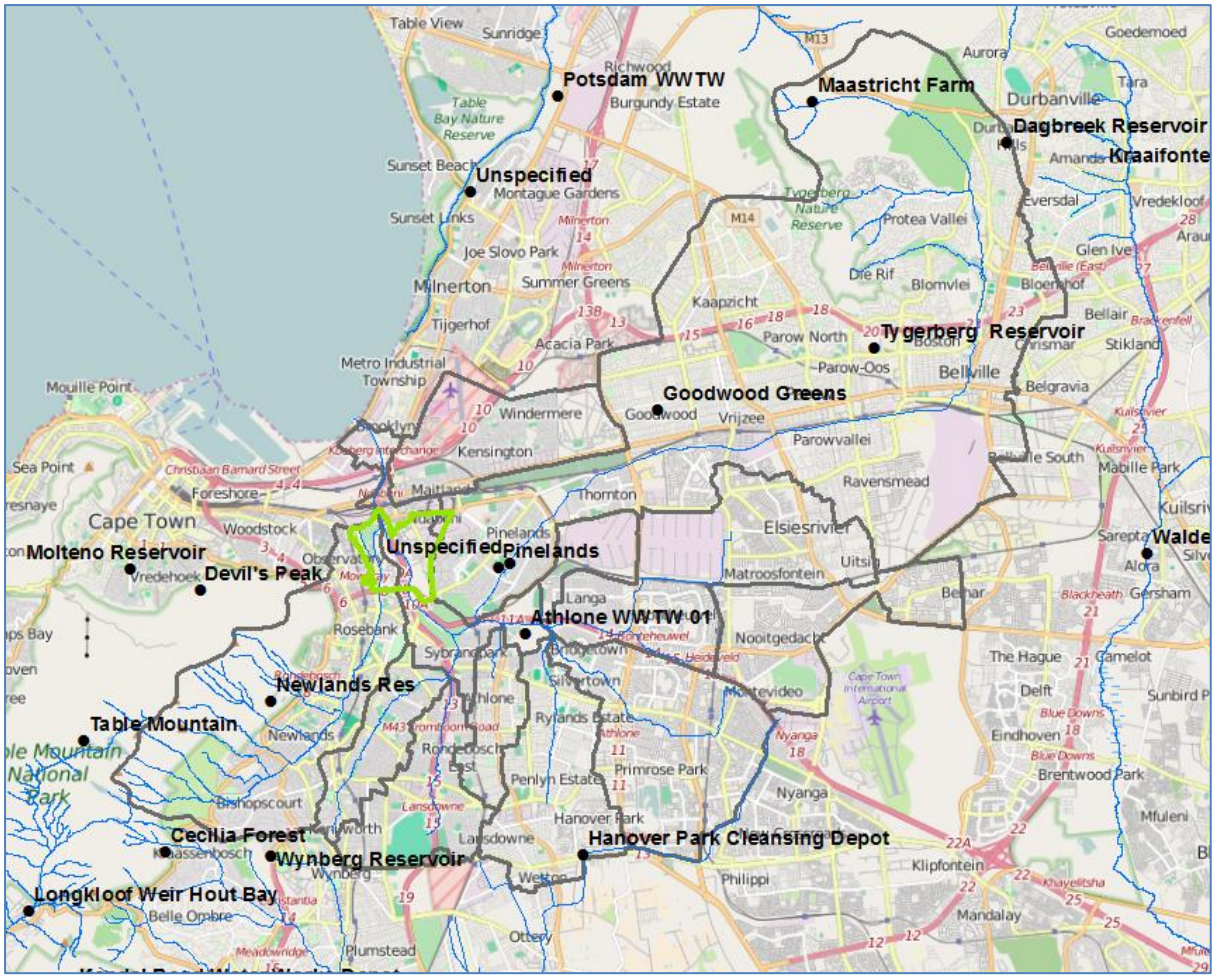


Figure 2: Rainfall stations in and around the Salt River catchment (Background: Open Street Map and contributors)

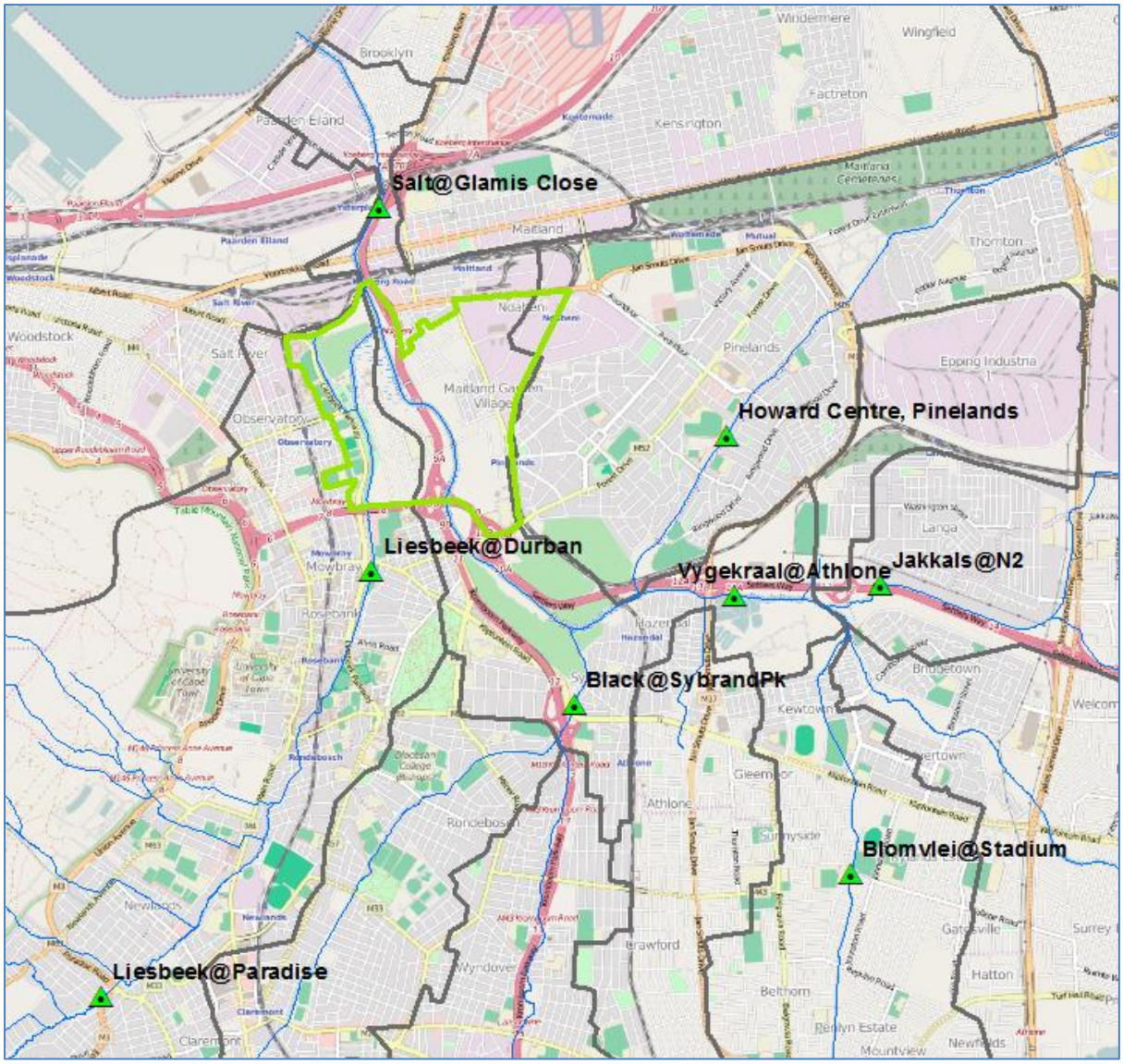


Figure 3: Stream flow gauges in and around the Salt River catchment (Background: Open Street Map and contributors)

Stormwater network and canal data

GIS shape files of the stormwater network were provided by the City of Cape Town, including conduits (i.e. pipes, culverts and open channels), inlets and outlets, junctions (i.e. manholes, junction boxes and other chambers), pump facilities, rivers and open watercourses and storage facilities or storage ponds. Data included conduit sizes and invert levels where available, but as noted by SRK (2012), there was much missing data.

A drawing of the Salt and Black River canals including a long section and cross-sections at various points was provided. The elevation datum of this drawing was, however, not provided.

Elevation data

Both ground and non-ground LiDAR survey points of the area to be modelled were provided. A 50 m x 50 m elevation grid for the remainder of the Salt River catchment was also provided. 2008 Aerial photography was also provided.

Reports of previous investigations

Several reports were obtained from the City of Cape Town, consultants and land owners. Key amongst these for the flood modelling are:

- Final Report: High Level Stormwater Masterplan for the Salt River Catchment (SRK consulting, 2012), which described the previous models; and
- City of Cape Town Climate Change Think Tank Marine/Freshwater Theme: Marine Inputs To Salt River Flood Model (Prestedge Retief Dresner Wijnberg, 2010), which was recommended by the City of Cape Town as the source for sea levels for use in the modelling.

4 Base Model Setup

4.1 Software and overall approach

The area to be modelled was defined in the terms of reference as shown in Figure 4. In addition to the Liesbeek and Black Rivers within TRUP, the model area includes the Salt River canal, the Zoarvlei (into which overland flow would spill during a flood event) and the suburb of Paarden Eiland and parts of the Salt River suburb and harbour area which may be flooded during extreme floods in the Salt and Liesbeek Rivers.

Design conditions for modelling were defined in terms of estimated probability that a certain flood will be exceeded in any year: e.g. 1:100 year flood has a 1/100 chance of being exceeded in any year. The same return period was assumed in both the Liesbeek and Black Rivers, as well as for local rainfall and local stormwater runoff. The occurrence of flooding primarily in one catchment was not investigated.

Exacerbating circumstances such as the blockage of downstream bridges or the failure of upstream dams were not considered.

The model setup was a one dimensional – two dimensional (1D-2D) approach one-dimensional model of the rivers and bulk stormwater network and a two-dimensional representation of overland floodplain flow. A one-dimensional model assumes flow only in the downstream direction and the same water level at any time across the channel or culvert, which is a reasonable approximation.

The software specified in the terms of reference is PCSWMM or compatible software. PCSWMM is designed primarily for the modelling of stormwater networks. Its numerical set up is not a real 2D model, but a quasi (or pseudo) 2D model, representing flows between cells as if they would flow through canals, based on hydraulic properties. Bridges are not represented similar to their real physical cross-sections but with openings of the same hydraulic properties. The consultant therefore recommended other software for this urban planning project. During the early stages of the modelling, the consultants were requested to provide a quotation for the alternative use of 3Di software, but this quotation was not accepted. The consultants also proposed the use of HEC-RAS software, which is designed for river modelling. The consultants suggested that the SRK (2012) HEC-RAS model could be used as the basis for the development of a 2D HECRAS model, needed for the urban planning advice in the area of interest. However, the 2D HEC-RAS cannot be imported into PCSWMM at the time of writing of this report, and this would therefore not meet the requirement for compatibility with PCSWMM. This requirement was suggested in the TOR and insisted on, as this is the hydrodynamic modelling software which the CCT currently uses in the Stormwater Branch.

Figure 4: Modelling area marked in yellow, TRUP project area marked in green

Figure 5 is an example of the model which shows how the two-dimensional component of PCSWMM is quasi-2D in that overland flow is simulated as a network of rectangular channels, with the blue marked connection showing on the right as a canal and its properties.

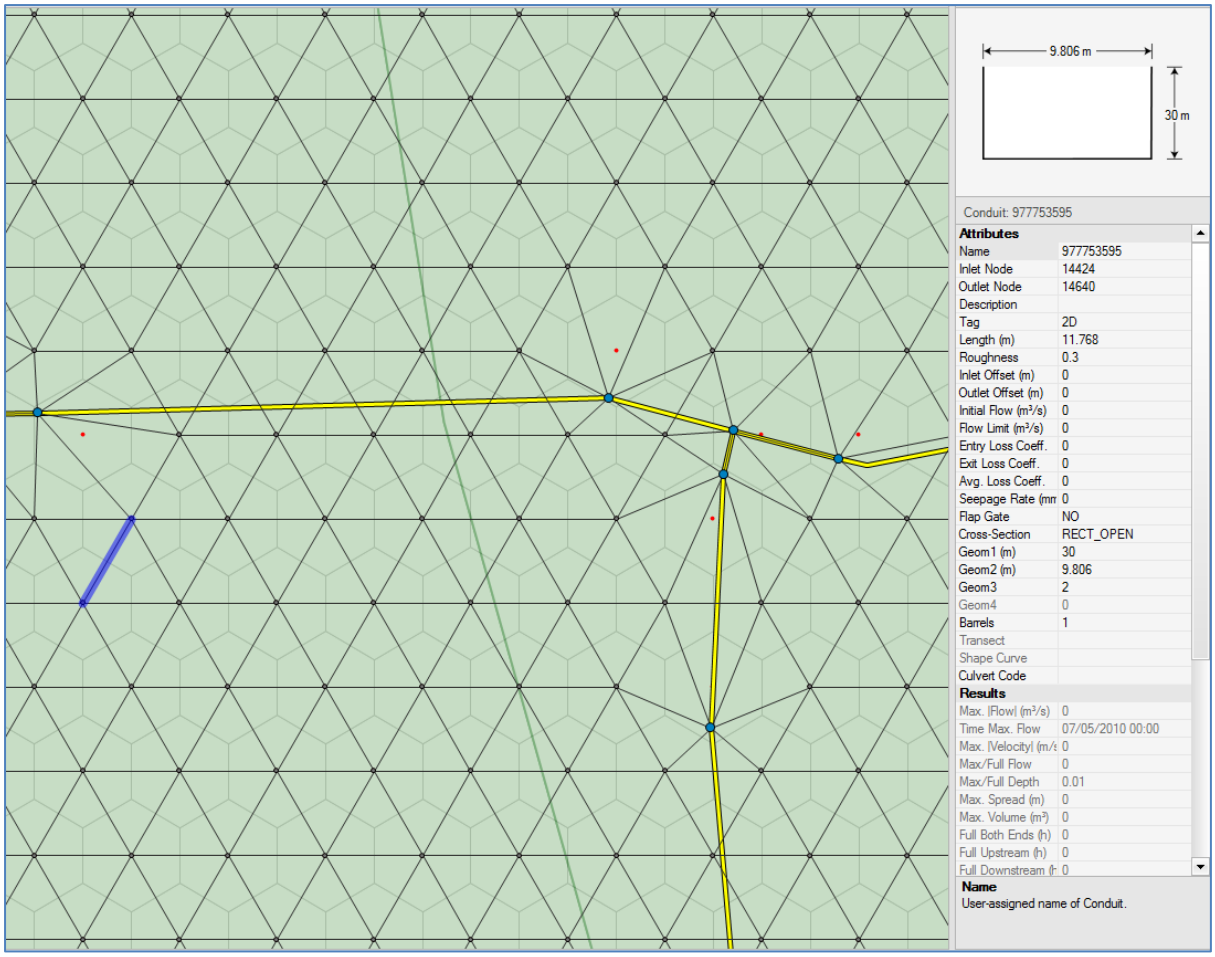


Figure 5: Representation of overland flow as a quasi-2D network in PCSWMM, with on the right the canal representation of the blue marked line

The basis of the model was the previous PCSWMM-models, which were set up by SRK (2012). However, this PCSWMM-model was not used by SRK (2012) for the delineation of floodlines. Rather, predicted peak flows in the river were exported from the PCSWMM model to a separate HEC-RAS model for the rivers only, to do backwater analysis to determine flood levels and plot flood lines.

The three models set up by SRK (2012) for the three (3) downstream sub-catchments (Liesbeek, Salt-Vyge and Maitland-Kensington) were initially combined into a single model of the bulk stormwater and river network, which was refined as required within the model area. Catchment parameters and rainfall were derived from the original models of SRK (2012). The SRK (2012) models for these downstream catchments already included inflows from sub-catchments further upstream. These upstream sub-catchments had been modelled separately by SRK (2012).

As a result of comments on the modelling approach received in January 2017, which related much to the upstream model that was not in the scope, in models G1 to G9 the network upstream of the modelled area was removed from the model and inflows were taken from the relevant points in the Liesbeek, Salt-Vyge and Maitland-Kensington sub-catchments models of the SRK (2012), apart from the 1:50

year flood for the Maitland-Kensington model that SRK had not yet run and for which new model runs were done. As shown in Figure 6, it appeared that the version of PCSWMM software changed the model results[3].

The larger flow hydrographs calculated by running the updated SWMM engine (v5.1.007, 2014) were selected for the Liesbeek sub-catchment (and also the 1:50 year flows for the Maitland-Kensington sub-catchment) for all further modelling (including model G1), since:

- The analysis in the software has probably been improved based on user experience;
- No calibration seemed to have been performed for the original flows; SRK consulting (2012) verified their Liesbeek model with flows at the Durban Road and Paradise flow gauges for the flood of 17 August 2005. The downstream Durban Road gauge is closer to TRUP and therefore more relevant. Peak flow at Durban Road was over-estimated for this event by 18% when using a synthetic time distribution of the rainfall and by 20% when using the rainfall as recorded. The peak flow also did not occur as quickly as was modelled. It would, however, appear that no calibration was performed.
- The higher flows are preferred since the flood levels already lower than in other studies as pointed out by Aurecon; and
- The omission of overland flow paths in parts of the upstream catchment would tend to reduce flood peaks, thereby counteracting the effect of selecting a higher peak.

[3] A SWMM engine does the analysis within PCSWMM software. Different versions of the SWMM engine may be selected. SRK (2012) models were originally run using SWMM Engine Build 5.0.021 (30 Sept 2010), while the TRUP models were run using Engine v5.1.007 of 2014. It was noted that in the Liesbeek catchment in particular, there is a significant difference in the flows predicted by the two software versions as shown in Figure 6. Difference appears to be primarily due to the runoff calculation in sub-catchments Salt_Lie_036 to Salt_Lie_045 at Kirstenbosch gardens and the mountain slopes above it. The effect of the change in inflow on the TRUP model was investigated by running one model (G2a) with the original 1:100 year flows and one model (G2b) with the flows modelled by re-running using the latest software version.

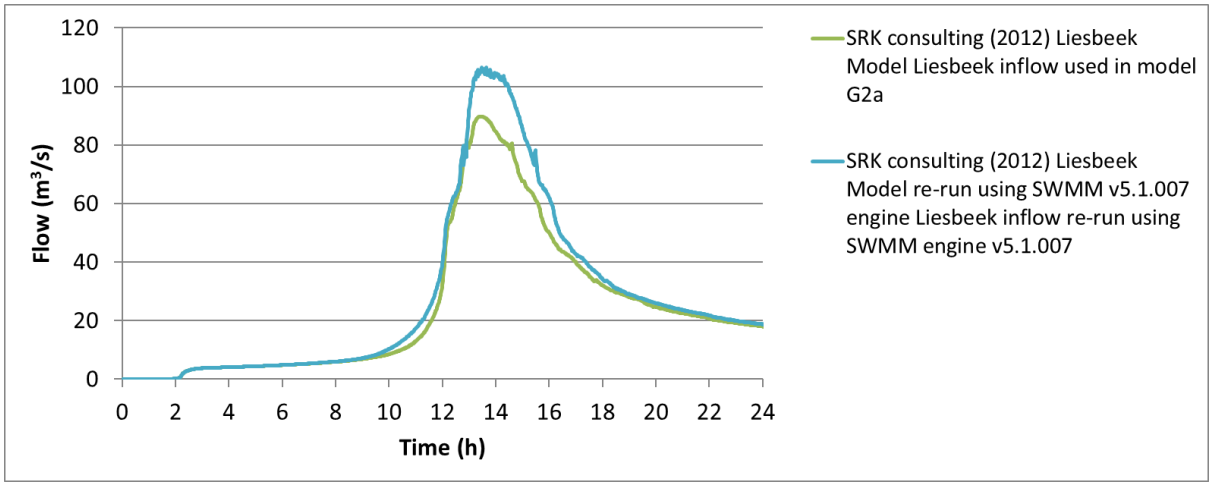


Figure 6: Effect of SWMM version on 1:100 year Liesbeek flows

4.2 Rainfall

For uniformity throughout the model, the same model rainfall distribution was used for new catchments as was used by SRK (2012).

For each return period, a 24-hour rainfall surface was interpolated between the 24h rainfall depths from the University of KwaZulu-Natal rainfall grid, which are commonly used in South Africa for design rainfall. For each catchment, the 24-hour rainfall depth was taken as the average value of this surface over the catchment. This 24-hour rainfall depth was rounded to the nearest 10mm.

The basis of the time distribution of rainfall is the SCS-SA Type 1 rainfall distribution (Schmidt and Schulze, 1987), which is illustrated in Figure 21 on page 60. This is one of four Chicago-type rainfall distributions derived by Smithers and Schulze (1987) based on South African rainfall patterns. In Chicago type rainfall distributions, peak rainfall depths for various durations are nested within each other so that for any given duration, the peak rainfall depth for a given return period occurs within that duration. The consultant agrees with the Peer Review by the Dutch team (March 2017), that this is not a realistic rainfall distribution, as in such a large catchment, the effect of the assumed short peak is limited. In the River Study Workgroup meeting of 06 April 2016 this was discussed and the consultant was nonetheless advised to use the 24-hour rainfall event as in the SRK study, as this is the standard in Cape Town and assumed conservative. This choice is also in accordance with the recommendations of the City of Cape Town Stormwater planning and design guidelines for new developments (2002), which state that "in a study by Berg et al (2000) a number of recorded storms were analysed and it was concluded that the use of the Type 1 storm distribution is most appropriate for [Cape Town]."

To get the time distribution of rainfall over any catchment, the SCS-SA type 1 rainfall distribution was scaled according to the 24-hour rainfall for that particular catchment and return period.

4.3 Runoff

As for rainfall, for uniformity throughout the model, the same runoff method, and where appropriate the same runoff parameters, were used for new catchments as was used by SRK (2012). Runoff was estimated within PCSWMM software using the SCS method (USDA-SCS, 1972) for infiltration losses and surface flow routed over the catchment using Manning's equation based on an average flow depth and an estimated typical catchment width and flow path length. The catchment parameters selected are shown in Table 2.

Table 2: Catchment parameters

Parameter	Value
Area	Measured for each sub-catchment
Catchment width	Selected for each sub-catchment
Slope	Measured for each sub-catchment
Per cent impervious	Selected for each sub-catchment
SCS curve number (representing infiltration)	Selected for each sub-catchment. New sub-catchments vary from 69 to 90, but some sub-catchments adopted from SRK (2012) had curve number as low as 39. With the exception of one catchment on the slopes of Devil's Peak, the soil type used in deriving the curve numbers for the new sub-catchments was group B, which corresponds to moderately low storm flow potential.
Depth of depression storage in impervious areas	2.8mm. Some of the SRK catchments have higher values.
Depth of depression storage pervious areas	6mm. Some of the SRK catchments have lower values.
Percentage of impervious area with no depression storage	25% (Software default)
Manning's *n* for impervious areas	0.015 (Assumed)
Manning's *n* for pervious areas	0.3 (Assumed)

4.4 Topographic Survey

The necessity for topographic survey as outlined in the terms of reference (TOR) was reinforced by the fact that previous models did not include any survey, and dimensions and levels were not entirely consistent with site observations.

A survey was conducted along the Liesbeek, Black and Salt rivers from the N2 freeway to the ocean, and included:

- 83 channel cross-sections across the rivers;
- 25 bridges;

- Two weirs across the Liesbeek;
- A pipe crossing over the Black River; and
- Pipe sizes and invert levels for stormwater outfalls along the rivers.

Additional survey requirements were initially specified, but the survey specification was later revised due to constraints on the survey budget. Items removed from the specification included:

- River cross-section and structure surveys upstream of the N2.
- Additional cross-sections within TRUP;
- Key points along the stormwater network other than the outfalls; and
- Stormwater attenuation ponds.

A long-section along the lowest point of the river / canal bed produced from the survey is shown in Figure 7. Irregularities in the long-section are explained in Figure 8.

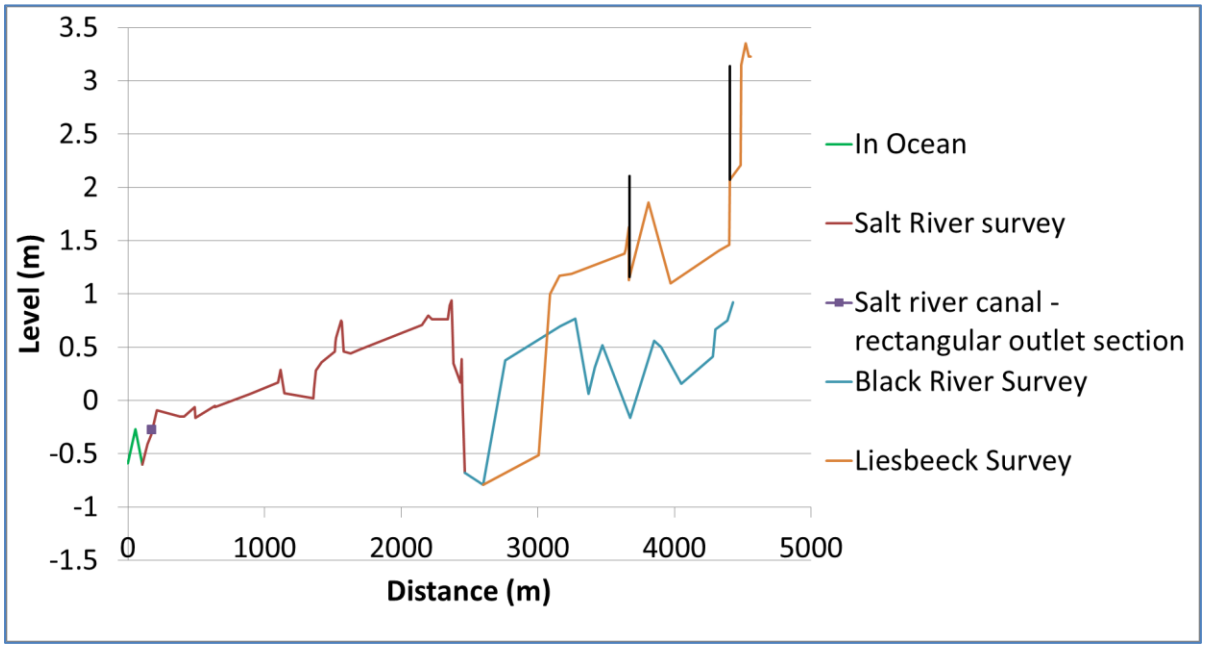


Figure 7: Surveyed long-section along the lowest point along the channel bed

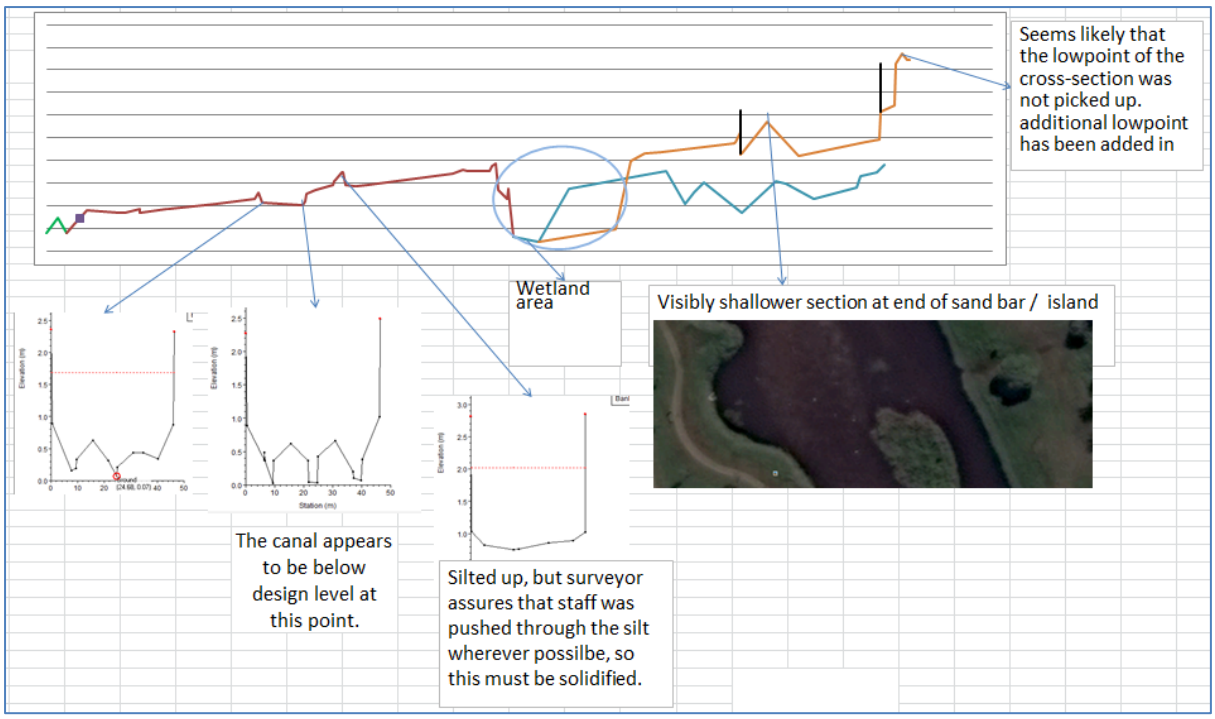


Figure 8: Explanations for irregularities in the surveyed channel long section

Figure 7 shows that the bottom of the Salt canal itself is more than a meter higher than the bottom of the lower end of the TRUP area (1-1.5 m difference). The bottom of the TRUP area is below sea level (-0.7 to -0.8 m).

4.5 Modelling of River channel and structures

The river channel up to the surveyed top of bank was modelled in one dimension. Cross-section and bridge geometry was set up using HEC-RAS software. Channel cross-sections provided by the surveyor were used directly. In order to connect to the 2D overland flow model between the surveyed cross-sections, additional cross-sections were interpolated between the measured cross-sections at a maximum spacing of 50 m using HEC-RAS. Bridge geometry was also captured in HEC-RAS using surveyed dimensions. An example is shown in Figure 9.

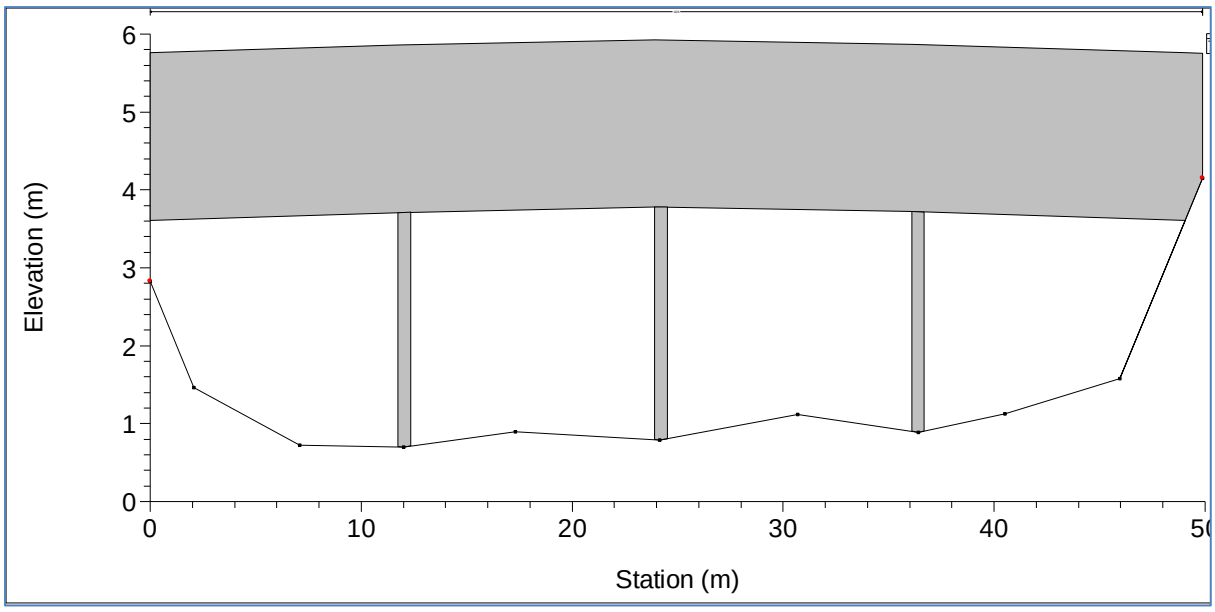


Figure 9: Example of bridge representation in HEC-RAS

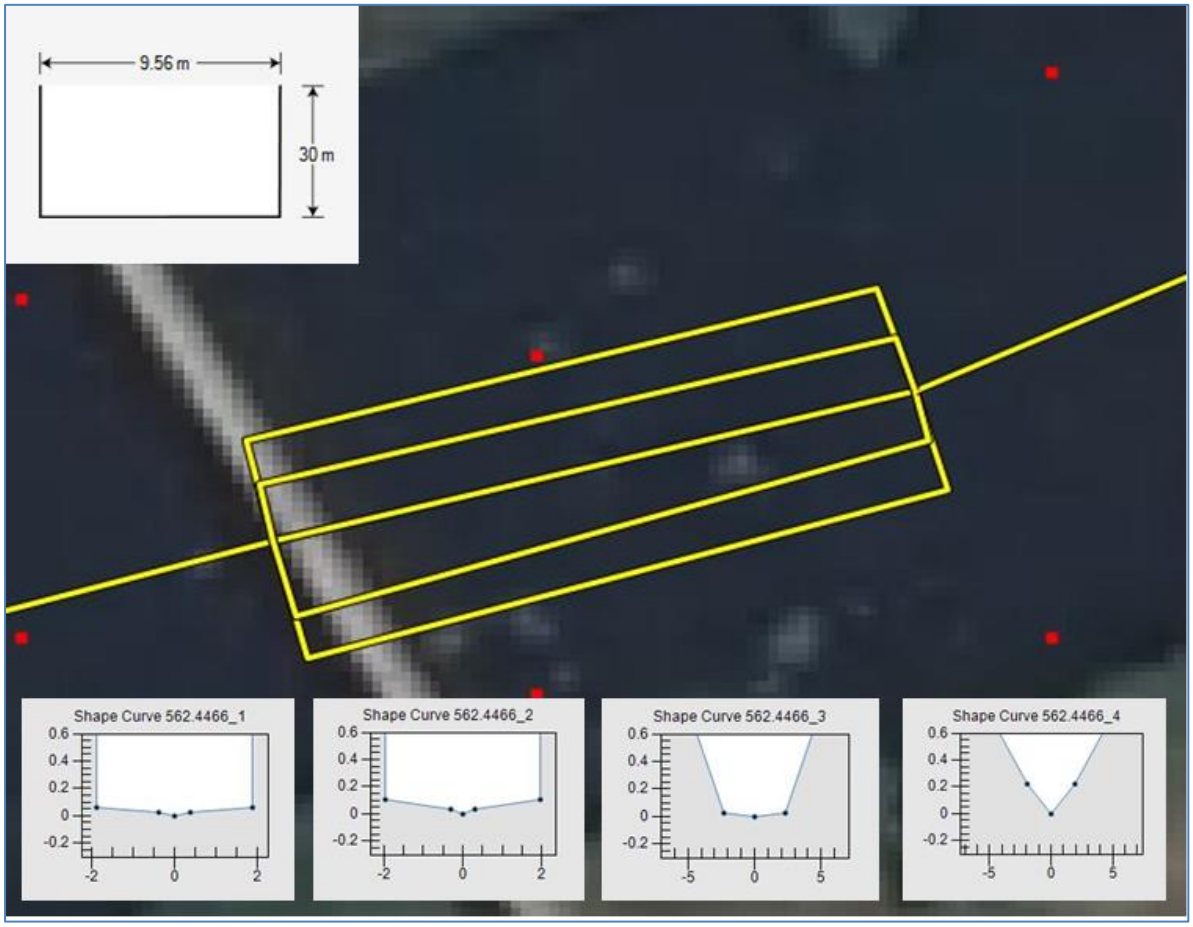


Figure 10: Example of bridge representation in PCSWMM (Same bridge as in Figure 9.)

Assumptions on debris blocking

Since openings are likely to be blocked with debris during a flood, railings, barriers and balustrades were modelled as solid obstacles i.e. assuming that there is no flow through the openings. This is analogous to the situation for guard rails. Any small flow through the openings would be compensated by the energy loss due to the flow expansion and contraction as it passes over the railings.

No debris blocking the openings between piers was assumed, since debris blocking has already been taken into account by assuming no flow through the railings or balustrades and further blocking was assumed to be too conservative.

Impact of PCSWMM imports from HEC-RAS

Cross-sections and bridges were then imported from HEC-RAS into PCSWMM. The representation of the openings between bridge piers is changed during import into a symmetrical opening with the same area and wetted perimeter for each water level, as shown in Figure 10, which is the PCSWMM representation – based on hydraulic radius and flow area - of the same bridge shown in Figure 9. It is therefore not possible to see the actual opening shape in the PCSWMM model. For this reason, for later reference the HEC-RAS model geometry is also being provided to the Client in the electronic deliverables of this project.

During the modelling of development scenarios and flood mitigation options it was found that PCSWMM did not correctly import bridge deck levels. The bridge deck cross-sections were imported with the correct levels, but some of the vertical offsets from the lowest point on the upstream cross-section to the bridge deck did not import correctly from HEC-RAS. PCSWMM uses these offsets in the analysis rather than the actual levels. This (as well as various other specific errors as listed in Appendix H) was corrected for the runs of the proposed scenarios and mitigation options (except for the run for the canal and bridge widening), but the existing scenario was not re-run with these corrections. The proposed development scenario was run both with and without these corrections for comparison, and shows that the corrections have little effect.

Impact of initial water levels

For the initial runs for the existing situation as well as the canal widening, the river channels (as well as all conduits) were modelled as dry at the start of the simulation. This is not strictly the case in reality, as there are known permanent water areas along the channel, most notably the wetlands at the junction of the Liesbeek and Black rivers. An improvement for the Task 2 model runs is to set the initial water level throughout the model as equal to the initial downstream tide level (1.72 m for the 1:100 year flood and 1.64 m for other return periods). This initial water level was applied throughout the model wherever the ground level was below this level, which included almost the entire river channel. This revised initial water level is

sufficient to wet all along the river channels. Because the adopted rainfall hyetograph starts at the beginning of the simulation, but only peaks after 12 h, initial runoff allows low points along the river channel to be filled before the peak flow. Comparison with the later models shows that the assumption of initially dry channels does not affect the flood peaks and extents along the Black River. However, along the Zoarvlei only, inflows are largely determined from upstream, and the maximum water level and flood extents there are affected by the initial water level.

Correction at Liesbeek weir for runs F1 to G9

In PCSWMM, flow is lost from the modelling system where the water level exceeds the rim elevation of a node, unless this node is connected to a 2D cell where the water can become overland flow. PCSWMM allows for only one 1D-2D connection per 2D cell, therefore, some of the 1D nodes could not connected to the 2D network where there was already a 1D-2D connection within the same cell. This was the case at the Liesbeek weir, as well as in the adjacent upstream nodes close by that together describe the weir. The node could therefore not be connected, as a limitation of PCSWMM. The rim elevation was correct in terms of the physical circumstances, but the model 'leaked', and the consultant was not aware that PCSWMM had this limitation of not respecting the water balance.

Therefore, water leaking of the numerical model was identified at a late stage (in December 2016, at the review of the Client of the final draft report) at the Liesbeek weir upstream of Observatory Road.

In order to evaluate the effect of this 'leaking', we generated hydrographs for cross-sections upstream and downstream of this weir as shown in Figure 3. The blue line represents flow upstream of the weir, while the red line represents flow downstream of the weir. The resulting loss does indeed appear to be considerable for the 1:50 year flood. The effect will only appear in the higher floods when the rim is exceeded considerably. For these higher floods, with the specific node being so close to confluence, the impact of back water levels from the Black River is considerable, and therefore the mistake was more difficult to notice from the long-sections.

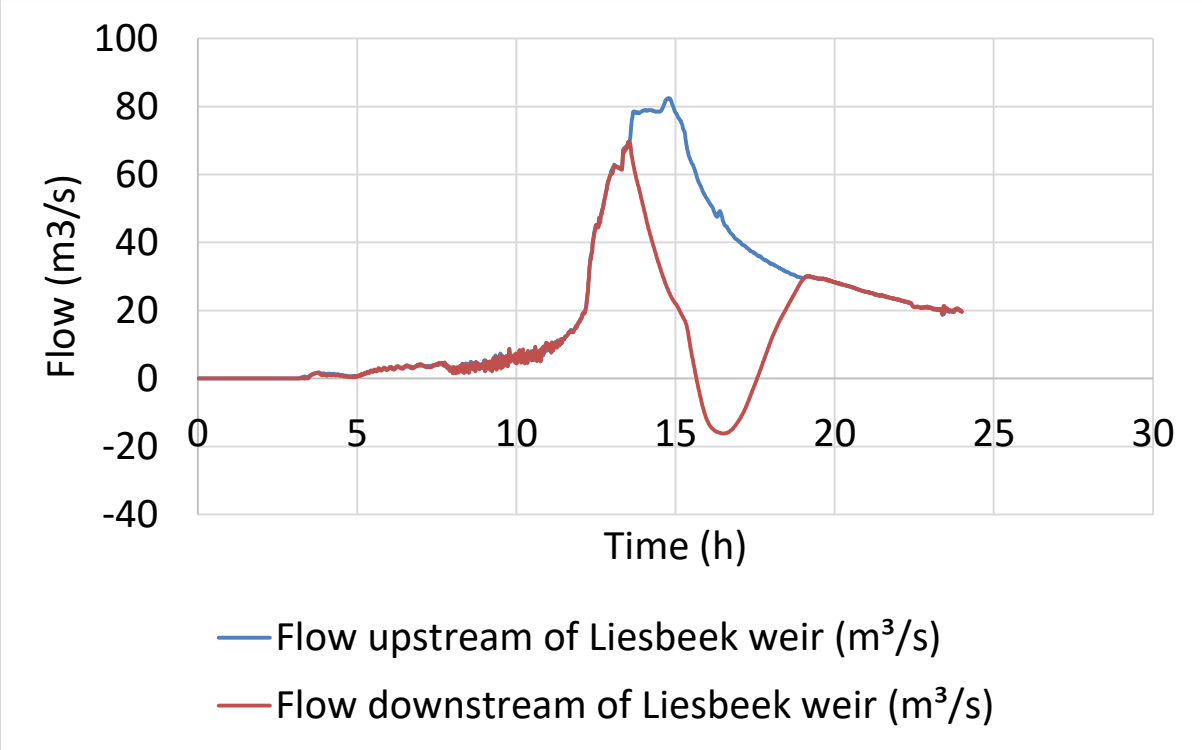


Figure 11: 1:50 year flows upstream and downstream of Liesbeek weir before correction of the loss

The rather artificial solution was to increase the node height so as to raise the rim elevation above the maximum flood level. Two node rim elevations had to be raised at this location: node J843.7469 and node J840.851.

4.6 Bulk stormwater network (culverts and open channels)

Much of the effort of this project was in improving on the bulk stormwater network, in particular in the downstream Paarden Eiland. Although the consultant did not consider this of high importance for advising the TRUP design team, it was a requirement in the terms of reference and therefore executed.

The bulk stormwater network was included in the model as a one-dimensional network. A map of the bulk stormwater network used for models B1 to E9 is shown in Figure 12. The main assumptions for refinement are noted in Table 3. Detailed assumptions on input values that were used at specific locations are included in Appendix F. As noted in Table 3, outside or upstream of the modelling area defined in Figure 4, the stormwater network as modelled by SRK (2012) was accepted as provided.

Table 3: Main assumptions for the modelling of the stormwater network

1	The stormwater network outside of the modelled area will be accepted as modelled by SRK (2012), which was based on the assumptions stated in their report, which generate inflows to the modelled area.
2	Use surveyed levels and conduit sizes where measured at river outfalls.
3	Stormwater conduit sizes and levels in the City of Cape Town Geodatabase will be accepted. Where there appears to be an inconsistency, this has been recorded.
4	Where the Geodatabase indicates several parallel conduits of the same size, these are indicated in PCSWMM as a single conduit with a specified number of barrels. This is necessary, since only a single junction is allowed within each 2D cell. Small conduits in parallel with larger conduits are deemed not to be part of the bulk network, and are excluded.
5	Where areas for which there is no data in the Geodatabase have been modelled by SRK (2012), conduit sizes and levels from the SRK (2012) model will be adopted, except where inconsistencies are noted and assumptions need to be made (as in Appendix F). Sizes from the Geodatabase will, however, take precedence.
6	Open channel cross-sections not previously modelled were extracted from the LiDAR, for parts that are not under water in the LiDAR record, or estimated based on photographs or Google Street view.
7	Any stormwater manholes, headwalls or channel cross-sections are represented as junctions in PCSWMM. Each junction may also be connected to the two-dimensional overland flow network. The rim elevation is the highest elevation of the structure or cross-section, above which it will spill out of the model. For one-dimensional modelling, the rim elevation is generally the top of the structure or channel cross-section. Where the level of the top of the structure was not available, the rim elevations were taken as the greater of adjacent ground levels from the LiDAR or the top of the connecting stormwater pipes or channels, calculated as the invert level plus the depth or diameter. Where a junction is connected to the two-dimensional overland flow network, an arbitrary 30m is added to the elevation to allow for overland flow to be contained within the height of the junction.

8	Where there is no culvert size available, the size of culverts was assumed to be the same as that of the upstream and downstream conduits. Where the upstream and downstream conduit sizes differ, the change in size was assumed to occur downstream of significant inflows. In certain cases, a size in between that of the upstream and downstream conduits was assumed.
9	Where the invert level is missing (as noted by negative or zero values in the Geodatabase), a level was interpolated between available levels upstream and downstream. Where there are levels available only upstream or only downstream, available slopes were approximately projected.
10	Where the manhole rim elevations are incorrect (as can be seen from the fact that the pipe segments/culverts/channels coming into and out of the manhole are clearly larger than the depth of the manhole), the ground level from the LiDAR survey was used. If this was still too low (i.e. depth is still incorrect), then the rim was assumed to be 0.3 m above the highest soffit of all culverts joining into the manhole.

The bulk stormwater networks in the areas indicated by red ellipses in Figure 12 are outside of the Salt River catchment, and were therefore not modelled by SRK (2012).

However, these areas would be connected to the Salt River by overland flows during extreme floods, and were therefore added into the model. These areas, as well as the Zoarvlei, have their own catchments, which were delineated approximately based on stormwater network data and the limited topographic data available in these areas. Catchment parameters affecting runoff were estimated for these catchments.

The most significant changes made to the bulk stormwater network are as follows:

- At Northgate Business Park along Section Road just east of the M5, the SRK model showed a series of pipes with diameters 1.65m, 0.3m and 0.9m from upstream to downstream. The diameter of the middle pipe (conduit 260) was changed from 0.3m to 0.9m to match the downstream pipes. However, it still seems unusual that a series of 1.65m diameter pipes would flow into a series of 0.9m diameter pipes, and it is suspected that the diameters may still be incorrect.
- Sizes of the main outfall from the canal near the N1/R27 interchange to the harbour were taken by assuming similar dimensions to the upstream canal (conduits 2543 to 2547).
- The same canal near the N1/R27 interchange crosses under the N1 freeway a short distance upstream. This opening was only corrected when modelling the scenarios and mitigation options (Task 2). It was intially represented by three separate culverts (1.2x2.1m, 1mø and 1.2mø), but was replaced by 3 off 3.6m x 2.1m, which is similar to the dimensions of the upstream channel, because the

open channel size remains similar near the N1/R27 interchange, and no constriction is visible at the start of the culvert. This correction was not applied to the models of the exisitng situation, or to the 1:100 year model for the canal widening.

- The diameter of conduit No. 2 along Oude Molen Road in Ndabeni was changed from 1.0m to 0.6m, which was the diameter in the GIS shapefiles received from the City of Cape Town.
- For bridges over road or rail, the ground LiDAR usually picks up the ground at the bottom of the bridge opening. However, there were three bridges (N1 over rail at Paarden Eiland, R27 Marine Drive over rail at Lagoon Beach and N1 over the West Coast Busway) where the opening was not picked up. These bridges were therefore represented as rectangular culverts with estimated dimensions. Because the first two of these bridges would allow floodplain flows to pass through towards the sea, a tidal boundary was added at the edge of the sea downstream of these bridges.

Figure 12: Modelled areas of the bulk stormwater network outside the Salt River catchment

Within the model area, the network was checked and refined using available data as indicated in the assumptions mentioned in Table 3. Detailed assumptions on input values that were used at specific locations are included in Appendix F. In reviewing

the TRUP models in January 2017, Aurecon noted that there are discrepancies between conduit sizes used for the TRUP models and a previous study of the Salt River suburb undertaken by Element Consulting Engineers - formerly Wouter Engelbrecht (Pty) Ltd. They also noted *that "conduit sizes and routes in the Maitland area … do not correspond with a previous study of the area undertaken by Africon as part of CCT's 2002 Salt River Catchment Study."* Neither of these two previous studies was made available to the TRUP team, and there may well have had more accurate or complete information on the conduits in their respective areas. This report was finalised without provision by the Client of this information, or a request to get hold of this information.

4.7 Further Corrections to the Bulk Stormwater

Several further corrections were made to the bulk stormwater network after initial runs, and these are described in Appendix H.

In addition, in models G1 to G9, the upstream areas from the SRK (2012) models were removed from the TRUP model and replaced with inflows, giving the revised bulk stormwater network in Figure 13. The derivation of inflows is discussed in section 4.1.

Figure 13: 1D conduits as revised for models G1 to G9 excluding upstream areas

4.8 Two-dimensional modelling of overland flow

Digital elevation model

To represent the elevation of the floodplains, a digital elevation model was created from the LiDAR ground points.

After the first computational runs, comparison with the survey indicated that the LiDAR was generally lower than the corresponding ground survey, and the level of the LiDAR points had to therefore be raised by 0.25 m. The LiDAR levels were therefore raised by 0.25 m throughout the entire model. Since the complete channel section is generally from the survey and not the LiDAR, raising of the LiDAR levels does not affect the channel capacity. At the mouth, where the top of bank was not surveyed, the flow depth is from the surveyed bed level up to the assumed tide level, so the flow area is also not significantly affected by the raising of the LiDAR. It

was later confirmed by the City of Cape Town surveyor that a correction in the order of 0.25m was deemed necessary in other studies in the TRUP area as well.

Since the rivers and open stormwater channels below the top of bank level were included in the one-dimensional model, these needed to be excluded from the two-dimensional model. The ground level in the digital elevation model within the rivers and channels was therefore set equal to the level of the top of bank (on the higher side). The full depth of flooding within the channels is therefore not shown in the 2D results.

A Triangulated Irregular Network (TIN) was first generated from the LiDAR points, and then converted to a 1m x 1m elevation grid for use in PCSWMM. The TIN is necessary to interpolate between points.

Table 4: Comparison of LiDAR and surveyed levels before raising the LiDAR levels by 0.25m

Survey level (m)	LiDAR level (m)	Diffe-rence (m)	Survey level (m)	LiDAR level (m)	Diffe-rence (m)	Survey level (m)	LiDAR level (m)	Diffe-rence (m)
5.313	4.627	0.686	3.661	3.322	0.339	10.781	10.525	0.256
5.312	4.587	0.725	3.888	3.265	0.623	8.851	8.606	0.245
5.214	4.654	0.560	2.498	2.223	0.275	9.025	8.792	0.233
5.219	4.649	0.570	2.842	2.470	0.372	6.641	6.409	0.232
5.293	4.432	0.861	3.060	2.684	0.376	6.423	6.147	0.276
5.309	4.181	1.128	3.773	3.376	0.397	6.106	5.834	0.272
5.363	4.533	0.830	3.043	2.836	0.207	6.199	5.980	0.219
5.033	4.439	0.594	4.481	4.157	0.324	5.506	4.698	0.808
5.020	4.416	0.604	4.709	3.019	1.690	4.854	4.626	0.228
4.456	3.381	1.075	4.520	1.094	3.426	5.094	4.797	0.297
4.464	3.605	0.860	4.413	3.117	1.296	5.202	4.504	0.698
4.384	3.922	0.462	4.748	2.835	1.913	5.880	5.149	0.731
4.378	3.902	0.476	4.820	3.960	0.860	6.033	5.731	0.302
4.452	3.922	0.530	4.782	4.085	0.697	6.545	6.093	0.452
4.505	4.050	0.455	4.712	4.156	0.556	6.950	5.279	1.671
4.429	4.052	0.377	4.717	4.175	0.542	6.660	5.291	1.369
4.499	3.932	0.567	4.619	4.236	0.383	6.656	5.121	1.535
3.985	3.667	0.318	4.623	4.185	0.438	6.924	5.185	1.739
3.932	3.634	0.298	5.146	4.892	0.254	5.697	4.294	1.403
3.852	3.585	0.267	5.222	4.981	0.241	5.777	3.247	2.530
3.679	3.477	0.202	4.998	4.754	0.244	5.686	5.377	0.309
3.744	3.555	0.189	4.912	4.679	0.233	4.394	4.099	0.295
3.503	3.229	0.274	4.974	4.707	0.267	4.585	4.166	0.419
4.296	2.620	1.676	5.151	4.811	0.340	4.486	4.178	0.308
4.262	2.278	1.984	11.102	10.685	0.417	4.146	3.823	0.323
4.276	2.853	1.423	11.348	10.613	0.735			
3.472	3.219	0.253	10.902	10.629	0.273			

The point density was high, but there was some irregularity in the accuracy of the LiDAR compared to ground survey as shown in Table 4. This would have affected the model accuracy, but the errors are generally local, and small compared the coarse mesh resolution. The inaccuracy would therefore not have a great effect on the model results, particularly when compared to the effect of the coarse mesh.

A TIN and 1 m x 1 m elevation grid was also generated including non-ground points, which was useful for understanding the above-ground terrain, but this was not used directly in the model, as the buildings and boundary walls were either modelled as obstructions or the mesh of the model was too coarse for the above-ground terrain obstructions and therefore incorporated as 'roughness' (see below).

A question was raised at the River Study Workgroup meeting of 05 May 2016 whether the openings in the Liesbeek that were recently cut were represented in the LiDAR. These were, in fact picked up in the LiDAR, but in order to better represent the berms in the model, it was decided to:

a) Use a smaller 3 m x 3 m mesh on and around the berms; and
b) To represent the southern break in the berm and the downstream channel as a 1D channel

Area categorisation (roughness)

The two-dimensional model area was divided into polygons representing different land uses, and each was assigned a suitable hydraulic roughness, represented by Manning's *n* values. The values adopted are shown in Appendix E. Buildings and solid boundary walls (except those modelled as obstructions) were accounted for by increasing the roughness value.

- Streets within the model area are paved, although many sidewalks are grassed, and were allocated a Manning's n roughness of 0.2.
- Flow in the rail yards is obstructed by the ballast and grass, as well as the fact that the rails run perpendicular to the flow. A Manning's roughness coefficient of 0.09 was estimated using Cowan's Equation.
- Walled street blocks (in Observatory, Brooklyn and Rugby) were assigned a Manning's n value of 0.8
- The dense industrial areas of Maitland and Paarden Eiland, where many buildings are built adjoining each other, or with narrow gaps between them, were assigned a Manning's n value of 0.8.
- Grassed areas, sports fields and golf courses were assigned a typical Manning's n value of 0.03.
- Reed wetlands were assigned a Manning's n value of 0.2, based on comparison with other South African reed wetland areas.

- The harbour, which is largely paved, but has many obstructions, was assigned a Manning's n roughness of 0.08.
- For Valkenburg and the South African Astronomical Observatory, where there are buildings within large open spaces, a Manning's roughness of 0.05 was estimated using the Hejl (1977) equation as presented by Jarrett (1985).
- For Mowbray, which has mostly walled blocks but has a few openings between houses, a Manning's roughness of 0.5 was assumed.

Connection to external catchments and flows

As mentioned in section 4.6, the stormwater network upstream of the modelled area within the Salt River catchment was included from the SRK models. Street or overland flow was represented in these models as rectangular channels 30 m wide in parallel with the pipe flow beneath. While the pipe network was connected directly to the pipe network within the model area, street or pipe flow was connected to a few of the nearby 2D nodes, to spread the overland flow. The last conduit upstream of the modelled area was duplicated (with a reduced width) as required to connect to several 2D nodes. As pointed out in the review by Aurecon, there are some areas in which SRK (2012) did not provide for overland flow. This would have to be addressed in future stormwater modelling for the whole Salt River Catchment.

Catchments in PCSWMM can only be connected to a single node. Where external catchments were added, these were connected directly to the nearest node of the stormwater network, except along the Zoarvlei. Should the capacity of the stormwater network be exceeded, these nodes would overflow into the 2D network. Since the Zoarvlei was represented only by the 2D network, catchments draining to the Zoarvlei were connected directly to a 2D node. The entry point for distributed catchment inflows to the river network (including the Zoarvlei) was generally taken towards the upstream end of the section of river into which that catchment flows, in order to avoid under-estimating the flow along that section of the river.

Obstructions to overland flow

Bridge decks, sections of the traffic barriers (particularly along the M5), as well as certain solid walls were modelled as flow obstructions as indicated in Figure 14. The figure also indicates the 2D flow area outlined in purple, TRUP boundary in green and obstructions in red.

The 2D nodes and mesh needed to be re-generated or manually edited whenever the obstructions were edited.

Note that the wall around the properties between the PRASA depot and Voortrekker Road was put in as a solid obstruction. However, some conduits still passed through and allowed flow through this wall. Some changes were made between different runs, affecting the flood extents in this area.

Figure 14: Obstructions to overland flow included in the model (2D flow area outlined in purple, TRUP boundary in green and obstructions in red; bridges in red are not full obstructions, as the bridge openings are modelled)

Overland mesh

An overland mesh with a high resolution is more useful for determining flow paths and increases the accuracy. The consultant had to make a choice between a model of a higher resolution and the computational challenges coming with a higher resolution, as a strong PC has difficulties with handling PCSWMM computations and one run took several days to compute with the chosen resolution,

or sometimes stopped halfway. This is partly due to the fact that the model area, including Paarden Eiland, is quite large.

An overland mesh was generated by PCSWMM software as described in section 4.1. Generally, an adaptive mesh was used, which uses hexagons, which may be larger in areas where there is little change in elevation. This was not done for the streets, where a denser hexagonal mesh was used. The resolution of the mesh in each area was as follows:

- Berms East of the Liesbeek River: 3m to a maximum of 10m where the change in elevation is less than 0.1 m
- Streets: 10m
- Above the top of bank of rivers and stormwater channels: 30m
- All other areas: 50m, up to a maximum of 200m where the change in elevation is less than 0.1m

Connection to 1D network

Wherever possible, all 1D nodes within the area of the 2D network were connected to the 2D network. Connections were made using the 'direct connection' approach, which moves the relevant 2D nodes to the nearest 1D node.

However, only one 1D node can be connected to each 2D cell. For the bulk stormwater pipes, parallel conduits were combined into single conduits with multiple barrels but single junctions. These could be connected to the 2D network. Along the rivers, stormwater nodes to be connected to the 2D network were individually selected. This was done to ensure that only one node was selected for connection within each cell, as that is required by PCSWMM.

4.9 Tidal boundary condition

In general, in upstream river areas, there is no influence of the ocean downstream, and flood levels are determined entirely by river flow. In the upstream case, the frequency of occurrence of a certain flood level is the same as the frequency of occurrence of the corresponding river flow. Where the river enters the ocean, the water level is equal to the sea level, and the river flow has no influence on the flood levels. The flood water level is equal to extreme sea levels occurring at a defined frequency. The TRUP area, being so close to the sea, is influenced by the sea level during extreme flood events.

In such an intermediate zone, a flood level can be achieved by many combinations of river flow and sea level, each of which has its own probability of occurrence. While TRUP is not really an estuary anymore, as in normal circumstances this influence does not anymore exist, for flood modelling computations a choice has to be made on a combination of river flow and sea level during floods.

The TRUP site is at a location where flood levels across most of the site are primarily determined by river flow. Therefore, it was decided that, in modelling a flood with a defined probability of occurrence, the river flow with the same probability will be adopted.

The sea level modelled is a combination of the tide level, the storm surge (due to wind setup and atmospheric pressure), the wave setup and the projected sea level rise. Prestedge Retief Dresner Wijnberg (Pty) Ltd (2010) give estimates of storm surge and wave setup corresponding 1:20 year, 1:50 year and 1:100 year frequencies, as well as best and upper estimates for sea level rise in 2035 and in 2060. It was agreed with the City of Cape Town to use sea level estimates presented by Prestedge Retief Dresner Wijnberg (Pty) Ltd (2010).

The tidal component is unrelated to weather conditions, and extreme storms can occur at any tide level. Both the timing of the flood peak relative to the twice daily tidal cycle and the tidal range of that cycle are important.

The preliminary 1:100 year run presented at the workshop on 05 May 2016 used the 2010 sea levels (i.e. without climate change) with the 1:100 year storm surge and wave setup. For the river flows however climate change up to 2060 was already taken into account, as this was the only available scenario in the SRK studies. Following the workshop on 5 May 2016, it was agreed with the CCT stormwater representative to use the following estimates, the values for which are presented in Table 5:

- The 2060 best estimate for sea level rise in view of the long-term nature of the TRUP development;
- The 1:50 year storm surge and wave setup for the 1:100 year flood, and the 1:20 year storm surge and wave setup for more frequent floods;
- The 90th percentile high tide (which is superseded once in 10 times during high tides, but as we were modelling a 1:100 year flood event and the river flows were already 1:100 years, the combination with a tide with a lower frequency would make the probability of exceedance well below 1:100 year); and
- To do a sensitivity analysis on the timing of the tide.

The effect of the timing of the tide would be investigated by modelling both the highest and lowest tide coinciding with the peak inflow to the model area. The decision on 5 May 2016 thus changed the TOR to account for climate change in all outstanding runs, including the base scenario. The timing of the peak river flow relative to the tidal cycle was modelled as agreed, but a sensitivity test proved this did not have much influence on the TRUP area.

The tidal boundaries (the locations were the flooded area interacts with the ocean) were more extensive than originally thought. Therefore, a new computational mesh

was generated and a tidal boundary was also added in the area near the Northern end of the Zoarvlei where there appeared to be some overflow into the ocean. Levels include the 90th percentile high tide plus the best estimate sea level rise for 2060.

Table 5: 90th percentile tidal ranges and tidal levels as per Prestedge Retief Dresner Wijnberg (Pty) Ltd Table 6.1, for 2060

Return period (years)	Low Tide (m above land levelling datum)	High Tide (m above land levelling datum)
20	0.83	2.45
50	0.90	2.53

4.10 Use of water levels rather than energy grade levels

The City of Cape Town floodplain and river corridor management policy states that "All floodlines must be based on the theoretical energy level as opposed to the water surface level," but this was not a requirement of this project, as this project is not meant to generate floodlines but to advise on urban planning. However, this seemed to become a concern for the City of Cape Town in a late stage of the specialist study (written communication November 2016 and December 2016). This section explains why in the case of a 2D PCSWMM model for unsteady state conditions (flows changing over time) used in this specialist study, the use of water levels is more suitable. Normally for floodlines 1D steady state modelling is used, and then the use of energy grade levels (EGLs) makes sense, which may explain the policy. The modelling of this specialist study can be (and will be) used to update the floodlines, but that is an added benefit, not the determinant of the methodology.

The use of energy grade levels is not suited to two-dimensional modelling in general and the use of PCSWMM in particular, for the following reasons:

- In the 2D model, the edge of the floodplain is defined by the boundary between a cell which becomes wet and one which remains dry. In the dry cell, there is by definition no flow, and therefore no velocity. Therefore, the energy grade level is the same as the water level. Therefore, use of the energy grade level cannot make any dry cell become wet and cannot make any difference to the flood extents. Note that although the software does output some maximum water level in the dry cells, this is clearly not correct since theoretically, the water level is undefined.
- Other than the definition above, the only possible estimate for the EGLs in dry cells adjacent to the floodplain edge would be the same EGL as in adjacent wet cells. The first difficulty would be defining which of the adjacent wet cells to use to determine the energy grade level in each dry cell. Once this is decided, it would be possible (although impractical) to calculate the EGL in each wet cell along the boundary. This calculation would need to be done for each time step (because maximum velocity generally occurs when the

cell is filling) and the maximum EGL identified. This would be an impossibly complex procedure with very little benefit.

- As mentioned, velocities reduce towards the edge of the floodplain, and must theoretically tend towards zero towards the edge of the floodplain where the depth tends to zero. This again implies that the water level and EGL are the same at the edge of the floodplain.
- In unsteady hydraulic models (even in 1D models), instabilities first manifest as spikes in the EGLs or velocities. These spikes mean that the maximum of the calculated EGLs is not a realistic estimate.
- PCSWMM is based on momentum equations, and the help-guide implies that the computed EGL, which can be displayed only on longitudinal sections is inaccurate in unsteady state conditions:
  "*The limitations on the plotted quasi-energy grade line (QEGL) are even greater than those for the [water level or Hydraulic Grade Line] HGL. The US EPA SWMM5 program isn't amenable to producing reasonable EGL plots under many conditions. One limitation is that the output results only contains the "average" velocity along a conduit, not the velocities at the upstream/downstream ends which should be used to compute E. The average velocity is the flow divided by an area that is computed from the average depth at the two ends of the conduit. One can usually infer what the depths at the upstream/downstream ends of a conduit should be from the junction depths, except when there is a free drop due to an offset. Another limitation is the computed depth results are computed for nodes, which are not at the same physical location as the computed velocities. A third limitation is that SWMM's dynamic wave flow routing routine is based on maintaining conservation of mass and momentum within a conduit and a continuous depth at junctions. There is no explicit conservation of energy requirement (just like there is no conservation of flow requirement at non-surcharged junctions). An option for plotting a Quasi-EGL has been included in PCSWMM due to user-demand for a way to mimic the results of the simpler steady flow methods that are based on energy conservation, like the culvert equations or the step-backwater calculations (as used in HEC-RAS). This may not be possible for unsteady flows (a fairer test would be to compare results only for steady flows).*" This again proves there are several limitations to PCSWMM as also alluded to in Section 4.1.

It was therefore decided to plot all flood extents based on computed water levels.

4.11 Summary of assumptions

The main modelling assumptions and their effect on flood extents are summarised in Table 6.

Table 6: Summary of assumptions and their effect on flood extents

Increase flood extents	Neutral	Decrease flood extents
Peak rainfall preceded by 12h of heavy rain as per SCS-SA type 1 rainfall distribution	Peak rainfall over entire catchment at the same time	No debris blocking bridge openings or weirs
All sub-catchments have peak rainfall of the same return period	Best estimate runoff parameters	No additional blockage by sediment transported during flood
No flow through barriers and balustrades (solid)	Best estimate channel and floodplain roughness	
Existing "solid" sediment remains during flood	Quazi-1D flow	
Sea level: High storm set up with near spring tide	Low tide coinciding with peak inflow	
Rainfall includes extra 15% for climate change		
Best estimate sea level rise to 2060 included		

5 Model checking

5.1 Stability and software warnings

As an initial check on the results, PCSWMM software calculates separate flow balances for runoff and flood routing. The runoff loss shows the error in the balance of rainfall, runoff, infiltration and catchment depression storage. The stated runoff error for the final models was -0.1%. The routing error is the error in a balance of inflows, outflows at ocean boundaries or nodes within the model and storage for the flow network. The routing error in the final models was 1.2%. These imbalances are acceptable for a model of this extent, bearing in mind the model objective to test the effect of flood mitigation options.

Model instability appears to be during the initial wetting (and drying) of certain floodplain areas and the corresponding culverts, which is difficult for most other 2D hydraulic modelling packages. According to the Dutch Peer Review, PCSWMM is known to be not one of the most stable hydraulic modelling packages. Later in this chapter, this instability is shown, such as in Figure 16 and Figure 18.

The software also presents warning messages for several of the conduits. There are some 1D and 2D conduits where the slope is less than the minimum specified by the software, but these are areas where, according to available information, gradients are indeed flat. The other warning is for 2D conduits where the elevation drop exceeds the culvert length, which is physically impossible. However, this results from the artificial lengths generated by the software for the 2D conduits. The online help of PCSWMM explains this as follows:

"*All 2D junctions are connected to adjacent 2D junctions with 2D links or conduits … The lengths and widths of conduits are adjusted according to a ratio dependent on the number of links connected to the node. This ratio was determined from a large number of tests to give expected wave speeds under a wide range of scenarios*".

In the light of the above, the model setup was accepted.

5.2 Calibration

Aims and possible parameters to adjust

In general, the aim of hydraulic model calibration is use a historic event or historic evidence to improve on (=calibrate) model parameters.

Although some upstream areas from the previous SRK models were included, the intention of this project was to model the TRUP area, as well as downstream areas affecting flow through TRUP as indicated in Figure 4. The intention is not to re-model or calibrate upstream models.

Since the model includes both rainfall-runoff analysis and water level analysis, either flows or water levels could be calibrated if suitable data are available.

In terms of flows, calibration could be used to adapt model parameters to better simulate the downstream hydrograph for given river inflow hydrographs to the TRUP area. Adjustments could be made to:

- Runoff coefficients for local stormwater catchments within or draining into TRUP; and
- Channel and floodplain roughness, which would affect the attenuation of flows within the area.

In terms of levels, channel and floodplain roughness could be adjusted to give known water levels for a measured flow hydrograph.

Assessment of flow data

Out of the available flow data described in section 3.2, flood events were investigated for possible use in model calibration. From newspaper cuttings / internet, recent flooding of the River Club area has occurred on August 2004, 6 and 7 July 2012 and 30 August 2013. From information received from PRASA on another project, the Salt River Depot of PRASA, which is at a higher elevation than the River Club, experienced flooding problems on 9 August 2009 and 12 April 2012.

For the flood events of 12 April 2012, 6 and7 July 2012 and 30 August 2013, no flows were recorded at the Salt River gauge, as indicated by zero values throughout the flow record. The flood of 9 August 2009 was completely outside of the time period for which data were provided.

Records for the flood of August 2004 included the Salt River (at Glamis, downstream of TRUP), the Liesbeek River (at Durban Road, upstream of TRUP) and the Elsieskraal River (at Pinelands). However, no record was available for the Black River, which is the tributary with the largest catchment area. (As noted in Table 1, flows in the Black River at Sybrand Park are available for two days in October 2004 only.) This introduces an uncertainty which is compounded by the unknown inflows between the gauges on the Liesbeek and Elsieskraal rivers and TRUP.

Figure 21 shows that in flood situations, inflows from stormwater outlets within TRUP are very small in comparison to river inflows to the area. Furthermore, the predicted timing of the local inflows relative to the peak river flow means that local inflows add only 3.8% to the flood peak. As this change is less than the uncertainty of the unmeasured flow from the Black River and other catchments between the flow gauges and TRUP, calibration based on those flows which are known flows would be meaningless. It was therefore concluded that no calibration for flows was possible.

Assessment of water level data

In order to calibrate using water levels, predicted and measured water levels should be compared for a known flow. The only information on historical water levels within the model area is:

- The known flooding of the River Club and PRASA sites mentioned above; and
- Gauge records for the Salt River flow gauge at Glamis already mentioned.

Model predictions do indeed show extensive flooding of the River Club. The PRASA site is shown to be just at the edge of the 1:100 year floodplain, and local stormwater runoff likely contributes to flooding of the site when river levels are high. As mentioned previously, flow data are not available for any of these events to be able to do calibration.

Available records for the Salt River gauge at Glamis are in terms of flow rather than water level, but water levels could be back-calculated using the gauge rating curve. The levels so calculated should be the original recorded levels, although some inaccuracy would be introduced through the process. The alignment of the Salt River canal, near the Glamis Close monitoring point, was modified when the M5 / Koeberg Road intersection was upgraded in around 2010. It is unclear whether this had any effect on the gauge rating curve.

It would be possible to compare water levels derived from the TRUP model at the gauge position with measured water levels for the gauged flows. However, this would essentially be comparing the TRUP model with the model used to derive the rating curve, since flows from the rating curve model would be fed into the TRUP model to get back to the water levels. This was not seen a useful means of calibration, since the TRUP model already includes recent survey and floodplain details which were absent from the rating curve model and is therefore already expected to be the more accurate of the two models.

It was therefore concluded that no calibration could be done. While calibration would improve model confidence, we believe that the un-calibrated model provides a sufficient basis for comparing different flood mitigation options and assessing a level of impact of these flood mitigation options, which is the main purpose of this study. A sensitivity analysis could be done on the calibration parameters, within a realistic range, on whether the influence would be large on flood levels and extents and flood hazard (in which velocity and depth and duration also play a role) to find out if calibration could influence planning decisions. However this is beyond the scope of this study. Also, it is recommended to move to another model for more detailed design of the area. A strong recommendation of this study, repeated in Section 16.4, is to start measuring flows and water levels.

Nonetheless, further information on the August 2004 flood is provided below for information, as it was further studied to assess if calibration would be possible, at the request of the representative of the CCT stormwater branch.

August 2004 flood record

For the August 2004 flood event, recorded flows are plotted in Figure 15. The relative timing of the flood peaks appears to be unrealistic in that the downstream Salt River gauge peaks before both upstream gauges. By the time of the peak flow in the Elsieskraal River, the downstream flow in the Salt River has already reduced to less than the Elsieskraal River peak, which is clearly not possible.

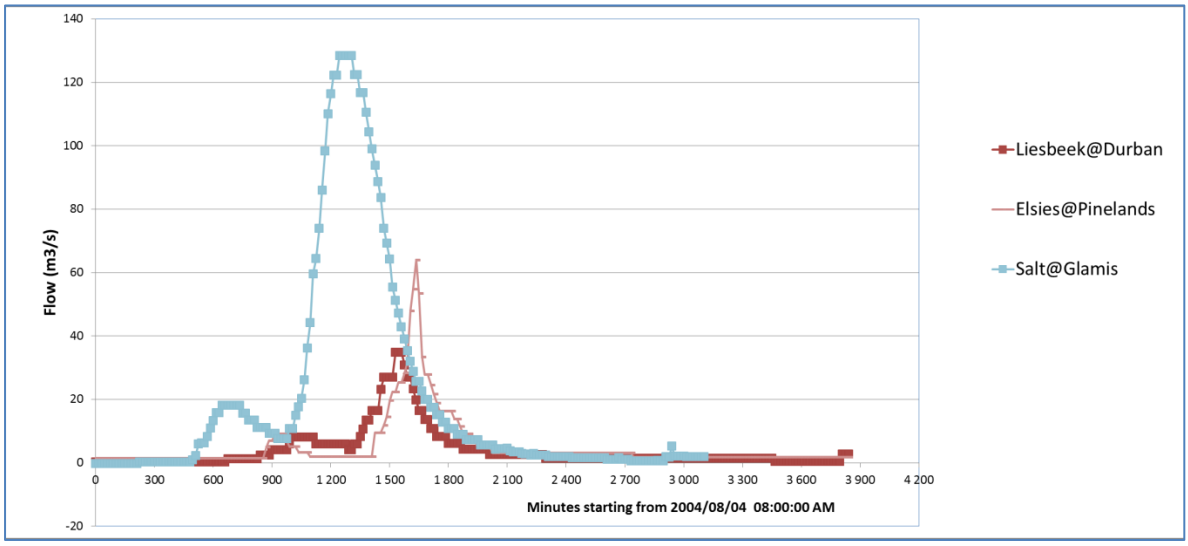


Figure 15: Peak flows recorded for the August 2004 flood

Peak flows and runoff volumes for this flood, as well as catchment areas based on sub-catchments delineated by SRK (2012) are compared in Table 7. Volumes are based on the sum of flows from 08:00 on 04 August 2004 to 24:00 on 06 August 2004, which is the same period plotted in Figure 15. In view of the relative catchment sizes, the increase in peak flow between the Elsieskraal River and the Salt River is more than would be expected, given the likely flood attenuation due to the small slope of the watercourses and wide floodplain.

Table 7: Summary of flow data for the flood of 02 to 05 Aug 2004

Gauge		Liesbeek @Durban	Elsieskraal @Pinelands	Salt @Glamis	Additional catchment between upstream flow gauges and Salt@Glamis
Catchment Area	km^2	22	90	204	91

Peak flow	m^3/s	35	64	128	1.62
Runoff volume	Mm^3	1.00	1.42	3.67	
Runoff volume relative to catchment area	mm	45	16	18	18

Daily and 5-minute interval cumulative rainfall on 5 August 2004 for various rainfall stations in the catchment are shown in Table 8. Pinelands rainfall station is very close to the flow gauge on the Elsieskraal River, so it is expected that this rainfall would contribute both the measured flood in the Elsieskraal River and to the downstream flood in the Salt River. The 5-minute interval data show that there is indeed a higher rainfall event measured in Pinelands compared to other stations, which could explain the higher peak and flood volume in the Salt River, although not the relative timing.

Pinelands is the only rainfall station for which both daily and 5-minute interval rainfall data are available. However, there is a major inconsistency between the two data sets. For Pinelands, the 5-minute dataset gives a total rainfall of 61 mm on 05 August 2004, whereas the daily data gives less than 12 mm.

Table 8: Summary of rainfall data for the flood of Aug 2004

		2004/08/03	2004/08/04	2004/08/05	2004/08/06	2004/08/07
Daily data		**Rainfall (mm)**				
For Black River						
	Athlone	0	10.8	60.6	3.6	24
	Observatory	0	46	40.5	0	0
	Pinelands	0.2	15.4	11.6	5	19.2
	Groenvlei	0	47.4	22.4	9	70.4
For Liesbeek						
	Newlands	1.2	66	36	18.8	90.8
	Cape Town	0	53.3	29.8	0	0
	Kirstenbosch	0.8	102.5	24.5	16	0
5 min interval data						
For Black River		**Total Daily rainfall (mm)**				
	Pinelands			61.2		
	Tygerberg			17.2		
	Dagbreek			20.2		
	Maastricht			14		

5.3 Sensitivity to timing tide relative to inflow

As explained in section 4.9, the effect of the timing of the tide was investigated by modelling both the highest and lowest tide coinciding with the peak inflow to the model area. This test was done using the preliminary models before implementing the corrections in Appendix H, and using a time step of 0.3s instead of the final time

step of 0.2s. The respective hydrographs for outflow from the Salt River canal to the sea for the 1:100 year flood are compared to the tide levels in Figure 16 and Figure 18. Note that the outflows plotted are only those through the Salt River Canal, and not through stormwater outfalls.

The difference in flood extent is as indicated in Figure 17. There is a very marginal increase in flood extent when the peak inflow to the modelled area coincides with the lowest tide, most likely because the travel time from upstream of TRUP to the ocean is of the same order of magnitude as the 6h required for the tide to rise. The extended duration of the peak outflow limits the impact of the timing of the tide, and the timing of the peak outflow is itself influenced by the timing of the tide. Based on these findings, it was decided to time the tide peak at 9 h and 21 h after the start of the simulation, i.e. so that the low tide coincides approximately with the inflow to the modelled area for all further modelling.

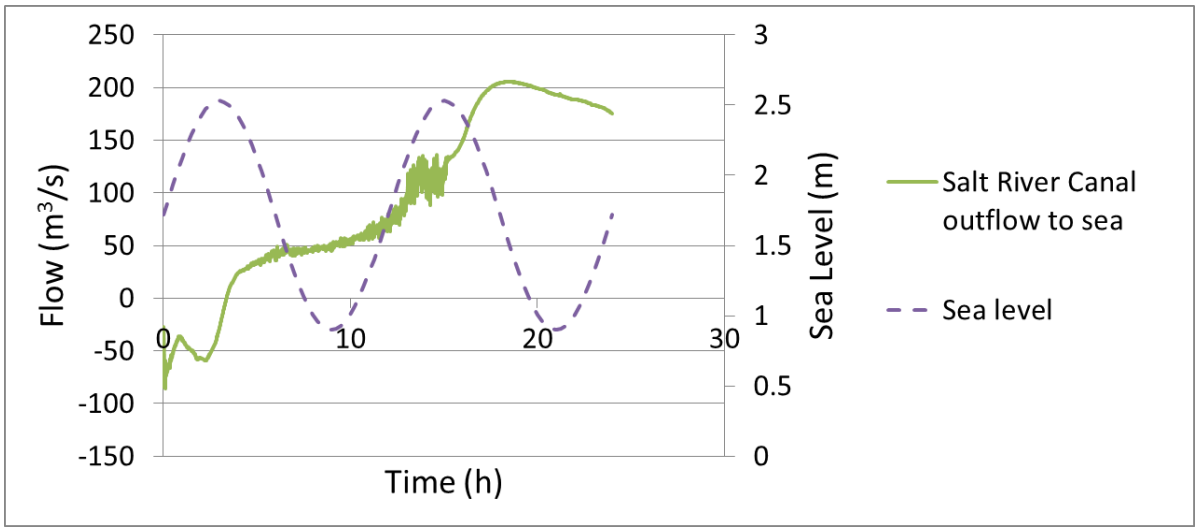


Figure 16: Predicted 1:100 year outflow to sea from the Salt River Canal for highest tide coinciding with peak inflow

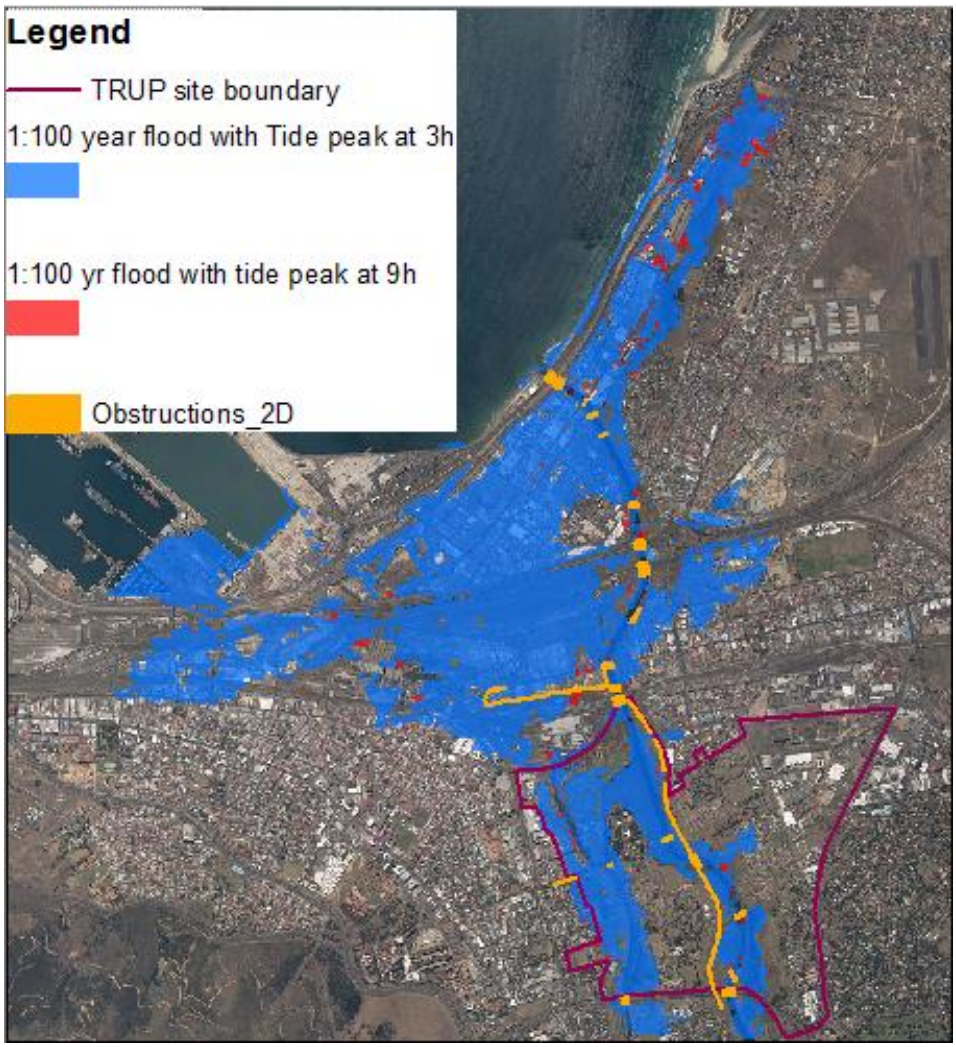


Figure 17: Effect of timing of tide on 1:100 year flood extents indicated in bright red

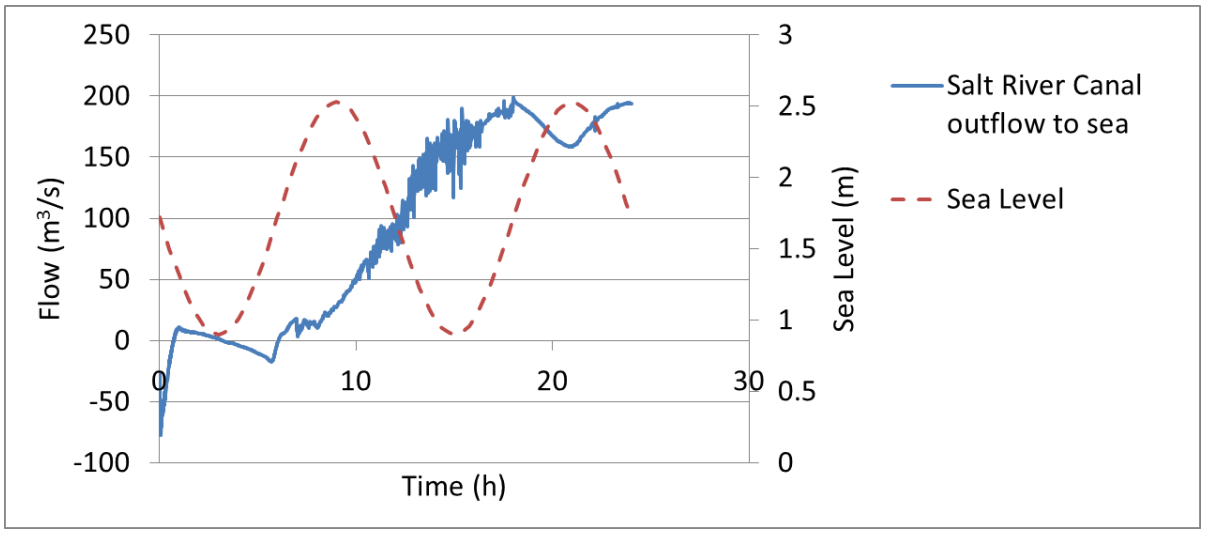


Figure 18: Predicted 1:100 year outflow to sea from the Salt River Canal for lowest tide coinciding with peak inflow

5.4 Sensitivity to upstream cross-sections

During preliminary runs, the Raapenberg Road was backing up floods from the Black River. This was investigated, and it was discovered that higher river bed levels in this area, taken from the cross-sections of SRK's original model, caused the backing up. The first surveyed cross-section was immediately upstream of the N2 Bridge, while the higher bed levels were several metres upstream of this. Knowing that these bed levels had not been surveyed, and seeing no reason why there would be higher bed levels in this area, these bed levels were lowered to an interpolated minimum level between that in the SRK-model further upstream and the surveyed levels adjacent to the N2 bridge. This removed the backing up in this area. This real bed levels should be verified in the field, which was beyond the scope of this contract, therefore is added as a recommendation in Section 16.4.

5.5 Corrections to the model after the initial runs

The original base models were runs B1, B2, B3 and B4 for the 1:100 year, 1:50 year, 1:20 year and 1:10 year floods respectively. As discussed above, certain corrections were made to these models as described above and in Appendix H. The respective 1:100 year, 1:50 year and 1:20 year base models were re-run for all these changes as model runs G1, G5 and G6 respectively. As agreed with the Client, the 1:10 year model was not re-run, and the 1:10 year results include significant errors and are therefore not consistent with the other return periods. Nonetheless, the preliminary 1:10 year results are shown in Maps 5 and 6 in appendix A.

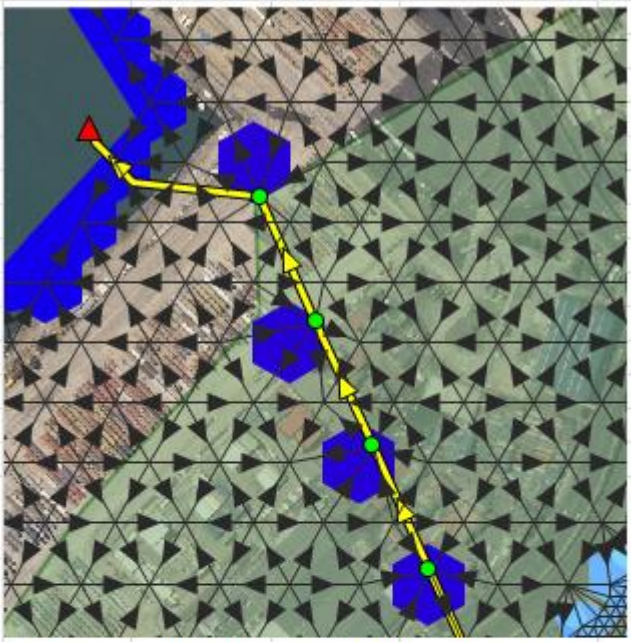
The effect of the corrections on flood extents for the 1:20 and 1:100 year return periods is shown in the maps in Appendix H.

6 Base scenario: Results

6.1 Flood extents

The TOR refers to flood extents as results, not flood depths, or other flood hazards (time of exposure, velocity). Flood extents for the different model runs and comparing different model runs have been added in Appendix A and Appendix H, with Appendix A having the final results and Appendix H showing maps that show the effect of corrections.

An important issue with PCSWMM which must be taken into account in interpreting results maps is that those 2D cells which are directly connected to the 1D bulk stormwater and river network are indicated as wet even when they are dry. This is because the maximum water depth in a cell which is connected to the 1D network is calculated above the invert level of the 1D node. A typical example is the cells along the Woodstock and Salt River stormwater outfall from to the harbour shown in Figure 19. The results are taken from the final 1:100 year model with Scenario 7 development footprint (G7). The four cells connected to the nodes along the bulk stormwater network are shown as wet. However, for all four cells, the maximum water surface elevation is below the cell elevation, so the cells are actually dry. Maps 6, 32 to 37 and 58 only have been redrawn using depths calculated manually by subtracting the cell elevations from the calculated water surface elevations, thereby avoiding this effect. However, this is also not ideal, as isolated low points may be shown as wet.



The four wet cells (from upstream to downstream) have

Name	Maximum Water Surface Elevation	Cell Elevation	Maximum Depth (calculated by PCSWMM from the bottom of the 1D conduit
S38940	2.83	3.406	3.33
S38883	2.76	3.476	3.37
S38833	2.69	3.847	3.41
S38767	2.62	3.376	3.43

Figure 19: Cells at harbour outfall erroneously shown as wet

In **Appendix A**, Predicted flood extents for the 1:10, 1:20, 1:50 and 1:100 years are included. Note that most maps are in pairs – one termed "*Modelled*" showing the entire area within which the model was revised and a larger scale map of *TRUP* only.

Map 1: Modelled flood extents for various return periods – with outline of proposed development - Scenario 7 superimposed (but not used in the model)

Map 2: TRUP flood extents for various return periods – with outline of proposed development – Scenario 7 superimposed (but not used in the model)

Map 3: TRUP flood lines for various return periods

Map 4: Comparison of TRUP 1:100 year flood lines with previous flood lines of SRK (2012)

Map 5: Modelled flood extents for 1:10 year flood (preliminary extents without model corrections described in Appendix H)

Map 6: TRUP flood extents for 1:10 year flood (preliminary extents without model corrections described in Appendix H)

Map 7: Modelled flood extents for 1:20 year flood

Map 8: TRUP flood extents for 1:20 year flood

Map 9: Modelled flood extents for 1:50 year flood

Map 10: TRUP flood extents for 1:50 year flood

Map 11: Modelled flood extents for 1:100 year flood

Map 12: TRUP flood extents for 1:100 year flood

Map 60: Comparison of TRUP 1:50 year flood lines with previous flood lines of SRK (2012)

Map 61: Comparison of TRUP 1:20 year flood lines with previous flood lines of SRK (2012)

In **Appendix H**, the following maps show the effect of the corrections described in Appendix H on the base models:

Map 13: Effect of corrections to base model on modelled 1:20 year flood extents

Map 14: Effect of corrections to base model on TRUP 1:20 year flood extents

Flood extents within TRUP for the 1:50 and 1:100 year floods are similar to those predicted by SRK (2012). However, flood extents for the 1:20 year flood differ significantly in the area west of Liesbeek Parkway, where the TRUP analysis predicts only minimal flooding for this return period, while the SRK modelling overtops to the other side of the Liesbeek Parkway.

Flooding from the bulk stormwater network was not considered by SRK (2012), and has also not been included when drawing the new flood lines. However, flood extents for flooding from the bulk stormwater are shown in the maps of the various flood extents. The following areas are predicted to be flooded due to the capacity of the bulk stormwater network being exceeded:

- Northern parts of Maitland Garden Village during the 1:20 year storm (maps 7 and 8);
- The area around Eastman Road and between Berkley Road and Frere Road, even during the 1:10 year storm (Map 6); and

- Hartleyvale Stadium along Liesbeek Parkway, which was predicted by SRK (2012) to be within the river floodplain for the 1:10 year flood is now predicted to be flooded only by local stormwater during the 1:10 year and 1:20 year floods (Map 6).

Note that in the first two of these areas, the upstream extent of the flooding is beyond the area which was modelled for 2D overland flow. The area modelled was as per the terms of reference only. The flooding along these paths is therefore not shown further upstream.

The open area of the M5 / N2 interchange is mainly predicted to be flooded in the 1:20 year flood (see maps 7 and 8), but not in the smaller floods (i.e. not in the preliminary 1:10 year flood without Appendix H corrections - Maps 5 and 6). This area could therefore be used to provide flood attenuation storage.

Comparison with the development proposals for TRUP (Scenario 7) shows significant development within the floodplain along the Liesbeek River and on the River Club Island. The proposed docking feature and the adjacent building are also within the Black River floodplain. These development proposals were included in later modelling as described in Chapter 7.

According to the 1:20 year 2D model, the river water only leaves the TRUP area through the Salt River Canal. The water flooding the PRASA depot is basically coming from the bulk stormwater network further to the west, except for direct flooding of a narrow strip along the bank of the Liesbeek. However, during the 1:50 and 1:100 year floods, water is predicted to leave TRUP across the PRASA depot as well as along the Salt River Canal. A hydrograph for predicted flow across the PRASA yard from the 1:100 year base model is shown in Figure 20.

It is further noted that the PRASA site has many fine topographic features, including the rails themselves, which would affect flow. This detail is far beyond what would be practical for modelling for the total model area, and would in any case change significantly with the current re-development of the PRASA site. Flooding of the PRASA site is nonetheless indicated in the maps at the model scale. From our work for PRASA at the Salt River depot we have indications that flooding at the PRASA site is also due to limitations on discharge from PRASA's local stormwater network (which has not been modelled) during high river levels. .

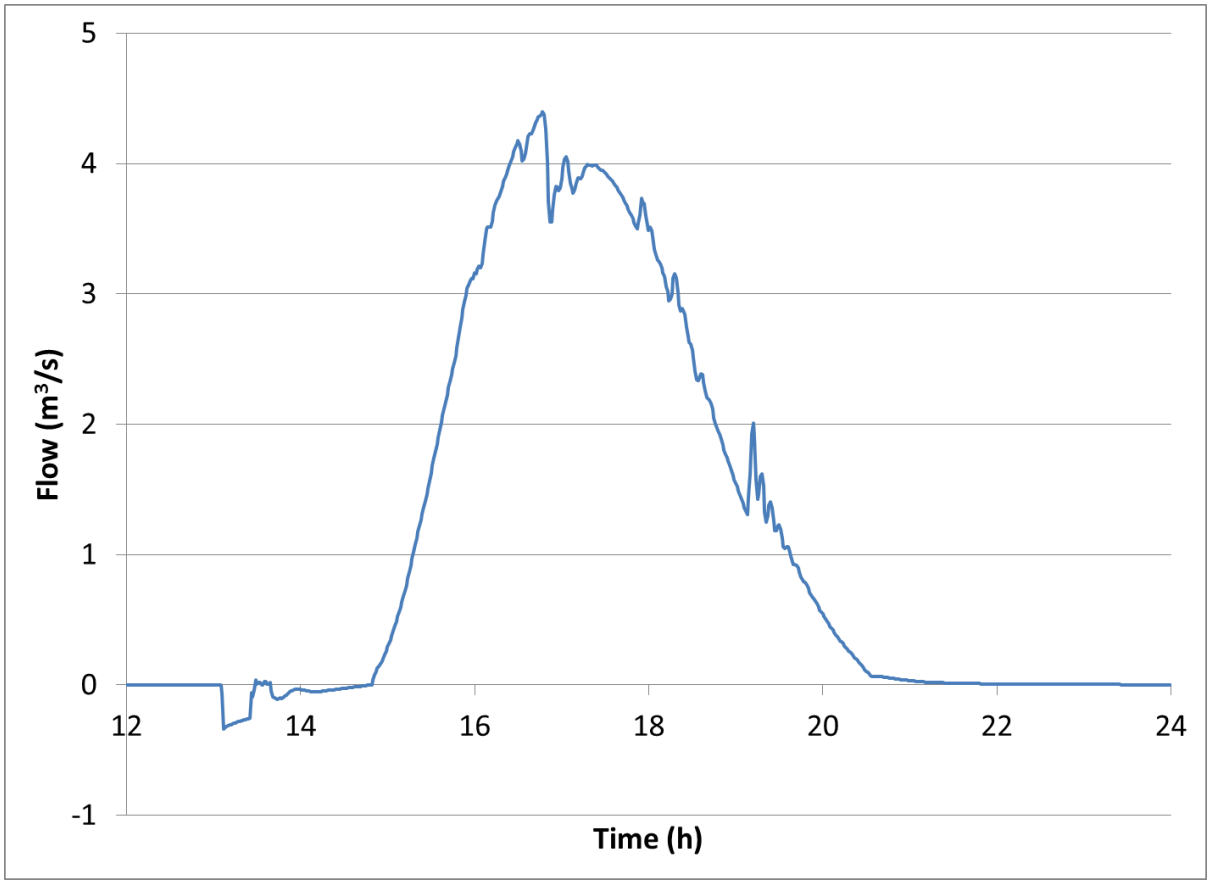


Figure 20: Modelled flow through PRASA yard during the 1:100 year flood (Base model G1 illustrated in maps 11 and 12 in Appendix A)

Modelled 1:100 year hydrographs for inflows to TRUP and outflows from TRUP are shown in Figure 21 and Figure 22 respectively.

The normalised SCS Type 1 rainfall distribution is also shown in Figure 21. As discussed in section 4.2, this rainfall distribution was used for all catchments and return periods, scaled according to the 24-hour rainfall expected in each catchment for each return period. The base distribution is therefore dimensionless. Note that the rainfall in the first four hours, although relatively low, is sufficient to initiate river flow.

Attenuation of the 1:100 year inflows within TRUP is predicted to be 13%, from a peak inflow of 303 m^3/s to a peak outflow of 264m^3/s. (The change in flow across-the TRUP area is a reduction in flood peak of 11%, which is equal to the attenuation less catchment and stormwater inflows.)

Volumes under the 1:100 year hydrographs are compared in Table 9. Outflow volumes are lower than inflow volumes because of water remaining in storage at the end of the 24 h simulation.

Table 9: Total flood volumes under the modelled 1:100 year hydrographs

Flow	Total Volume (thousand m³)
Liesbeek River Inflow to TRUP	2 380
Black River inflow to TRUP	8 201
Stormwater inflows to TRUP and runoff from local catchments	394
Total Inflow to TRUP	**10 974**
Outflow from TRUP along Salt River Canal	9 942
Outflow from TRUP across PRASA yard	47
Total outflow from TRUP	**9 990**

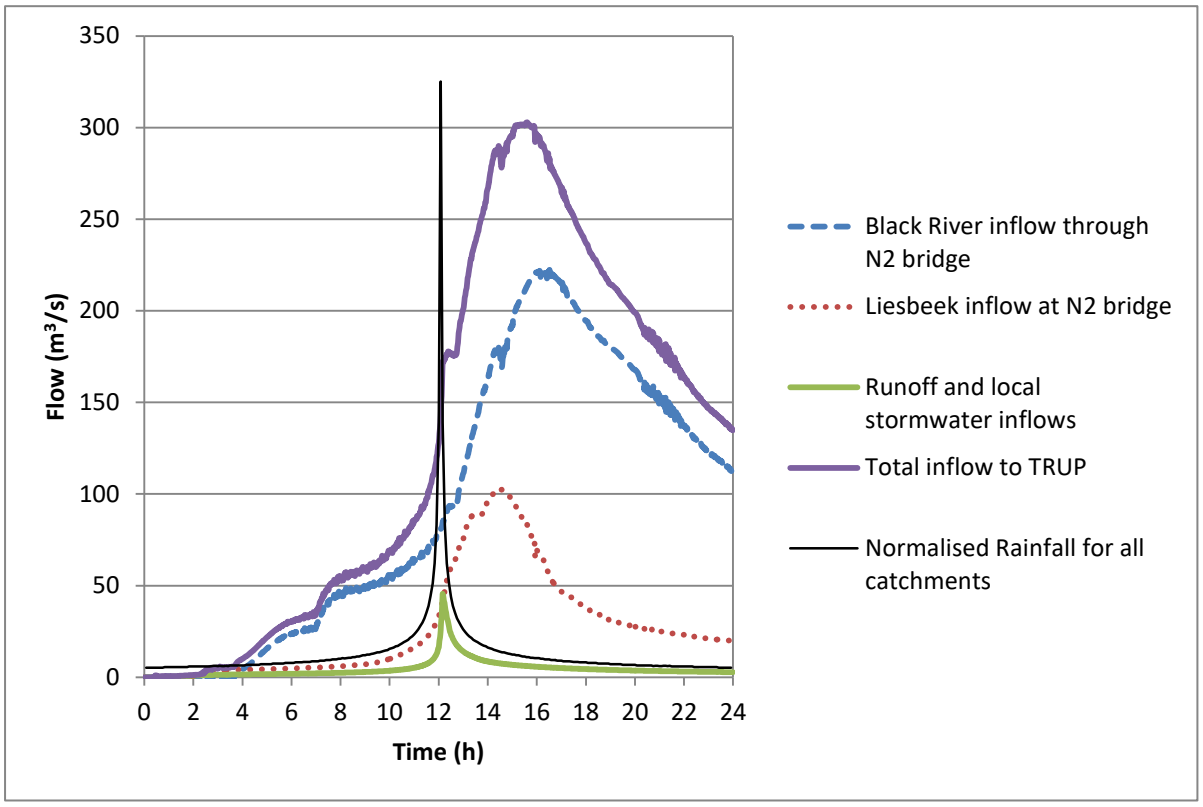


Figure 21: Modelled 1:100 year inflows to TRUP.

The normalised rainfall is dimensionless.

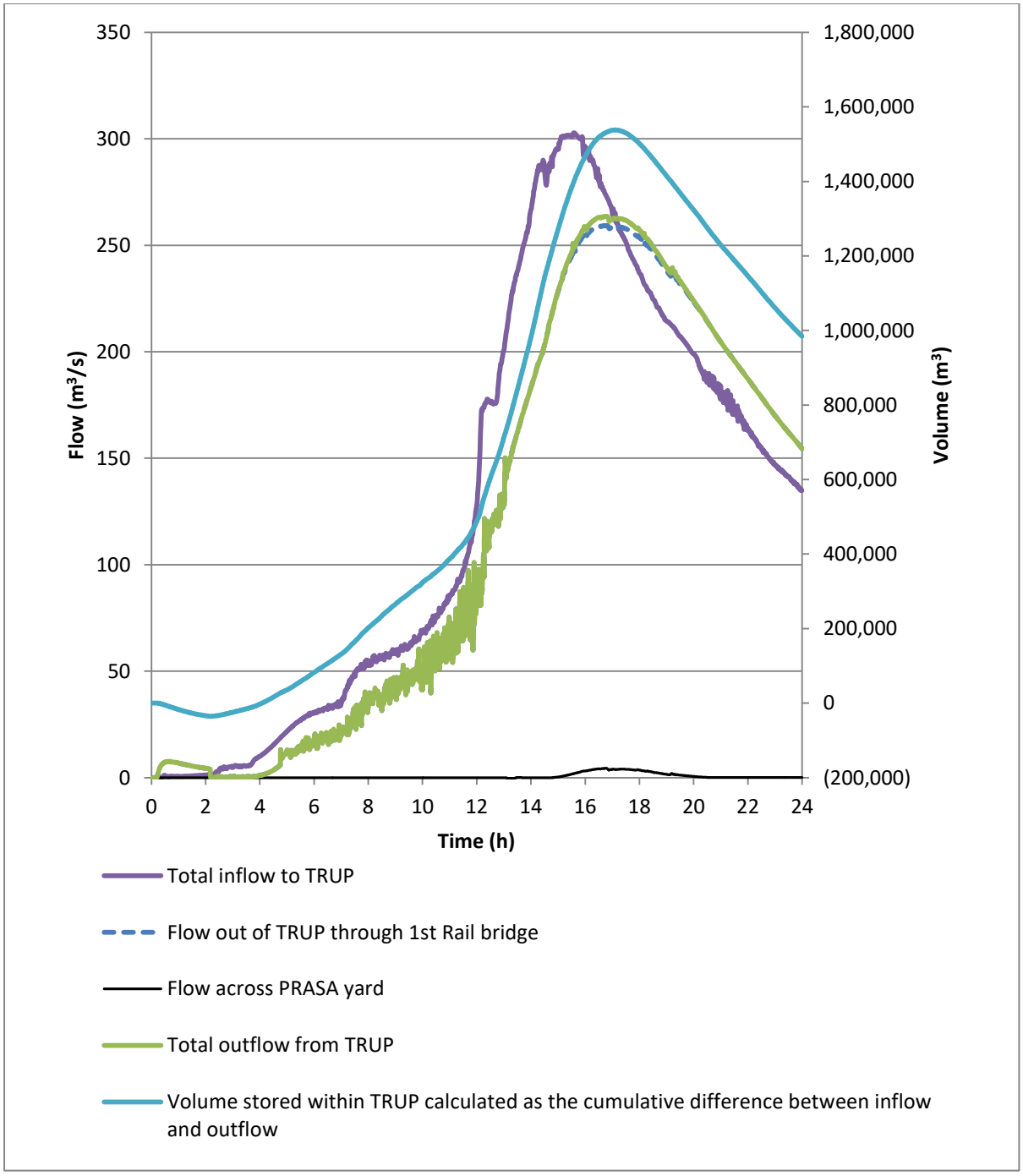


Figure 22: Comparison of inflows to and outflows from TRUP for the 1:100 year base model, and volume temporarily stored in TRUP

PART B: MODELLING EFFECT OF PROPOSED DEVELOPMENT

7 Modelling of proposed development scenarios

As per the terms of reference, "*the service provider will be required to propose, agree and model further development scenarios in the area of interest, as well as flood mitigation interventions*".

7.1 Urban development scenarios

Several proposed development scenarios were considered by the urban planning team.

The latest scenario at the start of flood modelling Task 2 was identified as scenario 6, and this was incorporated in initial development models.

An updated development scenario termed scenario 7 was developed in parallel with the modelling. Scenario 7 includes less development within the floodplain than Scenario 6, except for some additional development on the northern end of the River Club Island.

There has been some discussion regarding development on the River Club Island, which is likely to be one of the first areas to be re-developed within TRUP. While scenario 6 considers development of only a narrow strip of floodline along the western edge of the River Club Island, initial proposals by the property owner consider more extensive development. In order to investigate the effect of maximum development on the River Club Island, it was decided to model the entire property as an area that could not be flooded, in a separate model run.

The layouts of these scenarios are shown in Appendix I.

The proposed development includes buildings, green areas and parking areas. As green areas and parking areas do not form an obstruction to overland flow, only the potential buildable areas were incorporated into the model. These potential buildable areas were added as areas that could not be flooded (as if walls were built at their boundaries, obstructing any flow into these areas). It was agreed not to use any extra space around the development at the river study meeting of 05 May 2016. As per the minutes, "It was agreed that the footprint would just be used as an obstacle and no extra space (or a defined extra m) would be used for further levelling of the ground;" The reasons for this is that the width to be allowed around each developable area will depend on the design, which is yet to be done. Also, depending on the final design, not all developable areas will be solid fill or buildings. There might very well still be spaces within the developable areas which will allow some flow through. This would compensate for blockage by surrounding berms.

Two new bridges are also being considered as part of the proposed development. However, the effect of proposed bridges on flooding would depend on the bridge design, which is still to be determined. From the perspective of flood risk, it would be

recommended that the bridge openings be wide enough and high enough that the bridges do not constrict flood flows.

Table 10: Effect of proposed development on 1:100 year flood levels

Model No.		G1	G2 (b)	G7	G8	G1 vs. G2(b)	G7 vs. G1	G8 vs. G7	G8 vs. G1
Scenario		Base model	Scenario 6 Development	Scenario 7 Development	River club infill plus Scenario 7 Development	Effect of scenario 6 development	Effect of proposed development scenario 7	Effect of River Club infill	Combined effect of River Club infill and Scenario 7 development
Location	Node	9-minute average peak water level (m)				Difference (m)			
Black River									
Downstream of N2	J1789.755	5.05	5.08	5.06	5.10	0.03	0.01	0.03	0.04
Downstream of Valkenburg Bridge	J1253.188	4.89	4.93	4.91	4.95	0.04	0.01	0.04	0.06
Downstream of M5 bridge	J771.3945	4.53	4.57	4.56	4.61	0.04	0.03	0.04	0.07
Liesbeek River									
Downstream of N2	J1681.983	5.16	5.15	5.19	5.16	-0.01	0.04	-0.04	0.00
Downstream of weir near Wild Fig	J1573.841	4.63	4.63	4.60	4.70	0.01	-0.03	0.10	0.07
Downstream end of ponds near Wild Fig	J1279.92	4.53	4.57	4.59	4.65	0.03	0.05	0.06	0.11
Upstream of weir near Observatory Road	J846.0473	4.53	4.56	4.58	4.64	0.03	0.05	0.06	0.11
Downstream of Observatory Road	J807.905	4.53	4.56	4.58	4.64	0.03	0.05	0.06	0.11
Junction with Black River	93285	4.49	4.54	4.53	4.57	0.04	0.03	0.05	0.08
Salt River									
Downstream end of TRUP before rail bridge	4	4.43	4.48	4.47	4.48	0.05	0.04	0.01	0.05
Downstream of rail bridges near PRASA yard	J2353.102	3.95	3.97	3.97	3.97	0.01	0.02	0.00	0.02
Upstream of Section Road Bridge (Bridge 6)	J1143.123	3.22	3.22	3.22	3.23	0.00	0.00	0.00	0.01
Mouth	J56.34234	2.53	2.53	2.53	2.53	0.00	0.00	0.00	0.00

Note that the effect of development on local stormwater runoff is considered negligible in comparison with river flows, and was not modelled.

The maximum effect is along the Liesbeek River, where the obstruction presented by the proposed development causes backing up.

The combination of Scenario 7 development and the River Club infill causes backing up upstream of the River Club of up to 11cm.

7.2 Results: Flood extents

The effect of the proposed development scenarios on modelled flood extents is shown in **Appendix J**, which includes maps 17 to 23, 46 to 50 and 56 to 59.

Map 17 Rev 1 and Map 18 Rev 1 compare the 1:100 year flood extents with and without Scenario 6 development for the entire model area and for TRUP respectively. **Map 17 Rev 2 and Map 18 Rev 2** compare the 1:100 year flood extents with and without Scenario 7 development for the entire model area and for TRUP respectively.

In both cases, the development blocks off some of the flow towards the western side of Liesbeek Parkway, resulting in a decrease in flood extents to the west of Liesbeek Parkway. This effect is more pronounced for Scenario 6 development. In the 1:50 and 1:20 year runs for Scenario 7, as displayed in **Map 21 Rev 2 and Map 22 Rev 2**, the flood extents are the same for with and without Scenario 7 development.

The proposed development leads to an increase in the flood extents at the PRASA yard and between the PRASA yard and Voortrekker Road during the 1:100 year flood (Map 17 Rev 1 and Rev 2 and Map 18 Rev 1 and Rev 2). Flow in this area is increased due to the loss of upstream attenuation within TRUP. This is illustrated in Figure 23. The storage is calculated as the cumulative difference between inflow to and outflow from TRUP. This becomes negative at first as the water making up the assumed initial water level flows out of TRUP.

A useful comparison is the difference in storage within TRUP with and without the proposed Scenario 7 development. The maximum difference in storage volume is 94 000m^3 and occurs after 19 hours and 10 minutes. This is an indication of the storage volume which would be required to compensate for the storage lost due to the development.

There is also an increase in flooding along the stormwater main through Maitland Garden Village where this enters the Black River floodplain, due to a slight increase in water levels along the Black River from the Salt River.

The changes at the extreme western end of the model around the Woodstock bulk stormwater main are probably due to regeneration of the mesh. The maximum flood level here occurs at about 22.5 h, and is due to the combination of tidal backwater (from the tide peak at 21h) and the receding stormwater flow from both the Salt River and Woodstock bulk stormwater mains.

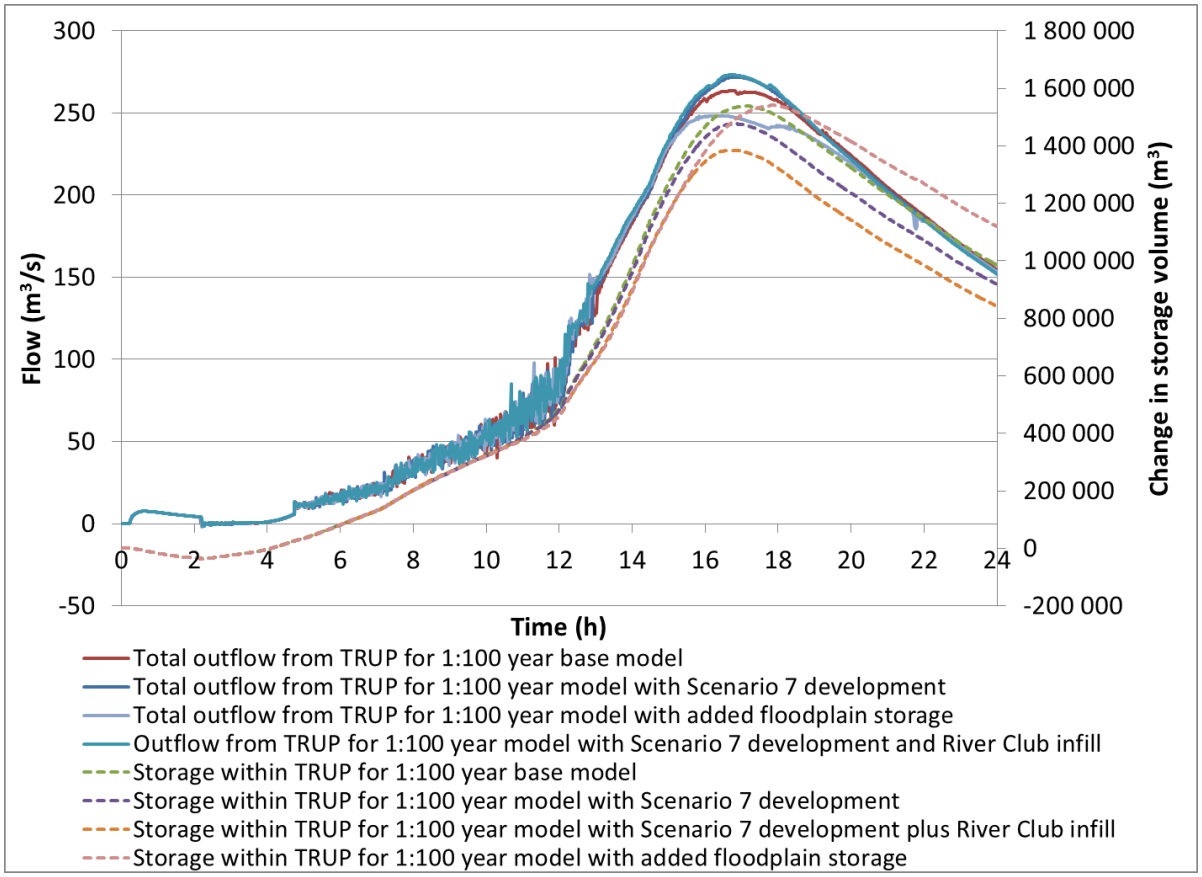


Figure 23: Effect of proposed development on storage within and outflow from TRUP

In **Appendix J:**

Maps 19 to 22 similarly show the effect of Scenario 7 development on modelled 1:50 year and 1:20 year flood extents. The effects are similar to those predicted for the 1:100 year flood.

Maps 56 Rev 2 to 59 show the predicted flood depths with Scenario 7 development for floods of various return periods.

Most of the model runs for Task 2 included only the corrections in Part 1 of Appendix H, and not the more significant effects of the corrections in Part 2 of Appendix H. The flood extents shown in the respective maps in Appendices J and K are therefore preliminary, but nonetheless give a good indication of the effect of the respective development options and mitigation measures. A few of these models were re-run with the Appendix H Part 2 corrections as indicated in Table 15 on page 101.

Map 23 shows the effect of a slightly higher tide (1:50 year setup) on the modelled preliminary 1:20 year flood extents (without Appendix H Part 2 corrections) with Scenario 6 development. As expected, there is a slight increase in flood extents in the downstream area, but no effect within TRUP.

Maps 46 and 47 show the effect of Scenario 6 development on preliminary 1:5 year flood extents (without Appendix H Part 2 corrections) with widening of the Black River channel. The proposed Scenario 6 development again leads to some

additional flooding downstream, although some of these differences are due to a model error which interrupted overland flow from Maitland.

Map 48 shows the additional effect of the River Club infill on 1:100 year flood extents. There is a slight increase in flood extents in the immediate vicinity of the River Club and at the sports fields adjacent to Liesbeek Parkway due to backing up from the River Club infill.

Similarly, **Maps 49 and 50** compare the preliminary flood extents for the 1:5 year flood with and without infill of the River Club Island, in addition to the Scenario 6 development and without the corrections in Appendix H Part 2. The River Club infill again has little impact on flood extents within TRUP. Map 49 shows a few additional cells flooded during the 1:5 year flood in the vicinity of the River Club, along the Liesbeek upstream and in the grassed area south of Salt River Station. The changes in Maitland are due to the fact that overland flow was incorrectly cut off near Van Wyk Street in the model without River Club infill.

Maps 15 and 16 in Appendix H examine the effect of the corrections listed in Appendix H on the modelled 1:100 year flood extent with Scenario 6 development. It compares the flood extent with and without the corrections listed. There is a significant increase in flood levels and extents, mainly because flow is no longer lost at Liesbeek weir. The flood extents in the Zoarvlei are slightly increased due to the initial water level of 1.72m introduced. A similar comparison for the base scenario with and without the corrections for the 1:20 year flood is presented in maps 13 and 14.

PART C: HYDRAULIC EVALUATION OF FLOOD MITIGATION OPTIONS

8 Introduction to flood mitigation options

Flood mitigation concepts are presented in Appendix B, and were discussed in detail during a workshop held on 05 May 2016.

After a presentation by the flood modelling team, each participant was given the opportunity to rate up to five options as preferred for modelling and up to five options as not preferred. Participant ratings are also included in Appendix B, with the number of participants favouring each option in the green circles and the number of participants not preferring each option in the orange circles.

Minutes are included in Appendix C, and give some of the comments made on the various mitigation options. These comments and ratings are from a general project perspective. However, as discussed during a teleconference on 15 September 2016 (minutes of which are also in Appendix C), all options still need to be considered in terms of their flood reduction potential from a hydraulic perspective.

Following the initial ten model runs of Task 2, further discussion was held during the workshop on 02 November 2016. Minutes are also included in Appendix C.

In the following chapters, hydraulically similar options are grouped together and discussed from the perspective of hydraulics. The identified groups of options are:

- Flood protection berms, infill and construction(discussed in chapter 9);
- Channel modification (Chapter 11);
- Flood water storage(Chapter 12); and
- Outflow improvement: (Chapter 13).

Combinations of the various measures will be discussed in Chapter 14.

Lists of all model runs and maps are provided in Chapter 15.

9 Flood protection berms, infill and construction

Existing flood protection dykes or berms in TRUP are shown as red lines in Figure 24.

Figure 24: Existing flood protection berms

Wherever development is proposed within the floodplain, the buildings themselves with either be designed to be resilient to floods (for example, see Figure 25), or they will need to be protected either by constructing these areas of the development on fill or by constructing berms (dykes) around them for flood protection. Hydraulically (and therefore for the modelling in this specialist study), all these options are similar, since they all keep the flood water out of the area. The modelling of the proposed development has already taken this into account by considering the proposed development buildable area boundaries as an obstruction to flow into these buildable areas.

It is noted that the City of Cape Town floodplain and river corridor management policy (2009a) states with regard to filling within the floodplain that "In exceptional circumstances minor 'smoothing' of the 50 /100 year flood line may be considered, provided equivalent compensatory stage storage volume is provided within the development precinct."

Figure 25: Example of flood resistant buildings and buildings on infill, Hamburg harbour district, Germany

10 Reducing catchment inflows

If peak flows and volumes entering TRUP from upstream (along the Black River, the Liesbeek or both) can be reduced, flooding within TRUP (as well as downstream of TRUP) will correspondingly be reduced.

10.1 Catchment stormwater harvesting, detention and infiltration

Ninham Shand (2004) identified a number of potential detention areas in each sub-catchment. Their report should be consulted for before further consideration of specific measures, the consideration of which is beyond the scope of this study that focuses on the TRUP area itself. Potential storage volumes or peak flow reduction were, however, not quantified by Ninham Shand (2004).

A rough indication of what might be achievable in the Liesbeek is given by Lloyd Norman Fisher-Jeffes in his PhD (Fisher-Jeffes, 2015). He suggests that for the Liesbeek catchment, stormwater harvesting could reduce the peak runoff from urbanised parts of the catchment for most 1:20 year storm events by between 10% and 30%.

On the basis of this information, it was agreed with the representative of the stormwater branch of CCT to consider the effect of catchment flow reduction measures by modelling a reduction in the 1:100 year peak flow in both the Liesbeek and Black rivers by 15%, giving the flood peaks given in Table 11.

Aside from the peak flow, the effects of various measures on the hydrographs differs.

In reality, in the case of stormwater harvesting, the flow volume is completely removed from the system, while in the case of detention, the flow is delayed so that it no longer contributes to the peak, but is still released later. For infiltration, part of the flow will still eventually reach the river, but also does not contribute to the flood hydrograph. For simiplicity for the modelling, it was decided to just truncate the hydrograph peak by 15%.

10.2 Results

The effect of the flow reduction on preliminary flood extents is shown in **Maps 18 and 19** for the entire modelled area and TRUP respectively. Note that both the models with and without the flood reduction used for this comparison are preliminary models with Scenario 6 development footprint and with the corrections of Appendix H Part 1 only. The only area in which a more noticeable reduction in flood extent is predicted is along the Liesbeek upstream of the N2. The reduction in flood extents within and downstream of TRUP is very marginal.

An indication of the effect of other reductions can be gained by comparing the flood extents for other different return periods (shown on maps 1 to 4 in Appendix A) with the corresponding flows indicated in Table 11. A 15% reduction of the 1:100 year flood peak is similar to reducing the 1:100 year flood peak to the the peak of the 1:50 year flood. Therefore, the final 1:50 year flood extents give a reasonable approximation to the effect of this reduction. Because of the availability of this

comparison and because there was in any case very little effect, it was agreed that it was unnecessary to re-model the reduction with the final model corrections.

In the base scenario, the largest reductions in flood extents are for the 1:20 year and 1:10 year floods. The peaks of these floods in the Black River are respectively 38% and 49% less than the 1:100 year flows(Table 11). Therefore, to reduce the 1:100 year flood extents to the current 1:20 year flood extents ,a peak flow reduction of 38% would be needed on the Black River. This magnitude of reduction is highly unlikely to be achievable with moderate upstream measures, given that the upstream area is also further densifying at the same time.

Table 11: Peak river inflows to TRUP

Flow scenario	Liesbeek inflow peak (m^3/s)	Black River inflow peak (m^3/s)
1:100 year flood(1)	106	224
1:100 year flood(2)	103	233
1:100 year flood reduced by 15% (3)	87	198
1:50 year flood (1)	92	203
1:20 year flood (1)	72	166
1:10 year flood (4)	53	144

(1) Final inflows from re-run SRK models used in models G1, G2b and G3 to G9 and taken approximately 150m upstream of the N2 bridge over the Liesbeek River and approximately 50m upstream of the Raapenberg Road bridge over the Black River

(2) Preliminary model D1 with Scenario 6 development footprint used for flow reduciton estimate

(3) Used in preliminary model D2 for assessment of effect of inflow reduciton.

(4) Preliminary base model B4

10.3 High flow diversion of Elsieskraal River

Another technically possible intervention, which was considered for reducing the inflow to TRUP from the Black River, is the construction of a flood channel to divert flood flows in the Elsieskraal River to the ocean as indicated in Figure 26. The 1:100 year flow hydrograph for the Elsieskraal River at the proposed diversion is compared with the hydrograph at the downstream junctions in Figure 28. All flows shown are those predicted by SRK Consulting (2012). In addition, there is a parallel canal north of the Elsieskraal River draining Goodwood, which joins into the Elsieskraal River further downstream. This canal would have to be crossed by the diversion, and would therefore also form part of the diversion. 1:100 year flood hydrographs predicted at the diversion point by SRK Consulting (2012) are shown in Figure 27. The total predicted 1:100 year flood peak at the proposed diversion point is 119m^3/s.

Figure 26: Possible flood diversion for Elsieskraal River

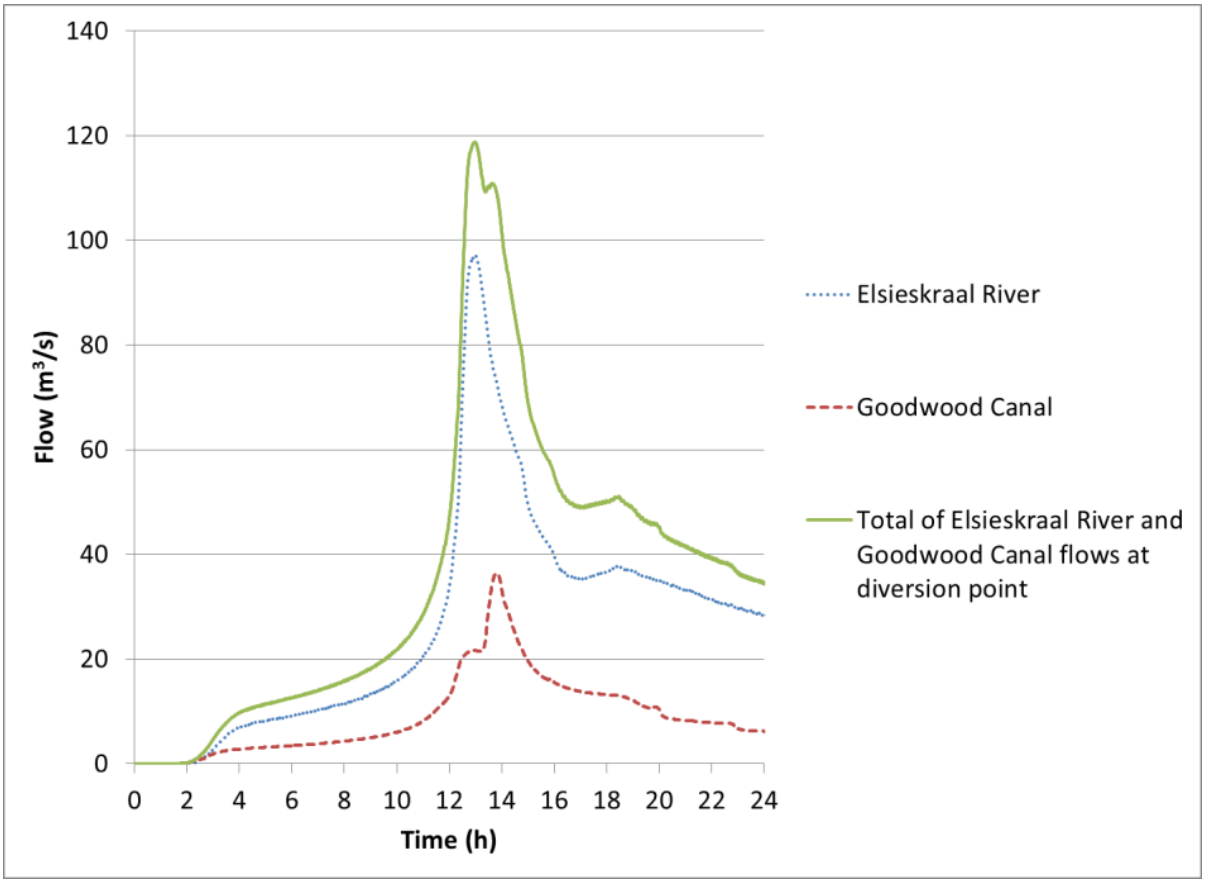


Figure 27: Flows at possible Elsieskraal River diversion point

The effect of the diversion on the flood peak entering TRUP is influenced by the timing of flood peaks across the junctions downstream of the proposed diversion, as

shown in Figure 28. Across the junction of the Elsieskraal River with the Vygekraal River, the Elsieskraal River peaks 31 minutes before the downstream flow in the Vygekraal River. Therefore, only the recession limb of the Elsieskraal peak contributes directly to the downstream peak. Nonetheless, the difference in timing is relatively small. There is also little difference in the timing of flood peaks across the junction between the Vygekraal and Black rivers. Therefore, the effect of the diversion of the Elsieskraal flow on the flood peak in the Black River could be expected to be somewhat less than the diverted peak flow, perhaps of the order of 50 to 100m^3/s. The Elsieskraal diversion could therefore have the potential to reduce the 1:100 year flood peak in the Black River at TRUP to something closer to the current 1:10 year or 1:20 year flood peak. From the comparison in **Maps 2 and 3** in Appendix A, the effect on flood extents along the Black River is expected to be small relative to the large reduction in the flood peak. On the other hand, there is also likely to be a positive effect on the Liesbeek River due to less large backwater effects from the junction of the Liesbeek and Black rivers.

The Elsieskraal diversion was not modelled due to the negative reaction to this option at the Workshop of 05 April 2016 (one positive and ten negative ratings). Nonetheless, it has the potential to significantly reduce flood peaks both within TRUP and in other areas along the Elsieskraal, Vygekraal, Black and Salt rivers.

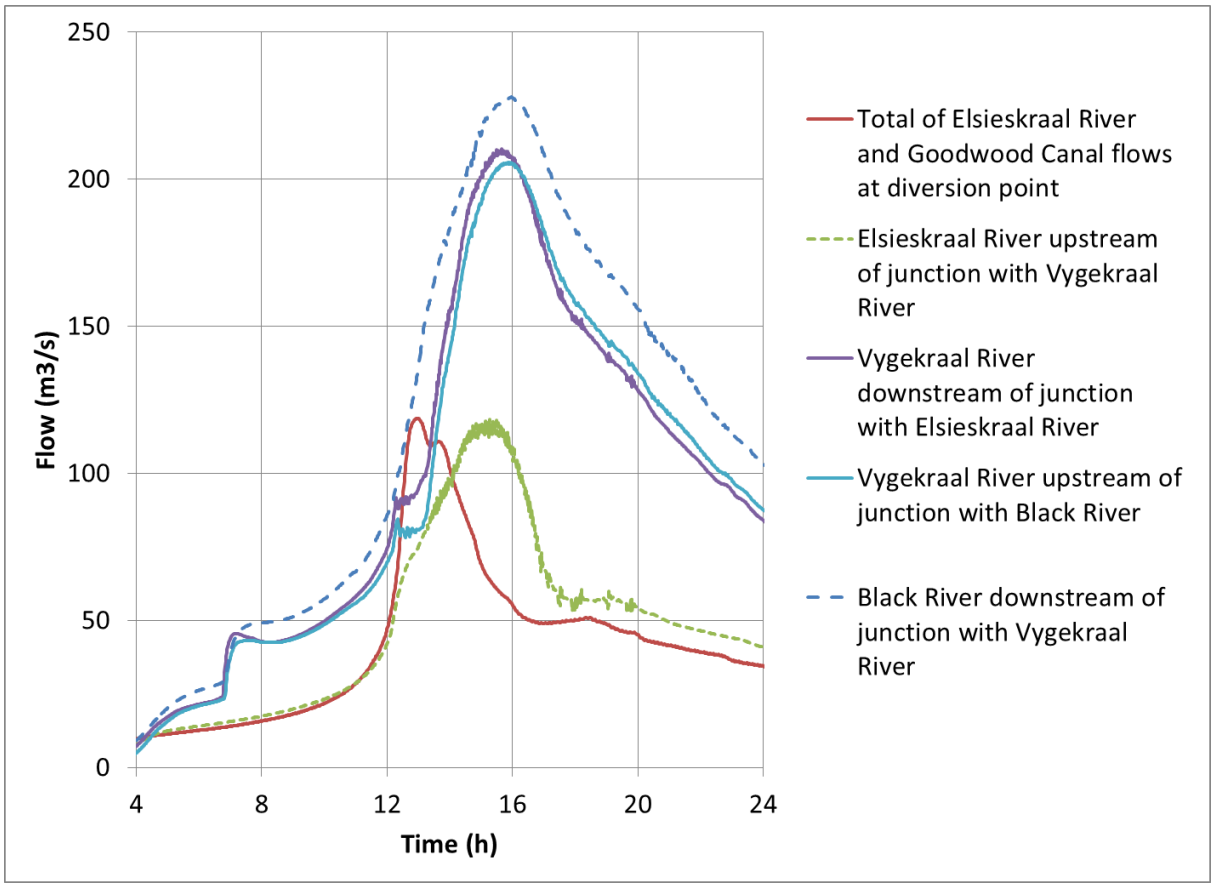


Figure 28: 1:100 year hydrographs predicted by SRK consulting (2012) for the Elsieskraal River and downstream junctions

11 Channel modification

Flow in the Salt, Black and Liesbeek Rivers occurs almost entirely in the channel, with the floodplain providing storage, but with negligible active flow, as is concluded from the velocity maps in the 2D model. Flood levels are controlled by the downstream tide level plus the change in level from downstream to upstream. The following features contribute to the change in level from downstream to upstream:

1. **Changes in the channel cross-section along the length**, particularly sudden changes, which can cause backing up. The rivers in TRUP are already almost uniform channels, without abrupt changes in cross-section.

2. **Bridges and weirs** within and downstream of TRUP are shown in Figure 29 and Figure 30 respectively. These structures will also cause backing up if they obstruct the flow. Generally, the bridge openings within TRUP cover the full width and height of the channel, and therefore cause little backing up. In certain cases, the bridge decks form some obstruction to flow above the top of bank level, while the weirs cause some obstruction to flow along the channel bottom.

   The long-sections for the 1:100 year flood (from the base model G1) are shown for the Black River and Salt River in Figure 31 and for the Liesbeek River in Figure 31. As noted in section 4.10, PCSWMM portrays an approximate quasi-energy grade line. As a result, the energy grade line, which would usually be used to assess the effect of obstructions, is approximate and shows unrealistic spikes. Nonetheless, consideration of the water and energy grade levels on the long-section clearly shows the effect of the bridges and weirs. A significant drop in water level or energy grade can be seen at bridges 6, 7 or 8, 9A, 10, 11, 15, 17, 19, 21 and 23, as well as at the weir near the Wild Fig Restaurant. It is also evident from the long-section that most of the change in water level along the Black and Salt rivers occurs at the structures. This gives an indication of the effect which removal of these structures would have on flood levels immediately upstream. However, the effect reduces with distance from the structure.

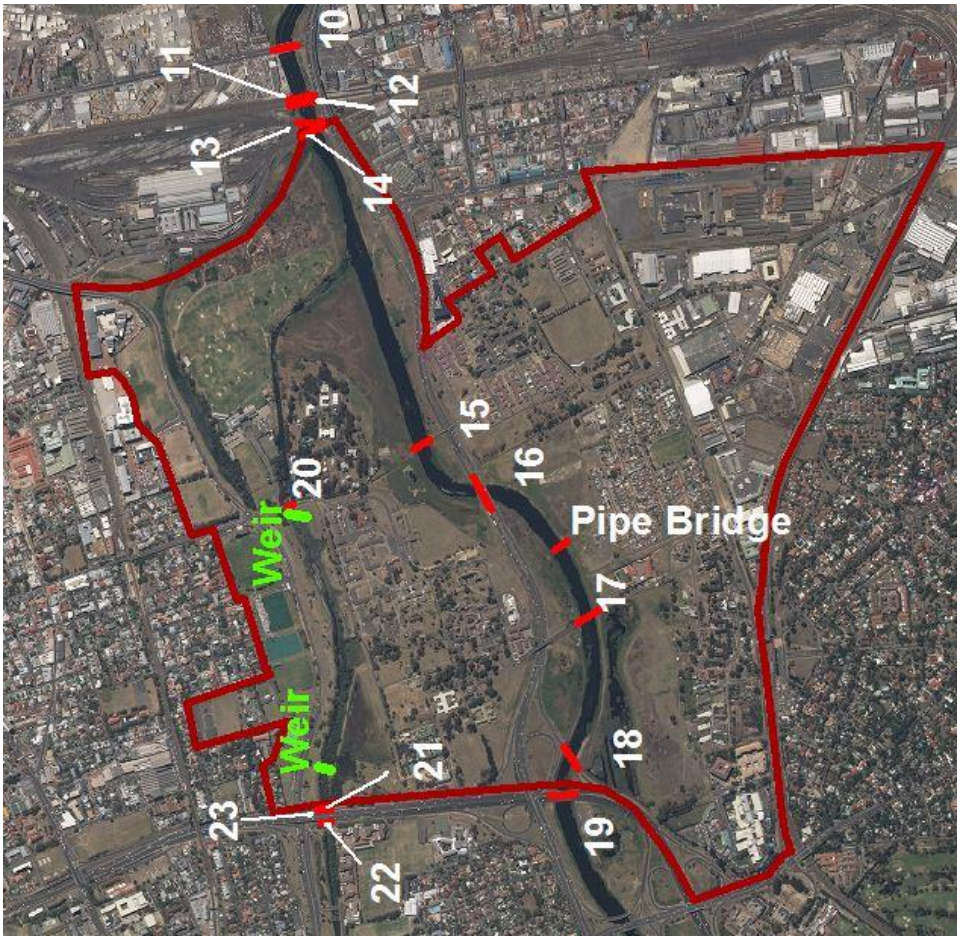


Figure 29: Positions of bridges and weirs within TRUP

Figure 30: Positions of Salt River bridges

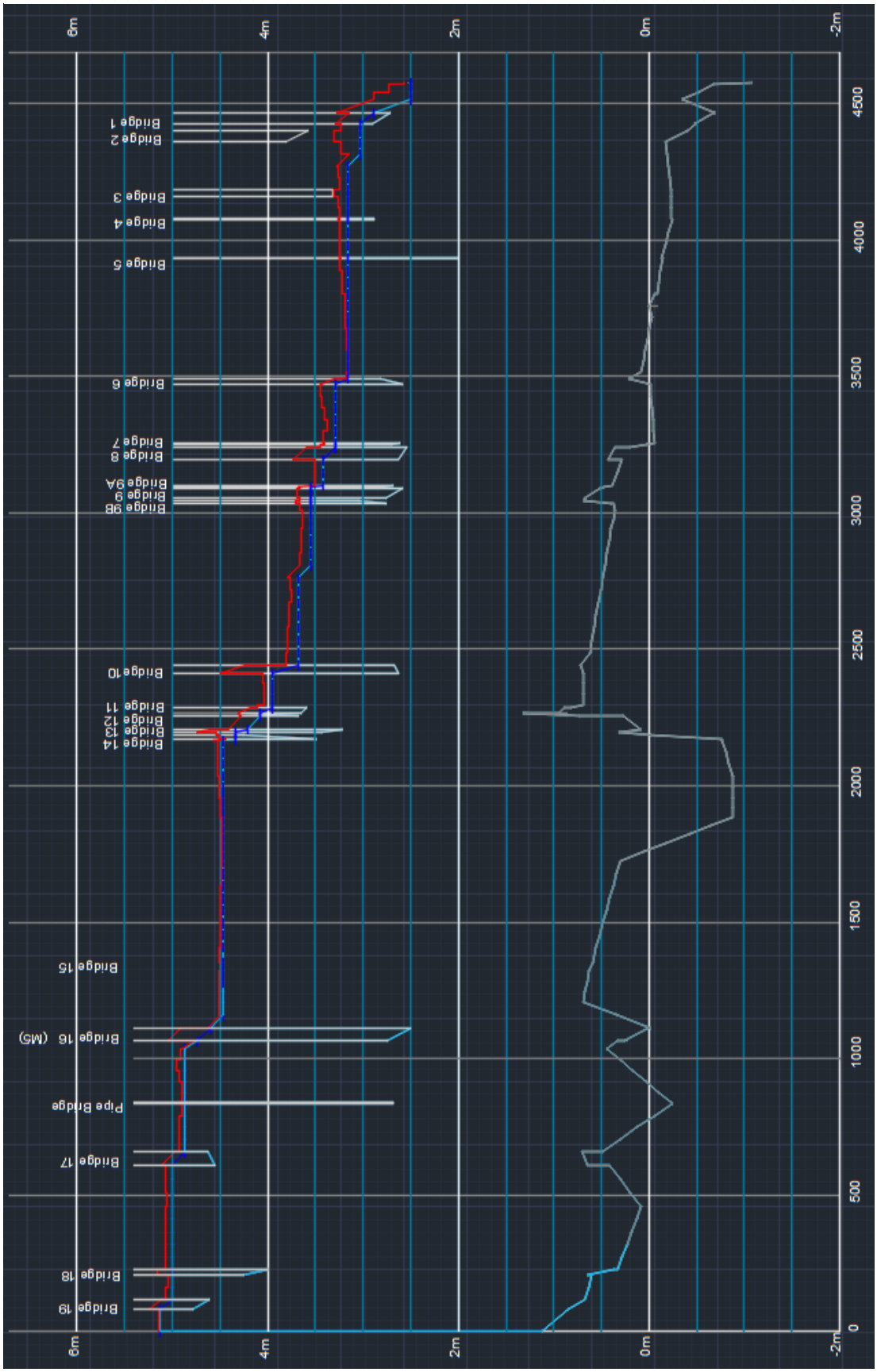


Figure 31: Section along Black River and Salt River canals with proposed development

Maximum predicted 1:100 year water level is shown by the blue line and approximate energy grade is shown by the red line.

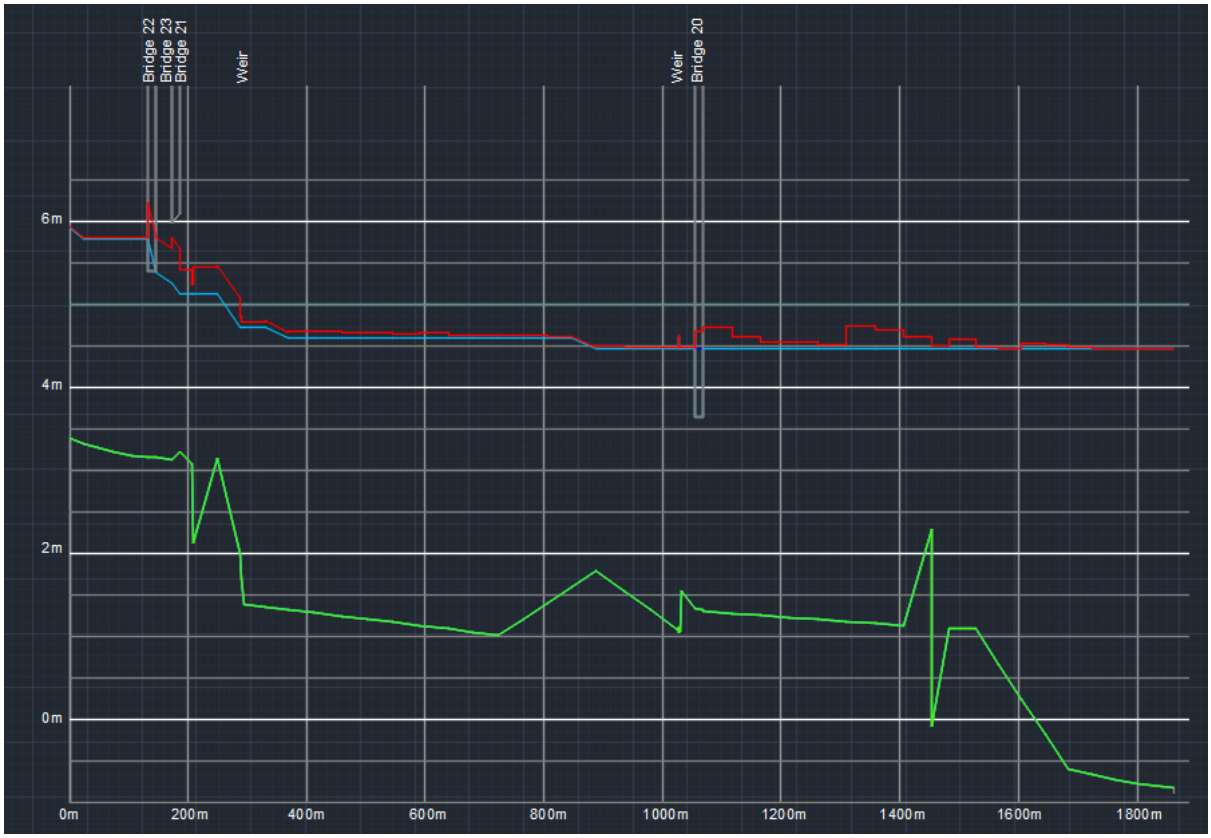


Figure 32: Section along Liesbeek River

Maximum predicted 1:100 year water level is shown by the blue line and approximate energy grade is shown by the red line.

Widening or raising of bridges within and downstream of TRUP was briefly considered. Raising or widening of bridges is costly. In particular rail bridges are very difficult to raise due to the vertical alignment requirements for railways. The effect of most structures on water levels is in any case relatively small, as indicated above. From a practical point of view, bridges cannot easily be removed, as they all form essential links for rail, road and non-motorised transportation.

The weirs are considered to be needed to retain water in the Liesbeek during low flow periods, or would need to be replaced by other flow regulating measures.

The effect of the removal of structures or the raising of bridges was therefore not modelled, as it was considered impractical. The effect of bridge widening will be considered together with channel widening below.

3. **The channel itself**: Flow along the channel naturally follows and also causes a slope from upstream to downstream. This will be considered in more detail below;

Change in level due to flow along the channel is dependent on flow area, channel roughness, the ground area in contact with the flow and the channel length. The measures of more naturalised banks as discussed in the Specialist Study Watercourse management and a Docking Station / Waterfront Feature (De Groen et al., 2017) would all have effect on the parameters discussed below. The discussion below can therefore serve for further design of these more naturalised banks, not as flood mitigation but to reduce the possible effect on flood increases when the designs are done.

(a) Channel roughness, which would include vegetation, obstructions and irregularities: A rougher channel slows flow, which backs up more upstream. The rivers within and downstream of TRUP are already relatively smooth straight channels. Some improvement in flood conveyance could be achieved by straightening, more regular dredging and vegetation removal. A straighter channel is not in accordance with the limitations of the ecological sensitivities and the ambitions of the landscape architects for the site. The CCT dredges when needed downstream of the confluence between Elsieskraal River and Vygekraal River and is not considered needed within TRUP. Although there is some sedimentation noted in the Salt River Canal, the survey did not indicate large scale sedimentation within TRUP. Reduced roughness was therefore not modelled.

(b) The ground area in contact with the moving flow: Contact with the ground also slows the flow and causes higher water levels upstream. Since the rivers in TRUP already have steep sides and the channel slope is very flat, there is relatively little which could be done to reduce the ground area in contact with the flow without also reducing the flow area.

(c) Channel length: A straight channel provides the shortest length, and therefore the lowest flood levels upstream, as the channel will then also have a steeper gradient. All rivers within TRUP are already relatively straight, so little improvement is possible in this regard.

(d) Cross-sectional flow area: More flow area allows more flow through and therefore a lower flood level). There are several ways to increase the flow area in the channel:

(i) Widening of the channel or an additional parallel channel increases the flow area in proportion to the width. For the maximum effect, the bridges and weirs would also need to be widened, since this would reduce the

backing up caused by these structures. In addition if only the channel is widened and not the bridges, the sudden change in flow area upstream and downstream of each bridge would also cause backing up. The greater the widening, the greater the effect on flood levels. This option was modelled, see further discussion below.

(ii) Deepening the channel also provides additional flow area, and could also be considered. The effect would be limited by the fact the downstream bed in the Salt Canal is already higher than the channel bottom at the downstream end of TRUP, and the influence of the sea level in flood situations. The effect is also somewhat counteracted by the additional area of the sloping banks in contact with the flow. The possible extent of deepening is likely to be limited by geology and stability. This option is therefore not recommended.

(iii) Channel benches (Figure 33 and Figure 34) provide additional flow area above the bench while maintaining the narrower channel for low flows. As explained above, the additional flow area reduces flood levels and therefore flood extents. This is part of the design of 'naturalised river banks' and is an option to be further considered.

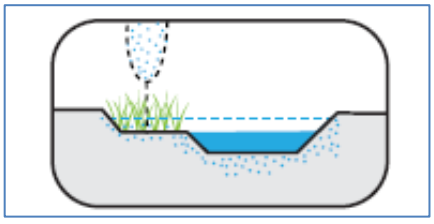


Figure 33: Schematic diagram of channel bench

Figure 34: Example of flood bench in Rotterdam (right) compared with a situation in Indonesia (left) from (picture Greater New Orleans Urban Water Plan, 2013)

Since benches increase the area only through part of the depth, while channel widening is across the full depth, channel widening provides the greater increase in area. Therefore, from a flood modelling perspective it makes sense to test the impact of widening and have other design considerations (behaviour of river in low

flow conditions) come in later, in case a larger cross-sectional flow area seems a promising intervention.

In order to ascertain the potential maximum effect of increasing the cross-sectional flow area, it was decided to model an additional rectangular channel in parallel with the existing Salt River and Black River channels, together with widening the bridges crossing these channels. A widening by 25m (i.e. approximately half the width of the existing Salt River Canal) was agreed with the representative of the CCT stormwater branch, as a realistic maximum. The modelling of an additional rectangular channel would represent the widening well, but was far less cumbersome to realise with the specific PCSWMM software and was therefore implemented.

The Liesbeek Dead Arm could potentially function as an additional parallel channel for the Liesbeek if it were connected to the Liesbeek at the upstream end. However, this would have limited benefit in terms of flood reduction, since, as seen from Figure 32, the water surface profile along the Liesbeek River below Observatory Road is very flat.

The effect of the channel widening on preliminary modelled water levels for the 1:100 year flood is shown in Figure 35. The preliminary flood extents (based on models without the changes described in Part 2 of Appendix H) are compared in **maps 24 to 29** in **Appendix K.** Immediately downstream of the N2 Freeway, there is a predicted reduction of 0.83m in the water level due to the widening. It is clear from both the water levels and the flood extents that the main effect is on the Black River towards the upstream end of TRUP. There is, however, also a predicted reduction in flooding from the Liesbeek at the PRASA site.

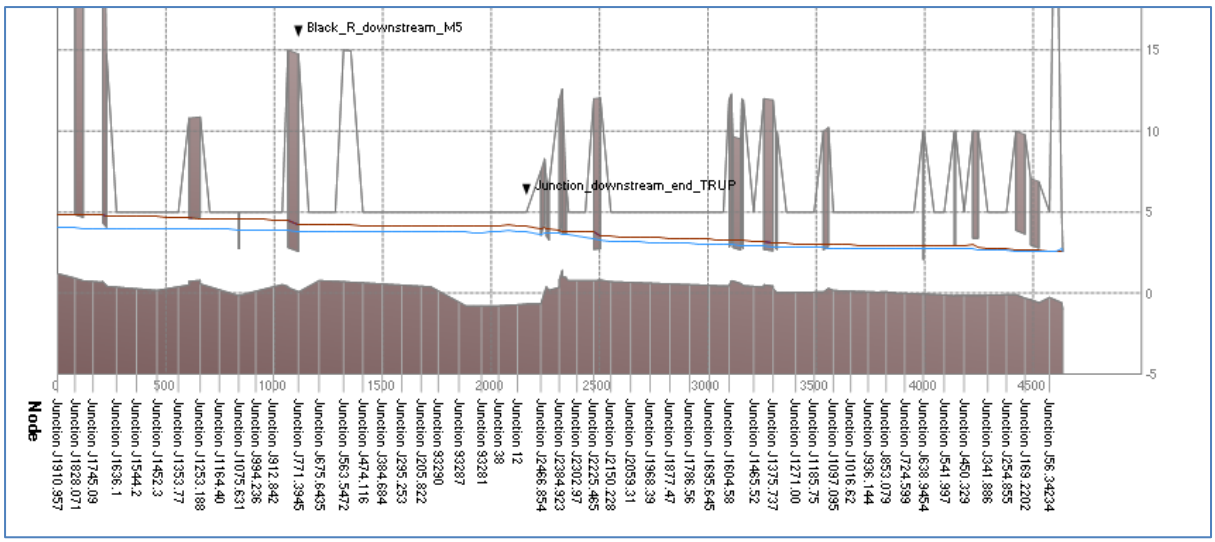


Figure 35: Preliminary Longitudinal Section along Black and Salt Rivers for Base scenario and channel widening
Left end is upstream i.e. southern boundary of TRUP. Right end is the mouth of Salt River canal.
Red line shows maximum water level for 1:100 year base scenario.
Light blue line shows the predicted maximum 1:100 year water level with channel widening.

Because the main reduction in flood extents was towards the upstream end of the Black River, widening or otherwise modifying for increased area only the Black River channel and bridges and not the Salt River Canal was considered. Further 1:100 year and 1:5 year runs were undertaken with a similar 25m widening of the Black River channel only, up to the junction with the Liesbeek River dead arm.

This option is far more practical since:

- It involves the widening of only four bridges;
- Work is within the TRUP area. This means that the shape, material and location of the channel enlargement could be designed to enhance the wider river corridor development in accordance with design objectives;
- Widening within TRUP could be an early visible manifestation of the development of TRUP.
- There would be no need to encroach on existing development close to the existing canal in Salt River, Maitland, Paarden Eiland and Brooklyn.

The effect of this widening on preliminary flood extents is shown in **Maps 38 to 41** in Appendix K. A significant reduction in flood extents is still achieved in similar areas to those achieved with the widening of both the Black and Salt River channels.

Note that energy losses at bridges, weirs and junctions were added to the final models F2 to G9 and not yet implemented in the models for widening the channels (Models D6, D7 and D8 and the base model D1 to which they are compared). Because the widening would also affect these energy losses, the widening would have more effect where these losses are taken into account. Adding the effect of energy losses on water levels to the previous effect of widening gives a potential maximum limit on the effect of widening. The actual effect will be somewhat less

than this potential maximum. The addition of the energy losses, although significant, does not change the order of magnitude of the widening effect.

Table 12: Potential effect of Salt and Black river channel widening on 1:100 year flood levels

Model No.		**F2 vs. F1**	**D6 vs. D1**	**TOTAL**
Location	**Node**	**Effect of addition of energy losses at bridges**	**Effect of widening of Black and Salt rivers[1]**	**Potential maximum effect of widening**
		Change in 1:100 year flood level (m)		
Black River				
Downstream of N2	J1789.755	-0.15	-0.84	-0.99
Downstream of Valkenburg Bridge	J1253.188	-0.16	-0.68	-0.84
Downstream of M5 bridge	J771.3945	-0.13	-0.41	-0.54
Liesbeek River				
Downstream of N2	J1681.983	-0.05	0.00	-0.06
Downstream of weir near Wild Fig	J1573.841	0.09[2]	0.02	0.10[2]
Downstream end of ponds near Wild Fig	J1279.92	-0.11	0.03	-0.08
Upstream of weir near Observatory Road	J846.0473	-0.12	0.00	-0.12
Downstream of Observatory Road	J807.905	-0.12	-0.03	-0.15
Junction with Black River	93285	-0.14	-0.37	-0.51
Salt River				
Downstream end of TRUP before rail bridge	4	-0.18	-0.29	-0.47
Upstream of Section Road Bridge	J1143.123	-0.18	-0.26	-0.44
Mouth	J56.34234	0.00	0.00	0.00

(1) With an additional parallel channel

(2) The unexpected positive values at the weir downstream of the Wild Fig Restaurant appear to be due to instability during overtopping of the existing attenuation pond in that area.

Comparing the widening of the Black River only against the results of the preliminary base scenario for the 1:100 year model, the peak water levels from the end of the widening downstream are slightly increased in the run with the widened Black River (by up to 0.08m), which may be expected given reduced upstream attenuation. However, this small effect is barely visible in **Map 39**.

As for the widening of both the Black and Salt rivers, adding the effect of energy losses on water levels to the previous effect of widening gives a potential maximum limit on the effect of widening. This is shown in Table 13.

Table 13: Potential effect of widening the Black River channel only on 1:100 year flood levels

Model No.		**F2 vs. F1**	**E1 vs. D1**	**TOTAL**
Location	**Node**	**Effect of addition of energy losses at bridges**	**Effect of widening of Black and Salt rivers with an additional parallel channel**	**Potential maximum effect of widening**
		Change in 1:100 year flood level (m)		
Black River				
Downstream of N2	J1789.755	-0.15	-0.53	-0.67
Downstream of Valkenburg Bridge	J1253.188	-0.16	-0.36	-0.52
Downstream of M5 bridge	J771.3945	-0.13	-0.09	-0.22
Liesbeek River				
Downstream of N2	J1681.983	-0.05	-0.01	-0.06
Downstream of weir near Wild Fig	J1573.841	0.09	0.01	0.10
Downstream end of ponds near Wild Fig	J1279.92	-0.11	0.03	-0.09
Upstream of weir near Observatory Road	J846.0473	-0.12	0.00	-0.12
Downstream of Observatory Road	J807.905	-0.12	0.00	-0.13
Junction with Black River	93285	-0.14	-0.01	-0.15
Salt River				
Downstream end of TRUP before rail bridge	4	-0.18	0.08	-0.10
Upstream of Section Road	J1143.123	-0.18	0.00	0.00
Mouth	J56.34234	-0.00	-0.03	-0.21

(1) With and additional parallel channel

The preliminary 1:5 year model results, in contrast, show thanks to the widening a reduction in flood extents and levels, not just in levels. The difference in level is 0.22m at the downstream end of TRUP and 0.51m at the upstream end of TRUP. This

difference occurs when the widening is applied to the preliminary 1:5 year model with (model E9) or without (model E4) the proposed development. The reason for this reduction in flood levels downstream of the widening is not because the peak flows are lower at these points, but because the hydrographs develop differently over time. As can be seen in Figure 36, the flow magnitudes are similar with and without the widening, but are far more stable with the widening. It is therefore thought that the lower water levels may be related to the reduction of the energy loss effect of waves and flow unsteadiness in the downstream reach, which is a model effect not a real flood effect. Unsteadiness is reduced due to the damping effect of the second channel, and a similar effect would be expected with benches. It is not clear to which extent this reduction in water levels is realistic, as it is probably a modelling effect only.

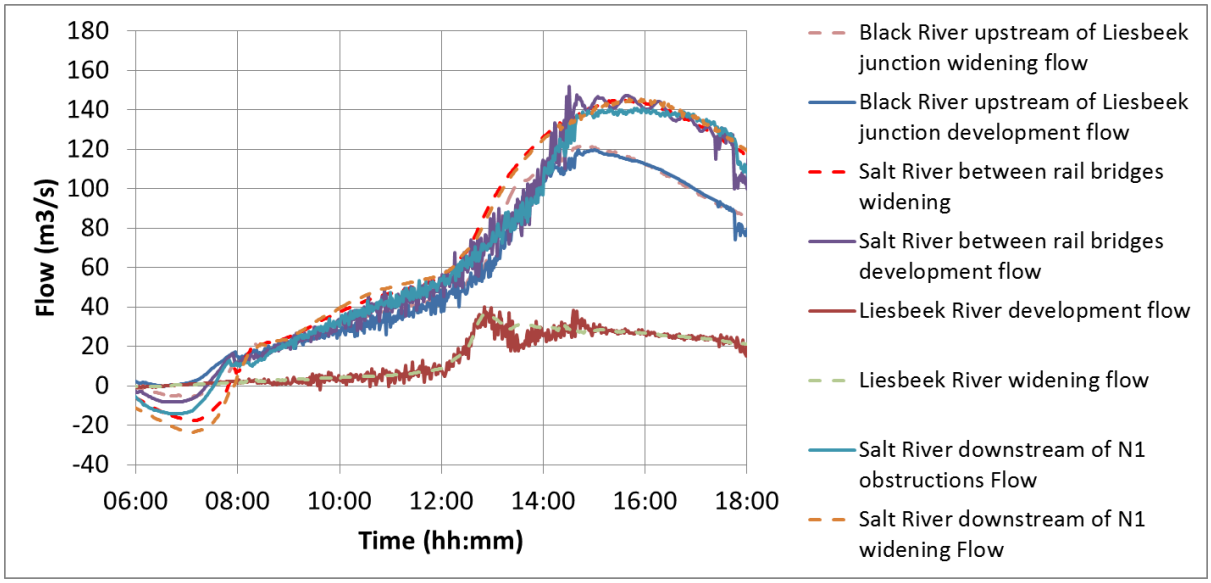


Figure 36: Comparison of preliminary modelled 1:5 year flows at different locations for runs with proposed development (run E2, solid lines) and with Black River channel widening (run E4, dashed lines). Note that both are preliminary runs without the corrections in Appendix H Part 2.

The widening of the Black River channel is thus expected to have a significant effect on flood levels. However, the main reduction in flood extents is in the area adjacent to Oude Molen, where no development is planned in Scenario 7.

12 Flood water storage

Storage of some of the flood water within the floodplain, channel or local stormwater system reduces flows and therefore water levels in the river downstream. Because water levels at the site are controlled from downstream, floodplain storage may also reduce flood levels adjacent to the storage sites themselves.

12.1 Floodplain Storage

Methods to provide space for water storage in the floodplain include lowering of floodplains, removal of flood protection berms or infill (termed depoldering) (both in Figure 37) and the introduction of flood storage ponds. Flood storage ponds are also lowered areas of the floodplain, but have inlets and outlets designed to control the flow of water into and out of the lowered areas.

In case of TRUP, possibilities of increased floodplain storage were discussed with the Landscape Architects. Floodplain storage was considered in front of Oude Molen and at the island of the River Club (in addition to the existing storage in front of the Wild Fig restaurant). See section 12.3.

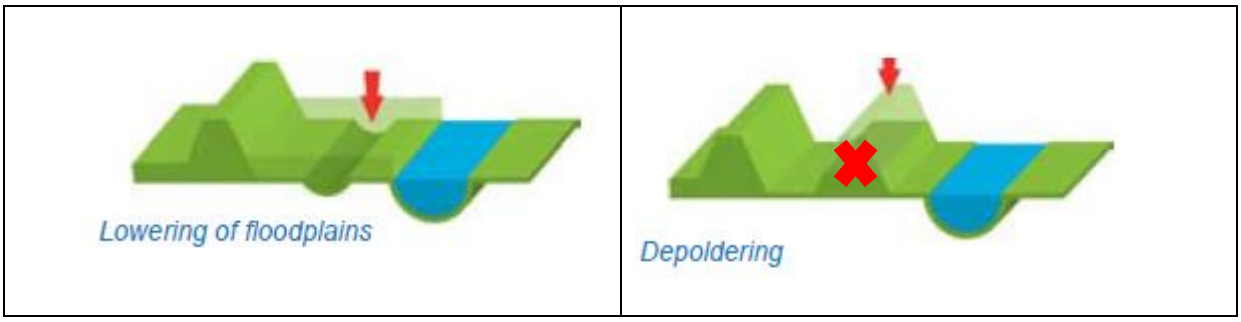


Figure 37: Schematic diagram of lowering of floodplain and depoldering

Source: Development Strategy 2030: River Warta (2012)

12.2 Stormwater attenuation: Ponds

Where flooding is due to local stormwater runoff, ponds can also be introduced to temporarily store and attenuate these flows. Within TRUP, the base model indicated potential flooding from the two bulk stormwater lines entering the Black River from the east (at Maitland Garden Village and adjacent to Berkley Road. Attenuation ponds could be introduced along these culverts. It needs to be realised that the stormwater attenuation is outside of the river floodplain itself and that it will not reduce the flooding in the river area in the 1:50 and 1:100 year situations, as the flow from local stormwater is relatively small in comparison to the river flows in these design situations. It will however be locally beneficial for flooding from stormwater, as the model indicates that the current bulk stormwater system at Berkley Road is likely to flood during the 1:5 year storm event, on the basis of the rainfall pattern used as

discussed in Section 4.2. If the inlets are designed to divert more frequent floods into the ponds then the ponds will be effective locally more regularly than mitigation measures for the larger floods.

12.3 Modelling and results

Potential storage areas were discussed with the consultants' project team and the representative of the CCT stormwater branch. After exclusion of some ecologically sensitive wetland areas, a final selection was made as shown in Table 14 and Figure 38. This included Extension of the existing wetlands within the M5 / N2 interchange and west of Oude Molen (SA1 and SA4), a new flood storage pond on the River Club Island (SA3) and two stormwater attenuation ponds (SP1 and SP2). Note that the existing attenuation pond on the Liesbeek River near the Wild Fig Restaurant was also retained, and is modelled implicitly as part of the overland flow network rather than being modelled explicitly as a storage area.

These storage areas were modelled for the 1:100 year flood with scenario 7 development and in preliminary 1:50 year and 1:5 year models (without Appendix H Part 2 corrections) with Scenario 6 development. For the 1:100 year model, the inlet level was set so as to start filling the storage just before the maximum water level, which is later than the flow peak, thereby having the maximum effect on the maximum water level. For the preliminary 1:5 year and 1:50 year flood models, the maximum water level coincided approximately with the maximum flow, and the inlet level was set to maximise the effect on the 1:50 year flood. This was done so as to investigate the effect of the same storage configuration on both floods.

Flow control structures, coupled with flow forecasting to control its operation, could increase the effectiveness of storage areas during higher frequency 1:5 and 1:10 year storms, but would require more maintenance, and are therefore not preferred by the CCT stormwater branch. It could also be decided to make the pond effective for 1:5 and 1:10 year and not effective during the more extreme floods, by decreasing the inlet level.

Table 14: Modelled flood storage areas (floodplain storage and ponds)

Number	Action	Area (x1 000m^2)	Outlet Level (m)
SA1	Extend existing wetland. Maintain connection to the river as is to maintain wetland function.	15	2.8
SA3	New flood storage pond	61	2.4
SA4	Extend existing wetland. Maintain connection to the river as is to maintain wetland function.	32	3.4
SP1	New pond for local stormwater	1	2.5
SP2	New pond for local stormwater	13[1]	3.4

[1] For the 1:100 year model G9, a larger storage area of 32 400m^2 was modelled, although the storage area is shown the same on the maps.

Storage area SA3 on the River Club Island

The effect of storage area SA3 on the River Club Island on flows in the Liesbeek River is shown in Figure 23 on page 66 and Figure 39 and Figure 40 below. The cause of the instability at around 22h has not been investigated, as it is long after the flood peak, and would therefore not influence peak flows, water levels or extents. Flooding and the effect of storage will depend on the relative magnitudes and timing of the flood peaks in the Liesbeek and Black Rivers.

The effect of the combined storage on the 1:100 year and preliminary 1:50 year flood extents is shown in **Maps 32 to 35**. The 1:100 year analysis used the latest models, but the 1:50 year analysis used preliminary models without the corrections described in Part 2 of Appendix H. There are local decreases in flood extents along the bulk stormwater lines, but the storage as modelled is not sufficient to prevent flooding from these lines. There is little change in flood extents along the Liesbeek and Black Rivers within TRUP, but storage within TRUP reduces downstream flows (Figure 23 on page 66), resulting in a decrease in flood extents in the PRASA yard and in the Woodstock or Foreshore area.

Connection of SA3 to the Black River rather than the Liesbeek River was also considered. However, the relatively far larger volume of the Black River flood would limit the effect of the storage on the flood hydrograph.

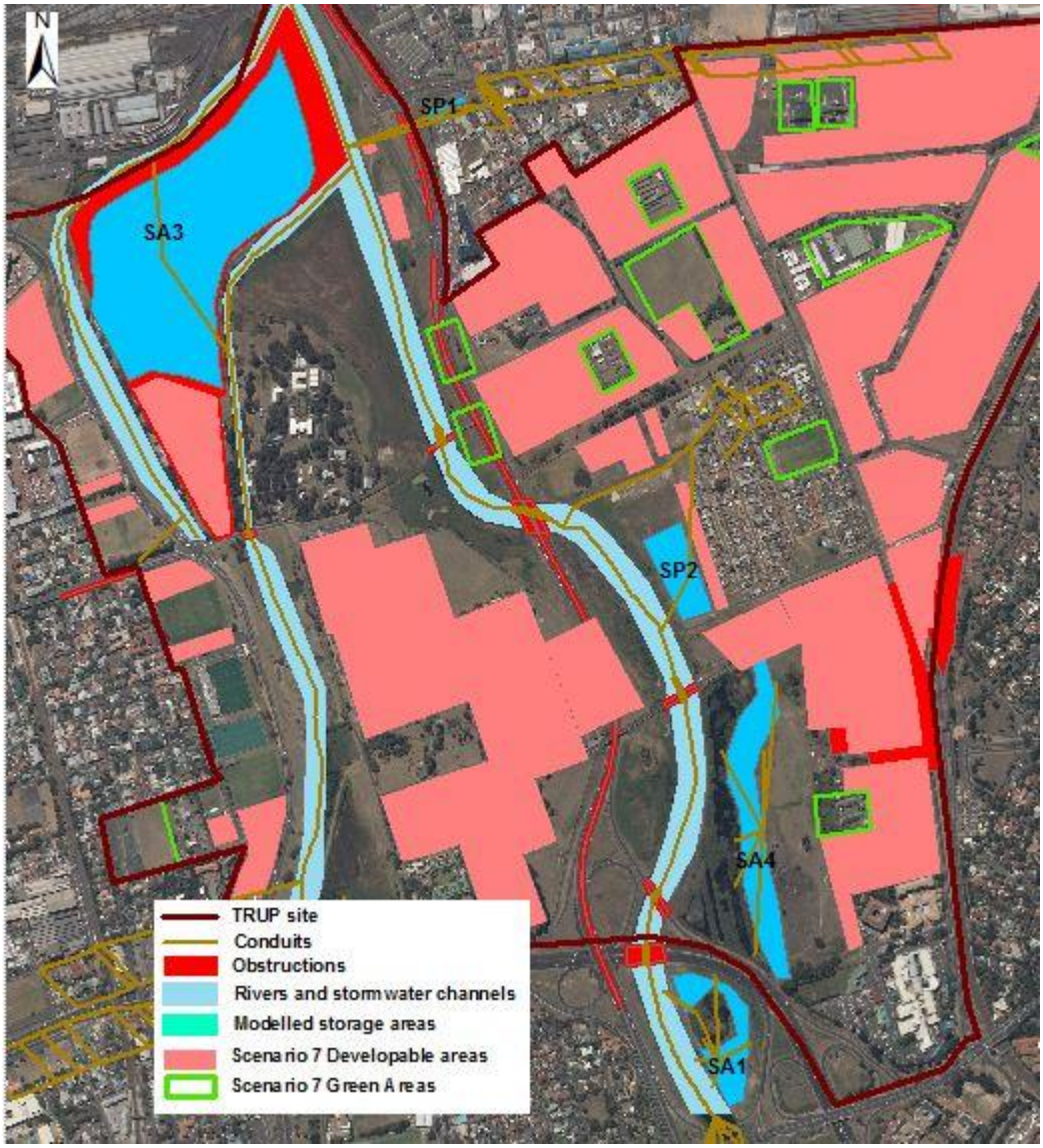


Figure 38: Modelled flood storage areas for the 1:100 year flood with Scenario 7 development

Note that for the preliminary 1:5 year and 1:50 year models, Scenario 6 development was instead used and that in these models, SA3 area differs slightly to accommodate the Scenario 6 development.

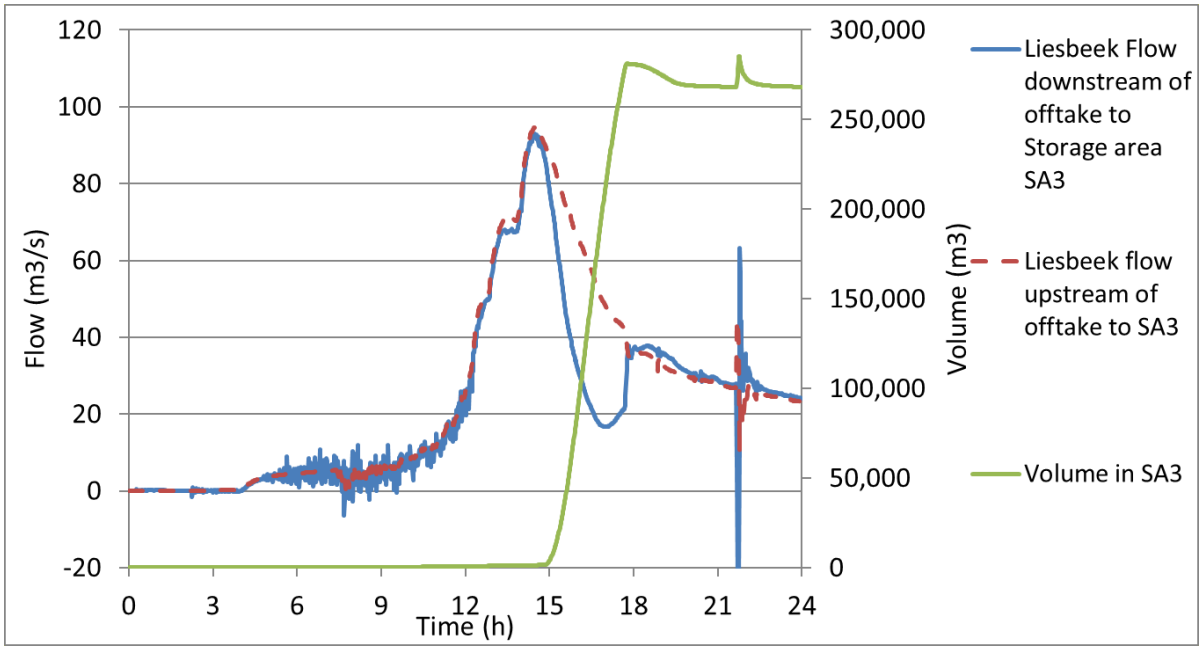


Figure 39: Effect of storage on the River Club Island on 1:100 year flood hydrograph in the Liesbeek River

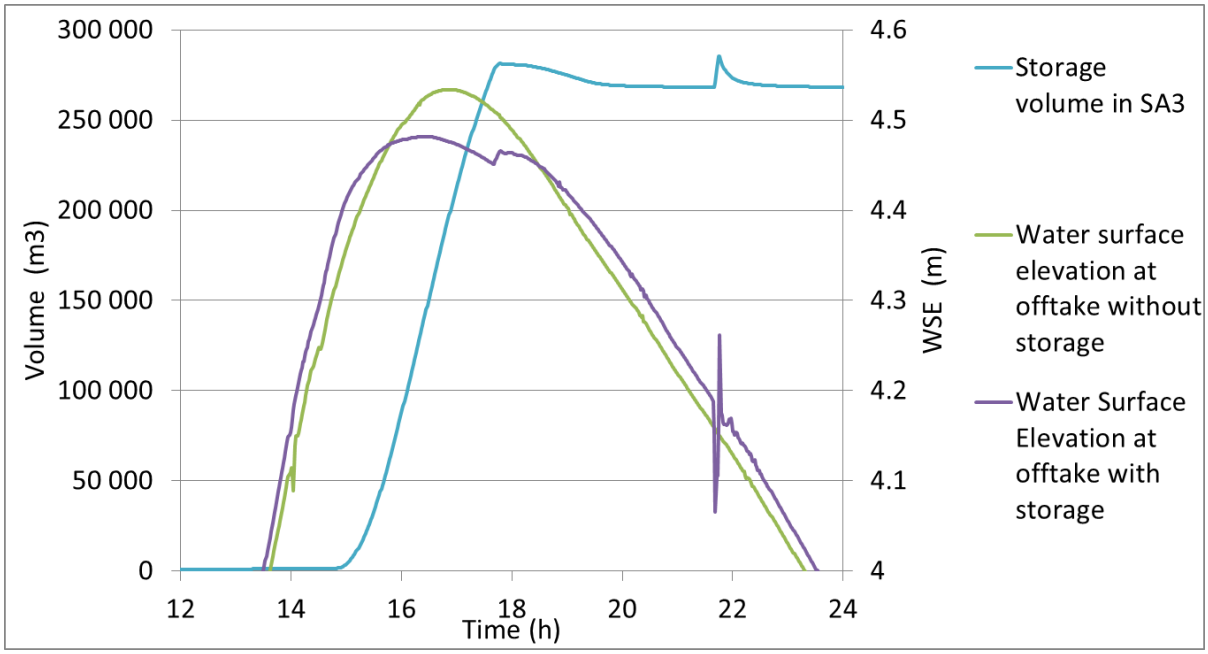


Figure 40: 1:100 year Water levels in Liesbeek River with and without floodplain storage

Extension of the Black River wetlands (SA1 and SA4)

The effect of floodplain attenuation along the Black River between Raapenberg Road and the Valkenburg Bridge (SA1 and SA4) is shown in Figure 41. Although the extensions of the wetlands store water during the flood peak, the storage volume is almost negligible in comparison with the flood volumes. The flow is predicted to be attenuated from a maximum flow of 225m^3/s at Raapenberg Road bridge to a peak

flow of 221m³/s at the Valkenburg Bridge. This includes all existing floodplain storage, as well as the modelled extensions to the wetlands.

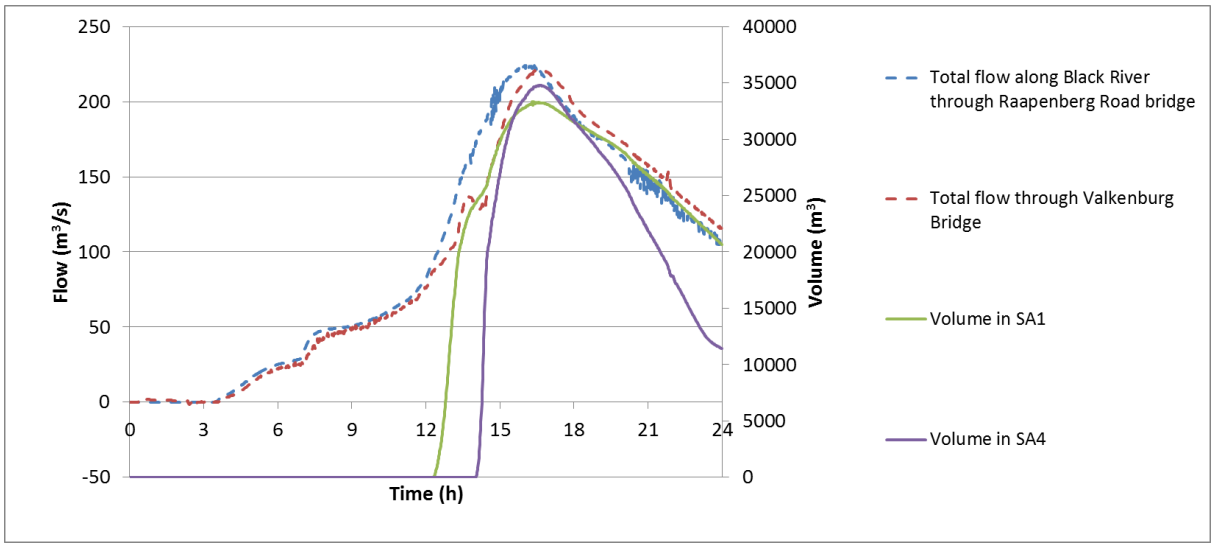


Figure 41: Effect of flood attenuation on 1:100 year Black River flows, including extension of wetlands SA1 and SA4, as well as existing storage

Stormwater attenuation pond SP1 at Berkley Road

The effect of the proposed stormwater attenuation pond on stormwater flows is shown in Figure 42 for the 1:5 year flood and Figure 43 for the 1:100 year flood.

While attenuation in SP1 is significant, it is not sufficient to prevent flooding along and adjacent to Berkley Road during any of the modelled flood events. The pond inlet level was fairly high, being set to maximise attenuation during the 1:50 year event. Both the preliminary 1:5 year and the 1:100 year flood peaks are reduced to approximately 1.8 to 1.9m³/s, which is the flow required to allow the pond to fill with this inlet level. This means that for the 1:5 year flood, very little of the stormwater is able to enter the pond, since it is restricted by the inlet. The pond is slowly filled by backflow from the Black River through the outlet, but never fills. In contrast, there is a great reduction in the 1:100 year flood peak, which fills the pond. The effectiveness for lower return periods could be increased by lowering the inlet level so that more of the flood enters the pond.

For the preliminary 1:50 year model only, the effectiveness of SP1 is reduced by the assumption that the pond was initially full to a depth of 2.5m (or an initial water level of 5.0m).For the 1:5 year model,

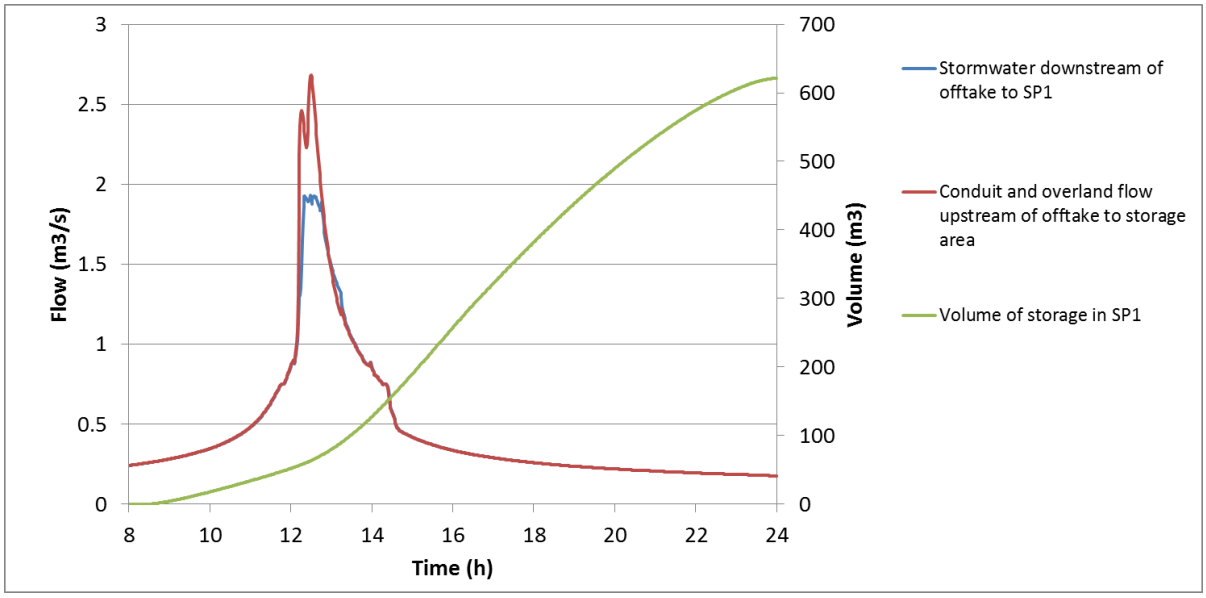


Figure 42: Effect of stormwater pond on stormwater conduit flows for the 1:5 year flood

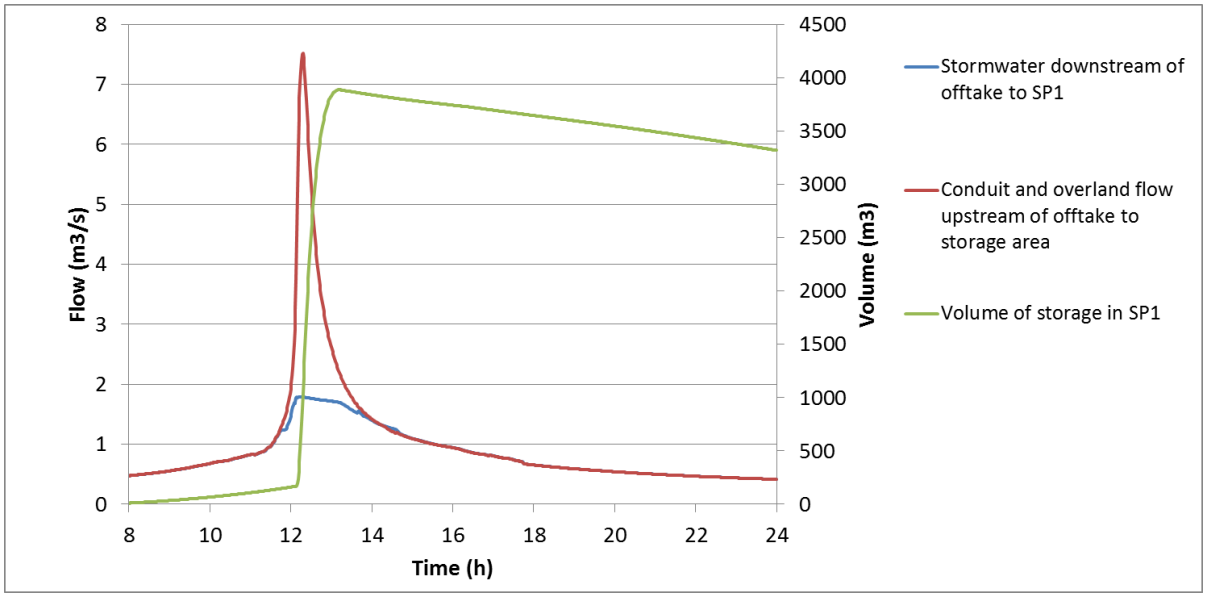


Figure 43: Effect of storage area SP1 on local stormwater hydrograph for the 1:100 year flood.

Stormwater attenuation pond SP2 at Maitland Garden Village

Although some other maps show some flooding of the northern parts of Maitland Garden Village along the bulk stormwater main during the 1:5 year flood, this is an artefact of the issue discussed in Section 4.1 that flooding is always shown at cells connecting to 1D conduits. Maps 6, 32 to 37 and 58 have been drawn based on the calculated water surface elevations, thereby avoiding this effect. They show that this area is only predicted to be flooded in the 1:20 year and larger floods, and not in the preliminary 1:5 year and 1:10 year floods. Stormwater attenuation pond SP2 was intended to alleviate this flooding. As shown on maps 34 and 36, the attenuation is sufficient to prevent flooding in this area during the 1:50 and 1:100 year floods.

As can be seen in the hydrographs in Figure 44 and Figure 45, the storage area continues to fill from the Black River once the local stormwater peak has passed.

For uniformity in the modelling, this pond was also included in the 1:5 year storage model, even though there no flooding of the area is predicted. In order to allow the pond to fill more quickly, the preliminary 1:5 year model used a lower inlet level (by 298mm) than the preliminary 1:50 year model.

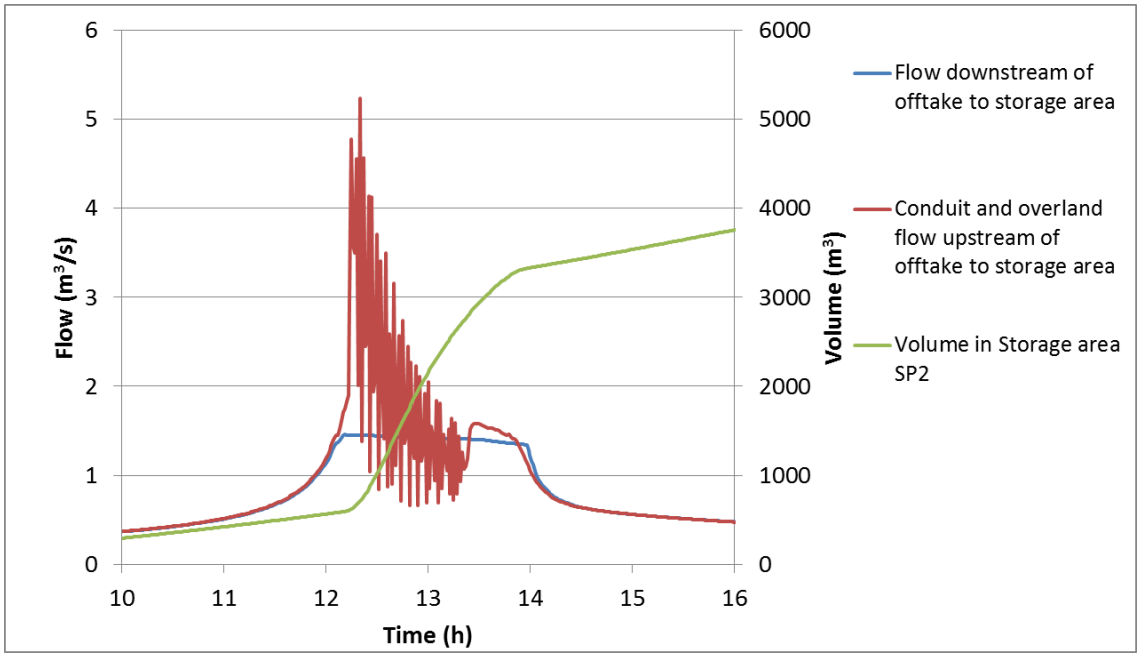


Figure 44: Effect of storage area SP2 on the 1:100 year local stormwater hydrograph. Note that inflow to and outflow from the storage area are not shown.

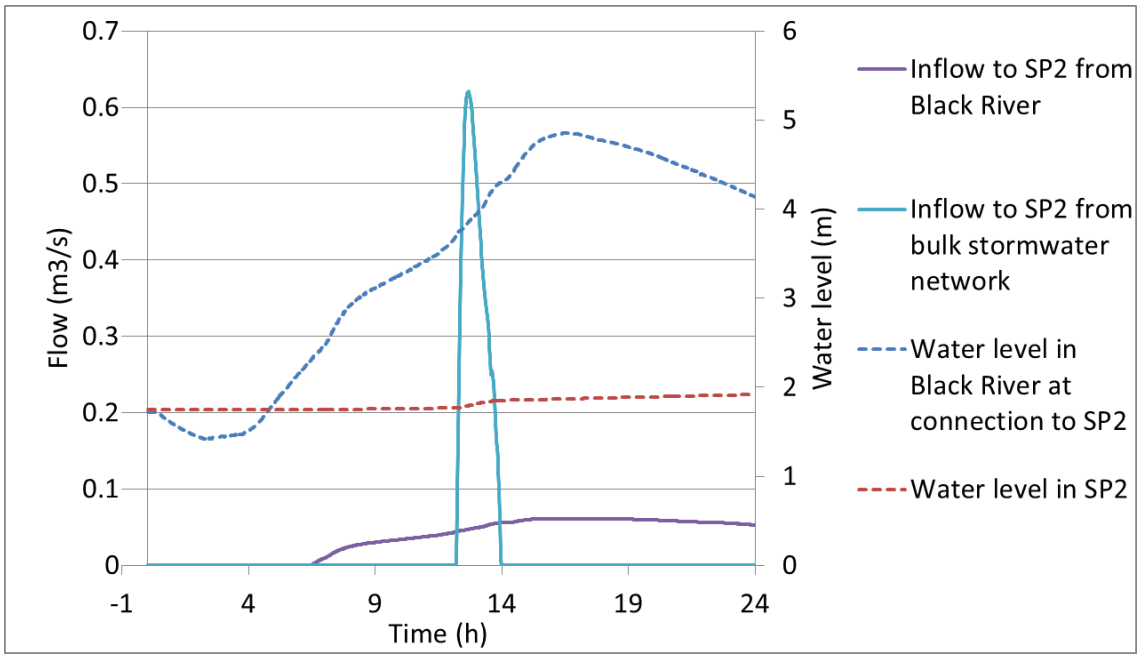


Figure 45: Inflows to attenuation pond SP2 during the 1:100 year storm

12.4 Storage upstream of TRUP

Storage could also be considered upstream of TRUP. In order to significantly affect flood flows and extents within TRUP, the storage areas should be as close to TRUP as possible, so as to capture flow from as much of the catchment as possible. The only large open space within the immediate upstream vicinity of TRUP is the open area of the Rondebosch and King David Mowbray golf clubs. Flood storage ponds covering the area of these two golf clubs could potentially provide a significant reduction in flood peaks along the Black River. Modelling of the effect of upstream measures is beyond the scope of this project, therefore this option was not considered for modelling.

In practise, it is uncertain whether the land could be made available for flood storage. Nonetheless, for reduction of flood extents and flood hazard, from an effectiveness perspective it could be considered to investigate the option of constructing flood storage ponds at the location of these golf courses, possibly in conjunction with the construction of an additional flood storage pond for the Liesbeek River on the River Club Island. It needs to be realised that considerable costs will be involved and the design would need to take into consideration ecological effects, apart from the fact that the option would affect the use of and view from the golf courses.

12.5 Channel: Meandering

Storage may also be achieved in the channel, either by channel modification as discussed in Chapter 11 or by increasing the channel length and width through the introduction of meandering. Meandering is discussed here as it was several times mentioned in the interactions with the landscape design team and other key role players.

Introduction of meanders into a river could provide additional storage volume within the longer channel. From a flood mitigation perspective, that is the topic of this specialist study, the flow itself slows down with additional meandering due to the decreased gradient. This storage further attenuates the flow, resulting in lower flood peaks downstream.

For the TRUP area itself, meandering would increase flooding. The project team considered it for aesthetic and ecological reasons. The specialist study on Watercourse Management and Creating a Docking Station / Waterfront Feature (De Groen et al., 2017), does not recommend a meander which has lower lengths and higher amplitudes as the morphological determinant 1:2 year flow would already try to make such meanders more straight, therefore the meanders would need to be highly engineered with high maintenance.

Introducing bends or meanders upstream of TRUP could in this way provide some flood mitigation. However, there is limited space available in the upstream catchment (except possibly in the golf clubs mentioned above) and the feasibility of extra meanders may have the same challenge as those that were considered in the TRUP area. Hydraulic modelling of upstream measures is beyond the scope of this project, so the effect on flood mitigation was not tested with the model.

Figure 46: Example of channel meandering: Kallang River

13 Outflow improvement: Zoarvlei outfall

Water from the Salt River spills over into the Zoarvlei. The Zoarvlei discharges to the Mouth of the Diep River through a stormwater culvert (Figure 47), which is reportedly often blocked. Water levels in the Zoarvlei could be reduced by improving the capacity of this outlet, by connecting the Zoarvlei to the Century City outfall.

Figure 47: Aerial image of Zoarvlei discharge

The long-section in Figure 48 suggests that the high ground between the Salt River Canal and the Zoarvlei has more influence on flow into the Zoarvlei than the water levels in the Zoarvlei. This assertion is also evidenced by the large change in predicted flood extents in the Zoarvlei depending on the initial water level. Therefore, a reduction in flood water levels in the Zoarvlei would not affect TRUP, unless an alternative lower level inlet to the Zoarvlei is also constructed. This would cause the Salt River to flow into the Zoarvlei more often, dependent on the design. This mix of Salt and Zoarvlei could interfere with the ecological functioning of the Zoarvlei as it is now (notwithstanding that the connection might historically have

been there). Even with a lower inlet to the Zoarvlei, given the distance to TRUP, the scope for reduction of flood levels in TRUP is very limited.

For the above reasons, it was decided not to model changes to the Zoarvlei outlet.

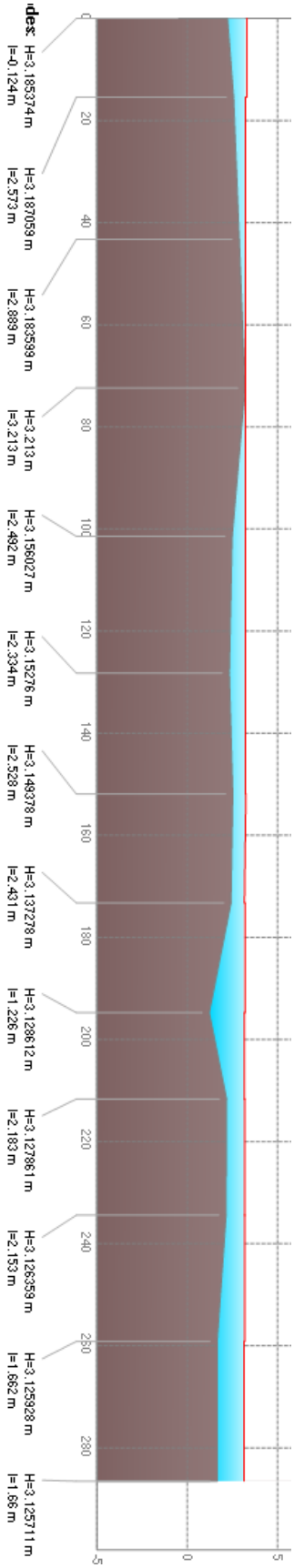


Figure 48: Section between Salt River and Zoarvlei showing maximum 1:100 year water level. H is the maximum water level and I is the ground level.

14 Possible combinations of mitigation measures

In view of the effects discussed above, the most likely combination of flood mitigation measures considered is:

- The channel could be widened or improved within TRUP as discussed in Chapter 11;
- Flood storage could also be provided within TRUP as discussed in Chapter 12. Of the storage options investigated, storage on the River Club Island has the most effect;
- Any proposed development which is still within the floodplain would need to either be constructed on fill, or be flood resistant or be protected by berms as discussed in Chapter 9.

Therefore, this combination of three mitigation measures was modelled for the extreme (1:100 year) and more frequent (1:5 year) flood events using the preliminary models without the Appendix H Part 2 corrections. The effect of the combination of the two mitigation options (Black River Canal widening and floodplain storage) on preliminary modelled flood extents is shown in **Maps 42 to 45** in Appendix K.

Both 1:5 year and 1:100 year flood extents are predicted to be reduced primarily in the upstream part of the Black River, and the effect is visible further downstream with the combination of measures, more than with the channel widening only. The 1:5 year flood extents are also reduced along the Liesbeek River. However, the area of the reduction is small compared to the area of the proposed storage areas, particularly the storage area on the River Club Island.

The effect for TRUP, given that the areas where the flood extents decreased are not identified as buildable areas in scenario 6 nor scenario 7, does not seem worth the extensive investment involved. The effect on Paarden Eiland also does not add much more benefit, so hardly adds to the argument in favour of such an investment.

15 Summary of Models and Maps

15.1 Summary of models

Model results of all model runs presented in Table 15 have been presented and discussed in this report. The maps produced from these models are also listed below. File names are provided for reference to the electronic model files, delivered to the Client as well.

Table 15: Summary of models

Geometry	Original Run or refinement No.	Run with part 1 corrections only(1)	Final version(2)	Return period	Model file Name(s)
Base Model	B1		G1	1:100 year	20160627_TRUP_100yr Model_G1_TRUP_100yr_base
	B2		G5	1:50 year	20160701_TRUP_50yr Model_G5_TRUP_50yr_base
	B3	C1	G6	1:20 year	20160624_TRUP_20yr 20160906_TRUP_20yr_revised_base_model Model_G6_TRUP_20yr_base
	B4			1:10 year	20160610_TRUP_10yr
Proposed development footprint	C2	D1	G7	1:100 year	20160712_TRUP_100yr_proposed_footprint_obstructions 20160808_TRUP_100yr_proposed_footprint_obstructions_corrected Model_G7_TRUP_100yr_Scenario7_footprint
		D3	G3	1:50 year	20161011_TRUP_50yr_proposed_footprint_obstructions_tide_083_245 Model_G3_TRUP_50yr_Scenario7_footprint
		D5	G4	1:20 year	20160911_TRUP_20yr_proposed_footprint_obstructions_tide_083_245 Model_G4_TRUP_20yr_Scenario7_footprint
		E2		1:5 year	20161104_TRUP_5yr_proposed_footprint_obstructions
		D2		1:100 year with river inflows reduced by 15%	20160810_TRUP_100yr_reduced_upstream_flows_with_proposed_development
		D4		1:20 year with 1:50 year tide setup	20160902_TRUP_20yr_proposed_footprint_obstructions_corrected
Widening of Salt River and Black River canals and bridges	D6			1:100 year	20160728_TRUP_100yr_widening_of_channel
		D7		1: 50 year	201610112_TRUP_50yr_widening_of_channel
		D8		1:20 year	20160906_TRUP_20yr_widening_of_channel

Geometry	Original Run or refinement No.	Run with part 1 corrections only[1]	Final version[2]	Return period	Model file Name(s)
Floodplain storage areas in combination with proposed development footprint		D9	G9	1:100 year	20161012_TRUP_100yr_proposed_footprint_obstructions_corrected_storage_areas Model_G9_TRUP_100yr_Floodplain_Storage_with_Scenario7_footprint
		D10		1:50 year	20161018_TRUP_50yr_footprint_storage_areas
		E3		1:5 year	20161107_TRUP_5yr_footprint_storage_areas
Widening of Black River channel only		E1		1:100 year	20161102_TRUP_100yr_widening_Black_only
		E4		1:5 year	20161108_TRUP_5yr_widening_Black_only
Proposed development with widening of Black River channel only		E9		1:5 year	20161110_TRUP_5yr_proposed_development_Black_widening_only
Proposed development plus infill of River Club		E5	G8	1:100 year	20161108_TRUP_100yr_proposed_footprint_obstructions_corrected_river_club_regen Model_G8_TRUP_100yr_Scenario7_footprint_plus_River_Club_infill
		E6		1:5 year	20161108_TRUP_5yr_proposed_footprint_obstructions_corrected_river_club_regen
Proposed development plus floodplain storage plus widening of Black River channel		E7		1:100 year	20161113_TRUP_100yr_Scenario6_Storage_widening_Black_only
		E8		1:5 year	20161112_TRUP_5yr_footprint_storage_areas._Black_widening
Intermediate 1:100 year models with Scenario 6 development showing the effect of individual corrections		F1[3]		1:100 year	20170118_Model_F1_TRUP_100yr_Scenario6_footprint_Liesbeek_weir_corrected
		F2[4]		1:100 year	20170128_Model_F2_TRUP_100yr_Scenario6_footprint_bridge_loss_corrected
		G2(a)[4],[5]		1:100 year	20170208_Model_G2a_TRUP_100yr_Scenario6__footprint_SRK_flows_original
		G2(b)[4],[6],[7]		1:100 year	20170208_Model_G2b_TRUP_100yr_Scenario6_footprint_SRK_flows_rerun

(1) Where proposed development is indicated in these runs, it is Scenario 6 development
(2) Version including all corrections. Where proposed development is indicated in these runs, it is Scenario 7 development.
(3) With corrections to prevent flooding losses
(4) With corrections to prevent flooding loss plus energy losses at bridges
(5) Using flows from SRK (2012) models
(6) Using flows from SRK (2012) models with Liesbeek model re-run with SWMM v5.1.007 engine (and flows for the appropriate return period)
(7) This is the final model for Scenario 6 development footprint.

15.2 Selected peak water level predictions

Predicted peak water levels from selected models at selected locations are presented in Table 16 for the preliminary models and Table 17 for the final models.

Table 16: Summary of selected peak modelled water levels from preliminary models

Model No.		B1	C2	D1	D2	E5	E7	E2	E3	E4	E6	E9	D6	E1
Return Period		1:100 year	1:100 year	1:100 year	1:100 year	1:100 year	1:100 year	1:5 year	1:5 year	1:5 year	1:5 year	1:5 year	1:100 year	1:100 year
Scenario		Base model	Proposed development (without corrections)	Proposed development (corrected)	15% Reduction in inflows to TRUP from the Liesbeek and Black rivers	Proposed development with infill f River Club Island	Proposed development with floodplain storage and Black River widening	Proposed development	Proposed development with floodplain storage	Widening of Black River only	Proposed development with infill of River Club Island	Widening of Black River with proposed development	Widening of Black and Salt rivers with an additional parallel	Widening of Black river only with an additional parallel canal
Position	**Node**	**9 minute average peak water level (m)**												
Black River														
Downstream of N2	J1989.755	4.77	4.78	4.78	4.60	4.78	4.24	3.94	3.88	3.43	3.95	3.43	3.94	4.25
Downstream of Valkenburg Bridge	J1253.188	4.57	4.57	4.56	4.42	4.56	4.19	3.84	3.78	3.41	3.86	3.41	3.88	4.20
Downstream of M5 bridge	J771.3945	4.19	4.19	4.18	4.13	4.18	4.09	3.73	3.66	3.38	3.74	3.38	3.77	4.09
Liesbeek River														
Downstream of N2	J1681.983	5.08	5.10	5.10	5.09	5.10	5.10	4.40	4.40	4.40	4.40	4.40	5.10	5.10
Downstream of weir near Wild Fig	J1573.841	4.50	4.51	4.51	4.47	4.51	4.49	3.86	3.82	3.74	3.88	3.72	4.53	4.52
Downstream end of ponds near Wild Fig	J1279.92	4.20	4.18	4.18	4.15	4.18	4.17	3.74	3.65	3.52	3.77	3.51	4.21	4.21
Upstream of weir near Observatory Road	J846.0473	4.03	4.03	4.03	4.03	4.03	4.03	3.70	3.58	3.46	3.73	3.45	4.03	4.03
Downstream of Observatory Road	J807.905	4.03	4.03	4.03	4.03	4.03	4.03	3.69	3.58	3.45	3.72	3.44	4.00	4.03
Junction with Black River	93285	4.07	4.07	4.05	4.03	4.06	4.05	3.64	3.56	3.35	3.65	3.35	3.68	4.05
Salt River														
Downstream end of TRUP before rail bridge	4	3.99	3.99	3.95	3.93	3.96	4.03	3.56	3.47	3.34	3.57	3.34	3.67	4.03
Upstream of Section Road Bridge (Bridge 6)	J1143.123	2.94	2.95	2.96	2.96	2.67	2.90	2.66	2.61	2.51	2.67	2.51	2.70	2.94
Mouth	J56.34234	2.53	2.53	2.53	2.53	2.53	2.53	2.45	2.45	2.45	2.45	2.45	2.53	2.53

Table 17: Summary of selected peak modelled water levels from final models

Model No.		F1	F2	G2(a)	G1	G2 (b)	G3	G4	G5	G6	G7	G8	G9
Return Period		1:100 year	1:100 year	1:100 year	1:100 year	1:100 year	1:50 year	1:20 year	1:50 year	1:20 year	1:100 year	1:100 year	1:100 year
Scenario		**with correction to prevent losses from Liesbeek weir**	**Proposed development with additional of energy losses (and Liesbeek weir correction)**	**Proposed Dev. with flows from SRK models**	**Base model revised 2017**	**Scenario 6 Dev. with flows from SRK models with Liesbeek model re-run**	**Scenario 7 Dev. with flows from SRK models with Liesbeek model and Maitland-Kensington models re-run**	**Scenario 7 Dev. with flows from SRK models with Liesbeek model re-run. Maitland-Kensington flows from our model**	**Base model revised 2017**	**Base model revised 2017**	**Scenario 7 Dev. with flows from SRK models with Liesbeek model re-run**	**River club infill plus Scenario 7 Development**	**Floodplain storage with Scenario 7 Development**
Position	**Node**												
Black River													
Downstream of N2	J1989.755	4.92	5.06	5.04	5.05	5.08	4.91	4.58	4.90	4.58	5.06	5.10	5.05
Downstream of Valkenburg Bridge	J1253.188	4.75	4.91	4.88	4.89	4.93	4.74	4.44	4.74	4.43	4.91	4.95	4.89
Downstream of M5 bridge	J771.3945	4.42	4.55	4.53	4.53	4.57	4.43	4.20	4.42	4.19	4.56	4.61	4.52
Liesbeek River													
Downstream of N2	J1681.983	5.12	5.17	5.10	5.16	5.15	5.13	5.00	5.13	5.00	5.19	5.16	5.16
Downstream of weir near Wild Fig	J1573.841	4.65	4.57	4.52	4.63	4.63	4.54	4.44	4.52	4.42	4.60	4.70	4.67
Downstream end of ponds near Wild Fig	J1279.92	4.44	4.55	4.51	4.53	4.57	4.47	4.27	4.42	4.24	4.59	4.65	4.55
Upstream of weir near Observatory Road	J846.0473	4.43	4.55	4.51	4.53	4.56	4.46	4.24	4.41	4.21	4.58	4.64	4.53
Downstream of Observatory Road	J807.905	4.42	4.55	4.51	4.53	4.56	4.45	4.22	4.40	4.19	4.58	4.64	4.52
Junction with Black River	93285	4.36	4.51	4.49	4.49	4.54	4.39	4.15	4.37	4.14	4.53	4.57	4.48
Salt River													
Downstream end of TRUP before rail bridge	4	4.27	4.45	4.42	4.43	4.48	4.33	4.07	4.31	4.06	4.47	4.48	4.36
Upstream of Section Road Bridge (Bridge 6)	J1143.123				3.95	3.97	3.91	3.76	3.89	3.76	3.97	3.97	3.92
Mouth	J56.34234	3.03	3.21	3.21	3.22	3.22	3.10	2.94	3.09	2.94	3.22	3.23	3.21

15.3 Summary of maps

Appendix A: Maps of Flood Extents existing situation (Task 1)

Table 18: Maps in Appendix A of existing situation

Return period	No. of Map of modelled area	No. of Map of TRUP area	Comment
1:20, 1:50 & 1:100 year[1]	1 Rev 2	2 Rev 2	Flood extents
1:20, 1:50 & 1:100 year[1]	3 Rev 1	4 Rev 1	Floodlines
1:10 year[2]	5	6	Preliminary flood extents
1:20 year[1]	7 Rev 1	8 Rev 1	Maximum depths
1:50 year[1]	9 Rev 1	10 Rev 1	Maximum depths
1:100 year[1]	11 Rev 1	12 Rev 1	Maximum depths
1:50 year[i]		60	Comparison with SRK (2012) floodlines
1:20 year[i]		61	Comparison with SRK (2012) floodlines

[1] Outline of proposed development - Scenario 7 superimposed (but not used in the model)

[2] Original results without Appendix H corrections

Appendix E: Map of Floodplain Manning's *n*

Map E1: Floodplain Manning's *n* values

Appendix G: List of Assumption for stormwater conduits

Map G1: Assumptions for stormwater conduits

Appendix H: Corrections to the model for scenarios and mitigation options

Table 19: Maps in Appendix H, comparing flood extents of two flood modelling scenarios

Flood modelling scenario 1		Flood modelling scenario 2		Return period	Map of modelled area	Map of TRUP area
Description	**Model**	**Description**	**Model**			
Base scenario	B3	Revised base scenario with Appendix H corrections	G6	1:20 year	13 Rev 1	14 Rev 1
Scenario 6 development without Appendix H corrections	C2	Scenario 6 development with Appendix H corrections	G2(b)	1:100 year	15 Rev 3	16 Rev 3
Scenario 6 development with Appendix H part 1 corrections only	D1	Scenario 6 development with Appendix H part 1 corrections and corrections to prevent flooding losses	F1	1:100 year	51	52
Scenario 6 development with Appendix H part 1 corrections only	D1	Scenario 6 development with Appendix H corrections except for the use of the SRK (2012) flows	F2	1:100 year	53	54

Appendix I: Scenarios of Urban Planning Development Footprints

Map of Scenario 6 buildable areas footprint

Map of Scenario 7 buildable areas footprint

Appendix J: Maps of flood extents for proposed development

Table 20: Maps in Appendix J comparing flood extents of two flood modelling scenarios

Flood modelling scenario 1		Flood modelling scenario 2		Return period	Nr of Map of modelled area	Nr of Map of TRUP area
Description	**Model**	**Description**	**Model**			
Base scenario	G1	Proposed development scenario 6	G2(b)	1:100 year	17 Rev 1	18 (Rev 1
Base scenario	G1	Proposed development scenario 7	G7	1:100 year	17 Rev 2	18 Rev 2
Base scenario	G5	Proposed development scenario 7	G3	1:50 year	19 Rev 1	20 Rev 1
Base scenario	G6	Proposed development scenario 7	G4	1:20 year	21 Rev 1	22 Rev 1
Proposed development[1]	D5	Proposed development with 1:50 year tide level[1]	D4	1:20 year	23[1]	-
Widening of Black River canal by 25m[1]	E4	Widening of Black River canal by 25m with proposed development[1]	E9	1:5 year	46[1]	47[1]
Proposed development	G7	Proposed development plus infill of River Club	G8	1:100 year	-	48 Rev 1
	E2[1]		E6[1]	1:5year	49[1]	50[1]
Scenario 6 development (Depths)	G2(b)	NONE		1:100 year	56 Rev 1	57 Rev 1
Scenario 7 development (Depths)	G7	NONE		1:100 year	56 Rev 2	57 Rev 2
Scenario 7 development (Depths)	G4	NONE		1:20 year		58
Scenario 7 development (Depths)	G3	NONE		1:50 year		59

[1] With Appendix H part 1 corrections only. The flood extents shown are therefore not final, but nonetheless give an indication of the effect of the proposed development on the flood extent for smaller floods.

Appendix K: Maps of flood extents for proposed mitigation measures

Table 21: Maps in Appendix K

Note that except where otherwise indicated, all models include Appendix H part 1 corrections only. The flood extents shown are therefore not final, but nonetheless give an indication of the effect of the respective mitigation measures on flood extents, since the runs with and without each mitigation measure are based on the same model.

Scenario 1		**Scenario 2**		**Return period**	**Map of modelled area**	**Map of TRUP area**
Description	**Model**	**Description**	**Model**			
Base scenario[1]	B4	Widening of Black River and Salt River canals by 25m[1]	D6	100	24	25
None		Widening of Black River and Salt River canals by 25m plus other changes as per Appendix H part 1	D7	50	26	27
Revised base scenario	C1	Widening of Black River and Salt River canals by 25m	D8	20	28	29
Proposed development	D1	Proposed development with 15% reduction in peak inflows from the Liesbeek and Black rivers	D2	100	30	31
Proposed development[2]	G7	Proposed development with floodplain storage[2]	G8	100	32	33
Proposed development	D3	Proposed development with floodplain storage	D10	50	34	35
Proposed development	E2	Proposed development with floodplain storage	E3	5	36	37
Base scenario[1]	B4	Widening of Black River Canal only by 25m	E1	100	39	38
Proposed development	E2	Proposed development plus widening of Black River Canal only by 25m	E9	5	40	41
Proposed development	D1	Proposed development with floodplain storage and widening of Black River Canal only by 25m	E7	100	42	43
	E2		E8	5	44	45

[1] Without any Appendix H corrections

[2] With all Appendix H corrections

16 Evaluation, conclusions and recommendations

The terms of reference require that "*conclusions and firm recommendations are required, with at least some indication of cost, as well as legal, planning, ownership, etc. implications.*"

The terms of reference also requires identification of the "*way forward*" for implementation of the recommended measures, including "*required steps*."

16.1 Effect of proposed development

Within TRUP, the proposed urban development Scenario 7 is assessed to have little influence on flood levels and extents. Expected changes in the 1:100 year flood levels are 0.05m or less. A slight backing up is predicted along the lower reaches of the Liesbeek and Black rivers. The 1:100 year flood extent does not change significantly, the only differences that can be noted are close to Maitland Garden Village, through the PRASA yard and a positive effect of the proposed development on flood extents on the west side of the Liesbeek for 1:100 year floods only, thanks to blocking by the buildings of the flooding. The effect of the proposed development on the volume stored in TRUP is 94 000 m^3 for the 1:100 year flood. This is an indication for a compensation needed elsewhere, in case a "builder compensates" principle will be applied.

The additional effect of infill of the River Club Island on predicted 1:100 year flood levels is expected to be up to 0.1m. All development within the floodplain (and even for some height above the predicted floodplain) should be designed for flood resilience.

16.2 Evaluation of flood mitigation measures

(i) Effectiveness

The arguments and models presented in this report suggest that the following measures would have hardly any effect on flood water levels and extents:

- Improvement of the Zoarvlei outlet;
- Catchment stormwater harvesting, detention and infiltration sufficient to reduce peak inflows, tested for a 15% flood peak reduction; and
- Extension of the existing wetlands within the M5 / N2 interchange and below Maitland Garden Village.

The effectiveness of other more promising measures will be discussed below.

Stormwater detention

Local flooding due to capacity of the bulk stormwater network being exceeded by the 1:5 year flood is predicted in the northern parts of Maitland Garden Village, in the area around Eastman Road and between Berkley Road and Frere Road.

a) ***Stormwater detention adjacent to Berkley Road***

Routing the Iflow from the stormwater main along Berkley Road through a detention pond (SP1) would help to reduce flooding downstream of the off-take to the proposed detention ponds. Although there is no proposed development downstream of pond SP1, other than the possible docking / waterfront feature, flooding along Berkley Road at the M5 interchange could potentially be reduced. This would be desirable, since a freeway interchange should not form part of the major stormwater drainage network.

Since the pond inlet was set for larger floods, model results show no effect for the 1:5 year storm (Map 37), but some reduction in flooding for the 1:50 year (Map 35) and 1:100 year (Map 33) storms. Berkley Road is still flooded during all modelled storm events. A flow control structure, coupled with flow forecasting to control its operation, could increase the effectiveness during higher frequency 1:5 and 1:10 year storms but would require more maintenance and are therefore not preferred by the CCT stormwater branch. It could also be decided to make the pond effective for 1:5 and 1:10 year and not effective during the more extreme floods, by decreasing the inlet level.

An alternative option would be to increase the capacity of the piped stormwater network along Berkley Road, as the local TRUP inflows during high return periods will have limited effect on downstream river flooding.

b) ***Stormwater detention below the northern parts of Maitland Garden Village***

Since this area is predicted to be flooded only during the 1:20 year flood, the flood protection benefits of stormwater attenuation would seldom be realised. Nonetheless, model results show that the pond could help to alleviate flooding during the larger storm events. Nonetheless, provision of a flood water escape route will still be required.

Both ponds could also be used for stormwater harvesting.

Flood storage

The effect of conversion of the River Club to a flood retention pond would differ, depending on whether a particular flood is primarily from the Liesbeek, the Black River or both and how the flood retention pond is used. High flows in the Liesbeek only (generated by a storm primarily over the Liesbeek catchment) could be greatly reduced by a flood retention pond on the River Club Island. For the situation

modelled, which is a widespread storm over the entire catchment of the Liesbeek and Black Rivers, there is still a large effect on flows in the Liesbeek River, but the effect on flood extents is very limited. This is partly because the storage is small relative to the larger flood volumes in the Black River and partly because the effect of floodplain storage is mainly downstream of TRUP.

Benefits to TRUP in this case cannot justify the conversion of the River Club Island to a stormwater attenuation pond, since the reduction in flood extents is far smaller than the area of the island itself.

More effective flood reduction for flooding either from both rivers or primarily from the Black River would require separate flood storage on the Black River. The extension of existing wetlands would have negligible influence on flows and therefore flood extents in the Black River. The conversion of the Rondebosch and King David Mowbray golf clubs to flood storage basins could be considered, but has not been investigated in this study.

Benefits of combined flood storage in the Black and Liesbeek Rivers are expected to be largely downstream of TRUP.

Channel enlargement

The most effective measure modelled to mitigate the effect of river flooding within TRUP is channel enlargement, together with accompanying bridge widening. Most of this effect is achieved by widening of the Black River channel only, and the additional effect of widening both the Black and Salt Rivers cannot be justified considering the large number of bridges involved. However, in either case, the main reduction in flood extents is in the area adjacent to Oude Molen, where no development is planned.

Various options for "nature-friendly" river banks are considered in the associated watercourse management specialist study, including wet banks, boggy banks and gradual slope banks. Re-shaping of the banks would go hand in hand with the enlargement of the channel, and the suggested wet, boggy or gradually sloping banks would form part of the enlargement. However, as noted in the watercourse management study, an increase in roughness (Manning's *n* values) would be expected with more naturalised banks. The increase in roughness would counteract the flood reduction effect of the larger channel.

The Liesbeek channel could effectively be enlarged by reconnecting the Liesbeek Dead Arm as a parallel channel. However, as for storage on the River Club Island, this would not be effective when flooding is also from the Black River, and has therefore not been investigated.

High flow diversion of Elsieskraal River

Diversion of high flows in the Elsieskraal River could potentially reduce the 1:100 year flows and levels in the Black River to approximately what are now the 1:10 year or 1:20 year flows and levels. It is an option which could be considered and further studied.

(ii) Cost

Stormwater detention

Stormwater detention ponds are common, and their cost is generally not prohibitive. The excavation volumes required for the modelled ponds are estimated as 7 000m^3 and 56 000m^3, based on the raised LiDAR levels. A capital cost of several million Rand would be expected for the two ponds.

The cost of the pond at Berkley Road is probably comparable to cost of upgrading the stormwater pipe in this area, but the pipe upgrading could be more effective if appropriate dimensions are selected, particularly in view of the fact that flooding from local stormwater usually does not coincide with river flooding.

Flood storage

The conversion of the River Club Island to a stormwater detention pond would involve large capital costs. The main capital costs would be:

- Land acquisition, which could be high, particularly in view of the development potential already identified by the owner
- Design, procurement and contract administration and inspection
- Excavation: An excavation volume of the order of 280 000m^3 has been estimated based on the raised LiDAR levels. It is possible that part of the excavated material could be used to fill in areas where development is planned in the floodplain, but a spoil site would have to be identified for most of the material. At a typical rate of R30/m^3 for soft material including limited haul, the excavation cost would be of the order of 8 to 9 million Rand. A considerable increase could be expected if hard material is encountered or if the spoil site is some distance away.
- Inlet structure: A wide inlet structure would be needed to accommodate the flood peak. The model considered an inlet width of 70m, although this full width may not be necessary.
- If the natural ground between the storage area and the Liesbeek River and that between the storage area and the Liesbeek Dead Arm are structurally unsuitable, a retaining wall may be required, which would be a major additional cost.
- Minor costs would include the outlet structure and pipe, and lining of the pond area (possibly with grass).

Similar costs, approximately in proportion to the area, would be expected for other flood detention areas.

Channel enlargement

Channel enlargement would also involve significant capital costs. These would include widening of the existing bridges across the Black River, as well as excavation, landscaping and vegetation of the channel.

The largest cost would be the widening of the M5 Bridge. This would likely be between the costs estimated by Whittemore and Naidoo (2004) of R13.8 million for the Voortrekker Road Bridge over the Salt River Canal (escalation with 6% per annum making it 29.4 MR in 2017) and the R 21.9 million for the N1 bridges over the Salt River Canal (escalation with 6% per annum making it 46.7 MR in 2017) excluding VAT, P&Gs, Contingencies, and Design Fees. Allowing for inflation, a cost of the order of R 50 to R60 million for widening of the M5 bridge alone seems likely.

Bridges 15 and 17 are pedestrian bridges, similar to the Salt River pedestrian bridge for which Whittemore and Naidoo (2004) estimated a widening cost of R 400 000 excluding VAT, P&Gs, Contingencies, and Design Fees (escalation with 6% per annum making it 0.85 MR in 2017). Allowing for escalation and additional costs, the cost for widening of the two footbridges would likely be of the order of R3 million.

Widening of the N2 interchange ramp bridge, which is right at the upstream end of TRUP could be omitted.

The modelled 25m rectangular widening over a length of 2.14km and an average depth of 3 to 3.5m would require excavation of the order of 170 000m^3. The actual quantities would depend on the channel shape and dimensions adopted. In view of the desire to naturalise the channel, the improvements are unlikely to be concrete-lined, but some alternative lining may be required. Whittemore and Naidoo (2004) estimated a cost of R15,9 million altogether for improvements to the Vygekraal and Black River channels, which were envisaged as the construction of a trapezoidal concrete-lined canal (escalation with 6% per annum making it 33.9 MR in 2017).

High flow diversion of the Elsieskraal River

Diversion of the Elsieskraal River would require a new channel approximately 6.8 km long. For a 2.2km long, 30m wide x 1.5m deep concrete-lined relief canal in the Culemborg area, Whittemore and Naidoo (2004) estimated a rough preliminary cost of R300 million (escalation with 6% per annum making it 640 MR in 2017). This is a similar size to that which would be required for diversion of the Elsieskraal flood flows. Scaling this up for the 6.8km length and allowing for VAT, P&Gs, Contingencies, Escalation and Design Fees, the probable cost would be of the order to three to four billion rand. There is no suitable vacant land to traverse Rugby, and either expropriation of or tunnelling under existing properties here would likely be required.

Several road crossings would be required including the N1, the M5 and the R27, as well as a rail crossing. All of these factors would add to the cost.

(iii) Legal, planning and ownership implications

Stormwater detention

Stormwater detention is a requirement for developments in terms of the City of Cape Town Management of Urban Stormwater Impacts policy (2009b). However, the requirement is to control runoff from the development site itself rather than flows from the upstream stormwater system. Therefore, instead of stormwater detention ponds close to the river, other types of buildings (water roofs) or streets (permeable) could be considered.

Proposed development is planned on the slopes below Maitland Garden Village which is predicted to be flooded around the stormwater pipes during the 1:20 year and larger floods. A stormwater detention pond was therefore proposed in this area.

The City of Cape Town in any case does not prohibit development of areas flooded from the stormwater system, so development here would be permissible without the stormwater detention pond. However, the requirement is for a flood water escape route, which may be:

- "*Public open spaces provided along drainage route;*
- *Underground system capacities upgraded to cater for 50 year storm events.*
- *Flood escape servitudes must be registered over private properties. Servitudes must prohibit all surface development, including construction of solid boundary walls,*" City of Cape Town (2002).

The proposed detention pond SP2 (see Appendix I) would take flow, but space has been allowed around the stormwater pipe between the identified developable areas, which could function as a flood escape route, which may make stormwater detention unnecessary.

The area of SP2 itself is not included as a developable area in the TRUP Scenario 7 development plan. If it is desired to construct this pond, some offset arrangement might be possible where limited storage is provided by the developer for the upstream stormwater instead of detention for local stormwater runoff.

Since this is part of the municipal bulk drainage network, it would be recommended for ownership of the storage pond to be transferred to the City of Cape Town.

Potential storage SP1 is located mainly on a currently grassed corner of the property of M5 office Park. Since this is an existing development, and the pond is for the municipal bulk system rather than for stormwater runoff from the property, the land would most likely need to be expropriated and the pond constructed by the City of Cape Town, in line with the stormwater management planning and design guidelines:

"Responsibility for the operation and maintenance of the stormwater system normally rests with the local authority," (City of Cape Town, 2002).

Channel enlargement

Channel enlargement would be located along and within the same land parcels as the existing canal.

Permitting requirements and implications of channel or bank modification are discussed in the TRUP watercourse management specialist study (de Groen et al. 2017). As noted in this study, so long as the watercourse can be proved to be enhanced and to have a higher level of societal and ecological functionality, without compromising the environment either upstream or downstream, interventions are likely to be supported.

In view of the sensitivity of the western side of the Black River, channel widening would most likely be through the wetland in front of Oude Molen, and further mainly along the eastern side of the existing channel. However, the enlargement would need to extend westwards into the Raapenberg Bird Sanctuary adjacent to the South African Astronomical Observatory, where space on the eastern side is limited by the M5 freeway.

Flood storage

Use of the River Club as a flood storage area would be in direct conflict with the current development plans of the landowners. Recreational use of the area would still be possible with appropriate landscaping. Permitting would need to be integrated into the TRUP Environmental Impact Assessment and Water Use License Application processes.

The management of both the river channel and flood storage areas would need to be integrated with the broader Green Corridor Management Plan being developed as part of this project.

High flow diversion of the Elsieskraal River

High flow diversion of the Elsieskraal River would require significant space along the Wingfield Aerodrome (or possibly along the M25 road) and Ysterplaat Aerodrome, both of which would present constraints related to ownership and the effect on existing infrastructure. As mentioned previously, tunnelling through or expropriation of properties in Rugby would likely be required.

The wetland between Acacia Park and Century City would also be affected, which could be an environmental constraint.

16.3 Recommended flood mitigation measures

All development within the floodplain (and even for some height above the predicted floodplain) should be designed for flood resilience.

Upgrading of the bulk stormwater network in the vicinity of the M5 and Berkley Road interchange is recommended to reduce the frequency of flooding of Berkley Road at the M5 interchange. Construction of a small stormwater detention facility at location SP1 in Figure 38 could form part of this upgrade. The inlet should be designed to accommodate more frequent flood events. A covered underground facility could be considered to allow the grassed cover to remain.

In the previous Scenario 6 development plan, the property at building L3 on the map in Appendix I lies in the overland flow path of the major stormwater drainage system. Thanks to earlier versions of this specialist study, the development plan has been revised so that in Scenario 7, although the buildings in this area are still within the floodplain, space has now been left for a flood water escape route. We therefore feel that a stormwater attenuation pond in this area would have little benefit with regard to flood prevention.

Proposed development as per TRUP development plans will have negligible effect on flood levels and extents. It could be classified as "minor smoothing" of the floodlines as per the Floodplain and River Corridor Management Policy and therefore permitted. However, other effects including ecological and public safety implications also need to be considered as part of the respective precinct plans and green corridor management plan.

Where "nature-friendly" river banks are envisaged, the cross-sectional area of the channel should also be increased to compensate for the higher channel hydraulic roughness. However, general widening, and particularly widening of the M5 bridge is unlikely to have sufficient cost benefit, and would damage sensitive habitats.

Two alternatives have been considered for the River Club Island: either flood storage or infill and development. Flood storage on its own has little effect, therefore should be considered only in combination with storage at the Rondebosch and King David Mowbray golf courses, which has not been modelled. If this storage combination is not envisaged, then limited development of the River Club in line with the TRUP proposals would have little effect on flood levels and flood extents. In either case, as indicated in the botanical study done for this project (Helme, 2016), there is some natural vegetation on the downstream end of the island which may be disturbed,

High flow diversion of the Elsieskraal River, although potentially of benefit, would have prohibitive costs and numerous legal challenges.

16.4 Recommendations for the way forward

The effect of the combined flood storage in the River Club, the King David Mowbray Golf Club and the Rondebosch Golf Club could be evaluated using the hydraulic model. If the results are promising, then a cost-benefit analysis should be undertaken in conjunction with a geotechnical investigation. A preliminary design for the stormwater upgrade at the M5 Berkley Road interchange should be prepared. In case a stormwater detention pond is needed as part of this upgrade, negotiations with the owners of the M5 Office Park could be initiated concerning the construction of a regional stormwater detention facility.

The flood water escape route for the overland flow below Maitland Garden Village should be indicated on the TRUP development plans.

A river corridor management plan is to be developed as part of the Green Corridor Management Plan within this project. Changes to the river channel should be considered at a high level within this plan, in case "nature-friendly banks" are to be developed, or other changes in the river channel and floodplain. The river corridor management plan should also identify further design work required to confirm and detail the selected interventions, particularly the river channel concept.

Permitting requirements for all proposed flood mitigation measures need to be integrated into the TRUP Environmental Impact Assessment and Water Use License Application processes.

The bed levels in the Black River upstream of Raapenberg Bridge were lowered from the SRK model. This is outside the project area and therefore was not part of the survey of this project. In case of further modelling, this should be verified in the field (See Section 5.5).

As explained, we started the project with the recommendation to use another 2D Hydraulic model than quasi 2D PCSWMM (See section 4.1). Many model runs and problems with the software later, we still recommend CCT to not use PCSWMM for any further 2D Hydraulic Models to be made for other catchments, and even for more detailed design of this project area.

Last but not least, while useful flow data were not available for calibration of the models in this specialist study, they will be important for further consideration of interventions in the watercourses in TRUP and in other areas in the same catchment (such as decision on effluent reclamation), for the full season, not just for high flows. It is therefore recommended to CCT to improve on gauges, rating curves, data collection and/or database management, whichever is needed to have a good continuous set of flow gauging data.

References

Berg R, Braune M, van Rensburg J, Howard G, Görgens A, and Shand M (2000), 'Salt River Hydrological Study - Phase 1", CMC project number: WR 3/99; NS project number: 8453; NS report number: 3004/8453 as quoted by City of Cape Town (2002).

City of Cape Town (2002) Stormwater Management Planning and Design Guidelines for New Developments V1.0, July 2002.

City of Cape Town (2009a) Floodplain and River Corridor Management Policy v2.1 No. C58/05/09

City of Cape Town (2009b) Management of Urban Stormwater Impacts policy v1.1 No. C58/05/09

De Groen, M. et al. (2017) *Two Rivers Urban Park specialist study: Watercourse management and creating a docking/waterfront feature: Final Report.* Western Cape Government (WCG): Department of Transport and Public Works (DTPW) Regeneration Programme in partnership with the City of Cape Town (CCT) Development and Facilitation Unit

Fisher-Jeffes, L. N. (2015). *The viability of rainwater and stormwater harvesting in the residential areas of the Liesbeek River Catchment, Cape Town*. PhD dissertation University of Cape Town.

Hejl H.R. Jr. (1977) *A method for adjusting values of Manning's roughness coefficients for flooded urban areas.* U.S. Geological Survey, Journal of Research, v. 5, no. 5, p. 541-545.As quoted by Jarrett (1985).

Helme, N. (2016) *Specialist botanical and ecological scoping phase input: proposed Two Rivers Urban park development framework*, Cape Town. (DRAFT, July 2016, this project)

Jarrett R.D. (1985) *Determination of Roughness Coefficients for Streams in Colorado*. US Geological Survey Water-Resources Investigations Report 85-4004.

Ninham Shand (2004) *Salt River Catchment: Quantification of Flood Risk and evaluation Of Ameliorative Measures: Final Report.* Ninham Shand Report No. 3718/400582, Dec 2004.

Prestedge Retief Dresner Wijnberg (2010) City of Cape Town Climate Change Think Tank Marine/Freshwater Theme: Marine Inputs to Salt River Flood Model. Report No. 1063/1, City of Cape Town.

Schmidt E.J. and Schulze R.E. (2002) *Flood volume and peak discharge from small catchments in Southern Africa, based on the SCS technique*. Water Research Commission Report No. TT31/87, Water Research Commission, Pretoria.

SRK Consulting (2012) Stormwater Infrastructure Asset Management Plan (Phase 2A) Rainfall Analysis and High Level Masterplanning: Salt River Catchment: Final Report: High Level Masterplanning. Report Number 403343/03, City of Cape Town.

USDA-SCS (1972) United States Department of Agriculture - Soil Conservation Service National Engineering Handbook, Section 4: Hydrology. Washington, D.C.

Whittemore, C. and Naidoo, J. (2004) *Salt River Catchment: Quantification of Flood Risk and Evaluation of Ameliorative Measures*. Ninham Shand Report No. 3718/400582, Dec 2004. City Of Cape Town Catchment, Stormwater and River Management Branch.