The Appropriateness of the drainage strategy which underpins the Arden Structure Plan

at

49-51 Henderson Street & 62-70 Gracie Street, North Melbourne

by

Professor Peter Coombes

ABN: 47093645777

70 Howden Street
Carrington NSW 2294

With modelling support

from

Andrew Allan

February 2022

Version 1: 7 February 2022

70 Howden Street Carrington NSW 2294 ▪ Tel: 0419 439 124 ▪ thecoombes@bigpond.com ▪ www.urbanwatercyclesolutions.com

About the Author

Professor Peter Coombes

Peter Coombes is a director of Urban Water Cycle Solutions, an honorary and visiting professor in Crawford School of Public Policy at the Australian National University, a Fellow of Engineers Australia and Certified Practicing Engineer in Civil and Environmental Engineering, Leadership and Management at the Engineering Executive (EngExec) level. He was awarded the 2018 GN Alexander Medal for scientific contributions to Hydrology and Water Resources and the 2019 Presidents Medal for his role as a lead editor of the Urban Book of Australian Rainfall and Runoff. Peter holds a PhD in Civil and Environmental Engineering, and degrees in Civil Engineering (hons) and Surveying (Hons). Peter Coombes has also almost completed a B. Econ/Dip. Law qualification.

Peter was most recently the Associate Dean (Education) and Professor of Water Resources Engineering at Southern Cross University. He is a Member of Systems Research Steering Committee at Imperial College London and is an editor the Urban Book of Australian Rainfall and Runoff published by Engineers Australia. He has held senior academic positions at University of Newcastle, University of Melbourne and Swinburne University. Peter was a Chief Scientist in the Victorian Government and contributed to inquiries into stormwater management and flooding by the Senate of the Australian Parliament and into water resources by the Productivity Commission.

Peter was a managing director of Bonacci Water, a member of the water advisory group to the Prime Ministers Science, Engineering and Innovation Council, the advisory council on alternative water sources for the Victoria Government's Our Water Our Future policy, a member of the advisory panel on urban water resources to the National Water Commission, an advisor on alternative water policy to the United Nations and a national research leader of innovative WSUD strategies in the eWater CRC. He has generated over 250 scientific publications and designed more than 120 sustainable projects including settlements that generate all of their water resources and manage flooding. Professor Coombes was also a co-author of Australian Runoff Quality and a former chair of the Stormwater Industry Association. More information can be found at http://urbanwatercyclesolutions.com

Expert Witness Statement

The above discussion highlights that I have the appropriate qualifications, skills and experience to make this expert witness report. I do not have any private or business relationship with the parties that are the subject of this report.

I have made all of the enquiries that I believe are desirable and appropriate and that no matters of significance which I regard as relevant have to my knowledge been withheld from the Tribunal.

Professor Peter Coombes

7/02/2022

Executive Summary

A. The Arden Macaulay Precinct at North Melbourne includes land owned by RSA Holdings Pty Ltd and Rockford Constant Velocity Pty Ltd within a sub-precinct bounded by Henderson, Gracie, Fogarty and Green Streets. The **sub-precinct** is impacted by the proposed flooding overlays and drainage strategy that underpins the Arden Street Structure Plan and the draft Planning Scheme Amendment C407.

B. The **sub-precinct** has been nominated as a new integrated stormwater open space which changes the status of land owned by RSA Holdings and Rockford Constant Velocity. HWL Ebsworth Lawyers engaged Professor Peter Coombes from Urban Water Cycle Solutions to consider the reports, including expert witness reports, and facts to provide expert evidence on the appropriateness of the drainage strategy and whether the land in the sub-precinct is required for drainage purposes. Urban Water Cycle Solutions engaged Andrew Allan from Afflux Consulting to provide modelling support to this investigation.

C. This report provides an investigation of historical flooding and tide behaviours, the expected climate change impacts on tidal behaviour, sea levels and heavy rainfall, the characteristics of the site, and published reports relied on by the Victorian Planning Authority. This information was used to assess the appropriateness of the predicted flooding impacts and proposed drainage strategy. The consequences of the insights from this investigation were examined using a hydraulic model of the sub-precinct and surrounding areas.

Tides and sea level rise

D. The tidal tailwater levels used in the published modelling were too high due to an assumption that the 1% AEP maximum high tide will occur at the same time as a 1% AEP rain event. This assumption cannot be made because rare rainfall, floods and high tides are independent in Southern Australia. This assumption may have resulted in a 1 in 10,000 year (0.1% AEP) flood event that is outside to the 1 in 100 year (1% AEP) statutory standard required for the planning scheme.

E. The tidal datum at Williamtown is also 0.524 metres higher than the land datum (AHD) at the Arden Macaulay Precinct and this correction may not have been applied to modelling underpinning the proposed planning scheme amendment. These issues have resulted in assumed tailwater levels used in analysis of expected flooding that are 0.6 – 0.8 metres higher than they should be. The best estimate of the tailwater that should be used for existing conditions is 0.62 m AHD and the tailwater level used in the high emissions climate change scenario in 2100 should be 1.29 m AHD.

Historical flooding

F. The Moonee Ponds catchment has experienced strong population growth and continuous additions to stormwater management that has provided ongoing improvements in the severity and intensity of flooding in the North Melbourne area since the 1800s. The time for the peak flow in Moonee Ponds Creek to arrive at North Melbourne was recently observed to be 6 to 18 hours and more intense short duration (15 – 20 minutes) local rain events were seen to apply to the low lying section of Langford Street adjacent to the sub-precinct.

G. Significant historical observations of local flooding (such as the event on 6 March 2010), rainfall depths, tide levels and flows in Moonee Ponds Creek are available. However, there is no reporting of the use of this information to ensure the assumptions in the hydrology and hydraulic models are based on the local reality.

Hydrology

H. Little or no information was provided about the hydrology model of Moonee Ponds Creek that used in the investigations and there is no evidence that selection of critical storm durations and patterns was based on local observations. The selection of a two hour design storm for use in the hydrology and hydraulic models presents as arbitrary and is inconsistent with the observed characteristics of the Moonee Ponds Creek catchment. There is no evidence of the essential industry best practice of using real rainfall and flow data to select the critical storm duration and pattern.

I. The modelling underpinning the various reports is based on the superseded 1987 version of Australian Rainfall and Runoff, and has not used the more valid data and methods underpinning the current 2019 version of Australian Rainfall and Runoff.

J. The 1% AEP peak flows used in the model are significantly higher (18.6%) higher than the observed 1% AEP peak flow at the Mount Alexander Road stream gauge. The assumed climate change multiplier of 18.5% for the 2100 high emission scenario is also higher than the most likely multiplier of 14%.

K. It is possible that the assumed climate change multiplier was applied twice to the historical 1% AEP flows in Moonee Ponds Creek which will produce higher flood levels and a greater need for stormwater drainage solutions.

Hydraulic model and flood depths

L. However, a more severe impact on expected flood levels is the assumption in the models that peak flows from Moonee Ponds Creek arrives at the precinct at the same time as the local peak stormwater runoff and maximum tide levels. This assumption is improbable and has produced significantly higher flood levels that would reasonably be expected and may have profoundly altered the perceived flood dynamics of the Precinct.

M. There are also anomalies in the stormwater flow paths and local accumulation of stormwater in the hydraulic model that adversely effects the estimated flood levels in the sub-precinct. It is difficult to understand that pumps designed to manage flooding are assumed to fail during flood events in the models and the presentation of flood extents of 0 – 300 mm may be misleading.

N. No evidence has been provided that shows the benefits of a flood storage in the sub-precinct. However, upgrades of levees, pumps and pipe drainage networks provides strong benefits. Incorporation of the corrections revealed in this investigation in a model of the precinct with operating pumps provides that properties in the sub-precinct in not heavily impacted by existing flood events.

O. Whilst there is moderate depth of stormwater over short periods on Langford, Gracie and Green Streets, there are limited flooding impacts on the properties within the sub-precinct. Greater flood depths across the precinct are driven by the 1% AEP Moonee Ponds Creek flood which breaches the levees upstream of Macaulay Street which ultimately transfers flood waters towards the sub-precinct at the bottom of this flooding mechanism.

P. The operation of the existing pumps limits the impacts of these processes and these results demonstrate that a flood storage in the sub-precinct would provide minimal stormwater management benefits. However, the proposed upgrades to levees, pumps and pipe drainage infrastructure will mitigate the potential for existing and climate change flood impacts.

Q. Addition of a sub-precinct and building flood emergency warning protocol to the proposed drainage strategy could also avoid any risks that might be posed by rare events (less than 1 in 10 years) whilst maximising the value of land in the sub-precinct on every other day. This strategy will also ensure that the sub-precinct has maximum value and is prepared for the potential future impacts of the high emissions climate change.

Response to expert opinion

R. The report extensively addresses the substance of expert opinion of Warwick Bishop from Water Technology and Paul Clemson from Engeny. This investigation revealed general agreement with aspects of past reports and comments from the expert witnesses but reveals concerns about the detail of assumptions.

S. The Arden Macaulay precinct is subject to flood risks from rare events and proposed drainage strategies include some residual flood risks. This report agrees with comments that incorporation non-structure measures can address residual flood risks and enhance the value of the land in the Precinct.

T. Most of the flooding impacts are limited to streets with private properties largely avoiding significant impacts. The stormwater mitigation opportunities are limited by some excessive assumptions and a need to consider the dynamic nature of flooding. A storage at the sub-precinct will provide little or no benefit.

Table of Contents

1 Introduction

1. HWL Ebsworth engaged Professor Peter Coombes from Urban Water Cycle Solutions to provide expert evidence on the appropriateness of the drainage strategy and whether their land located within the Arden Macaulay Precinct is required for drainage purposes.
2. The Arden Macaulay Precinct at North Melbourne includes land owned by RSA Holdings Pty Ltd and Rockford Constant Velocity Pty Ltd within a **sub-precinct** bounded by Henderson, Gracie, Fogarty and Green Streets.
3. This **sub-precinct** is impacted by the proposed flooding overlays and drainage strategy that underpins the Arden Street Structure Plan and the draft Planning Scheme Amendment C407. The sub-precinct has been nominated as a new integrated stormwater open space which changes the status of land owned by RSA Holdings Pty Ltd and Rockford Constant Velocity Pty Ltd.
4. The extent of the Arden Macaulay Precinct and elements of the proposed drainage strategies including the Integrated Stormwater Management Open Space are presented in Figure 1.
5. This report provides an investigation of the key elements required to understand the suitability of the proposed drainage strategies for managing potential flood risks, including historical flooding and tide behaviours, expected climate change impacts on tidal behaviour, sea levels and heavy rainfall, the characteristics of the site, and published reports relied on by the Victorian Planning Authority.
6. A "simple" hydraulic model was also developed, in partnership with Andrew Allan from Afflux Consulting, using publicly available information and data, and the current version of Australian Rainfall and Runoff to examine implications of the key findings of this investigation.
7. These considerations were used to assess the appropriateness of the predicted flooding impacts and proposed drainage strategy with respect to flooding and requirement for flood storage in the **sub-precinct**.

Figure 1: The Arden Macaulay Precinct with proposed stormwater drainage elements (VPA, 2021)

8. Figure 1 shows the proposed retarding basin (flood storage) in the sub-precinct bounded by Henderson, Gracie, Fogarty and Green Streets that is adjacent to the

pump at the low point in Langford Street, Moonee Ponds Creek and nearby to Arden Street.

9. The following published reports and guidelines were considered during this investigation:

VPA, (2021), Arden Structure Plan, Victorian Planning Authority

VPA, (2021), Arden Precinct Background report, Victorian Planning Authority

Engeny, (2021), Arden Macaulay Precinct Flood Management Strategy, Report for Melbourne Water

IPCC (2021), AR6 Climate Change 2021: The Physical Science Basis, Chapter 9: Ocean, cryosphere and sea level change, Intergovernmental Panel on Climate Change.

GHD, (2020), Modelling Assumptions & Implications, Memorandum to Melbourne Water Corporation, 30 July 2020.

Engeny, (2020), Arden Macaulay Precinct & Moonee Ponds Creek Flood Modelling, Model Build Report, Report for Melbourne Water and City of Melbourne.

CSIRO and Bureau of Meteorology (2020), Climate Change in Australia, Projections for Australia's NRM Regions. Technical Report, CSIRO and Bureau of Meteorology, Australia.

DELWP, (2019), Guidelines for development in flood affected areas, The State of Victoria Department of Environment, Land, Water and Planning.

Engeny, (2019), Arden Macaulay Precinct - Langford St Flood Storage Investigation, Report for Melbourne Water and City of Melbourne.

Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors), (2019), Australian Rainfall and Runoff: A Guide to Flood Estimation, © Commonwealth of Australia (Geoscience Australia)

Coombes, P., and Roso, S. (Editors), (2019), Runoff in Urban Areas, Book 9 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia, © Commonwealth of Australia (Geoscience Australia), 2019.

Engeny, (2017), Arden Macaulay Precinct Drainage Investigation. Revision 2, 7 February Report for Melbourne Water and City of Melbourne.

Engeny, (2016), Arden Macaulay Precinct stages 1 and 2. Revision 0, 29 February Report for Melbourne Water and Metropolitan Planning Authority.

Engeny, (2016), Melbourne Water Metropolitan Planning Authority Arden Macaulay Precinct stage 1. Version 0, 26 January Report for Melbourne Water

AECOM, (2013), Hydrologic and Hydraulic Modelling of Arden Street and E-Gate, Department of Transport, Planning and Local Infrastructure Victoria.

SES, (2012), City of Melbourne Flood Emergency Plan, State Emergency Service, Victorian Government

Pilgrim, D, H., (1987), Australian Rainfall and Runoff. A guide to flood estimation. Volume 1. Engineers Australia.

The following expert witness reports are also considered in this report:

Bishop, W., (2022), Expert Opinion - Flooding and Drainage, Amendment C407 to the Melbourne Planning Scheme, Water Technology

Clemson, P., (2022). Planning Panels Victoria, Expert Witness Statement requested by Harwood Andrews, Engeny.

2 Discussion

2.1 The local environment

10. The sub-precinct includes low lying areas near Moonee Ponds Creek that is expected to experience flooding from 5% and 1% AEP events, and is subject to a Land Subject to Inundation planning classification.[1] In particular, there is a low point at 1.2 m AHD near the corner of Langford and Gracie Streets in North Melbourne, and Langford, Gracie and Green streets have significantly lower elevations than the surrounding properties as shown in Figure 2.

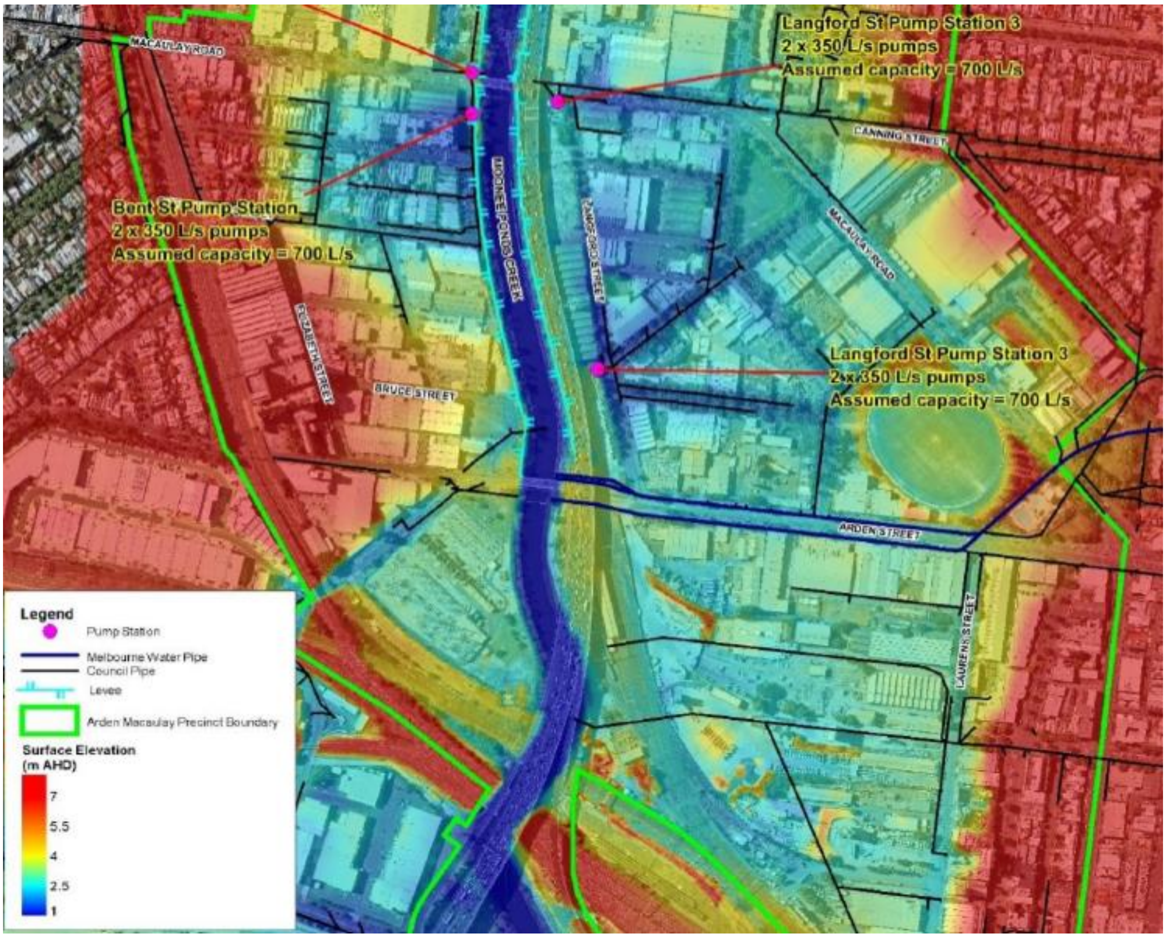


Figure 2: Land surface elevations and existing drainage infrastructure after Engeny (2016)

11. Figure 2 shows that the Arden Macaulay Precinct currently includes are range of stormwater management infrastructure such as pump stations near Langford Street, flood levees adjacent to Moonee Ponds Creek and pipe drainage networks.
12. There is limited discussion of historical flooding in the analysis and reporting of existing flooding underpinning the proposed drainage strategy for the Arden Macaulay Precinct. A picture of local flooding at Langford Street from 7 March

[1] Engeny, (2016), Arden Macaulay Precinct stages 1 and 2. Revision 0, 29 February Report for Melbourne Water and Metropolitan Planning Authority

2010 (this investigation established that the event occurred on 6 March 2010) is provided but no other records of flooding is presented for the precinct. There is little or no discussion of historical flooding and the discussions about existing conditions is limited to results from models.

13. An overview of the flood history and solutions is provided by Melbourne and Metropolitan Board of Works (Leight, 1981)[2] and the State Emergency Services (SES, 2012).[3] The history of the North Melbourne area includes strong population growth, filling of the West Melbourne "swamp", building embankments and channelisation of Moonee Ponds Creek. During the 1800s and early 1900s the area, now known as the Arden Macaulay Precinct, experienced flooding from local stormwater runoff and by overtopping of the embankments by flows in Moonee Ponds Creek.
14. Drainage mitigation measures of creating a channel and embankments for Moonee Ponds Creek were reported to decrease the severity and frequency of flooding. However, these solutions increased the likelihood of local flooding during high flow events in the Moonee Ponds creek channel because the local catchments could not discharge accumulated stormwater to the creek.
15. Inclusion of drains, catchment storages (including the Jacana retarding basin), levees, and pumps was observed to further mitigate the impacts of catchment and local flooding in the Moonee Ponds Creek catchment. More recently, the SES (2012) reported that peak flood flows in Moonee Ponds Creek take 6 – 18 hours to reach North Melbourne which can also impacted by stormwater runoff from intense short duration local rain events.
16. There are no recent reports of flooding impacts on the sub-precinct originating from Moonee Ponds Creek flows. However, SES (2012) report that intense short duration rainfall has sometimes created local flood inundation in the sub-precinct with stormwater depths up to 1.2 m at the low point in Langford Street.

Existing conditions summary

17. The local environment within and surrounding the sub-precinct has been shaped by population growth and responses to flooding from the Moonee Ponds and local urban catchments. The flooding impacts on the relatively low lying area close to Moonee Ponds Creek have diminished throughout history in response to a range of structural (such as drainage infrastructure) and non-structural (for example, flood emergency planning and land use policy) solutions.

2.2 Tides and sea levels

[2] Leight C. H., (1981), Development of Moonee Ponds Creek drainage system, Melbourne and Metropolitan Board of Works.

[3] SES, (2012), City of Melbourne Flood Emergency Plan, State Emergency Service, Victorian Government

18. The Arden Macaulay Precinct is adjacent to Moonee Ponds creek and is 2.1 km from the confluence of Moonee Ponds creek and the Yarra River as shown in Figure 3. It is located 7.7 km from the confluence of the Yarra River and Port Phillip Bay and about 10 km from Melbourne Williamstown tide gauge.

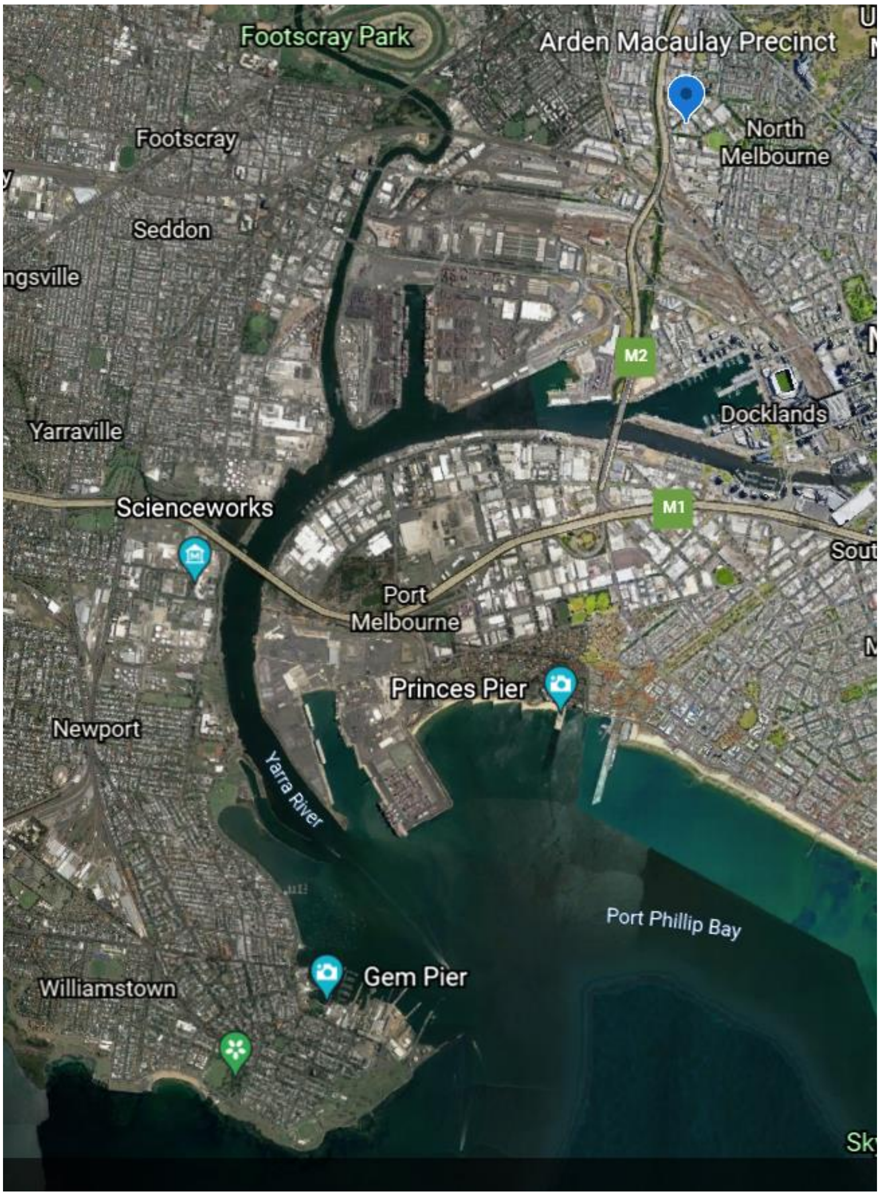


Figure 3: The Arden Macaulay Precinct, Yarra River, Port Phillip Bay and the Williamstown tide gauge

19. Tidal behaviours at the Melbourne Williamstown tide gauge define the water levels in Port Phillip Bay that act as tailwater levels in flood events that influence flood levels at the Arden Macaulay Precinct. The maximum, mean and minimum monthly tides recorded by the Victorian Regional Channels Authority (VRCA) and are available from the Bureau of Meteorology (BOM).

20. The Engeny (2020) Model Build Report assumes a cyclic tide with a 10% AEP (Annual Exceedence Probability) peak tide level of 1.975 m AHD (Australian

Height Datum) will occur during a rare flood event (1% AEP) which includes an assumed 0.8 metre sea level rise occuring in 2100.[4]

21. These assumptions were investigated by examination of the tidal behaviour at the nearby Williamstown tide gauge, advice provided by the Australian Rainfall and Runoff (ARR2019) 2019 guidelines and the sea level rise predictions by the Intergovernmental Panel on Climate Change (IPCC).[5] VRCA provides information about the astronomical tides and the relationship between the Chart Datum used for tides and the Australian Height Datum (AHD) used for land surfaces as shown in Table 1.[6]

Table 1: Tide heights at Williamstown gauge versus the Australian Height Datum

AHD (metres)	**Predominately diurnal tides**	**Chart Datum (metres)**
1.16	Highest recorded tide 24/06/2014	1.64
0.52	Highest astronomical tide (HAT)	1.04
0.42	Mean higher high water (MHHW)	0.94
0.00	**Australian Height Datum**	**0.524**
0.48	Lowest astonomical tide (LAT)	0.046

22. Table 1 provides some important considerations for this investigation, including that the chart datum used for tides is 0.524 metres higher than the Australian Height Datum, and the range of diurnal tides includes two high and low tides in each day. It is noteworthy that the highest tide only occurs over a six hour period on a given day in any year, and there are a range of lower tide heights at other times and days.

23. The Engeny (2020) report refers to the assumption of a 10% AEP high tide of 1.22 metres AHD with 0.8 metres sea level rise made in the AECOM (2013) report.[7] More recently, the GHD (2020) Memorandum expressed concern about selection of tidal tailwater levels that were too high with respect to Melbourne Water Corporation (MWC)'s designated flood levels.[8] The Engeny reports (2016, 2017, 2020, 2021) also refer to the use of a 1.4 m AHD tidal tailwater level (an

[4] Engeny (2020), Arden Macaulay Precinct & Moonee Ponds Creek Flood Modelling, Model Build Report, Report for Melbourne Water and City of Melbourne.

[5] IPCC (2021), AR6 Climate Change 2021: The Physical Science Basis, Chapter 9: Ocean, cryosphere and sea level change, Intergovernmental Panel on Climate Change.

[6] VRCA (2020), Vic Tides 2020, Edition 4, Victorian Regional Channels Authority

[7] AECOM (2013), Hydrologic and Hydraulic Modelling of Arden Street and E-Gate, Department of Transport, Planning and Local Infrastructure Victoria.

[8] GHD (2020), Modelling Assumptions & Implications, Memorandum to Melbourne Water Corporation, 30 July 2020.

increase from the previous assumption of 1.22 m AHD) in the analysis of flooding during existing conditions that was requested by MWC.[9] This tailwater assumption is substantially higher than all of the historical tidal observations provided in Table 1.

24. The GHD advice to MWC also expressed concerns about the unlikely joint probability of high tide levels occuring that the same time at rare rainfall generated floods. This follows a sequence of emails between GHD and MWC during March and April 2019 where MWC advise that the results from a different study (Skye Karingal project and Water Technology curves) that aligns 1%, 5% and 20% AEP rainfall events and tidal cycles with the same exceedence probabilities.
25. These probabilities of annual maximum tidal levels were investigated using monthly data for the years 1966 to 2020 sourced from the tide gauge 60780 Melbourne (Williamtown) provided by the BOM.[10] This data was corrected from the chart datum to the required AHD levels and the AEPs were derived from frequency an analysis using the TUFLOW FLIKE software. The results for the AEPs derived from Williamtown tide gauge are compared to the assumptions from the various studies in Table 2.

Table 2: Comparison between Annual Exceedance Probability (AEP) of maximum sea levels at the Williamstown tide gauge and assumed values in various reports

AEP (%), [1 in Y (year)]	Gauge AHD (m)	GHD AHD (m)	AECOM AHD (m)	Engeny AHD (m)	Water Technology AHD (m)
100, (1)*	0.62				
20, (5)	0.97	1.1			1.05
10, (10)	1.01		1.22	1.175	1.15
5, (20)	1.05	1.25			1.25
1,(10)	1.12	1.6		1.4	1.4

*This value is actually 1 in 1.01 years which is 99.75% AEP.

26. Table 2 reveals that the AEPs of the assumed values for maximum tide levels from various reports are significantly higher (0.13 – 0.48 metres) than the results derived from the Williamtown tide gauge. It is significant that the difference between the assumed values from various reports and the maximum gauge observations increases with rarer exceedence probabilities and it is likely that the 1.6 metre tide level for the 1% AEP tide has not been corrected to the Australian Height Datum (see Table 1).

[9] Ibid n4; Engeny, (2021), Arden Macaulay Precinct Flood Management Strategy, Report for Melbourne Water; Engeny, (2017), Arden Macaulay Precinct Drainage Investigation. Revision 2, 7 February Report for Melbourne Water and City of Melbourne; Ibid n1.

[10] BOM (2021), 60780 Melbourne (Williamtown) tide data

27. It is noteworthy that the assumed 1% AEP tide level is 0.48 metres higher than the level derived from the Williamstown gauge and the 10% AEP tide level assumed in the current flood studies is 0.165 metres higher than the levels derived from the gauge.
28. The source of the assumed 1.4 m AHD tidal level used for analysis of existing conditions is reported as Water Technology by GHD (2020) but this value is substantially higher than any recorded value and was apparently an estimate of the 1% AEP maximum tide level. Indeed this value is 0.78 metres higher the highest tide level that is likely to occur in any year (1 in 1 year event) and this value may not have been corrected from tide datum to the land datum.
29. However, as questioned by GHD (2020), is it valid to assume a strong joint probability of rare rainfall events and maximum tide levels that indicates a high level of dependence between these events?
30. This issue was examined by the Australian Rainfall and Runoff revision projects and a (in)dependence value of 0.98 was found for storm durations less than 12 hours and maximum tide levels at the Victorian coast near Melbourne (a value of 1 means completely independent and 0 indicates full dependence between rare storms and tides).[11] Dependence weakens further with decreasing storm duration.
31. The relationship between rare storm events and peak tides is almost independent. Thus rare high tides do not coincide with rare rain events and such an alignment is further unlikely for the chosen two hour design storm.
32. This high level of independence was found to be higher for shorter storm durations such as the two hour design storm that was assumed in the flooding investigations of the Arden Macaulay Precinct. The 1% AEP storm event creating flooding is independent of rare maximum tides at Williamtown and it cannot be reasonably assumed that these events occur together at the same statutory exceedence probability.
33. This situation of statistical independence between rare stormwater events with short durations and maximum tides is more suited to use of the total probability theorum rather than assuming rare high tides occur at the same time as rare rain events that generate flooding.
34. Using total probability theorum, the Engeny (2020) assumption of 10% AEP maximum tides coinciding with 1% AEP design storm events actually provides a 10% times 1% = 0.1% AEP (1 in 1000 year) design outcome.
35. Given this consideration, the 100% AEP (1 in 1 year) maximum tide of 0.62 metres should be used in this flood study as it expected to contribute to the

[11] Zheng, F., Westra, S., and Leonard, M., (2014), Coincidence of Fluvial Flooding Events and Coastal Water Levels in Estuarine Areas. Stage 3 Report, Australian Rainfall and Runoff. Engineers Australia.

statutory 1% AEP flooding outcome. In accordance with total probability theorum: 100% (tide) times 1% (storm) = 1% AEP flooding outcome.

36. This result is consistent with the GHD (2020) observation that a 0.6 metre tidal tailwater assumption provides a better match to MWC's designated flood levels in the Yarra River.[12] A flood event can occur during any tide conditions but is more likely to coincide with more frequent lower high tides (note that the mean higher high water value is 0.42 metres in Table 1).

37. The Engeny (2020) tidal tailwater assumption is likely to be 0.555 metres higher than the level required for the 1% AEP statutory flood level under current climate conditions.

38. The NASA sea level prediction tool provided by IPCC (2021) sixth report was utilised to determine the most likely sea level rise for high emissions scenarios in 2100 at Stony Point near Melbourne as shown in Figure 4.[13]

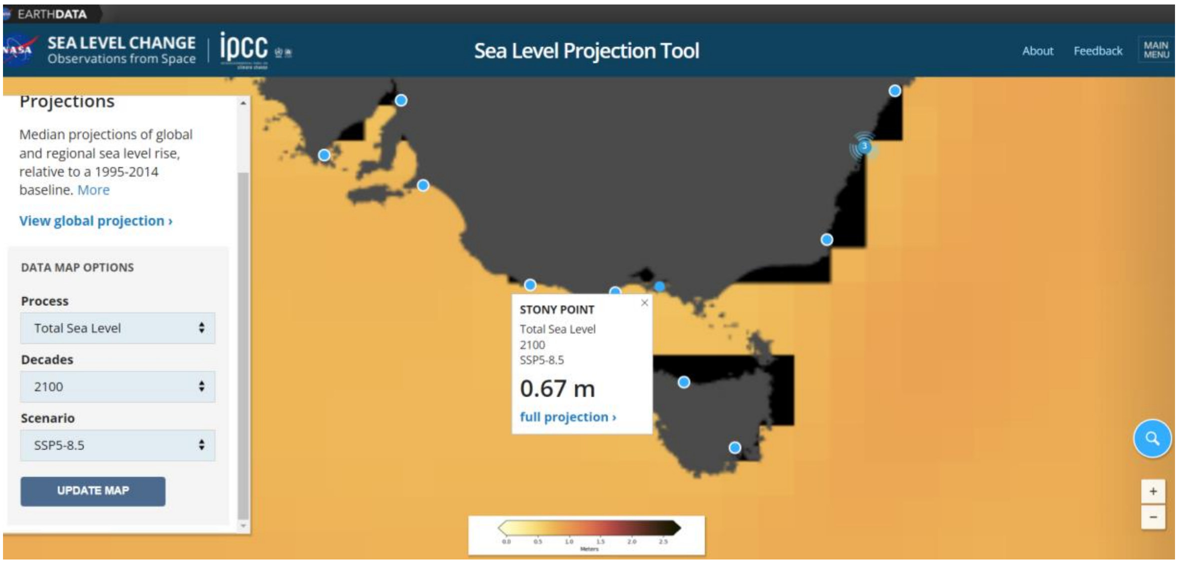


Figure 4: Likely sea level rise at Stony Point near Melbourne in response to high emissions scenarios.

39. Figure 4 reveals that the NASA climate change tool estimates the likely sea level rise at Stony Point near Melbourne as 0.67 metres in response to the high emissions climate change scenarios in 2100. This result indicates that the flood modelling should use a maximum tide level of 0.62 + 0.67 = 1.29 metres for the 2100 climate change conditions.

40. This is 0.685 metres lower that the 1.975 metres AHD value assumed in by Engeny (2020).

41. Clarification of the tidal conditions that are likely to occur at the time of a 1% AEP flood event under current and future climate change conditions indicate that

---

[12] Ibid n8

[13] Ibid n5, NASA Sea Level Rise Prediction tool

the **sub-precinct** properties bounded by Gracie, Henderson and Green streets in the Arden Macaulay Precinct may not to be unacceptably inundated by flood waters.

Tide and Sea Level Summary

42. The information and data about tidal behaviours and likely sea level rise in response to expected high emissions climate change scenarios reveals that the Engeny (2020) report may have over-estimated tailwater conditions used in flood modelling of existing and future scenarios. Utilisation of best practice and current knowledge from Australian Rainfall and Runoff 2019 shows that rare rain events creating flooding are not dependent on rare high tide events.
43. It is not valid to assume a maximum tide height (say 1% AEP) would coincide with a 1% AEP flood event as this creates a risk probability that is excessive in comparison to the statutory 1 % AEP policy for flooding.
44. These considerations indicate that the 1% AEP flood levels in the Arden Macaulay Precinct should be 0.6 – 0.8 metres lower than presented in the Engeny (2021) report. The properties in the sub-precinct many not be significantly inundated by flood waters from Moonee Ponds Creek.

2.3 Hydrology

45. The hydrology of the Moonee Ponds Catchment (about 139 km^2) has been estimated by multiple consultants using a "Moonee Ponds Creek RORB Model" provided by MWC.[14] It is important to clarify that a hydrology model was built using the RORB software and used to estimate the hydrology of Moonee Ponds Creek.
46. This model of the hydrology of Moonee Ponds Creek is reported by Engeny (2020) to include 17 sub-catchments and includes the Jacana Retarding Basin (2850 ML storage capacity) at the outlet of the upper catchment.
47. The model and reports describing the inputs and assumptions in the model are not available for consideration. Importantly, no information is provided about the calibration of this model to observed data from Moonee Ponds creek gauges which is an essential process for determining the critical storm duration and pattern. The RORB software is freely available but the model of Moonee Ponds Creek hydrology is not available for consideration.
48. The attributes of the Moonee Ponds Catchments provided by Engeny (2020) and collated from other public sources are summarised in Table 3. This information was also used to estimate the time of concentration (using the Bransby Williams formula) of catchment runoff to the outlet.

[14] Ibid n1, n4

Table 3: Attributes of the Moonee Ponds creek catchment upstream of the Mount Alexander Road gauge at North Melbourne

Catchment	Area (km^2)	Length (km)	Grade (m/km)	Fraction imperviousness	Time of concentration (minutes)
Upper	89	7	0.005	0 – 0.5	1350
Middle	39	12.6	0.009	0.5 – 0.55	720
Total to Mt Alexander Road gauge	128	19.6	0.006	0 – 0.55	1920

49. Table 3 reveals that the upper catchment of 89 km^2 includes rural and urban surfaces which discharge to the Jacana retarding basin at a time of concentration of approximately 1320 minutes (22 hours). The substantial volume in the Jacana basin will serve to buffer the changes in runoff from the upper catchment created by emerging urban development. The middle catchment of 39 km^2 is mostly developed and discharges to the Mount Alexander Road outlet at a time of concentration of 720 minutes.
50. The estimated time of concentration for the combined upper and middle catchments is 1920 minutes. Given that the Jacana Basin will delay runoff from the upper catchment, it is expected that longer storm durations will create larger runoff events.
51. The Moonee Ponds Creek RORB model was utilised by Engeny to derive the flows from the upstream catchment into the TUFLOW hydraulic model (9.4 km^2) used to estimate flooding within the Arden Macaulay Precinct.[15] The hydrology within the Precinct is estimated using a different local RORB hydrology model which is described in the reports by AECOM (2013)[16] and Engeny (2020)[17].
52. This local hydrology model of the 9.4 km^2 area includes 138 sub-catchments with areas ranging from 2 ha to 21 ha. The large scale nature of the sub-catchment assumptions may not be appropriate as inputs to define local flooding in parts of the hydraulic model.
53. The two RORB models used to estimate the hydrology of Moonee Ponds Creek and within the Precinct are based on superseded methods and design rainfalls from the 1987 version of Australian Rainfall and Runoff (ARR1987).[18] The ARR1987 guideline highlights in Chapter 1 that the latest science, data and guidance must be used in evaluation of flooding impacts:

[15] Ibid n1, n4,
[16] Ibid, n7
[17] Ibid, n4
[18] Pilgrim, D, H., (1987), Australian Rainfall and Runoff. A guide to flood estimation. Volume 1. Engineers Australia. Canberra.

54. *The use of new or improved procedures is encouraged, especially where these are more appropriate than the methods described in this publication. It is certain that within the effective life of the document, new procedures and design information will be developed.*[19]

55. The current national guidelines for estimation of flooding, Australian Rainfall and Runoff 2019 (ARR2019)[20], should be used in flood modelling to provide valid flooding results. It is noteworthy that the majority of the Australian Rainfall and Runoff guidelines, methods and data was released in 2016 (referred to as ARR2016) and was available for industry use throughout the analysis underpinning the Arden Macaulay Precinct.

56. These latest ARR2019 guidelines provide best practice methods and data that are vastly improved from the ARR1987 methods and data. For example, the ARR2019 guidelines (available in 2016) provide design storm intensities and patterns based on observations whereas ARR1987 approaches includes artificial storm bursts and patterns.[21]

57. The RORB hydrology models are described by Engeny (2020) as calibrated to Rational Method estimates. However, Rational Method estimates are based on the same assumptions used in the RORB models and this process cannot realistically be described as a calibration or verification of the hydrology models.

58. Calibration of the Moonee Ponds Creek RORB hydrology model, that includes assumptions used in the Rational Method, to peak flows estimated using Rational Method assumptions presents as a circular process and may not be ideal. As described by Goyen et al (2014)[22], in the ARR 2019 Revision Project, estimates of peak flows using the Rational Method are dependent on arbitrary values of runoff coefficient and times of concentration that do not account for catchment storages which are common to urban areas. These calculations are mostly dependent on engineering judgment and intuition.

59. Peak flows estimated using the Rational Method should not be used to calibrate hydrology and hydraulic models used in urban areas.[23] It is preferable to calibrate behaviours of hydrology models using observations of flows from gauged catchments.

---

[19] Ibid n18, Chapter 1, page 1, [6]

[20] Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors), 2019, Australian Rainfall and Runoff: A Guide to Flood Estimation, © Commonwealth of Australia (Geoscience Australia)

[21] Coombes, P., and Roso, S. (Editors), 2019 Runoff in Urban Areas, Book 9 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia, © Commonwealth of Australia (Geoscience Australia), 2019.

[22] Goyen A., Phillips B., and Pathiraja S., (2014), Rational Method developments. Urban Rational Method review. ARR 2019 Revision Project 13 Stage 3, Engineers Australia, National Committee on Water Engineering, Barton.

[23] Ibid n22

60. There are two flow gauges on Moonee Ponds Creek at Mount Alexander Road in North Melbourne and at the Jacana Basin. A flood frequency analysis was completed using TUFLOW FLIKE software on the observed flows in Moonee Ponds creek to obtain the 1% AEP (Annual Exceedance Probability) peak flows at Jacana Basin and at Mount Alexander Road. These results are compared to the published reports in Table 4.

Table 4: Results of flood frequency analysis of the BOM gauge flowrates versus RORB model outcomes in previous reports

Location	Peak flowrate (m^3/s) at 1% AEP			Difference (%)
	Gauge	AECOM	Engeny	
Jacana Basin	106	-	-	-
Mt Alexander Road	183	207	217	+18.6%

61. Table 4 reveals that the 1% AEP peak flowrates reported by Engeny (2020) are 18.6% higher that the observed values at the Mount Alexander Road gauge. This result indicates that the inflows to hydraulic model of the Arden Macaulay Precinct reported by AECOM (2013) and Engeny (2020; 2021) at Mount Alexander Road may be over-estimated by 18.6%.
62. The Engeny (2021) report also refers to a 1% AEP peak flow of 209 m^3/s at the intersection of Moonee Ponds Creek and Racecourse Road.[24] This location is near the Mount Alexander Road gauge on Moonee Ponds Creek and is also higher than the 1% AEP peak flow based on observed data.
63. Rainfall and streamflow gauge records were examined to understand the dynamics of hydrology in the Moonee Ponds creek catchment. Data from the Mount Alexander gauge on Moonee Ponds Creek reveals a runoff event in 2005 (2-3 February) that provided similar peak flows (185 m^3/s) to the 1% AEP peak flowrate derived from the flood frequency analysis. This information was combined with rainfall from the rain gauge at Melbourne Airport to investigate the characteristics of the runoff event that produces 1% AEP peak flows in Figure 5.

[24] Engeny, (2021), Arden Macaulay Precinct Flood Management Strategy, Report for Melbourne Water

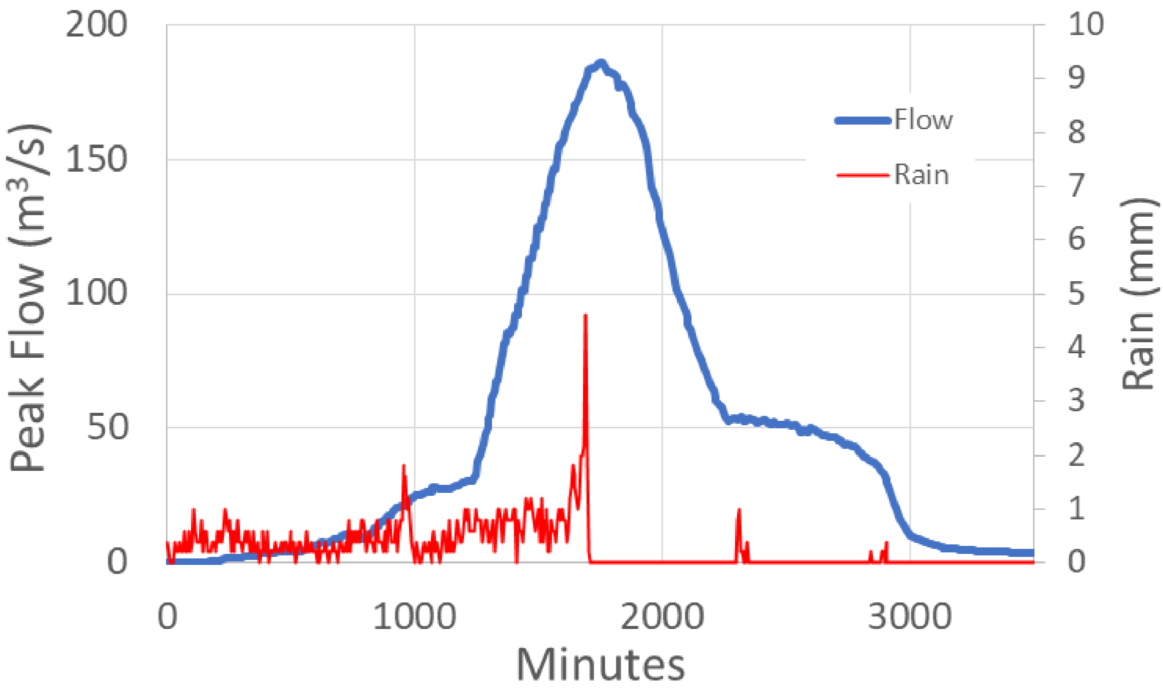


Figure 5: Characteristics of a historical 1% AEP runoff event in Moonee Ponds creek

64. Figure 5 reveals some important considerations about the hydrology of Moonee Ponds creek. The 1% AEP runoff event was created by 157 mm rainfall over a duration of 1690 minutes, and the time to peak flowrate was 1770 minutes.
65. The maximum rainfall intensity within the rain event was 46 mm/hour and the rainfall was a 1% AEP rain event at 18 (1080 minutes) and 24 (1440 minutes) hour durations in accordance with Australian Rainfall and Runoff (2019) definitions. The maximum depth of flows in Moonee Ponds Creek of 2.61 m was substantially less than the height of the levee (greater than 3.4 m) which indicates that Moonee Ponds Creek might not have overflowed into the precinct.
66. However, if the local pipe drainage outlets did not include one way flow arrangements (such as stop valves or gates) it is possible for water from Moonee Ponds Creek to enter the sub-precinct via stormwater pipe drainage infrastructure. Also the limits of the extent of the levees might have permitted bypass of the levees by flood waters.
67. The time to peak and rainfall duration that delivered the historical 1% AEP runoff event are consistent with estimates of the time of concentration in Table 3 of about 1920 minutes to the Mount Alexander Road gauge. These historical results indicate that that Moonee Ponds creek has a volume sensitive response to rainfall inputs and the response of the catchment is not limited to peak rainfall considerations.
68. The long time to peak also implies that runoff from smaller downstream catchments will discharge to Moonee Ponds creek prior to the arrival of the larger flows from the upper catchment. This insight is consistent with discussions by

the SES (2012) and is further highlighted by the peak flowrate arriving at the Mount Alexander Road gauge 180 minutes after the end of the rainfall event that had a duration of 1692 minutes.

69. However, Engeny (2020; 2021) and AECOM (2013) have reported that a design rainfall with a critical duration of 120 minutes was chosen for Moonee Ponds creek catchment by MWC. Engeny (2020) comment that the chosen critical duration as relatively short for this type of catchment and previously commented in Engeny (2016) that impacts of three different durations (15 minutes, 2 hours and 9 hours) was combined in the presented results.

70. This historical comment highlights that volume sensitive nature of the catchments and the need for calibration to observed events. The hydrology and hydraulic modelling for Moonee Ponds creek and the Arden Macaulay Precinct relied on a design storm with a two hour duration sourced from Australian Rainfall and Runoff 1987 to predict flooding. Two hour design storms are also known to contain unusually high internal rainfall intensities that are a product of their construction.[25]

71. Historical annual maximum peak flows in Moonee Ponds Creek at the Mount Alexander Road gauge were examined to locate the impacts of rain events with two hour durations. The highest recorded peak flow from a two hour rain event occurred in 1995 (5 January) and is shown in Figure 6.

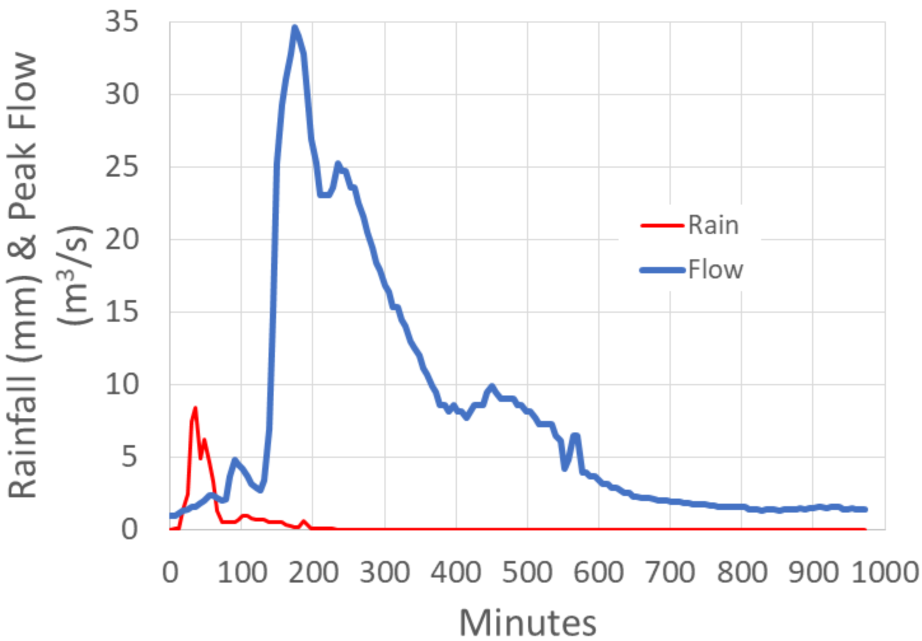


Figure 6: Maximum peak flows in response to a two hour rain event in Moonee Ponds Creek at Mount Alexander Road

[25] Retallick, M., Babister, M., Varga, I., Ball, J., and Askew. E., (2009), Do filtered temporal patterns resemble real patterns? 32nd Hydrology and Water Resources Symposium, Engineers Australia, Newcastle, Australia

72. Figure 6 demonstrates the response of the Moonee Ponds catchment to 47.24 mm of rainfall over a 120 minutes duration. The rainfall included a maximum rainfall intensity of 84 mm/hour and was classified as 1 in 35 year (3% AEP) rain event based on ARR2019 methods.
73. This rain event generated a peak flow of 34.9 m^3/s and it is difficult to consider that a 1% AEP rain event with a two hour duration will generate a peak flow of 217 m^3/s in Moonee Ponds Creek as described in the Engeny (2020; 2021) reports. The attributes of the 1% AEP two hour design storm for ARR2019 and ARR1987 are compared in Table 5.

Table 5: Attributes of two hour design storms from ARR2019 and ARR1987

Version	Storm Depth (mm)	Maximum peak rainfall rate (mm/hr	Number of storm events
ARR2019	59.6	65.58 – 146.69	10
ARR1987	61.74	123.7	1

74. Table 5 shows that the total rain depth in the two hour storm derived using the superseded ARR1987 guidance is slightly higher than the design storm from the current version of Australian Rainfall and Runoff (ARR2019). The maximum peak rainfall rate within the storm of 123.7 mm/hour is also within the range of the peak rainfall rates from the ARR2019 methods.
75. However, any similarity between the superseded ARR1987 and current ARR2019 design storm events must be considered in the context of the recommended method of applying these design rainfalls.
76. The ARR2019 method involves application of the ensembles of ten design storm patterns to select the average response and the ARR1987 process requires use of a single design storm pattern.
77. Whilst a sole focus on peak rainfall is also problematic for investigation of urban flooding due to volume effects, the maximum peak rainfall rates from an ensemble of ARR2019 design storms is compared to the single design storm (1987 AVM) and the average of the ARR2019 peak rainfalls in Figure 7.

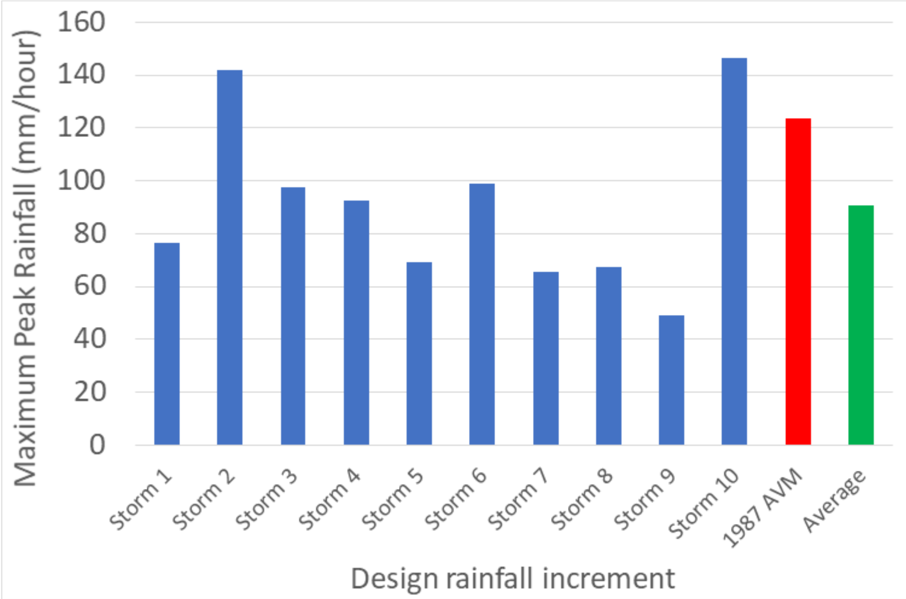


Figure 7: Maximum rainfall rates within design storm events with two hour durations for ARR1987 and ARR2019

78. Figure 7 demonstrates that the ensemble of ten design storm patterns from ARR2019 involve a wide range of maximum rainfall rates for a 5 minute increment versus of single maximum rate provided by the ARR1987 design storm.[26] Importantly, the ARR2019 methods involves choosing the storm pattern that produces to average response for the critical storm duration. This implies that average maximum rainfall rate of 90 mm/hour applies and this is considerably less that maximum rainfall rate of 123.7 mm/hour from ARR1987.
79. A further insight from Figure 7 is that storm pattern 4 might provide the critical temporal pattern for this location. This discussion highlights the vitally important consideration of the need for best practice methods to establish critical design storm durations and temporal patterns to understand flood risks in urban catchments with volume sensitive characteristics. This is done by choosing the critical duration and temporal patterns of design storms based on changes in flood elevations with verification by observed flood depths – not by selection of maximum peak rainfall and runoff.[27]
80. The absence of a current best practice approaches leaves open the need to determine the critical design storms and flood responses in the Arden Macaulay Precinct. This insight is underlined by comparison between the historical two hour storm (rain depth: 47.2 mm, peak rainfall rate: 84 mm/hour) that produces a peak runoff of 34.9 m^3/s (Figure 5) versus the assumed ARR1987 two hour design storm (rain depth: 61.7 mm, peak rainfall rate: 123.7 mm/hour) (Table 5) that is reported to produce a peak runoff of 217 m^3/s.

[26] Ibid n21, Chapter 6 Modelling Approaches, Figure 9.6.3
[27] Ibid n21, Chapter 6, Modelling Approaches, Figures 9.6.11 and 9.6.11

81. It seems unlikely that relatively small increases in rainfall depth (31%) and peak rainfall (47%) would produce more than six five times the peak runoff (217 m^3/s versus 34.9 m^3/s) for a two hour rain event that is reported for the Moonee Ponds Creek catchment at Mount Alexander Road.

82. These historical modelling investigations have also assumed that the local peak flooding, peak flows from Moonee Ponds Creek and peak high tides occur at the same time. The results in Figure 5 for hydrology and the discussion in the Tides and Sea Level section indicate that this assumption is unlikely.

Climate change

83. Flood studies by Engeny (2020; 2021) and AECOM (2013) apply increases of 18.5% to rainfall intensity in 2100 to account for climate change impacts. The assumed climate change multiplier of peak rainfall was 15.5% in the Engeny (2016; 2017) reports. The ARR2019 guidelines provide a methodology and the following equation for translating increases in expected average temperatures associated with climate change into increased rainfall depths[28]:

84. $$I_{p= I_{ARR}} 1.05^{Tm} \qquad (1)$$

where I_{ARR} is the rainfall depth from ARR2019 and T_m is medium temperature range.

85. Temperature ranges in 2090 for the RCP8.5 high emissions scenarios were sourced from the climate futures tool provided by the Australian government.[29] These results were also compared to the predictions provided in the sixth assessment by the Intergovernmental Panel on Climate Change (IPCC, 2021).[30]

86. For the high emissions RCP8.5 in 2090 scenario for the Southern Slopes region, the Climate Change Tool provides that 20 models predict increases in average temperature of 1.5ºC – 3ºC and 28 models predict increases of greater than 3ºC.

87. However, both the Climate Change Tool and the IPCC (2021) highlight that there is no model consensus for increases in heavy rainfall for Southern Australia or the Southern Slopes region that is relevant to the Precinct. There is no formal consensus on future increases in rainfall intensity in Southern Australia. Nevertheless, there is some evidence in peer reviewed research publications of a historical relationship between increases in maximum temperatures and higher rainfall intensity.[31] Local urban catchments can be subject to heat island effects

---

[28]Ibid n18, Chapter 6. Climate Change Considerations.

[29] CSIRO and Bureau of Meteorology (2020), Climate Change in Australia, Projections for Australia's NRM Regions. Technical Report, CSIRO and Bureau of Meteorology, Australia. Retrieved from www.climatechangeinaustralia.gov.au/en [http://www.climatechangeinaustralia.gov.au/en].

[30] Ibid n2

[31] Wasko, C. and Sharma, A. (2015), Steeper temporal distribution of rain intensity at higher temperatures within Australian storms, Nature Geoscience, 8(7), 527-529.

that can increase the intensity of short duration rain events and associated local stormwater runoff.

88. The results for expected increases in average temperatures in 2090 from the high emissions scenarios yields multipliers of rainfall depths of 1.12 and 1.16, and a weighted average multiplier of 1.14. Using this value indicates that the rainfall depth of the two hour design storm changes from 59.6 mm in 2019 to 67.9 mm in 2090.
89. The expected increases in rainfall depths are 14% in 2090 which is lower than the assumed 18.5% increases in peak rainfall intensities used by Engeny (2020; 2021) in the historical flooding investigations. These results can be compared to the "interim climate change multipliers" of 16.3% provided on the ARR2019 Datahub. It is noteworthy that these climate change multipliers apply to rainfall depths.

Hydrology Summary

90. No detailed information about the Moonee Ponds Creek hydrology model developed using RORB software, selection of critical storm events and patterns or calibration of the model to observed data was provided. The selection of the two hour design storm appears to be arbitrary.
91. The analysis of flooding and associated drainage requirements is based on superseded ARR1987 data and methods which may not be suitable for the intended outcomes. It is noteworthy that the majority of the updated design rainfalls and methods included in ARR2019 were released to the industry prior to 2016 and the ARR1987 guidelines requires the use of the latest best practice methods and data.
92. A two hour design storm was assumed for the Moonee Ponds Creek catchments which is inconsistent with observed historical behaviours for the Moonee Ponds Creek and local catchments. These issues must be resolved by calibration to observed flood levels in the sub-precinct.
93. Observed streamflow and rainfall data for the Moonee Ponds catchment indicate that the 1% AEP peak flow is 183 m^3/s. The 1% AEP runoff event is created by a rainfall event with long duration of 28 hours and the time for the peak flows to arrive at the Precinct is about 29 hours. Any local flooding in the precinct might be expected to discharge to Moonee Ponds Creek prior to arrival of the upstream peak flows.
94. The largest recorded peak flow in Moonee Ponds Creek created by a two hour storm is only 34.9 m^3/s which is one sixth of the assumed 1% AEP peak flow. The assumption that the peak flow from the entire Moonee Ponds Creek catchment arrives at the same time that the peak flows occur within the precinct is improbable. This implies that there may be two different types of flooding events (local and external from Moonee Ponds Creek) that impact on the sub-

precinct at different times which is consistent with the advice provided by the SES (2012).

95. Models describing the hydrological behaviour of the local and Moonee Ponds Creek catchments will need to be calibrated and validated using observed flood heights. The results of these industry standard calibration and validation processes must be publicly available to achieve confidence in the models.

2.4 Hydraulics

96. The determination of flood levels within the Arden Macaulay Precinct involves hydraulic modelling is described by a series of investigations from AECOM (2013) to Engeny (2016) to Engeny (2020; 2021). Each of the investigations utilised similar hydraulic models in TUFLOW software and inputs with varying assumptions that are critical to the flooding results as shown in Figure 8.

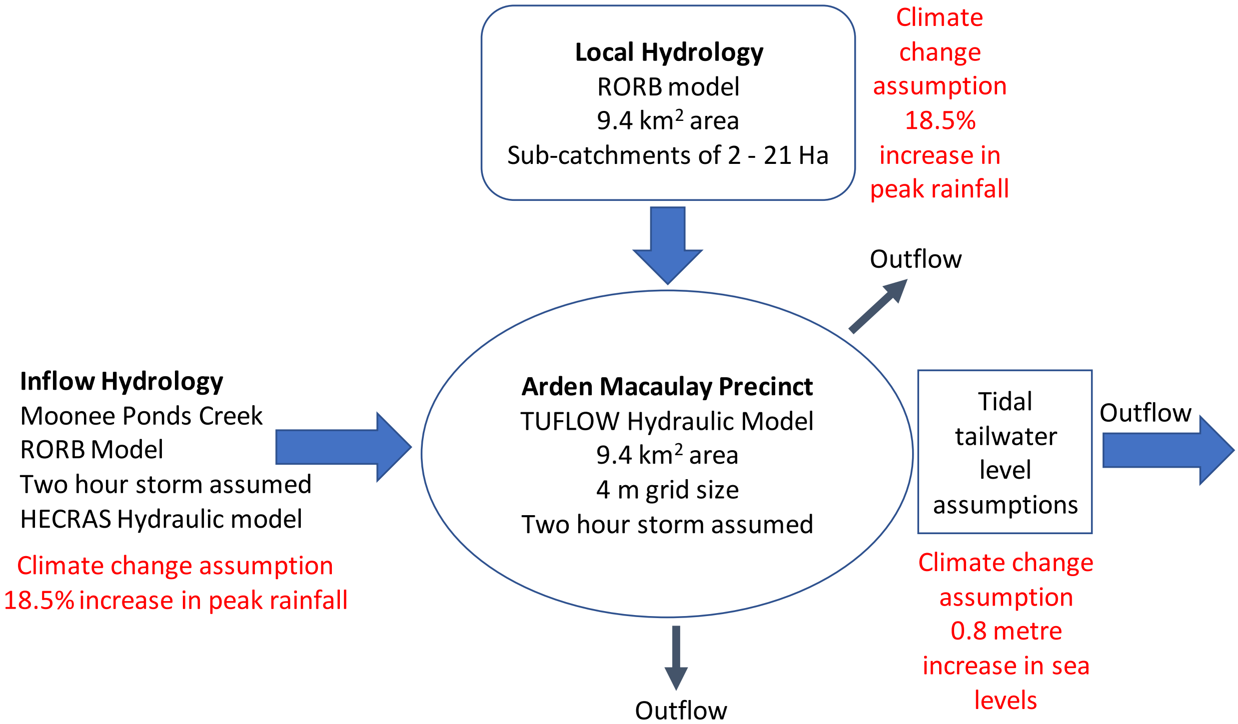


Figure 8: Overview of the critical processes involved in the historical hydraulic modelling of the Arden Macaulay Precinct

97. Figure 8 highlights that the hydraulic modelling to determine flood levels within the precinct is critically dependent on the inflow hydrology from Moonee Ponds Creek, local hydrology and tidal tailwater assumptions. These external assumptions drive the flood heights and durations within the precinct.

98. The limitations of inflow hydrology assumptions from the Moonee Ponds Creek catchment and with selection of tidal tailwater levels have been discussed in previous sections.

99. However, two key factors must be considered in the discussion of the local hydrology and hydraulics – the selection of a tail water level that may be 0.6 – 0.9 metres too height, and the critical inflow from upstream may arrive after the local flooding has dissipated.

100. This may profoundly change the behaviour of the local hydraulics. The extend of the hydraulics investigations, land surface levels, key infrastructure and Moonee Ponds Creek is presented in Figure 9.

101. Figure 9 also shows that the local topography and infrastructure were important considerations in the hydraulic modelling. The local hydrology was modelled using the RORB software based on large sub-catchments ranging from 2 ha to 21 ha that might not be suitable for flood analysis of an inner city area with flat grades.

102. This can lead to flow pathways and local accumulation of stormwater volumes in the model that are different to reality.

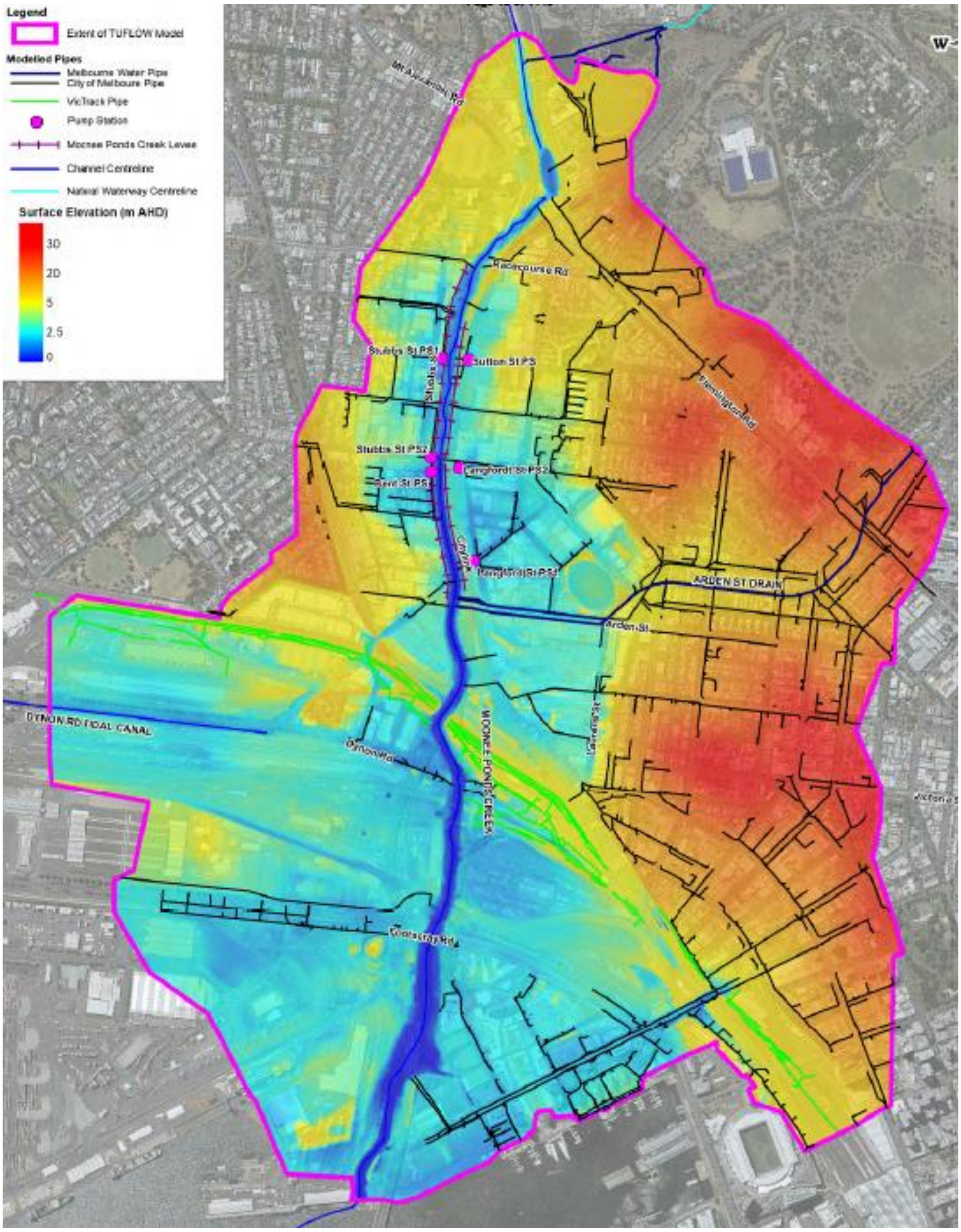


Figure 9: Overview of the catchment within the hydraulic models (after Engeny, 2020)

103. The local hydrology (RORB software) or the hydraulics models (TUFLOW software) were not calibrated or validated to observations of historical local flooding – for example at the low point in Langford Street during local flooding on 6 March 2010 versus rainfall and flows in Moonee Ponds Creek as shown in the Engeny (2021) report[32] and in Figure 10.

[32] Ibid n24

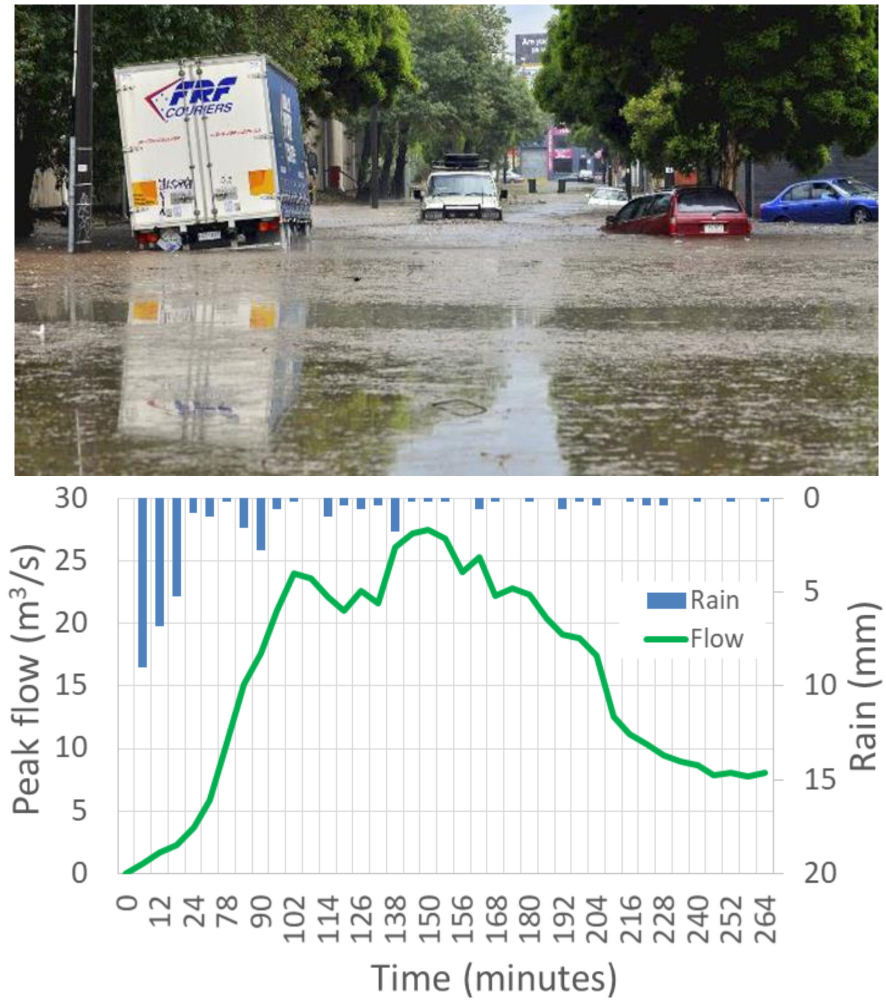


Figure 10: Flooding near the corner of Langford and Gracie streets on 6 March 2010

104. Figure 10 reveals that historical rainfall that created the flooding at the low point in Langford street included an intense 15 minutes of burst of rainfall (21 mm) which was determined to be a 5% AEP event.

105. This rainfall burst created a peak flow in Moonee Ponds Creek of 27.4 m^3/s. The peak depth of flow in Moonee Ponds Creek was 1.65 m which is less than the height of the levee (and 0.45 m higher that the low point in Langford Street) – there was no overflow from Moonee Ponds Creek into the sub-precinct from this event.

106. The localised flooding at the low point in Langford Street was created by local runoff (and perhaps some smaller flows from Moonee Ponds Creek into the sub-precinct via stormwater drainage pipes whilst permitted by water level differences).

107. There is also a possibility of stormwater from Moonee Ponds creek flowing into sub-precinct via connected pipe drainage networks. This may only be possible when the level of flows in Moonee Ponds is higher than the ground

levels and if the outlets of the pipe drainage networks do not include gate valves (which only permit outflow).

108. The intersection of Langford and Gracie Streets is the lowest point of 1.2 m AHD near the sub-precinct and is also the location of the pump station. In addition, Langford, Gracie and Green streets are significantly lower (up to a metre lower) than the surrounding properties.

109. The flood picture in Figure 9 shows the storage of stormwater in the low lying streets and highlights the importance of the pump at the intersection of Langford and Gracie streets, and any gravity connections (subject to tailwater conditions) for reducing the volume of stored stormwater in this area.

110. Engeny (2020, 2021) compared the local hydrology outputs to rational method assumptions which is not recommended best practice in accordance with ARR2019. These processes revealed a time of concentration of 15 - 24 minutes for the hydraulic catchment but a two hour design storm was used in the hydrology model that produced inputs to the hydraulic model.

111. Hydrographs from the hydrology model were applied to drainage inlet pits in the hydraulic model but this process may be inappropriate for the large sub-catchments utilised in the local hydrology model. Examination of the structure of the hydrology model in Figure F2 (AECOM, 2013) and Figure 3.2 (Engeny, 2020) also reveals stormwater from a large upstream Arden Street Drain catchment of 2.02 km^2 may have been directed down Macaulay and Gracie streets rather than to the Arden Street drain which drains this catchment as shown in Figure 11.

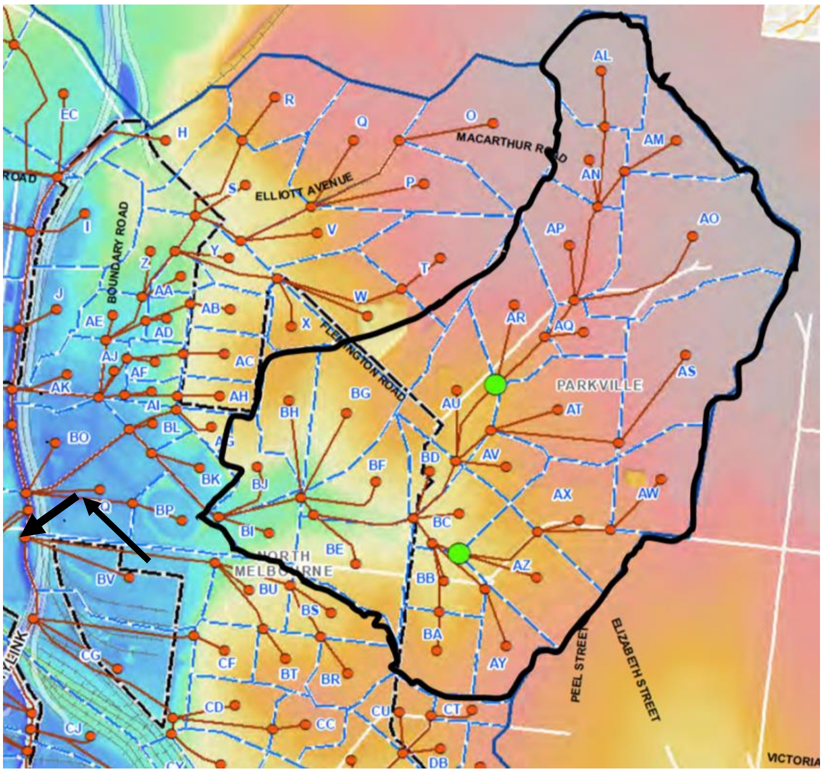


Figure 11: The portion of the upstream catchment, that should flow into the Arden Street Drain, directed to Macaulay and Gracie Streets in the local hydrology model

112. Figure 11 highlights that most of the Arden Street Drain catchment may have been directed down Macaulay and Gracie streets rather than the Arden Street Drain in the hydrology model – and therefore in the hydraulic model.

113. This will divert a substantial volume of stormwater (about 95 ML) from the Arden Street drain catchment into Macaulay and Gracie streets resulting in over-estimated flood levels in the sub-precinct. In addition, this will impact on the perceived effectiveness of the pump at the intersection of Langford and Gracie streets resulting considerably larger flood extents than reality throughout the adjacent "new stormwater management open space" sub-precinct bounded by Henderson, Gracie, Fogarty and Green streets.

114. The hydraulic model was used to determine local behaviour of stormwater runoff including flow velocities, and extents and depths of flooding. Use of 4 m grids in the model might be too large which could distort local flood effects. However, these differences are likely to be small relative to the impacts of selection of tailwater levels, assumed pump failures, misdirected stormwater flow paths and inflow hydrographs in Moonee Ponds Creek.

115. Greater uncertainties are expected from the use of inflows from relatively large sub-catchments (2 ha to 21 Ha) in the local hydrology model at drainage inlets in one dimensional networks representing drainage networks. This creates questions about flow directions and location of accumulated stormwater in the hydraulic modelling.

116. The flood levels in provided by the hydraulic modelling are also critically impacted by not permitting model outflows into rail tunnels and assuming that pumps do not work which will create higher flood levels.

117. Assumptions about pump failures do warrant examination as these assumptions imply that pump failures are absolutely correlated with the AEP of rainfall and flood events. Given that the pumps are specified to manage stormwater runoff and flooding, this assumption does appear to be unrealistic as it is counter to the design function of the pumps and this design will need to be rectified.

118. If pump failure is independent of the frequency and magnitude of rainfall events, then joint probability theorem will need to be applied to determine the actual statutory risk: for example (say) the pump has 1 in 10 (10% AEP) annual chance of failure and one is considering a 1 in 100 year (1% AEP) rain event, then the statutory probability is 10% times 1% which equates to 0.1% (1 in 1000 year) risk.

119. It would seem that altering the timing of events to align assumed high tailwater levels with peak local flooding and the arrival peak of upstream flooding from Moonee Ponds Creek could create unrealistic flood heights.

120. The colour schemes in published flood maps can be misleading with dark blue representing the lowest estimated flood depth from 0 – 300 mm. The selection of this display range might also provide unrealistic flood extents. Perhaps the first 50 mm of estimated flood depth in the 0 – 300 mm layer should be removed to account for topography, measurement and models errors? This can be achieved by industry standard geospatial filtering methods to produce a flood layer for 50 mm – 300 mm which will be a valuable planning asset.

Hydraulics summary

121. During period 2013 to 2021, a range of comprehensive investigations using similar models and assumptions to estimate the hydraulic and flooding behaviours in the Arden Macaulay Precinct in response to flows in Moonee Ponds Creek, local stormwater runoff and tidal behaviours that profoundly shape the results.

122. The hydraulic model of the Precinct has not been calibrated or validated to observed local flood behaviours, such as the 6 March 2010 event, and are based on superseded design guidelines and data.

123. Whilst the general hydraulic processes are likely to be valid, there strong uncertainty about magnitude and severity of the reported flood responses. It would seem that the flood impacts and consequences are over-estimated.

124. There are also questions about the stormwater runoff pathways and local accumulation in the models which impacts on estimated flood levels. For example, the routing of most of the Arden Street Drain catchment through the

sub-precinct in the model, use of large catchment inflows to nodes in the hydraulic model, assumptions about the operation of pumps and the omission of some of the stormwater outlets from the model (such as rail tunnels) heighten concerns about over-estimated flood levels.

2.5 Options

125. The investigation reviewed the modelling reports that provide models of current and future expected flood levels and potential options that could mitigate these expected flood events. These comprehensive reports provide a robust strategy to permit further development in the Arden Macaulay Precinct and development of the proposed planning scheme.

126. Nevertheless, the published investigations of options are significantly impacted by assumptions about existing and future conditions that are likely to significantly over-estimate flood levels (for example; tailwater levels that are 0.6 m – 0.9 m too high, over-estimated increases in peak intensities, and different arrival times for local and regional flood flows versus maximum high tides).

127. Indeed, it would seem that issues associated with local catchment runoff are more significant to the sub-precinct than any potential for impacts from overtopping of the levees from Moonee Ponds creek. It is noteworthy that the height of the levee above the creek invert (-0.2 m AHD) near the sub-precinct is more than 4 m.

128. The investigations underpinning this report also reveal that significant parts of the Arden Street Drain catchment may have been directed, in the models, along Gracie Street to the low point in Langford Street which will create higher than expected flood volumes and depths.

129. Nevertheless, analysis of the historical local flooding on 6 March 2010 reveal that local stormwater runoff can accumulate in the low lying areas in Langford, Gracie and Green Streets adjacent to the sub-precinct. The published modelling of options is analysed below to consider the potential impacts on the sub-precinct and the published need for flood storages within the sub-precinct.

130. A series of reports address the options for managing stormwater impacting on the Arden Macaulay Precinct, including reports by Engeny (2017; 2019; 2021).[33,34,35] Whilst there is uncertainty about the magnitude and severity of the reported flood impacts, the published analysis of options provides valid hydraulic processes and relativity of the performance of the different options.

---

[33] Engeny, (2017), Arden Macaulay Precinct - Drainage Investigation, Report for Melbourne Water and City of Melbourne.

[34] Engeny, (2019), Arden Macaulay Precinct - Langford St Flood Storage Investigation, Report for Melbourne Water and City of Melbourne.

[35] Engeny (2021), Arden Macaulay Precinct – Flood Management Strategy, Report for Melbourne Water.

131. The assessed management options for the high emissions climate change scenario in 2100 included raising of levees, creek widening and dredging which provide small benefits in the sub-precinct but the flooding processes were dominated by the high assumed tailwater levels in Port Phillip Bay, higher flows than expected in Moonee Ponds Creek and assumptions about the timing of these events. Most of the published reports found that upgrades to drainage strategies were required within the precinct.

132. The options considered for stormwater management within the precinct included flood storages at multiple locations, upgrades of pumps and pipes, inclusion of pressures pipes (such as Arden Street Drain) and raising of levees on Moonee Ponds Creek.

133. The Engeny (2019) results of the investigations of flood storages revealed considerable improvement in flood levels upstream from a flood storage assumed for the Citywide site within the sub-precinct.[36] This indicates that storages upstream from the sub-precinct and the linear storage between Langford Street and Moonee Ponds Creek has provided some mitigation in addition to the impacts of the raised levees and other measures (see Figure F5 in Engeny, 2017).[37]

134. The initial water level shown in Figure 4.2 of the investigation of local flood storages is about 1.2 m AHD which is considerably less than the assumed tailwater of 1.4 m AHD (existing) and 1.975 AHD (2100) described in the Engeny (2021) report.[38]

135. It is noteworthy that this investigation of the relative effectiveness of flood storages in the sub-precinct included most of the additional stormwater management measures such as upgraded pumps and pipes, raised levees and pressurisation of the Arden Street Drain in the models. These differences and inclusions add to the perceived benefits of the storage solutions at the sub-precinct.

136. Upgrading the capacity of the pumps and pipes in the precinct produces substantial benefits in reducing flood depths throughout the precinct, and within the sub-precinct. These substantial local benefits should be considered in the context of the proposed new local drainage infrastructure that connects to the Langford Street pump station near the intersection of Langford and Gracie Streets.

137. Inclusion of the pressure pipes in the Arden Street drain highlight the importance of limiting the conveyance of stormwater through the sub-precinct

[36] Ibid n28
[37] Ibid n28, Figure D5
[38] Ibid n28, Figure 4.2

for reducing flood levels as this solution provides strong reductions in flood levels.

138. A combined drainage solution provides substantial benefits in reducing flood levels in the precinct. A majority of these benefits can mostly be attributed to the local effects of the upgraded pumps and pipes, raised levees and the inclusion of pressure pipes in the stormwater drainage network.

139. The combined storages provide benefits upstream from the sub-precinct which indicates that the inclusion of flood storage within the sub-precinct (bounded by Langford, Gracie, Henderson and Green streets) is unlikely to provide significant additional benefit to the drainage strategy.

140. There is discussion of the downstream benefits of increasing flood storage but these benefits appear to be provided by the companion stormwater management measures outside of the sub-precinct, and these benefits are impacted by the barrier of Moonee Ponds Creek and associated structures.

141. The performance of the proposed drainage strategy without the flood storage proposed for the sub-precinct has not been compared to the overall performance of the drainage strategy. This comparison is needed to determine the relative contribution of the proposed flood storage in the sub-precinct.

142. In any event, the contribution of the proposed stormwater storage in the sub-precinct should be considered in the context of two scenarios, for managing local stormwater runoff and for managing inflows from Moonee Ponds Creek.

143. In a situation where inflows from Moonee Ponds Creek might overwhelm the sub-precinct with flood waters there is no opportunity for the flood storage in the sub-precinct to provide any benefits. If the management of local stormwater runoff is a concern, the various reports do not reveal the benefits of a storage in this location which might be insignificant in comparison to effective pumps and drainage, and upstream storages.

144. Discussions in the various reports from 2013 to 2022 have revealed a number of other important options that could substantially add to the viability of the Arden Macaulay precinct.

145. These opportunities, such as setting floor elevations, selection of building form (flood sensitive buildings) and using building or sub-precinct flood emergency response plans[39] are particularly valid for the sub-precinct that may be affected by an accumulation of local stormwater runoff that is impounded by the Levee at Moonee Ponds Creek and the low lying nature of Langford, Gracie and Green Streets relative to the significantly higher elevation of the surrounding properties.

---

[39] Stock Corporation Pty Ltd v Yarra CC [2020] VCAT 958 (4 September 2020)

146. This potentially impounded stormwater will have low velocity. The hazard management guidance from ARR2019 guidelines of flow depth versus velocity should be considered in this situation.[40]

147. Given that the proposed significant impacts of flood inundation on the sub-precinct are rare at greater than 1 in 20 years (5% AEP), there is value in considering the utility of this inner city urban area on every other day as discussed by the Victorian Civil and Administrative Tribunal in the *Stock* case.[41]

148. The value of land and the planning scheme should be considered in the context of assumptions about risk. The predictions from Bureau of Meteorology (BOM) could be utilised, with the existing City of Melbourne Flood Emergency Plan[42] and other resources to implement a sub-precinct and building flood emergency response plan to avoid flood risks.

149. As outlined by the SES (2012), and this investigsation, there may be 8 – 16 hours notice of flooding from the upper reaches of the Moonee Ponds Creek catchment and there is potential for local flash flooding generated by intense short duration events. These different types of events are rare but can be planned for in management strategies that can provide adequate warnings to permit evacuation or shelter in place to avoid risk from rare flash flooding or slower arriving regional flooding. For example, the BOM rainfall intensity maps and warnings can provide adequate notice of approaching storms that might create a flash flooding concern, and this potential risk could be managed by a range of management and planning protocols.

150. Some of the reports also highlighted the need to ensure that the outlets from drainage networks to Moonee Ponds Creek include one way flow valves (or check valves or tide gates).

151. These solutions will ensure that any high water levels in Moonee Ponds Creek do not flow via stormwater (pipe and channel) drainage outlets into the sub-precinct. These relatively inexpensive additions to the drainage network and pump systems will also improve the flood risks in the precinct. The analysis of hydraulic options should also include this type of infrastructure.

Options summary

152. During the period 2013 to 2021, a comprehensive series of options have been evaluated to manage expected flood impacts on the Arden Macaulay Precinct and to create a proposed planning scheme amendment.

153. Whilst there are concerns about the magnitude and severity of the estimated flood impacts, the relativity of the performance of the options in the various

[40] Ibid n20, Book 6, Chapter 7 Safety Design Criteria
[41] Ibid n37
[42] Ibid n3

reports is valid. The expected current and future flood levels and dynamics may be considerably less than reported.

154. The impacts of flooding on the sub-precinct are substantially reduced by raised flood levees, upgrades to pumps and drainage infrastructure, pressurising parts of the drainage network and provision of flood storages throughout the catchment (particularly upstream of the sub-precinct).

155. However, there has not been any relative evaluation of the benefit of the proposed flood storage in the sub-precinct, and it is unlikely that a flood storage located at the bottom of the catchment provides reductions in flood impacts.

156. It is also apparent that the sub-precinct might be subject to two different types of flooding impacts from local stormwater runoff and inflows from Moonee Ponds Creek. The timing and dynamics of these potential impacts would prompt different solutions.

157. The need to manage local stormwater runoff is more likely. It is important to recognise that the expected stormwater impacts are rare and that the sub-precinct will operate on a day to day basis. The value of the planning scheme can be maintained by managing risks from rare events by use of building and planning controls, including flood emergency response plans and flood sensitive buildings.

3 Modelling Results

158. A hydraulic model was developed using TUFLOW software to examine the consequences of this investigation on the Arden Macaulay Precinct and surrounding areas. The hydrology and hydraulic models underpinning analysis of the Precinct were not available and there are questions about the accumulation of stormwater and flow pathways that are examined in this section.

159. The model was set up using 3 metre grids and direct rainfall methods, existing infrastructure (pumps, levees and drainage networks) and is based on publicly available information as shown in Appendix A (full report). This analysis was undertaken using observed and verified public information, and the latest ARR2019 guidance to expand the insights of the investigation.

3.1 Validation to the observed event on 6 March 2010

160. The Arden Macaulay Precinct Flood Management Strategy (Engeny, 2021)[43] provides a picture of a local flood event on 6 March 2010. Observed rainfall, tide levels and streamflow in Moonee Ponds (see Figure 10) was used in the hydraulic model to verify the model results against the observed flood depths derived from the picture of historical flooding as shown in Figure 12.

Figure 12: Verification of the hydraulic model to local flooding records from 6 March 2010.

[43] Ibid n24

161. Figure 12 reveals that the hydraulic model predicted a flood depth of 1.14 m at the location of the car in the flood picture which is deeper than estimated observed flood depth of 0.6 – 0.7 m. This photograph of the flood depth at car might have been taken after the peak flood as indicated by the leaves on the bonnet of the car. The hydraulic model was capable of estimating the observed local flood depths at the sub-precinct and is suitable to examine the consequences of the insights of the investigation on the sub-precinct.

Historical flood depths at the sub-precinct

162. An overview of the predicted flood depths during the historical event is presented in Figure 13. The model results were filtered to remove flood depths less than 50 mm to account for input and model errors.

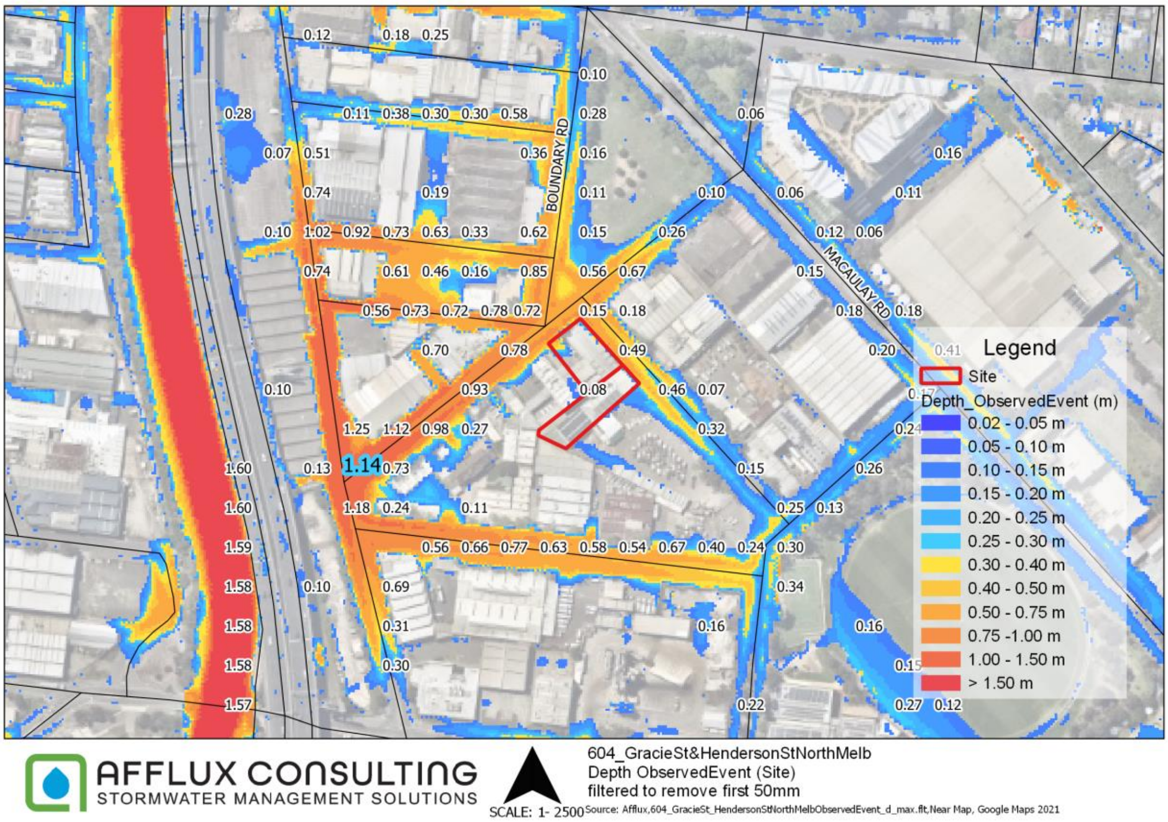


Figure 13: Estimated maximum flood depths from the 6 March 2010 event (filtered to remove floods depths less than 50 mm)

163. Figure 13 demonstrates that significant flood depths from the historical picture were mostly concentrated at the low point at the corner of Langford and Gracie Streets, and lesser flood depths were confined within the streets surrounding the sub-precinct. The properties in the sub-precinct were not subject to significant flood inundation.

3.2 Observed 1% AEP event in the Moonee Ponds Creek catchment

164. The historical 1% AEP flood event from 2-3 February 2005 (Figure 5) was created by a 28 hour rain event and the flood peak arrives at the sub-precinct 29 hours after commencement of the rain event. This rain event included 1% AEP storm bursts at 18 and 24 hour durations.

Historical flood depth from February 2005 at the sub-precinct

165. This observed Moonee Ponds Creek streamflow was utilised as an inflow to the model with a tailwater level of 0.62 m AHD and pumps were assumed to not operate to determine the impact on the sub-precinct as shown in Figure 14. Note that this scenario did not include the proposed upgrades to infrastructure, including levees, or local rainfall to isolate the impacts of a 1% AEP Moonee Ponds Creek flood event on the existing sub-precinct.

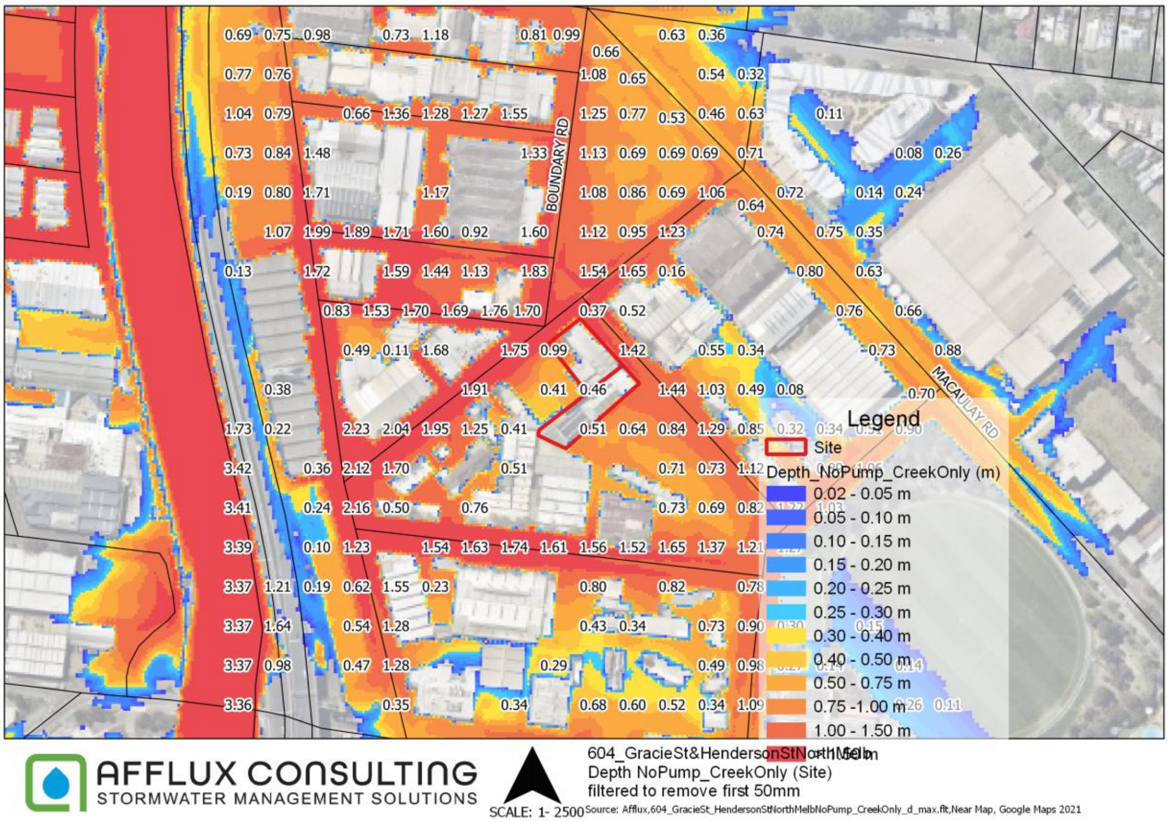


Figure 14: Impacts of the historical 1% AEP Moonee Ponds Creek flood event on the existing sub-precinct with pumps not operating.

166. Figure 14 reveals that the historical 1% AEP Moonee Ponds Creek flows would overtop the existing levees and create maximum flood depths ranging from 2.23 m at the low point in Langford Street to 1.12 m in Henderson Street. The properties within the sub-precinct are subject to flood depths less than 1 m.

February 2005 event with 2100 climate change impacts

167. The potential climate change impacts in 2100 were estimated by increasing

the local rainfall and the Moonee Ponds Creek flows by 14%, and by including a tailwater level of 1.29 m AHD in the model. The expected flood levels for the estimated climate change impacts without the proposed upgrades to infrastructure (existing conditions) are presented in Figure 15.

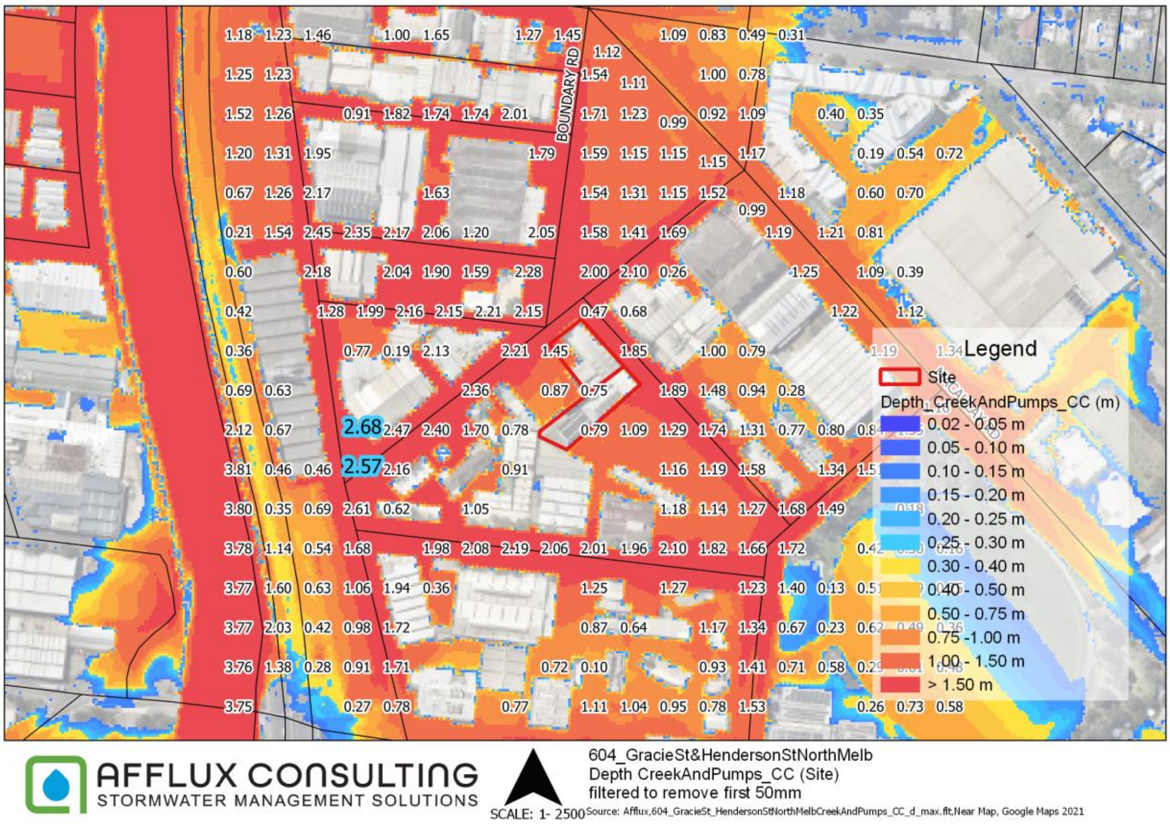


Figure 15: Flood depths from the 1% AEP Moonee Ponds Creek flows and rainfall with pumps operating subject to expected 2100 climate change impacts.

168. Figure 15 reveals that in the 2100 climate change scenario, the maximum depth of flood inundation is 2.68 m at the low point in Langford Street with lower flood depths in the streets surrounding the sub-precinct. The flood depths on the properties within the sub-precinct are less than the depths in the streets.

169. The addition of climate change effects and the local rainfall event results increases the flood depths. It should be noted that the climate change impact of simultaneously increasing rainfall intensity and streamflow may be unrealistic assumptions as it is not certain that these will occur in this manner.

Existing flooding processes from February 2005: no pumps

170. The 1% AEP Moonee Ponds Creek flows do create substantial flood depths at the sub-precinct with existing infrastructure and if the pumps are not operating. The mechanisms that drive this flood inundation are demonstrated in Figures 16 and 17.

171. It is estimated (Figure 16) that the levees are breached in three locations upstream of the sub-precinct at 24 hours into the hydrograph and this water fills the low lying areas adjacent to Moonee Ponds Creek to a maximum flood level at 30 hours. Then the flood levels recede leaving trapped water in low lying areas because the pumps are not operating.

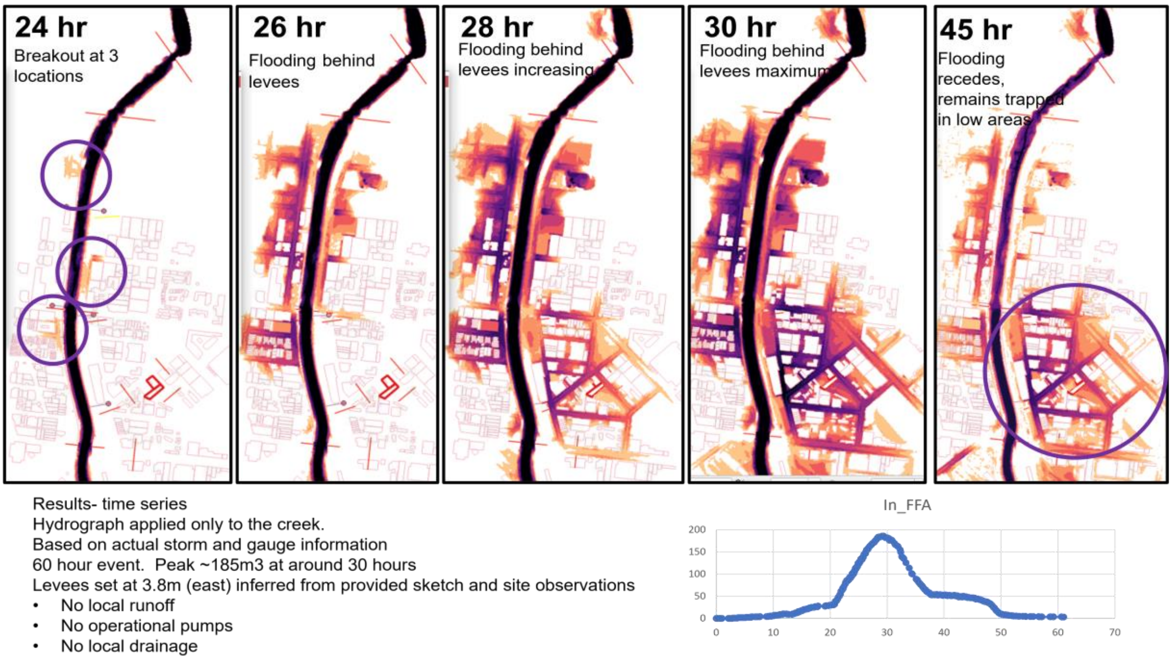


Figure 16: The flood behaviours created by the 1% AEP Moonee Ponds flows without pumps and subject to existing conditions

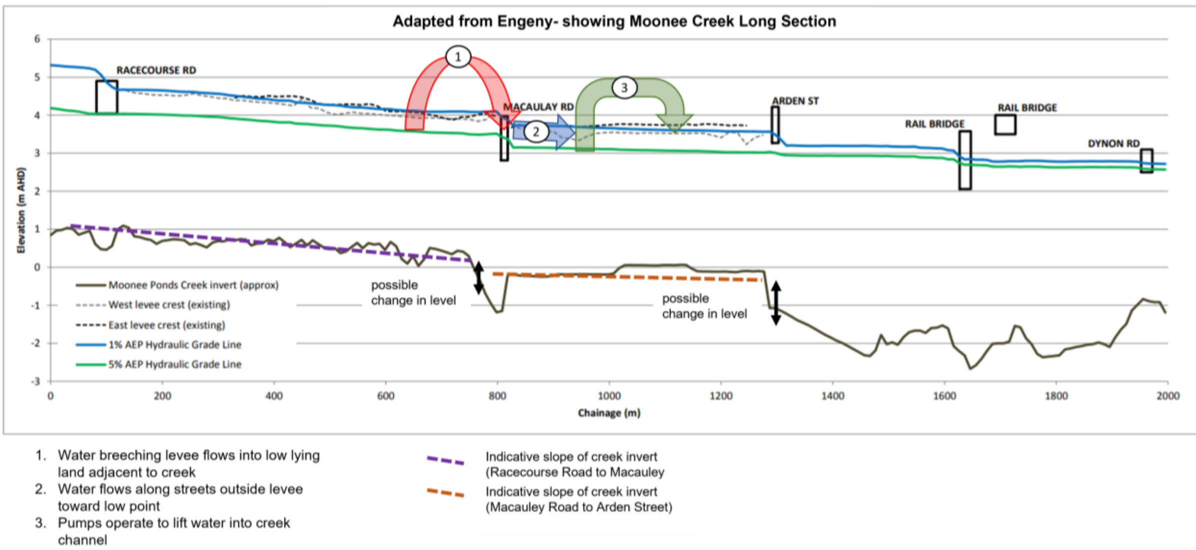


Figure 17: The mechanisms for local flood inundation driven by the 1% AEP Moonee Ponds flows without pumps and subject to existing conditions

172. Figure 17 reveals the flooding mechanism, in the context of the long section of the creek inverts, where water flows into the low lying areas near the creek, then flows along streets to low points where pumps could operate to lift water

back into Moonee Ponds Creek. These mechanisms and the locations of pumps are consistent with changes in invert levels in the creek.

173. The processes outlined in Figure 17 indicated that operation of pumps and upstream infrastructure improvements such as increases levee heights, improved drainage networks and linear storage adjacent to Moonee Creek will avoid impacts on the sub-precinct. The proposed storage within the sub-precinct is at the downstream end of this process and is unlikely to mitigate flood levels.

Mitigation Option: flooding processes.

174. Application of the 1% AEP rain event associated with the 1% AEP Moonee Creek flows with upgraded pumps and levees as proposed by Engeny (2021) creates the flood processes for existing conditions presented in Figure 18.

Figure 18: Flood processes from the 1% AEP Moonee Ponds Creek flows and rainfall with pumps operating with proposed upgrades to pumps and levees.

175. Figure 18 demonstrates that the proposed upgrades to the pumps and levees provides significant reductions in the transfer of stormwater from Moonee Ponds Creek and the local catchment towards the sub-precinct. The peak flood heights around the sub-precinct site are significantly reduced.

Mitigation option impacts at the sub-precinct

176. The expected flood levels for existing conditions with the proposed upgrades to pumps and levees are presented in Figure 19.

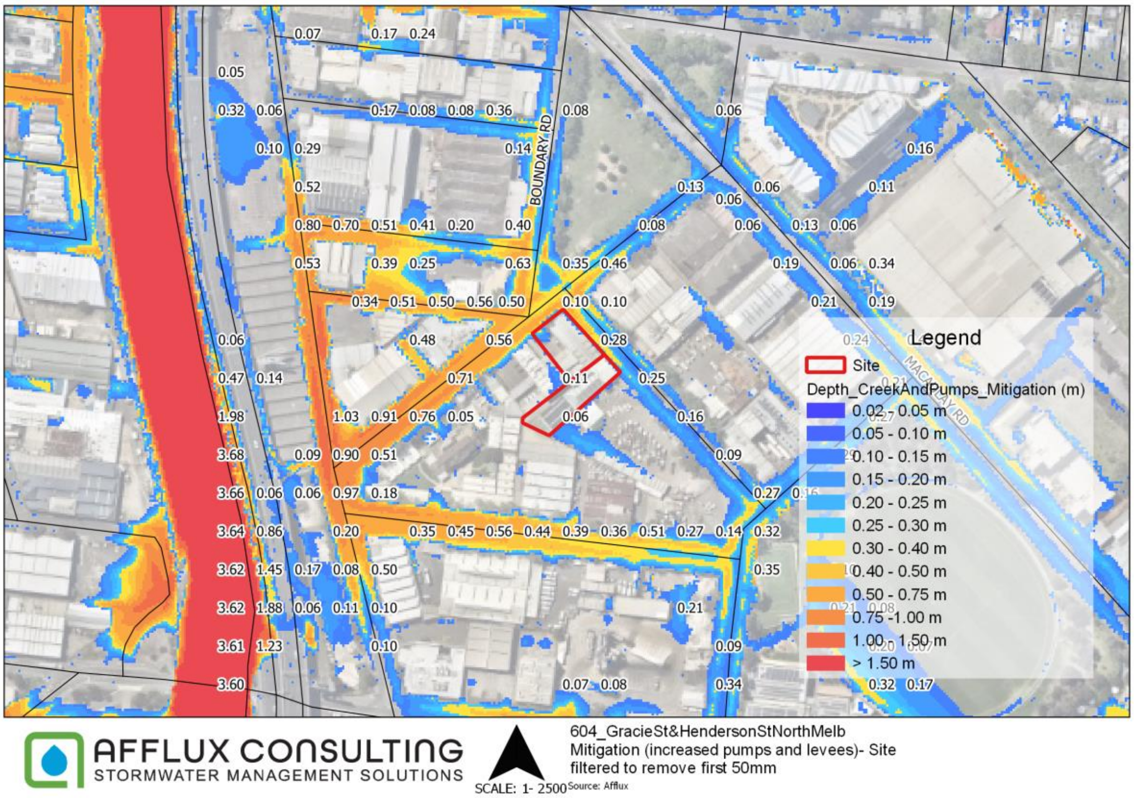


Figure 19: Flood depths at the sub-precinct from the 1% AEP Moonee Ponds Creek flows and rainfall with pumps operating with proposed upgrades to pumps and levees.

177. Figure 19 shows reductions in maximum flood depth at the Langford Street to 1.03 m as a result of increased levees and pumping rates. Maximum flood depths are generally reduced by over 1 metre and importantly the flood impact on properties is effectively eliminated in the vicinity of the site.

178. These results showing maximum depths obscures the dynamic nature of flooding in the area. Factors such as inundation time and duration are not possible to determine, and as such these plots do not provide an entire representation of risk and management options.

3.3 Local ARR2019 1% AEP 20 minute design rainfall with March 2010 Moonee Ponds flows

179. In this section, critical design rainfall inputs were derived from the ARR2019 data and methods and applied to the local catchment to coincide with historical Moonee Ponds flows from 6 March 2010. The flood depths from the critical ARR2019 1% AEP design storm pattern for existing conditions with pumps operating and 0.62 m tailwater levels is presented in Figure 20.

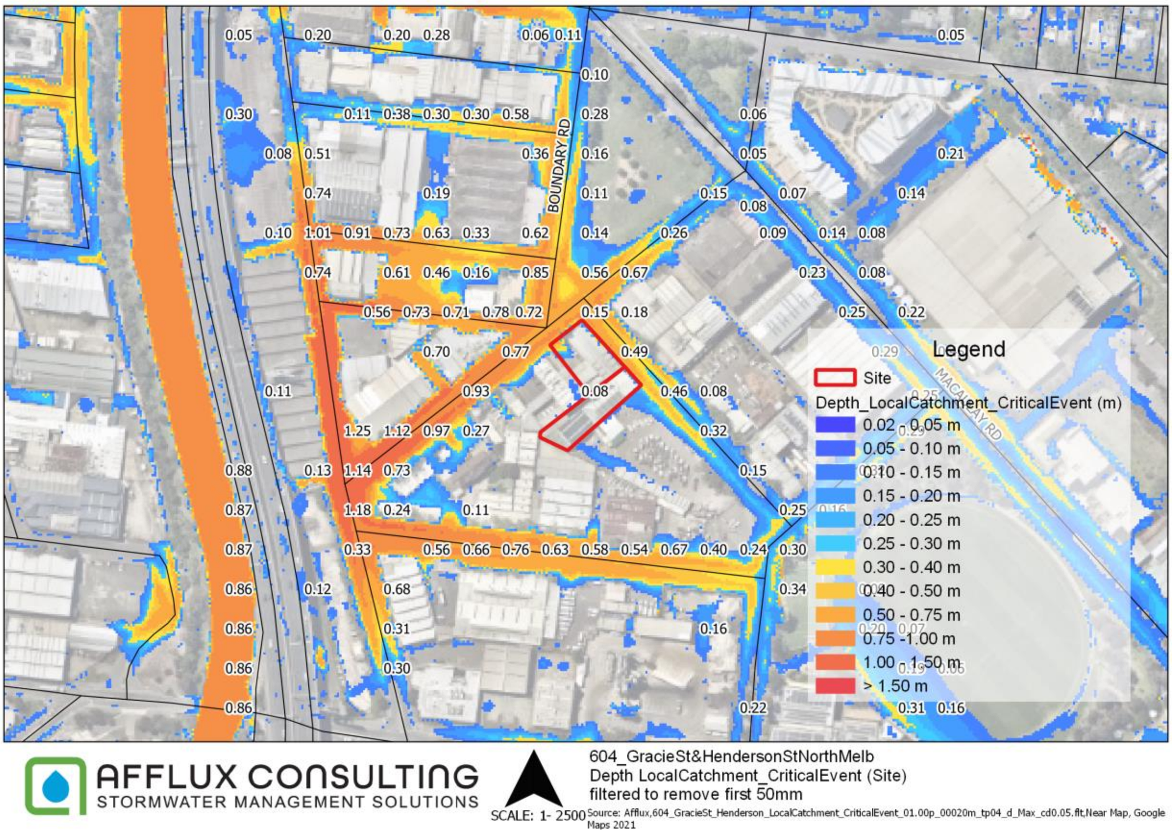


Figure 20: Flood depths from the critical 20 minute duration 1% AEP local rain event with inputs from the historical Moonee Ponds Creek flows from 6 March 2010.

180. Figure 20 demonstrates that the maximum flood depth of 1.25 m at the low point in Langford Street with significantly lower flood depths that are mostly contained within the roads. The properties in the sub-precinct are not subject to significant flood depths.

Climate change impacts on local runoff and flood depths

181. The high emissions climate change scenario was modelling by increasing the design rainfall and Moonee Ponds streamflow by 14%, and including a tailwater level of 1.29 m AHD. The expected flood depths are presented in Figure 21.

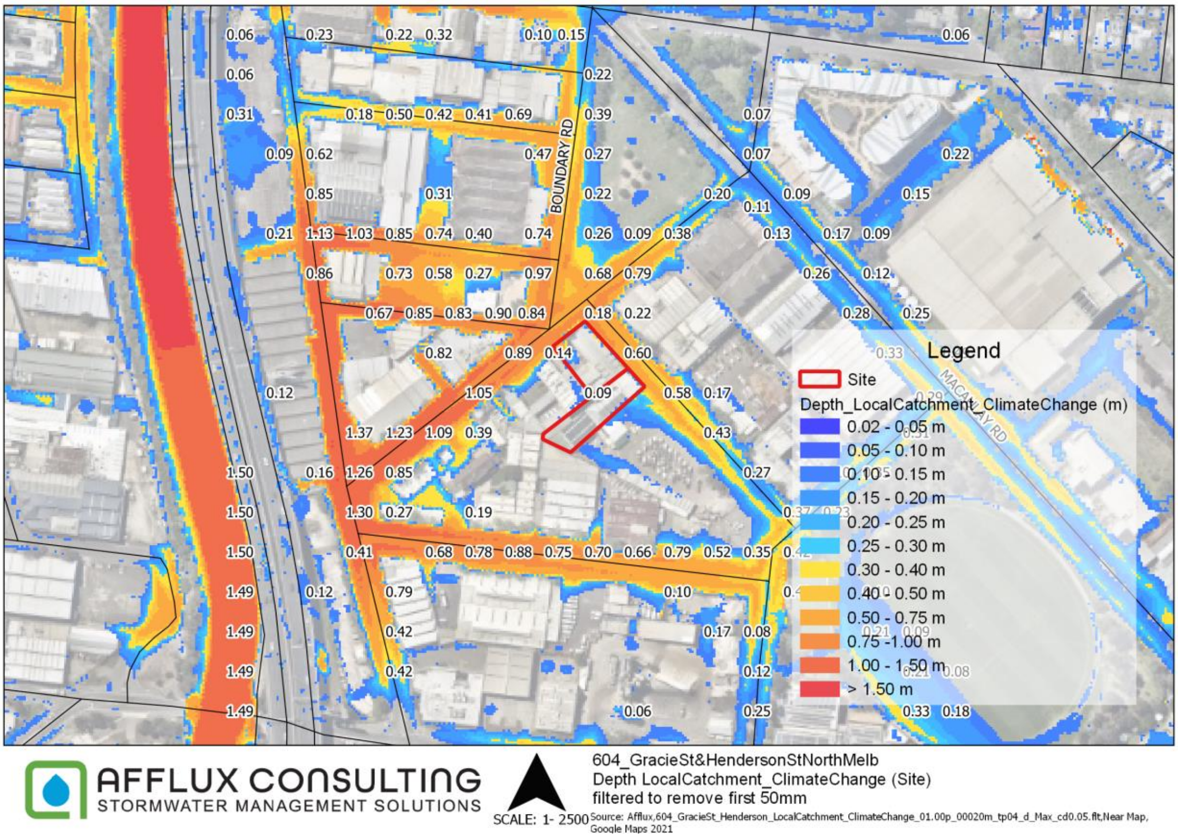


Figure 21: Flood depths from the critical 20 minute duration 1% AEP design rain event and Moonee Ponds flow for the 2100 climate change scenario with existing infrastructure.

182. Figure 21 reveals that a maximum flood depth of 1.37 m at the low point in Langford Street with lower flood depths that are mostly contained within the roads. The properties in the sub-precinct are not subject to significant flood depths in the 2100 climate change scenario.

183. The proposed upgrades of pump capacity, levee heights and pipe drainage infrastructure are expected to minimise the potential for future flooding of the sub-precinct and surrounds generated by the high emissions climate change scenario.

3.4 Modelling Summary

184. The hydrology and hydraulic models used to develop the reports underpinning the proposed planning scheme amendment were not available. This investigation has revealed considerable uncertainty the impacts of key input assumptions; tailwater levels, inflow hydrographs and timing of inputs; and the accumulation and flow paths of stormwater within the precinct. There was a need to test the impacts of these issues in a model.

185. A hydraulic model was created that included local information and infrastructure. This model was set up using ARR2019 methods and was based on a 3 m grid and included inputs of direct rainfall to understand local flow paths and historical observations of 1% AEP Moonee Ponds Creek flows.

186. The model used in this investigation was able to reproduce the historical flood depth at the low point in Langford Street and was considered to be suitable to test the consequences of the insights of this investigation in the context of existing conditions. There was a flood depth of 1.14 m at the low point in Langford Street (5% AEP historical rain event) with lesser inundation confined to streets but no significant flooding of properties in the sub-precinct.

187. Observation from the historical 1% AEP flood event from Moonee Ponds Creek was used as an input to the model with pumps not operating and no local rainfall. This revealed three potential breaches in the levees upstream from the sub-precinct and the flow of stormwater along streets to the sub-precinct which is located at the end of this flood transfer process. The ultimate accumulation of stormwater creates signification flood depths on streets (2.12 m at the low point in Langford Street) with lesser depths (less than 1 m) on properties in the sub-precinct.

188. The operation of the pumps and the proposed upgrades to pumps and levees overcome the impacts of flooding created by 1% AEP Moonee Creek flows and associated rainfall applied to the local catchment with a maximum depth of 1.06 m at the low point in Langford Street, lesser flood depths contained in streets and no significant flood depths on properties within the flood precinct. These results indicate that the upgrades pumps and levees are effective for mitigating flood depths and the proposed storage within the sub-precinct at the bottom of the flood transfer process is unlikely to provide significant mitigation.

189. The critical ARR2019 design storm for the local catchment produces maximum flood depth of 1.25 m for existing conditions and 1.37 m for 2100 climate change at the low point in Langford Street with significantly lower flood depths that are mostly contained within the roads. The properties in the sub-precinct are not subject to significant flood depths.

190. These model results indicate that the proposed upgrades to pumps, levees and pipe drainage infrastructure will mitigate the potential for current and future climate change driven flood depths.

191. It would seem that assumptions about higher tailwater levels and Moonee Ponds Creek flows, and the improbable alignment of the timing of maximum creek flows, maximum local runoff and maximum tide levels are the key differences in the reports underpinning the proposed planning scheme amendment.

4 Conclusions

193. The proposed flooding overlays, stormwater drainage strategies and planning scheme amendments that impact on the Arden Macaulay Precinct, and the sub-precinct, have been the underpinned by comprehensive investigations during the period 2013 to 2021. These investigations have utilised hydrology and hydraulic models constructed within industry standard software using similar assumptions throughout, and have importantly accounted for future climate change impacts.

Historical flooding and infrastructure

194. The north west of Melbourne within the Moonee Ponds Creek catchment has an early history of flooding and strong population growth with infrastructure and planning responses to respond to these challenges. The "West Melbourne Swamp" was filled and a channel with embankments was created as Moonee Ponds Creek. The increasing imperviousness of the Moonee Ponds catchment and concerns about local flooding motivated installation of levees, pumps, pipe drainage networks, catchment storages and town planning policies.

195. These actions provided ongoing improvements in the severity and intensity of flooding impacting on the North Melbourne area since the 1800s. The time for the peak flow in Moonee Ponds Creek to arrive at North Melbourne was recently observed to be 6 to 18 hours and more intense short duration (15 – 20 minutes) local rain events were seen as the challenge for low lying section of Langford Street adjacent to the sub-precinct. Significant historical observations of local flooding (such as the event on 6 March 2010), rainfall depths, tide levels and flows in Moonee Ponds Creek are available. However, these is no reporting of the use of this information to ensure the assumptions in the hydrology and hydraulic models are based on the local reality.

Impact of assumptions about tides and sea levels

196. Existing conditions has been defined as a model output rather than actual observations. The investigations are dominated by an assumption that the 1% AEP (1 in 100) flows from Moonee Ponds Creek will occur at the same time at 1% AEP maximum tide level and 1% AEP local stormwater runoff in the precinct. In the context of the 1% AEP statutory standard applied to management of flooding, this assumption implies that maximum tides are completely dependent on rare rainfall. However, there is a substantial body of evidence that tides are independent of rainfall in region that includes Port Phillip Bay – particularly for rainfall durations of less than 24 hours.

197. The assumptions about the alignment of maximum tides and rare rainfall in all of the reports underpinning the proposed drainage strategy and planning scheme amendments provide outcomes with greater 1 in 10,000 year (0.01% AEP)

probability which considerably greater than statutory 1% AEP standard. These assumptions drive an excessive outcome that is well outside of the jurisdiction of planning schemes and have strongly impacted on the choice of stormwater drainage options.

198. However, one must also consider that that tide datum at the Williamtown Gauge is 0.524 metres higher than the land datum (AHD) that is relevant to the Arden Macaulay Precinct. It is apparent that assumed tailwater levels in the various models may not have applied this correction to ensure that the tide datum is correctly related to the land datum. These issues with selection of tailwater levels used in the analysis of the precinct have most likely produced flood levels that are 0.6 m to 0.8 m too high in the analysis.

199. The maximum tailwater level used to examine existing conditions should be 0.62 m AHD (1 in 1 year maximum level) which is considerable lower than the 1.4 m AHD tailwater level that was utilised in the models. For clarity, there is no observed record of a maximum tide level of 1.4 m AHD that could be used as an "existing condition".

200. Global climate and sea surface models endorsed by the IPCC sixth report shown that the highest credible sea level rise in response to the 2100 RCP 8.5 high emissions scenario in Southern Australia is 0.67 m. The magnitude of expected increases in mean sea levels varies around Australia and the earth. In any event, the most valid tailwater level for 2100 under the high emissions climate change scenario is 1.29 m AHD. This is significantly lower than the assumed 2100 tailwater level in the various models.

Hydrology considerations

201. There are no reports describing the details and assumptions of the Moonee Ponds hydrology model, and no information is provided about the selection of critical storm duration and pattern. Importantly there is no published evidence that the hydrology model reproduces observed flows in Moonee Ponds Creek using observed rainfall which is standard industry best practice. It seems that the hydrology model is not linked to the reality of the Moonee Ponds Creek catchment. A verified historical relationship between model performance and real observations is vital for selecting critical storm durations and patterns.

202. It seems that section of a two hour design storm was an arbitrary process. Historical observations of the Moonee Ponds catchment indicate that the critical storm duration is 6 to 18 hours. Indeed, the observed rain event that generated 1% AEP peak flows at Mount Alexander Road North Melbourne had a duration of 28 hours (with embedded 1% AEP rainfall bursts of 20 and 24 hours) and the time to arrive (time of concentration) at North Melbourne was 29 hours. It is expected that this event would breach the levees on Moonee Ponds Creek at three locations near Macaulay Street which is upstream of the sub-precinct.

203. The construction of the hydrology model also utilised the superseded 1987 version of Australian Rainfall and Runoff guidelines which involved outdated data and methods. Indeed, current version of Australian Rainfall and Runoff includes more realistic design rainfalls and the selection of the critical storm and duration for analysis of urban areas should be based on selection of a storm duration and pattern that provides the maximum flood height that is verified by observed data (such as flood photos and levels in Moonee Ponds Creek).

204. The time of concentration for stormwater runoff from the local catchments that impact on the Arden Macaulay Precinct and the sub-precinct is 15 – 20 minutes. These local events may discharge into Moonee Ponds Creek before the arrival of the peak flows from the entire catchment. These observed differences in the timing of the hydrology influences changes potential impacts, performance of drainage measures and reveals opportunity for different management responses.

Hydraulic issues and flood depths

205. Hydraulic models utilised to determine the flood levels in the Precinct are severely impacted by assumptions about high tailwater levels, and the magnitude and timing of hydrological inputs. These issues profoundly change the dynamics of the local hydraulics and flood levels impacting on the sub-precinct. Similar to the issues impacting on the validity of the hydrology models, the performance of hydraulic model was not calibrated or validated using historical flood information – such as the observed flood levels at the low point in Langford Street on 6 March 2010. There is no evidence linking the hydraulic model to historical observations.

206. The efficacy of the hydraulic models is also impacted by concerns about the reliability of stormwater flow pathways and accumulation of stormwater that impacts on flood levels in the precinct and sub-precinct. The linking of hydrographs from large sub-catchments (2 – 21 ha) into nodes within the hydraulic model can create problems with the realistic location of stormwater in the model. Stormwater runoff from the Arden Street Drain catchment (2.02 km^2) also appears to be redirected through the sub-precinct via Macaulay and Gracie Streets to the pump at the low point in Langford Street. Together with assumptions that pumps fail and not permitting outflows of stormwater from the precinct (for example via rail tunnels), this creates higher flood depths than expected.

207. The assumption that pumps designed to manage flooding will always fail during flooding is an assumption that needs scrutiny. Presentation of a dark blue 0 – 300 mm flood layer in the various reports also has strong potential for misunderstanding. How much of the present flood extents is less than 50 mm deep and subject to considerable uncertainty due to a range of errors associated with data and models.

Consideration of stormwater management options

208. A substantial body of work has been created during the period 2013 to 2021 that is subject to considerable uncertainty due to range of assumptions that impact the overall magnitude of the published results – mostly flood levels that are too high and flood extents that may be excessive. However, the relativity between the comprehensive set of options that were evaluated provides valid comparisons due to robust nature of internal working of the chosen models.

209. An option to raise the flood levees surrounding Moonee Ponds creek provides substantial benefits to the precinct by mitigating the potential impact of flows from the entire Moonee Ponds catchment that are masked by the assignment of excessive tailwater levels. Provision of upgraded pumps and stormwater pipe drainage with pressurisation of parts of the pipe drainage network creates strong local benefits by significantly reducing flood levels throughout the Precinct.

Catchment storages

210. The investigation of the benefits of catchment storages includes most of the improvements that are discussed above and also appears to use a lower tailwater level than analysis of the other options. These assumptions lead to a perceived impact of storages as more beneficial than they might be. Comparison across the options reveals that the local storages provide limited benefit to the stormwater management strategy and those benefits are upstream of the sub-precinct. This insight was confirmed using a hydraulic model that demonstrates that flood impacts are driven by breaches of levees near Macaulay Street that is upstream of the sub-precinct.

211. These results suggests that the storages above the sub-precinct and linear to Langford Street provide some benefit but the proposed storage at the sub-precinct does not contribute. A relative analysis of the benefit of the storage in the sub-precinct versus the entire drainage strategy has not been done. There is no evidence of the need for this storage at the sub-precinct.

Inclusion of a local flood emergency plan

212. An important insight is that the properties in the sub-precinct have land elevations that are substantially higher than Gracie, Green and Langford streets, and the lowest point is in Langford Street at the pump and adjacent to Moonee Ponds Creek at an elevation significantly lower than the sub-precinct.

213. The expected flood events are rare (less than 1 in 10 year probability) and there an important opportunity to maximise the value of the planning scheme and the sub-precinct land on every other day by using a building and sub-precinct flood emergency plan. Similar to the decision in the *Stock* case at VCAT, a warning system can ensure that interaction between people, cars and potentially unacceptable depth of flood water is avoided by evacuation or shelter in place.

Using a model to investigate the consequences of investigation insights

214. Finally, a detailed "simple" direct rain model of the sub-precinct and surrounds that included the insights from this investigation, publicly available information, historical data and ARR2019 methods was employed to examine the consequences of the insights. The model reproduced the observed flood depth in Langford Street from the 5% AEP rainfall event on 6 March 2010. This process revealed lesser flood depths in the streets surrounding the sub-precinct, and no significant flood inundation of the properties within the sub-precinct.

215. The model in this investigation showed that historical 1% AEP flows in Moonee Ponds Creek (28 hour rain event) breached the existing levees in three locations upstream of Macaulay Street and these flood waters filled low lying areas and ultimately flowed along streets to the sub-precinct. Assumptions of pump failure permits flood depths to reach 2.12 metres at the low point in Langford Street with lesser flood depths on roads and up to 1 m flood depths on properties within the sub-precinct.

216. Assuming that the existing pumps do operate and upgrading pumps and levees reduces flood depths resulting from breaches of the levees and from local catchments. It revealed that the properties within the sub-precinct were not impacted by flood water from the historical 1% AEP Moonee Ponds Creek and the maximum flood depth at the low point at Langford Street was 1.06 m with considerably lesser water depths in Gracie and Green streets. There were insignificant flood impacts on properties. The climate change scenario increases the maximum depths.

217. Local stormwater runoff created by 1% AEP 20 minute critical storm with alignment historical Moonee Ponds Creek historical flows creates a maximum flood depth of 1.25 metres at the low point in Langford Street with lesser flood depths contained within streets and no significant flood depths on properties within the sub-precinct. The results from the climate change scenario were similar (maximum depth of 1.37 m in Langford Street).

218. These model results confirmed the insights of the investigation and indicate that the proposed upgrades to levees, pumps and pipe drainage networks will mitigate the expected flood events. The revealed mechanisms of flood inundation originating from Moonee Ponds Creek and from local catchments demonstrate that flood storages within the precinct (at the bottom of the flooding process) are unlikely to provide significant mitigation of flood depths.

219. The investigation of the impact of the assumptions about tailwater levels, timing of hydrology, pump failure and model structure (flow paths and accumulation of stormwater) using a model reveals that these assumptions have dominated to results underpinning the proposed planning scheme amendment with strong over-estimation of the stormwater flooding challenges.

220. These model results are consistent with insights of the investigation of reports that the flood depths in the sub-precinct are up to 1 m lower than published, the properties are not significantly inundated when pumps are operating and any accumulated stormwater dissipates rapidly from the sub-precinct.

221. A local flood storage at the sub-precinct is expected to provide little or no benefits and private property within the sub-precinct is largely unincumbered by expected existing flooding. The proposed upgrades to levees, pumps and pipe drainage networks, in combination with a sub-precinct flood emergency response plan, will most likely mitigate expected flooding from existing and climate change scenarios.

5 References

AECOM (2013), Hydrologic and Hydraulic Modelling of Arden Street and E-Gate, Department of Transport, Planning and Local Infrastructure Victoria.

Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors), (2019), Australian Rainfall and Runoff: A Guide to Flood Estimation, Commonwealth of Australia (Geoscience Australia)

BOM (2021), 60780 Melbourne (Williamtown) tide data, Bureau of Meteorology

Coombes, P., and Roso, S. (Editors), (2019), Runoff in Urban Areas, Book 9 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia, © Commonwealth of Australia (Geoscience Australia).

CSIRO and Bureau of Meteorology (2020), Climate Change in Australia, Projections for Australia's NRM Regions. Technical Report, CSIRO and Bureau of Meteorology, Australia. Retrieved from www.climatechangeinaustralia.gov.au/en [http://www.climatechangeinaustralia.gov.au/en].

DELWP, (2019), Guidelines for development in flood affected areas, The State of Victoria Department of Environment, Land, Water and Planning.

Engeny, (2021), Arden Macaulay Precinct Flood Management Strategy, Report for Melbourne Water

Engeny (2020), Arden Macaulay Precinct & Moonee Ponds Creek Flood Modelling, Model Build Report, Report for Melbourne Water and City of Melbourne.

Engeny, (2019), Arden Macaulay Precinct - Langford St Flood Storage Investigation, Report for Melbourne Water and City of Melbourne.

Engeny, (2017), Arden Macaulay Precinct Drainage Investigation. Revision 2, 7 February Report for Melbourne Water and City of Melbourne.

Engeny, (2016), Arden Macaulay Precinct stages 1 and 2. Revision 0, 29 February Report for Melbourne Water and Metropolitan Planning Authority.

Engeny, (2016), Melbourne Water Metropolitan Planning Authority Arden Macaulay Precinct stage 1. Version 0, 26 January Report for Melbourne Water

Gauging Station 229643A, Moonee Ponds Creek Upstream Mt Alexander Rd Flemington, Bureau of Meteorology

Gauging Station 229665A, Moonee Ponds Creek at Jacana Retarding Basin Jacana, Bureau of Meteorology

GHD (2020), Modelling Assumptions & Implications, Memorandum to Melbourne Water Corporation, 30 July 2020.

Goyen A., Phillips B., and Pathiraja S., (2014), Rational Method developments. Urban Rational Method review. ARR 2019 Revision Project 13 Stage 3, Engineers Australia, National Committee on Water Engineering, Barton.

IPCC (2021), AR6 Climate Change 2021: The Physical Science Basis, Chapter 9: Ocean, cryosphere and sea level change, Intergovernmental Panel on Climate Change.

Kuczera G., and Franks S., (2019), Chapter 2: At Site Flood Frequency Analysis, In Book 3 of Australian Rainfall and Runoff: A Guide to Flood Estimation, Commonwealth of Australia (Geoscience Australia)

Pilgrim, D, H., (1987), Australian Rainfall and Runoff. A guide to flood estimation, Volume 1, Engineers Australia. Canberra.

Retallick, M., Babister, M., Varga, I., Ball, J., and Askew. E., (2009), Do filtered temporal patterns resemble real patterns? 32nd Hydrology and Water Resources Symposium, Engineers Australia, Newcastle, Australia

SES, (2012), City of Melbourne Flood Emergency Plan, State Emergency Service, Victorian Government

VPA, (2021), Arden Structure Plan, Victorian Planning Authority

VPA, (2021), Arden Precinct Background report, Victorian Planning Authority

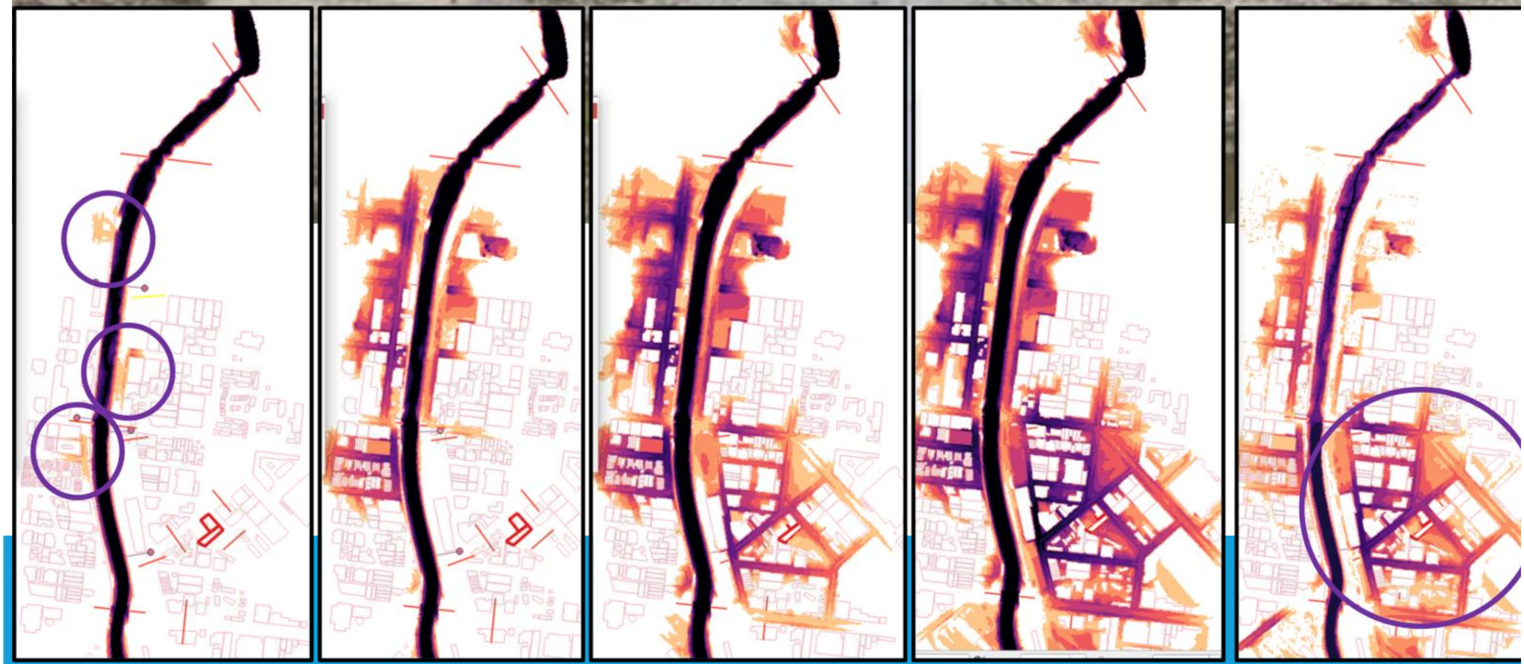
VRCA (2020), Vic Tides 2020, Edition 4, Victorian Regional Channels Authority

Wasko, C. and Sharma, A. (2015), Steeper temporal distribution of rain intensity at higher temperatures within Australian storms, Nature Geoscience, 8(7), 527-529.

Zheng, F., Westra, S., and Leonard, M., (2014), Coincidence of Fluvial Flooding Events and Coastal Water Levels in Estuarine Areas. Stage 3 Report, Australian Rainfall and Runoff. Engineers Australia.

Appendix A

Modelling Report



NORTH MELBOURBE AREA-FLOOD INVESTIGATION

49-51 Henderson Street and 62-70 Gracie Street, North Melbourne

Date	3 February 2022
Project No.	604_GracieSt&HendersonStNorthMelb
Version	01
Author	AA
Client	UWCS

Project History

Project Number	604_GracieSt&HendersonStNorthMelb
Author/s	AA
Checked	CMB
Approved	CMB
Issued to	UWCS

Document History

Version	Date	Description
01	4/2/2022	

Climate Change Statement

A wide range of sources, including but not limited to the IPCC, CSIRO and BoM, unanimously agree that the global climate is changing. Unless otherwise stated, the information provided in this report does not take into consideration the varying nature of climate change and its consequences on our current engineering practices. The results presented may be significantly underestimated; flood characteristics shown (e.g. flood depths, extents and hazards) may be different once climate change is taken into account.

Disclaimer

This report is prepared by Afflux Consulting Pty Ltd for its clients' purposes only. The contents of this report are provided expressly for the named client for its own use. No responsibility is accepted for the use of or reliance upon this report in whole or in part by any third party. This report is prepared with information supplied by the client and possibly other stakeholders. While care is taken to ensure the veracity of information sources, no responsibility is accepted for information that is withheld, incorrect or that is inaccurate. This report has been compiled at the level of detail specified in the report and no responsibility is accepted for interpretations made at more detailed levels than so indicated.

Contents

Tables

Figures

1. Introduction

Afflux Consulting have been engaged by UWCS to assist with an expert peer review of flood modelling in association with two locations in North Melbourne, namely 49-51 Henderson Street and 62-70 Gracie Street, North Melbourne (hereafter referred to as the site) as shown in Figure 1

Our engagement has been through Urban Water Cycles Solutions, and in particular to provide specialist flood modelling expertise to support an independent peer review that UWCS was commissioned to undertake by the primary client (believed to be Lawyers acting for the owners of the sites identified above). The purpose of UWCS engagement is understood to include an independent review into flood modelling that has previously been undertaken by the City of Melbourne and Melbourne Water.

The owners of the two sites have concerns which are believed to include the proposed extent of flooding as indicated in the proposed LSIO not being representative of the observed nature of flooding, and the implications for land value due to flooding or proposed mitigation options.

Afflux routinely undertakes flood modelling work, and staff are familiar with the requirements of various modelling packages to enable a realistic representation of flooding. Previous work with UWCS has led to the development of a flood modelling specification to address the specific needs of a local government agency. We are routinely engaged by clients to interpret planning scheme overlays and often develop localised models which are bespoke and able to offer an improved representation of the nature of localised flooding. Our principals have appeared as expert witnesses before VCAT and are familiar with the requirements of the tribunal in relation to standard of evidence and impartiality.

Although not engaged as the lead expert witness we can affirm that there are no conflicts of interests known to us that prevent us from assisting in developing evidence in relation to the site.

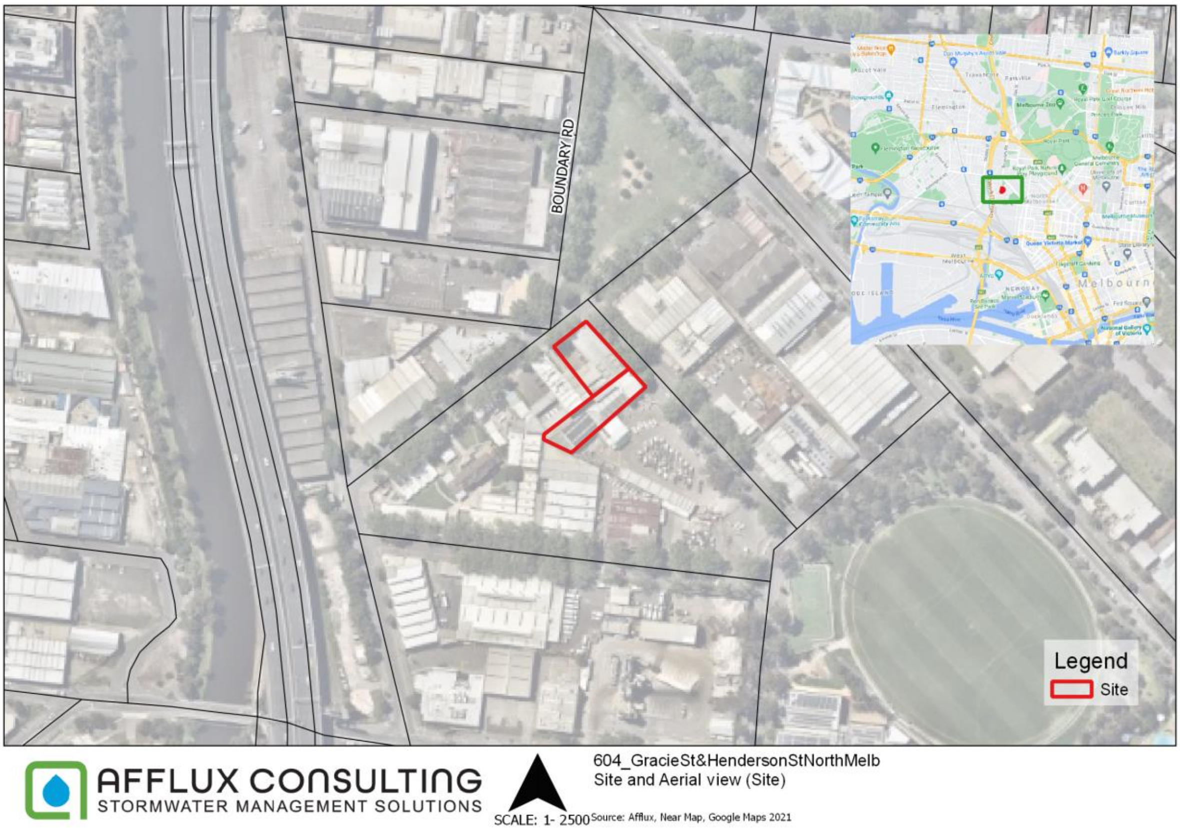


Figure 1.   Site location

2. Background

Urban Water Cycle Solutions has provided Afflux engineers with a succinct summary of key findings in relation to previous flood modelling and correspondences between various agencies and interested parties as the LSIO extents and mitigation options were being considered.

We have relied on these summaries from UWCS and collaboratively worked to develop a number of scenarios to 'test' through a modelling approach. We have been provided with a subset of reports to review and have independently identified a number of other reports through publicly available sources. These background documents, along with their relevance to this project are listed in Table 1.

Table 1. Documents reviewed

Document	Source	Relevance
Arden Macaulay Precinct & Moonee Ponds Creek Flood Modelling	Engeny August 2020	Provides information on TUFLOW models previously developed by consultants, including model extent, pump operational parameters, and determined flood characteristics
Arden Macaulay Precinct – North Melbourne Football Club Storage Investigation	Engeny November 2020	Provides information on flood mitigation options within the precinct surrounding North Melbourne Football Club
Interim AM STA 6200 Flood Mapping Projects	Melbourne Water (2021)	Outlines Authority expectations for flood mapping projects, including quality assurance.
Engeny_ArdenMacaulayPrecinct_ DrainageInvestigation_Rev2		
Flood Modelling Guidelines for Melbourne – Guidelines for Melbourne Catchments (Version 1.1)	Rain Consulting for City of Melbourne	Outlines authority expectations for modelling
City of Melbourne Flood Emergency Plan	City of Melbourne, Vic SES September 2012	Provides further information on nature of flooding within City of Melbourne, including historical accounts and known problem areas
Arden Macaulay Precinct levee failure preliminary analysis	Memo from Engeny to Melbourne Water, 23 March 2018	Provided information to determine levee levels and location
Arden Macaulay Precinct Cloudburst Management Plan	Engeny September 2018	Provides information on flood extents and potential mitigation options

Melbourne Water Arden Macaulay Precinct Flood Management Strategy	Engeny, August 2021	Provides analysis and recommendations for flood mitigation options in the Arden Macauley Precinct

Importantly, we have not been provided with specific information through the discovery process to assist in the development or assessment of technical modelling previously undertaken (i.e. access to flood models).

This has forced the modelling in this report to be produced independently, with fresh eyes and a sound first principles approach, however has required some judgement to be applied in determining a number of inputs into the model where these are not easily available.

3. Methodology

After discussion with UWCS it was decided to replicate the TUFLOW model for the area that was previously developed, initially by AECOM and then updated by Engeny, and presumably forms the basis for assessment of mitigation options.

TUFLOW is an industry standard model that is used for assessing flood behaviour across complex terrain. It is able to account for a range of real world processes including rainfall, stream behaviour and various infrastructure items such as pumps and pipes.

Once developed, the TUFLOW model would be run for a range of scenarios, including real world and 'approximate' events based on a range of inputs either derived from analysis of data or standard industry sources or provided via UWCS.

UWCS provided an independent review of all previous documentation that was made available and determined a number of refinements that would likely influence the representativeness of the modelling approach.

Importantly, the inclusion of all TUFLOW parameters were discussed with Afflux engineers and where these were found to be reasonable, were included in the modelling.

While the justification for the elements included in the model are more extensively documented in the UWCS report they are reproduced in Table 2 and where required, discussed in greater detail at the relevant sections in the report.

Table 2. Model input parameters and setup

Model parameter	Rationale	Outcome
Changing of tailwater levels in the model (i.e. at Port Phillip Bay)	UWCS identified discrepancies in datum used for land survey and sea level reporting	UWCS provided static tailwater levels for inclusion in existing and climate change scenario models
Update terrain information	State agencies collected aerial terrain surveys (i.e. LiDAR) in late 2017 and is the 'best available' information. It is available at a high degree of spatial resolution and has been processed to achieve 100mm vertical accuracy.	Adopted as the basis of the terrain model used
Inclusion of buildings	Buildings can have an important influence on local flooding as it influences the travel path of water across complex terrain	Building layers were sourced commercially in the vicinity of the site and represented in the flood model as obstructions extruded above ground surface
Levees inclusion	Levees along Moonee Ponds Creek are likely to be influential in containment of flows in this water body. Levees are unlikely to have been detected in LiDAR capture as they are under CityLink flyover, however levels	Levees were included in model as terrain modifiers along creek embankments based on information provided and visual inspection.

	and extents are available from previous reports.	
Model grid size	Model resolution was previously set at 4m grid size. This has been 'improved' to 3m and aligns with industry recommendations for modelling of this nature. Higher resolution grid will ensure complex terrain is better represented, including effects of buildings and levees.	A 3m grid was adopted as part of the modelling approach
Ensemble analysis	Previous studies are expected to have used single design storms and potentially selective analysis of these. Ensemble approaches recommended in latest guidance allow a range of storm events to be models to determine the critical combinations of event and temporal pattern impacting at locations of interest.	Ensemble analysis was undertaken for a simplified catchment to determine critical events using techniques recommended in guidance documents. Higher resolution models were run for the critical event identified through this process.
Event validation	A range of observed hydrological and rainfall inputs were included in model runs to determine extent and nature of flooding. This improves understanding of modelling when previous flood incidents are available for comparison.	Historical events (i.e. using real data) were included for model calibration and assessment of results against observations.
HPC model solver	TUFLOW is an industry standard model used for flood assessments of this nature. Previous studies are expected to have been repeated using 'Classic' versions of the software, however HPC model engines are available that allow more robust models to be run and dramatically improve run times.	HPC was adopted as standard for all model runs., and a range of Quality assurance checks undertaken.
Scenario Analysis	Previous reports presented limited scenarios and assumed simultaneous high tailwaters, creek flows and local runoff. It is unlikely these will occur at the same time, so our approach was to break issues down into component parts and model these in order to gain an appreciation of the flooding mechanism through the area, and	Scenarios were analysed for a range of creek flow rates (informed by hydrological analysis of the entire Moonee Ponds catchment), tailwater levels, climate change and pump operation. In total 6 scenarios were analysed.

	likely response under a range of conditions.	
Pumps inclusion	Previously pumps were omitted from the model. Up to 6 pumps operate in the area for the purpose of flood mitigation. Development of the area has occurred and it is considered that this infrastructure was provided for flood protection. Importantly, the review of different documents highlighted a requirement for pumping to be included as part of future flood mitigation scenarios, so it was considered reasonable to determine if these infrastructure items would be effective.	One model was run without operational pumps, with the remainder including pumps to operate at reported capacities and when water depth raised above 100- 200mm.

In aggregate, these changes represent a more comprehensive approach to developing an understanding of flood behaviour and impact.

By considering a range of catchment conditions a ‘narrative’ of flood mechanisms can be determined which is useful to understand risks and potential interventions to mitigate flood.

4. Limitations

Due to the scope time constraints and the availability of information there are a number of 'limitations' that may affect the fine grained assessment of specific flood behaviours, but (as the results will indicate) there are significant areas of agreement between the various approaches. For transparency, potential limitations are identified in Table 3 below along with commentary on how critical these may be.

Table 3. Potential limitations to be addressed

Limitation	Rationale	Response
Full inclusion of council pipe network	Without access to previous flood models it is not possible to fully represent the stormwater pipe network through the area. The City of Melbourne makes its drainage pipe network publicly through an Open Source platform.	A rudimentary pipe network was developed around the areas of interest based on Open Source information and basic assumptions around pipe cover and dimensions. The final result was a working flood model and pipes were checked to ensure that they were capable of receiving and conveying flood water, however there was no optimisation of this network. However, in large events the operation of the pipe network may not be sufficient to convey flows, and surface runoff will dominate. Where pumps are critical to relieve floodwaters, it is more important that water arrive at inlet locations, rather than the mechanism of arrival. In this regard the approach used would seem reasonable.
Choice of TUFLOW HPC modelling engine	The previous modelling works are expected to have been done in a previous version of TUFLOW, and as such concerns may be raised about the ability of the chosen approach to replicate. The HPC version of TUFLOW was developed to dramatically reduce runtime and improve model stability. It is particularly useful where there are sudden and complex changes in water level such as around buildings and direct rainfall application.	It is expected that the computational engine and adaptive time step approach used in HPC will have been robustly examined by the software vendors prior to releasing these software versions. MW technical specifications and TUFLOW manual recommend processes to check for time step changes affecting results.

	The model allows various hydraulic variables to be monitored throughout the model run and dynamically adapts the timestep to ensure correct model behaviour.	In addition, mass and volume balance checks can be examined. Our technical writeup will include Quality Assurance discussion supporting model suitability.
Bridges	Bridges were not explicitly modelled, however the terrain model derived from LiDAR was checked and the main creek channel was 'open' in areas where bridges were expected	The non inclusion of bridges may influence water levels in the main channel, however is unlikely to result in gross changes in flood levels. Bridges are not likely to have a direct influence on flood behaviour in areas of direct rainfall application.

5. Modelling Scenarios

In consultation with UWCS we have determined a number of scenarios to model as part of our engagement and are summarised as follows

- Existing conditions high flow Creek (no local runoff or operating pumps)
- Existing conditions high flow Creek (operating pumps and drainage infrastructure)
- Existing conditions modified for climate change (operating pumps and drainage infrastructure)
- Existing catchment with observed event
- Local catchment, ensemble storms to determine critical event
- Local catchment critical event (operating pumps and drainage infrastructure)
- Local catchment critical event modified for climate change (operating pumps and drainage infrastructure)
- Local catchment critical event (with drainage infrastructure but no operating pumps)
- Mitigation scenario with increased levees and pumping rates
- Quality assurance scenario to increase confidence in model performance

For model runs where matched sequences of streamflow and rainfall are available these have been used (i.e. Existing conditions). These models use a time varying flow (QT) upstream boundary, and static water level at the downstream boundary.

For model runs where rainfall inputs have been derived from the ARR datahub (i.e. local catchment) a different approach has been used and head versus time boundaries have been used for both upstream and downstream conditions. The upstream level has been generated from flow reporting lines inserted into the existing conditions high flow creek model just down from the inlet, and selected levels are approximate with elevated flow in the creek at a time when levees are expected to commence to breach as shown in Figure 2. Multipliers for input water level, rainfall and streamflow inputs have been used for climate change scenarios

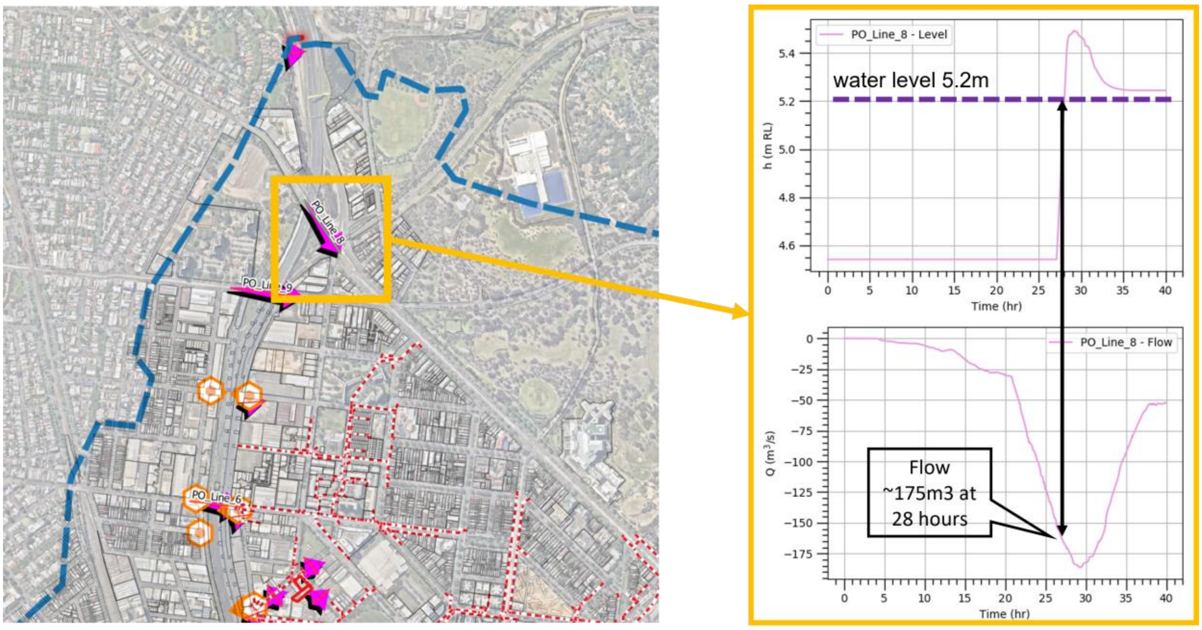


Figure 2. Stream level selection, Local catchment scenarios

This approach was adopted as a simplification to ensure models could be run in the time available, and is considered conservative as they imply that a peak flow in the creek coincides with a peak local rainfall event.

Given the vastly different times in catchment size (i.e. Moonee Ponds Creek versus North Melbourne) this is a highly improbable situation (significantly conservative).

Table 4 summarises the scenarios above and describes inputs used.

No	Scenario	Stream Flow used	Tailwater condition	Rainfall used	Piped included	Pumps operational	Other parameters
1	Existing conditions high flow Creek	Largest storm provided by UWCS based on FFA. 60 hours duration with 185.3m^3/s peak at 29 hours.	Tailwater corrected for AHD/ seal level datum anomalies as per UWCS. 0.62m AHD assumed for model duration	No rainfall input	No	No	Buildings and levees included as terrain modifiers. 40 hour run duration to clear peak and for floodwaters to start to recede (as per animation). 3m grid
2	Existing conditions high flow Creek	As per (1)	As per (1)	Rainfall sequence matched to creek as suppled by UWCS. Building omitted from Rainfall application around buildings and a corrective factor of 1.3 used based on scaling of building to application area.	Yes	Yes	As per (1)
3	Existing conditions modified for climate change	Flows in creek (1) adjusted for 2100 climate change scenario as per UWCS (i.e. by 14%, increases to 211 m^3/s at 29 hour)	Tailwater increased to 1.29m AHD as per UWCS 2100 climate change scenario	Rainfall used in (1) increased by 14% as per UWCS Climate change scenario. Correction for buildings as per (2)	Yes	Yes	As per (1)
4	Existing catchment	Flows corresponding to 2010 flood included. Peak of 26.75m^3/s at 2.6 hours,	Tailwater adapted at 0.8m based on	Rainfall corresponding to Melbourne Gauge at time	Yes	Yes	As per (1)

	with observed event	and a 10 hour sequence provided.	assessment of mean tide.	of flood event used, as provided by UWCS. Correction for buildings as per (2)			
5	Local catchment, ensemble storms	Flow introduced at 5.2m as constant level.	Tailwater at 0.62m adopted	Full ARR ensemble used for 10min, 20min and 1hour events	Yes	Yes	No buildings Model duration ranges from 1 hour to 2 hours to allow flow to propagate. Coarser grid (5m) to assist with run times.
6	Local catchment critical event	Flow introduced at 5.2m as constant level.	Tailwater at 0.62m adopted	Direct rainfall applied as per (1) but with 20min, tp04 storm selected for 1percent AEP event used as critical storm Correction for buildings as per (2)	Yes	Yes	Buildings and levees included 3m grid 2 hour runtime
7	Local catchment critical event modified for climate change	Stream level increased by 14% to 5.98m consistent with climate change assumptions	Tailwater increased to 1.29m	Direct rainfall for critical event multiplied by 1.14. Correction for buildings as per (2)	Yes	Yes	Buildings and levees included 3m grid 2 hour runtime
8	Mitigation Scenario. Existing conditions with high flow creek and	As per (2)	As per (2)	As per (2)	Yes.	Yes. All pumps increased by a factor of 10	Buildings and raised levees (0.7m increase) included. 3m grid 40 hour runtime

	operational pumps						
9	Local catchment critical event, but without pumps	As per (6)	As per (6)	As per (6)	Yes	No	As per (6)
QA	Quality Assurance run. Replicates Scenario 4	As per (4)	As per (4)	As per (4)	As per (4)	As per (4)	Model control number reduced to 0.8 to determine impact on results. All other parameters as per (4)

Table 4. Scenarios Modelled

6. TUFLOW Setup

Several TUFLOW variants were used for the modelling and are described in the Table above.

In summary

- Model variant 1 was used for Scenario 1 and was developed to not include pumps or pipe elements but included ‘building’ layer which contained levees. This had flow varying upstream boundary and a tailwater associated with discharge into the Yarra.
- Model variant 2 was used for Scenarios 2, 3 4, 8 and quality assurance runs. It includes buildings, pumps and local stormwater drains. This had flow varying upstream and rainfall application zones over the local catchment and wider catchment. Tailwater associated into the Yarra was used.
- Model variant 3 used for Scenarios 5,6 and 7. It used the same inputs as Model 2, however the inflow was adjusted to allow head versus time inputs upstream as well as downstream.

Figure 3 and Figure 4 show the TUFLOW setup for whole of model.

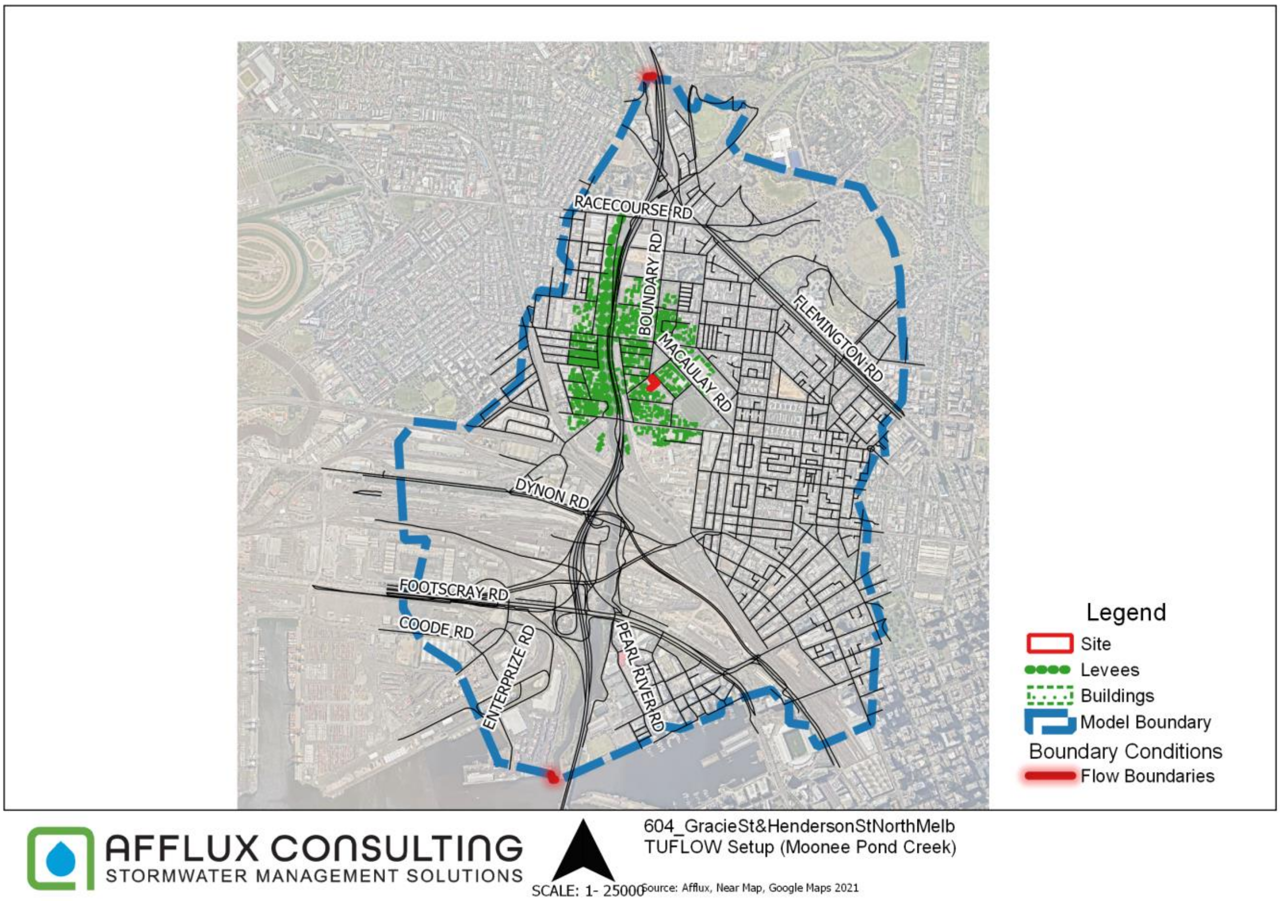


Figure 3. TUFLOW setup- Scenarios 1, Model scale

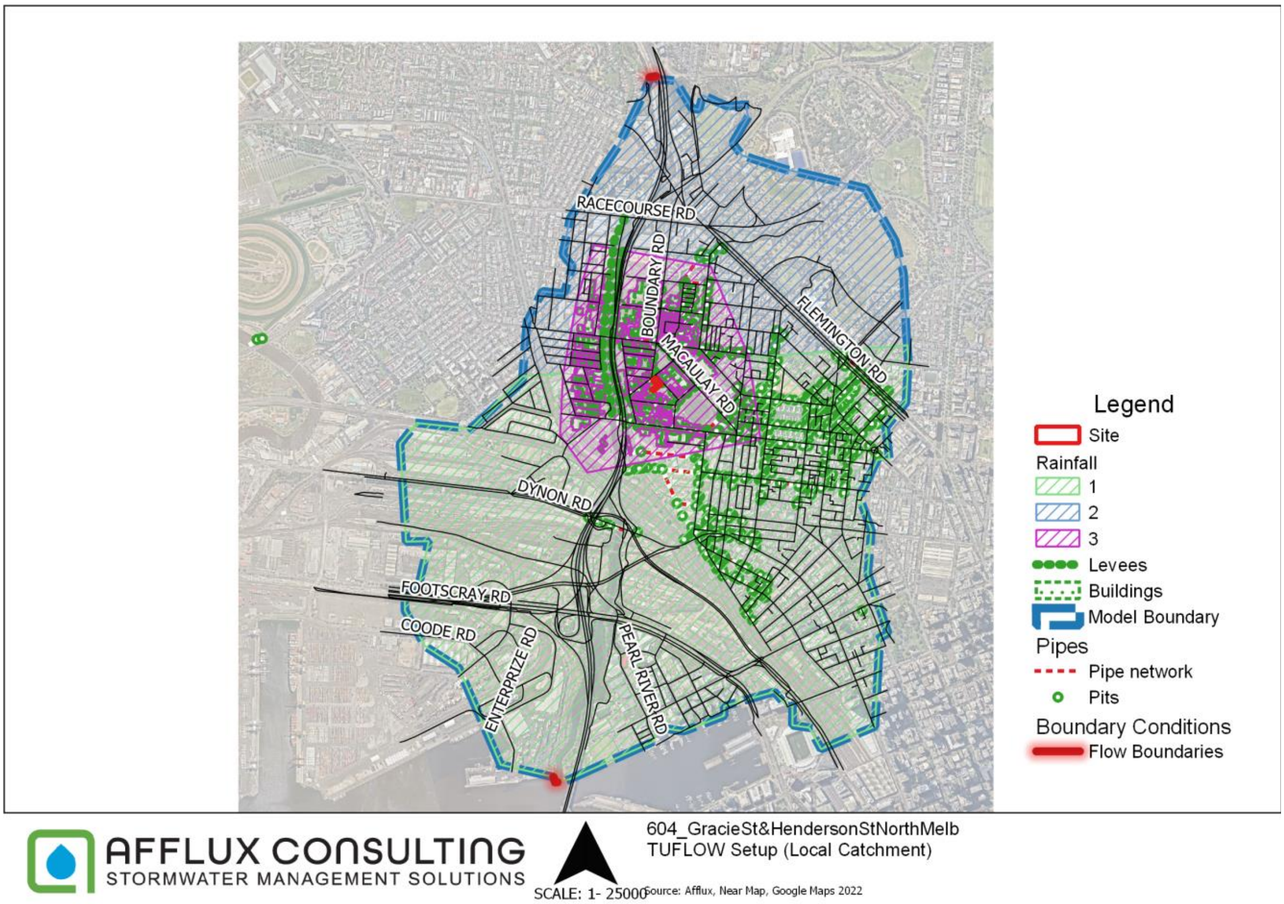


Figure 4. Model setup, Scenarios 2-7

Terrain used in the model was sourced commercially from LiDAR collected as part of the Melbourne wide collection project in late 2017 and early 2018, and the extent of the data obtained in shown below in Figure 5.

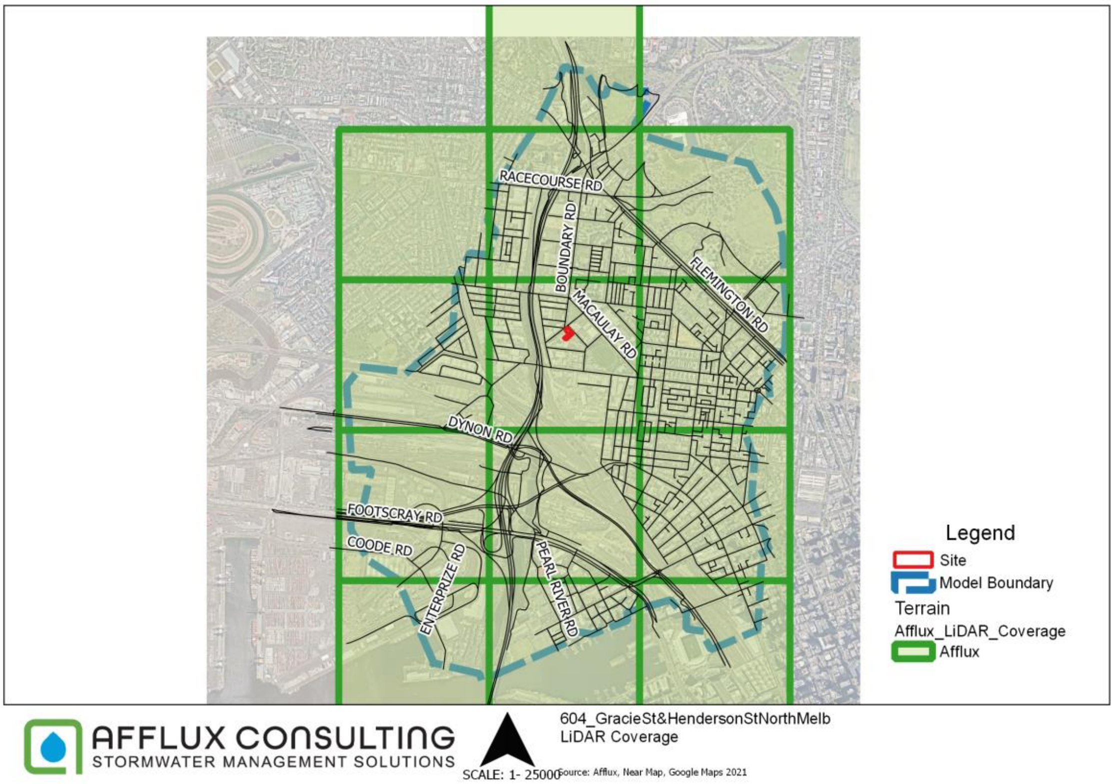


Figure 5. LiDAR coverage used in model

The information used is available at a gridded 1 metre resolution, and a vertical accuracy of 0.1m. From our examination of sample sections along the creek alignment it appears that the effect of the freeway flyover and bridges has been removed. Based on this, it was determined as prudent to explicitly represent the levees along the creek as a 'z shape' which allows the terrain to be locally raised to a nominated level which was based on information obtained from the Arden Macaulay Flood Levee failure report.

It is important to note that landform doesn't remain static over time, and that there have been several changes that have occurred in the catchment since the data capture (such as rail works in the vicinity of the North Melbourne station and the construction of a building on the corner of Canning Street and Vaughan Terrace).

The building information that was used was obtained commercially and comprises of building outlines which are digitised from remotely sensed imagery using a combination of automated and manual processes to identify, extract and orthogonalize objects resembling a building structure greater than 9m²

It is possible that the terrain used in the Planning scheme modelling was based on earlier LiDAR capture. While there is mention of survey being imported for the Moonee Pond creek there is no mention of terrain used in other parts of the model. Given the timing of the work, and the reliance on earlier models (i.e. AECOM) to provide a starting point it is probable that the 2008 LiDAR information collected across Melbourne would have been used.

Both this study and the Planning scheme studies have been commissioned to understand the broad nature of flood behaviour and it is not considered that changes to individual buildings will influence the 'type' of flooding. Indeed, the purpose of Planning Scheme Overlays is to take this catchment understanding and to use it to influence future development to reduce or manage risk or develop mitigation options.

7. Modelling Results

Results have been prepared for each Scenario 1- 8 are shown sequentially in Figure 6 to Figure 19 (Update) with a brief explanations provided where appropriate. Only depth has been reported as it is expected this is the most instructive parameter to understand what is happening with local flooding.

No results have been presented for other parameters (e.g. hazard or Water Surface Elevation).

Results are displayed for the model extent, and for an area buffering the site (approximately 200m).

Depths have been filtered to removed flooding depth below 50mm as per agreed approach with UWCS. It is difficult to substantiate flooding below this depth based on limitations with LiDAR and terrain sampling methods.

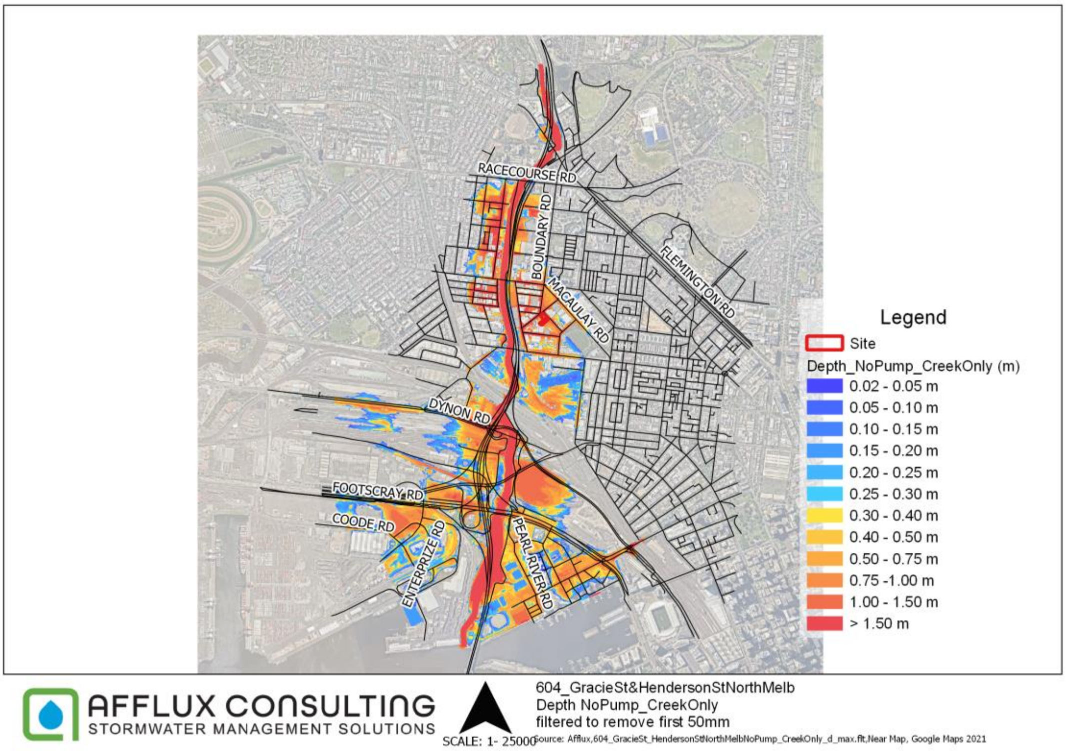


Figure 6. Scenario 1 flood depth- Model Extent

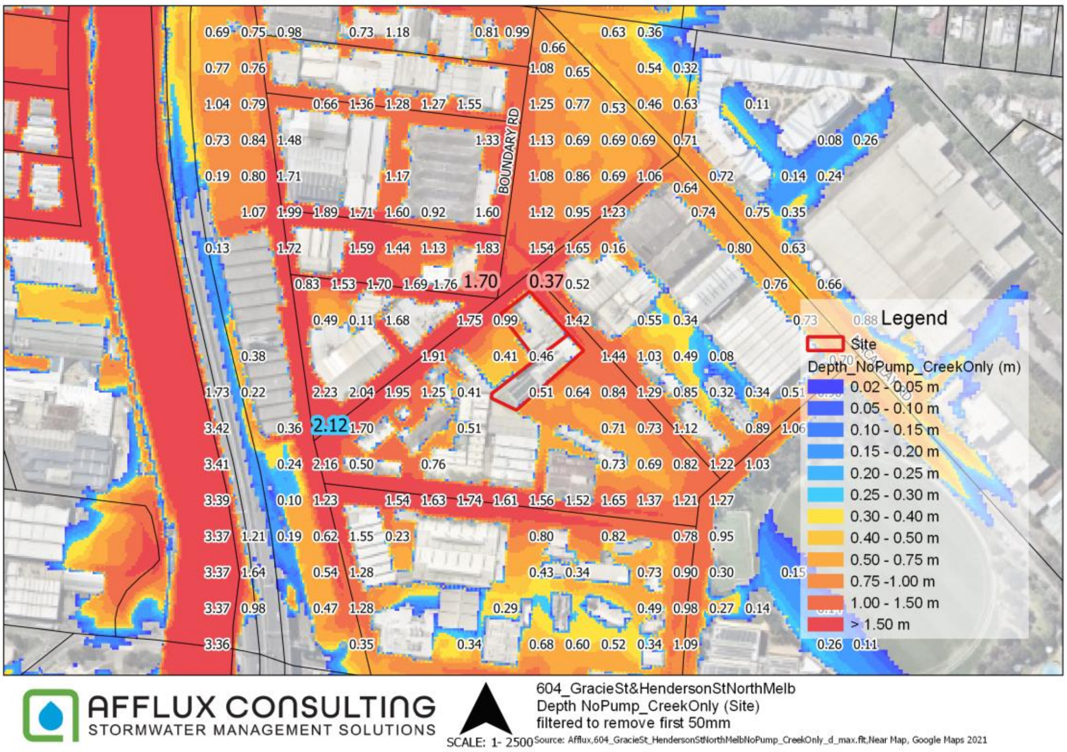


Figure 7. Scenario 1 flood depth- Site vicinity

Flooding is evident over much of the area to a depth of around 1.0- 2.0m in the vicinity of the site and extends to around 500 metres away from the main creek channel albeit at reducing depths.

This flooding is associated only with water escaping from the creek channel. It is expected that if local rainfall was applied the depths would increase slightly and the extent would be reach further into the surrounding catchment.

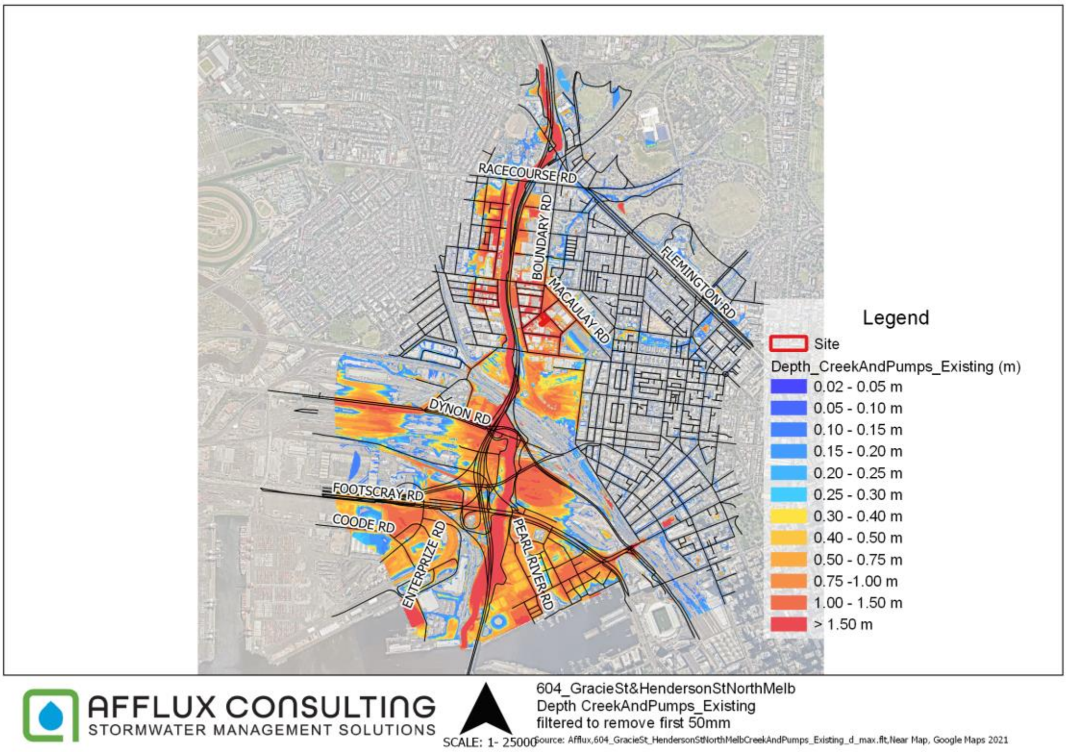


Figure 8. Scenario 2 flood depth- Model Extent

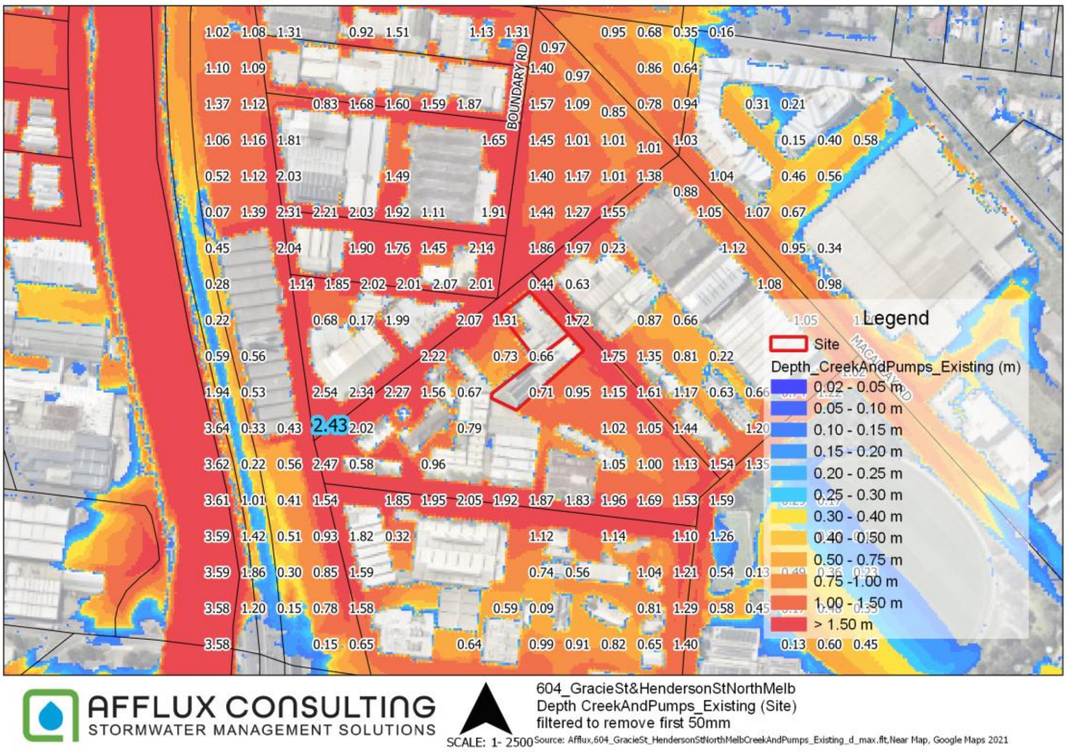


Figure 9. Scenario 2 flood depth- Site vicinity

With operating pumps the extent of flooding is similar to the no pump scenario, but increased in depth by around 200-300mm, likely as a result of runoff generated from the local catchment. The maximum depth as compared to the no pump scenario, has increased, and is largely contained within roadways. Deeper flooding is evident at some locations (e.g. the corner of Langford Street and Gracie Street which represents a low point, and is likely the reason pump stations have been located there). The pump rates however are not sufficient to cater for the combined effect of water passing behind the levee and runoff generated from the local catchment, and measure to address both these causal effects may be required to address the issue.

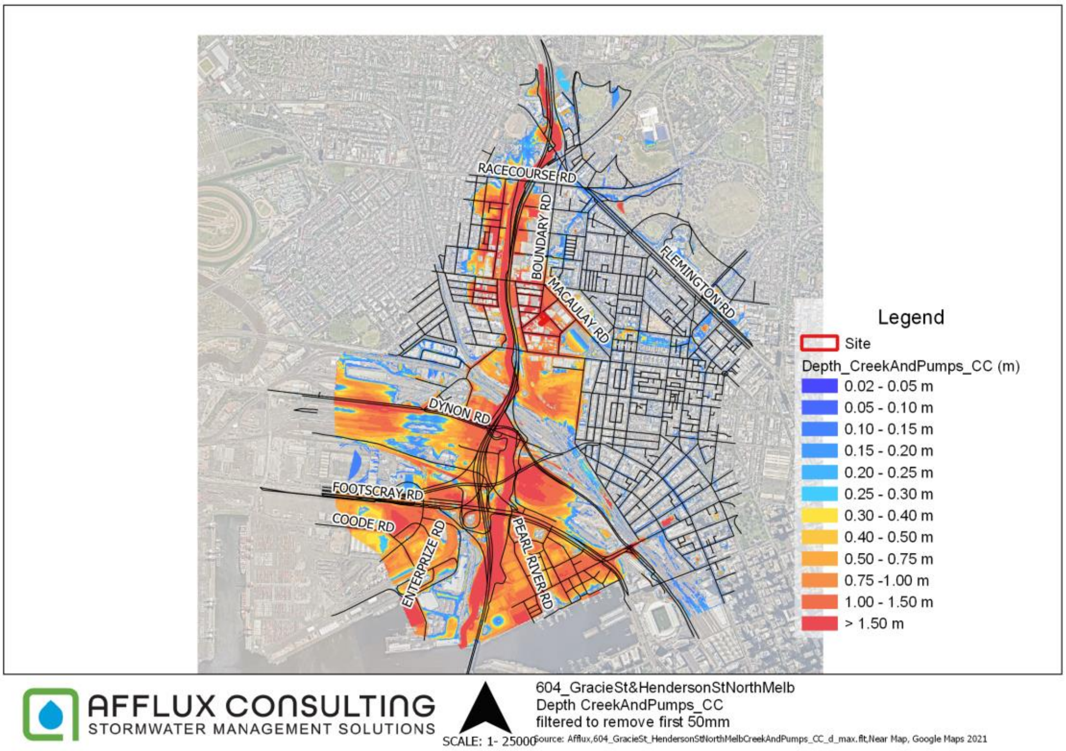


Figure 10. Scenario 3 flood depth- Model Extent

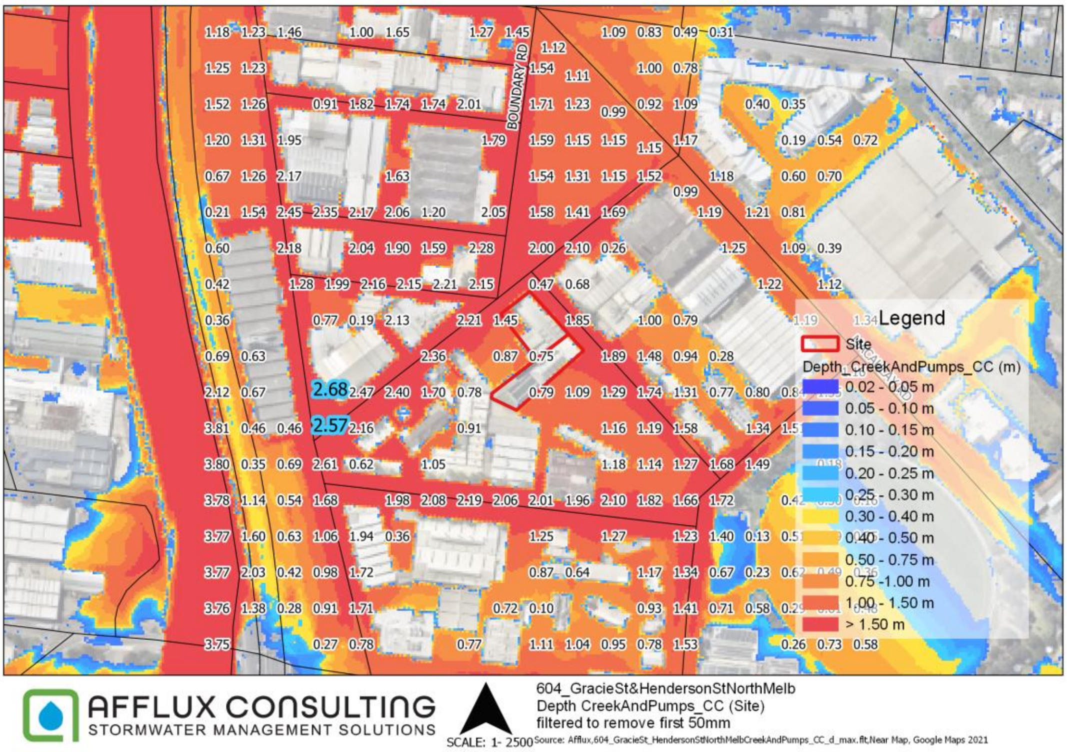


Figure 11. Scenario 3 flood depth- Site vicinity

Compared with the previous scenario flooding is of a similar extent, however increased by a further 150 – 200mm and suggest that climate change effects will worsen the situation.

It should be noted that the climate change impact of simultaneously increasing rainfall intensity and streamflow may be conservative as it is not certain that these will occur in this manner.

The same flooding mechanism is evident as in previous scenarios; given the planning horizon these longer trends in catchment condition should be factored into the assessment of risk and mitigation options.

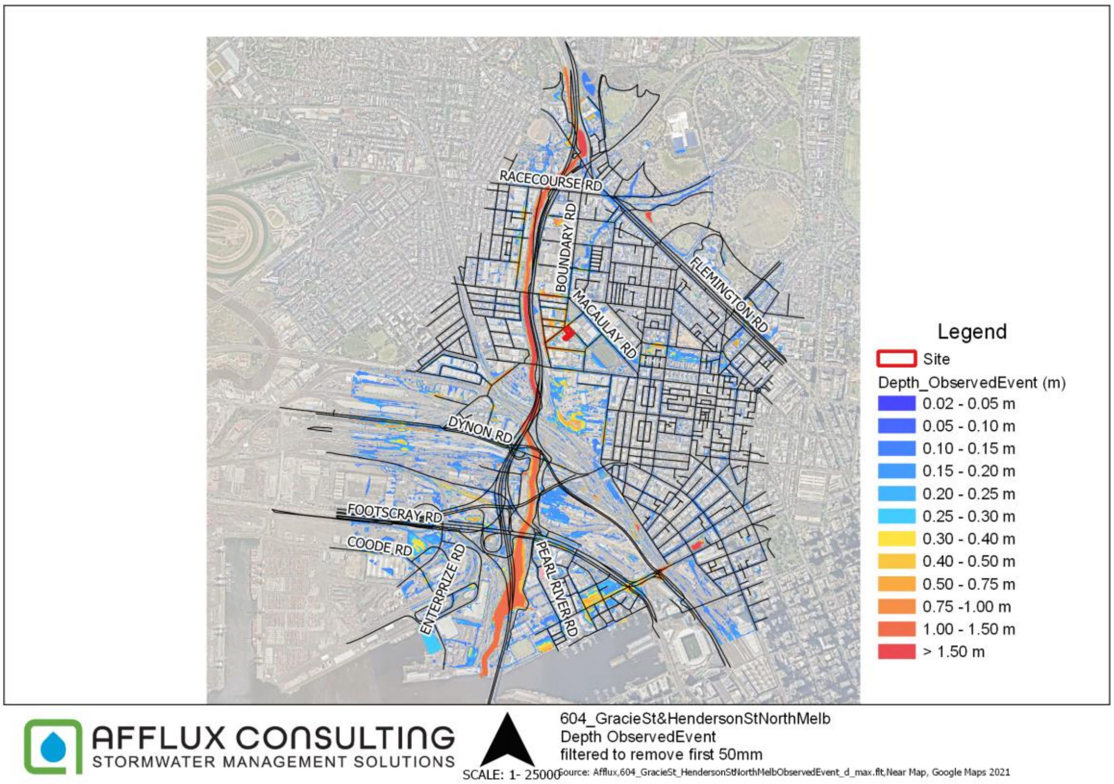


Figure 12. Scenario 4 flood depth- Model Extent

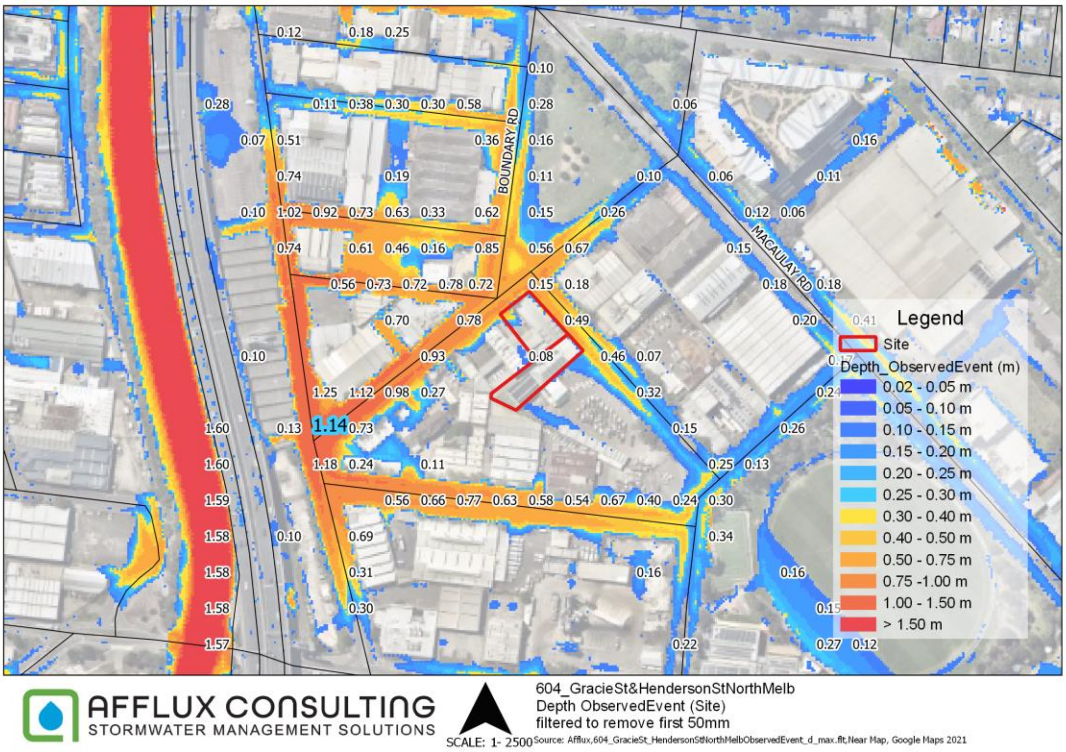


Figure 13. Scenario 4 flood depth- Site vicinity

The extent of flooding is reduced when compared with previous scenarios, and low spots at the end of Gracie Street are evident. The reduced flood extent is likely influenced by a shorter duration modelled event, significantly reduced water levels in the creek and lower intensity rainfall when compared with the 1%AEP critical event (i.e. observed rainfall cloudburst delivered around 20mm of rainfall in 45 minute period compared with 29mm in 20 minutes).

Water levels reported in these results can be compared with observed reports of flooding.

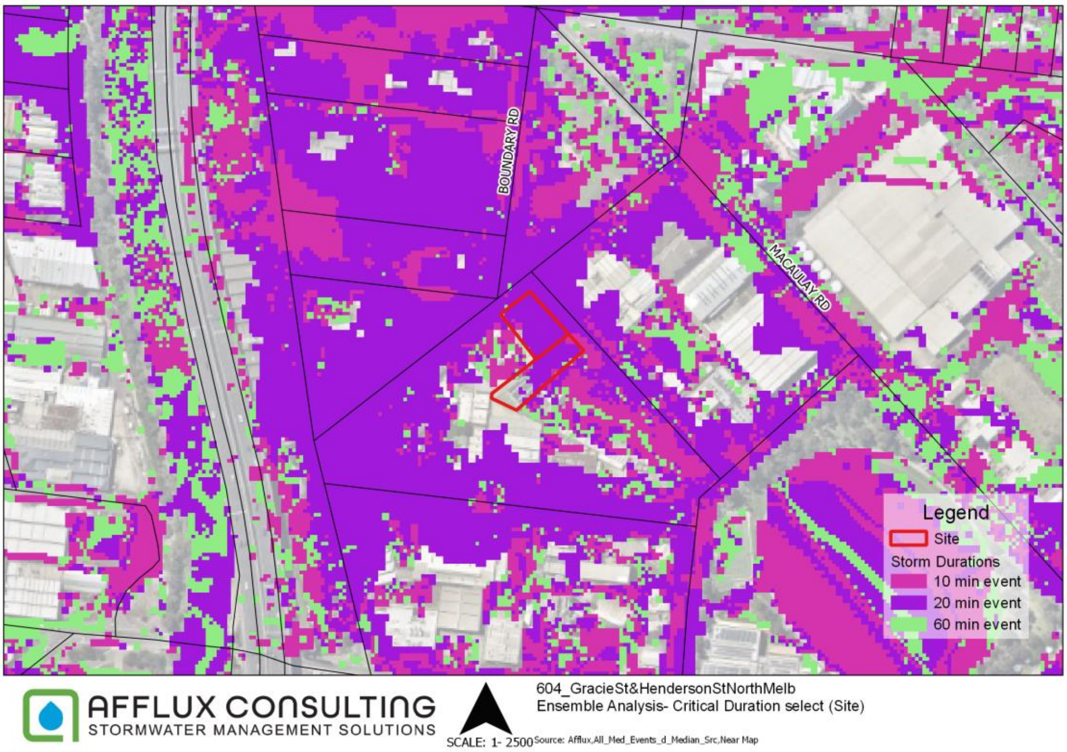


Figure 14. Scenario 5 Critical Storm analysis, Select duration

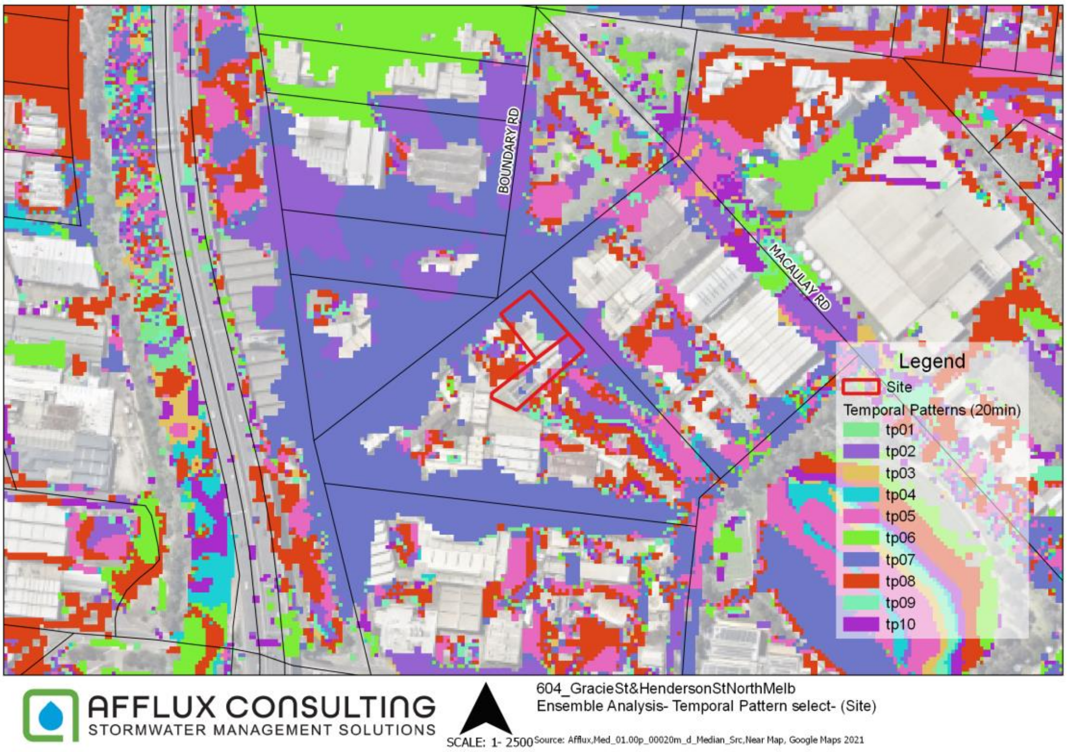


Figure 15. Scenario 5 Critical storm temporal pattern selection

The critical storm temporal pattern was selected using a median event approach as recommended in ARR2019 and using the TUFLOW utility functions. The critical storm duration of 20 minutes was selected first as being representative of flooding around the site. Temporal pattern 7 was selected from a median event analysis in the vicinity of the site, and was chosen because the majority of flooding is expected to (initially) occur in the road areas.

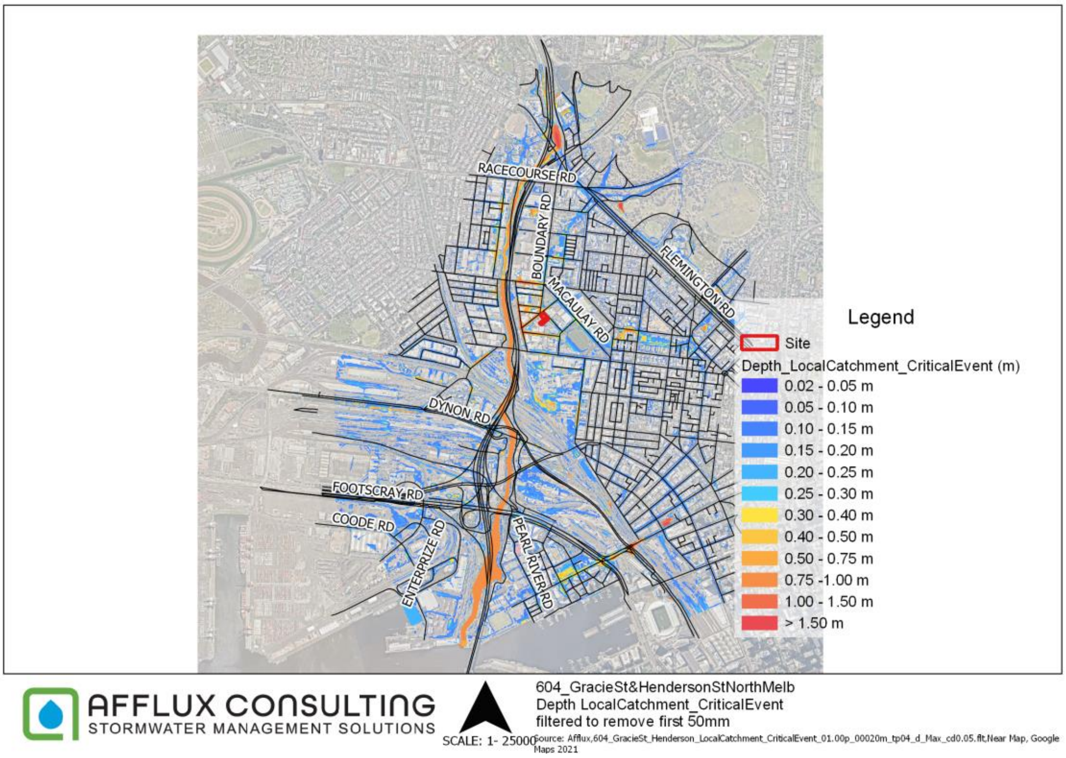


Figure 16. Scenario 6 flood depth- Model Extent

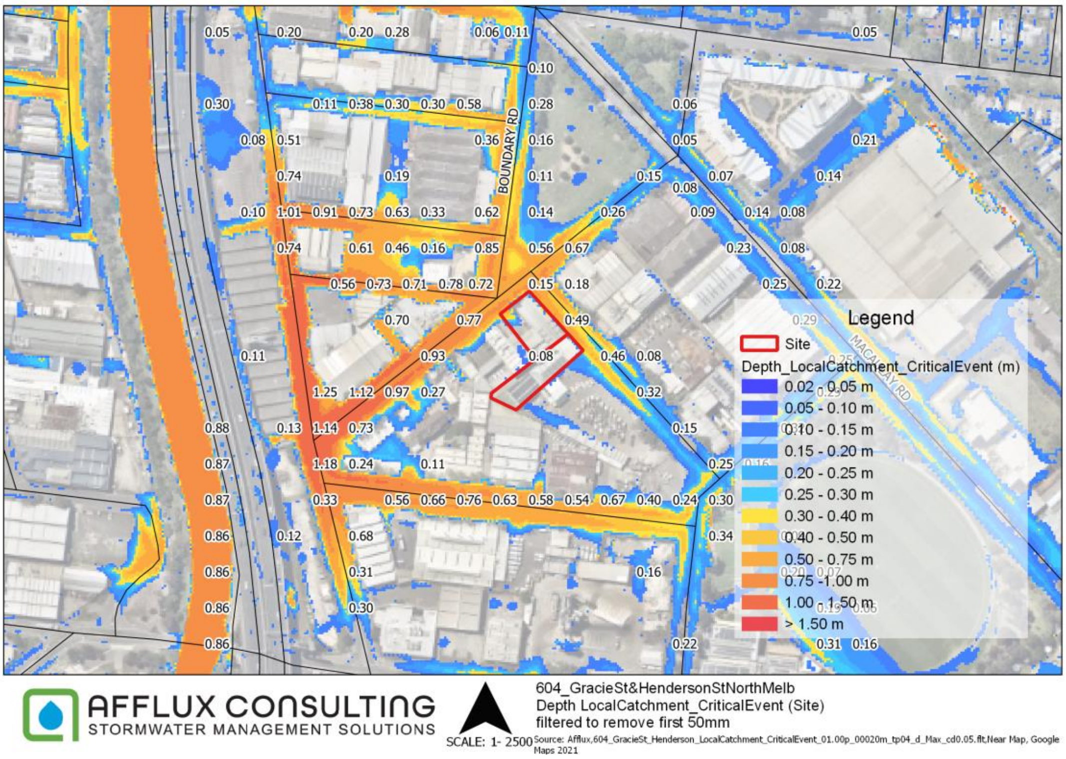


Figure 17. Scenario 6 flood depth- Site vicinity

Scenario 6 shows the elevated water levels in the channel commensurate with a higher flow event. Corrected tailwater and critical rainfall event applied over the catchment.

The results show that the floodwaters are generally contained along roads for the local catchment and that pumps operate to reduce flooding levels to around half that reported in previous scenarios.

The extent of floodwater impacting private property is generally reduced.

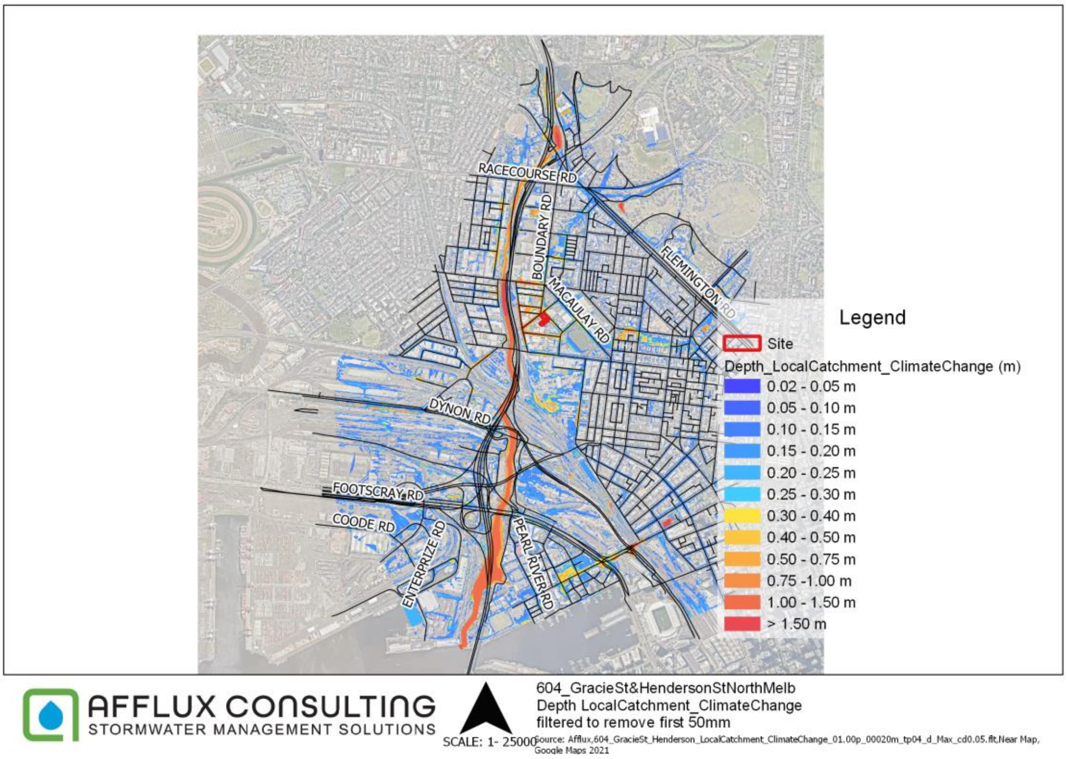


Figure 18. Scenario 7 flood depth- Model area

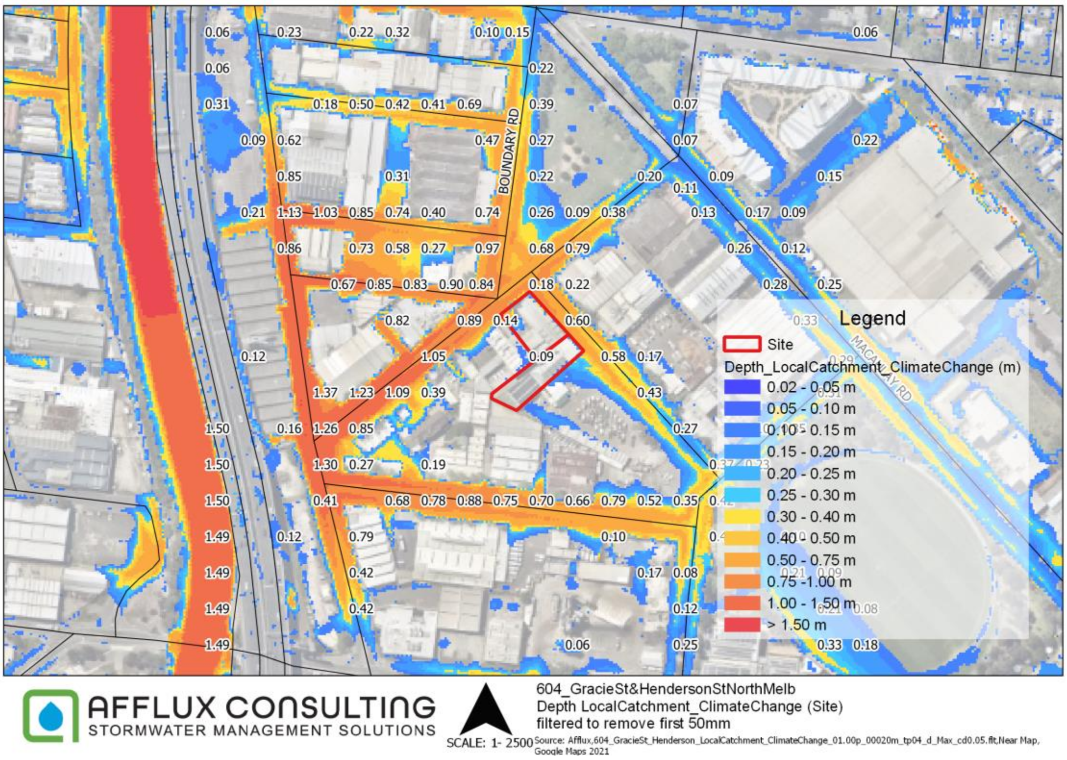


Figure 19. Scenario 7 flood depth- Site vicinity

Scenario 7 shows the effects of elevated water levels in the channel, elevated tailwater and estimated rainfall over the local catchment for the 2100 climate change event. Flooding is still well contained to road ways across the majority of the study area, and the pumps are able to prevent the broad inundation of areas behind levees.

Water affecting private land is reduced, suggesting that pumps may be a viable option for managing flooding occurring as a result of rainfall on the local catchment.

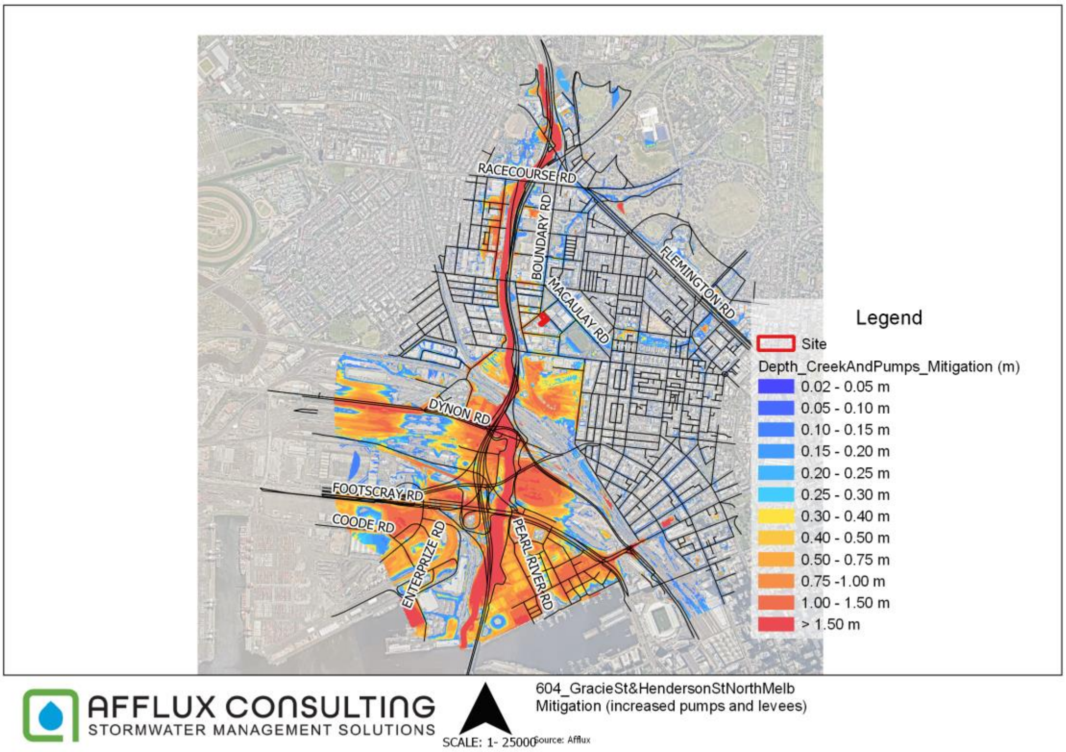


Figure 20. Scenario 8 flood depth- Model area

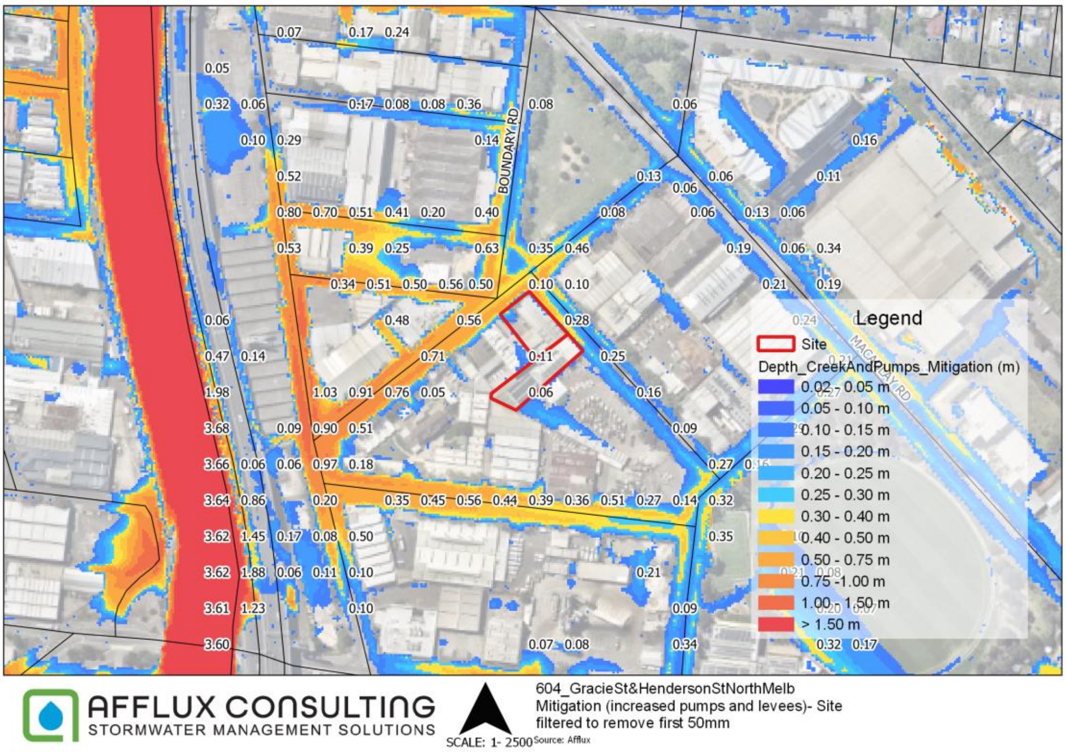


Figure 21. Scenario 8 flood depth- Site vicinity

The results show the reduction in maximum flood depth as a result of increased levees and pumping rates. The maximum flood depth is generally reduced by over 1 metre and importantly the flood impact on private property is effectively eliminated in the vicinity of the site.

It is important to note that these results show maximum depth reported, however the presentation obscures the dynamic nature of flooding in the area. Factors such as inundation time and duration are not possible to determine, and as such these plots do not provide an entire representation of risk and management options.

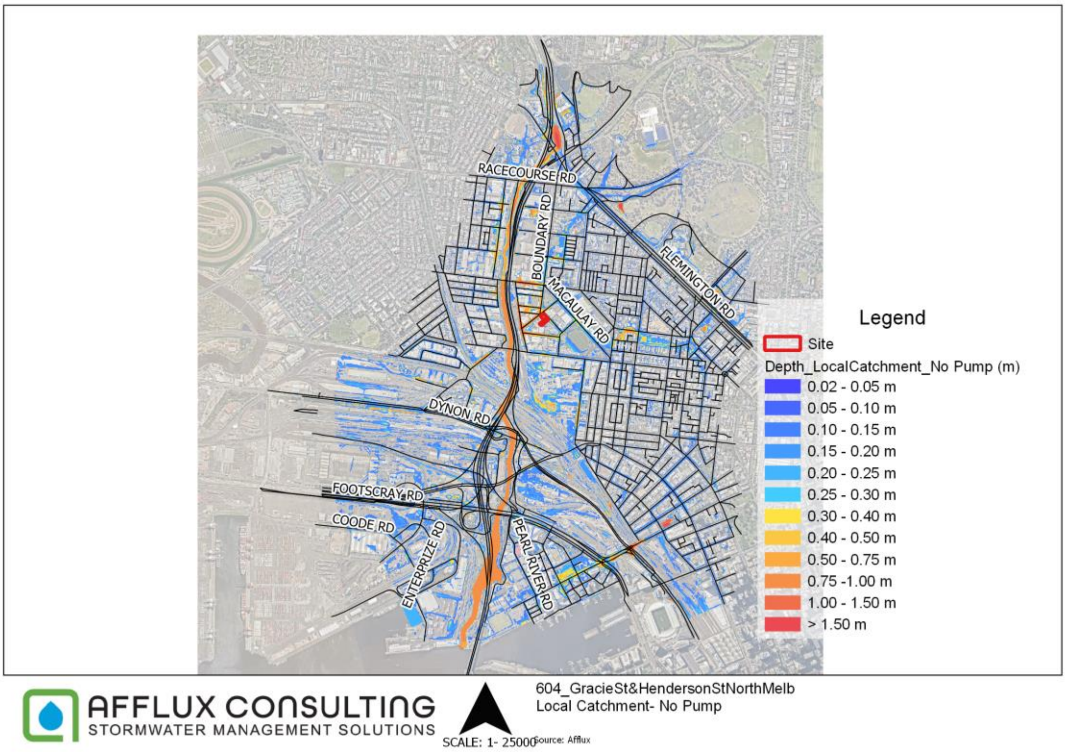


Figure 22. Scenario 9 flood depth- Model area

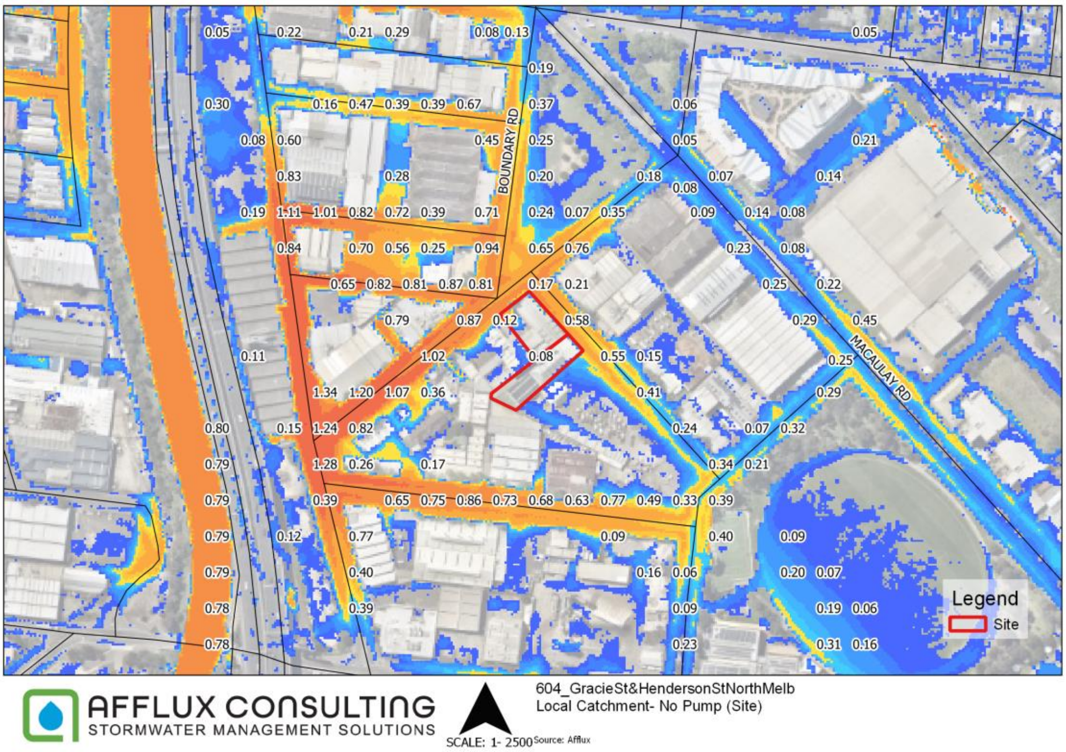


Figure 23. Scenario 9 flood depth- Site vicinity

Flood depths are increased slightly (by around 100mm) when pumps are not operational and local catchment runoff is able to impact. This scenario includes water levels in the creek which are known to be capable of breeching levees, while the capacity of pumps to address this are limited. The location of pumps and drainage have not been optimised, but it is of note that the flood mitigation options proposed by others suggest that these may be viable and desirable inclusions for the area.

8. Discussion

The modelling undertaken as part of this exercise serves to highlight a number of important factors in describing the flooding mechanism in the area, and serves to highlight the nature of flooding at different locations.

As with any modelling exercise the choice of input assumptions is a critical factor that can dramatically affect the end results. Just as importantly, the representation of results can influence the interpretation of results and decision that are subsequently reached.

The choice of flood depth as the main parameter to report was deliberate, it allows the nature of flooding to be interpreted intuitively- we all know the difference between a puddle and swimming pool with a shallow and deep end which all have differing levels of associated risk.

Broadly speaking, there is a high level of agreement between the higher flow scenarios reported in this study when compared with other modelling efforts that have been reviewed. The differences in absolute flood level can be explained by two key factors, the higher flow rates applied directly to the Moonee Ponds Creek (i.e. 185m³/s compared with around 260m³/s) and assumptions made around the operational statis of pumps. This study has been characterised by inputs derived by UWCS after an independent assessment of flood frequency and tailwaters at interfaces with the Yarra River at the lower end of the model. These have been accepted as starting assumptions, and we have not attempted to represent flood levels using previous parameters. We expect that if we were to do so, we would arrive at similar results, and as such the veracity of the model setup is considered robust.

The mechanism of flooding in the North Melbourne area and site vicinity appears to be dominated by two simple mechanisms, described as follows.

Breakout of flow from the Moonee Valley creek channel is directed into adjacent areas where it becomes trapped in low lying areas. Without the assistance of pumps this water remains trapped.

This is illustrated in Figure 24 and Figure 25 and Figure 26 below which shows a time varying snapshot of Scenario 1 (No pump) and Scenario 2 (with pump) and Scenario 8 (with mitigation options). Separate animations are available for each of these animations and can be supplied as required.

These show the effectiveness of operational pumps (and in the case of Scenario 8, levees) in reducing flood levels over time. In scenario 8 the increased levees are effective at preventing initial ingress of floodwaters into low lying areas around the site.

The plots suggest a dynamic nature of flood response could be entertained if safety and risk are adequately addressed. As long as the floor levels of buildings and entrances are adequately protected adequate warning time of advancing floodwaters could triggers a flood response. If access and egress routes are not unduly impacted by major flood warnings could be used to enact a flood response, and a management option could be included to complement traditional infrastructure approaches. Considering extreme flood events are rare sites could continue to operate productively in between flooding episodes, and Flood Response and Management Plan may provide another option.

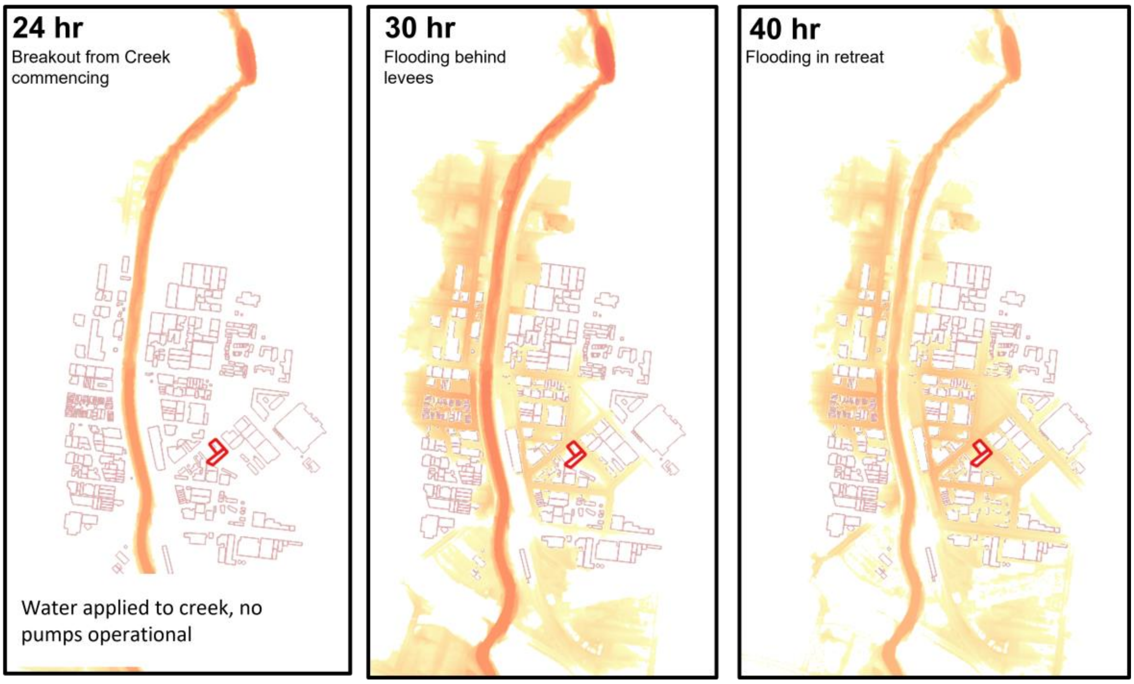


Figure 24. Flood advance and retreat without pumps

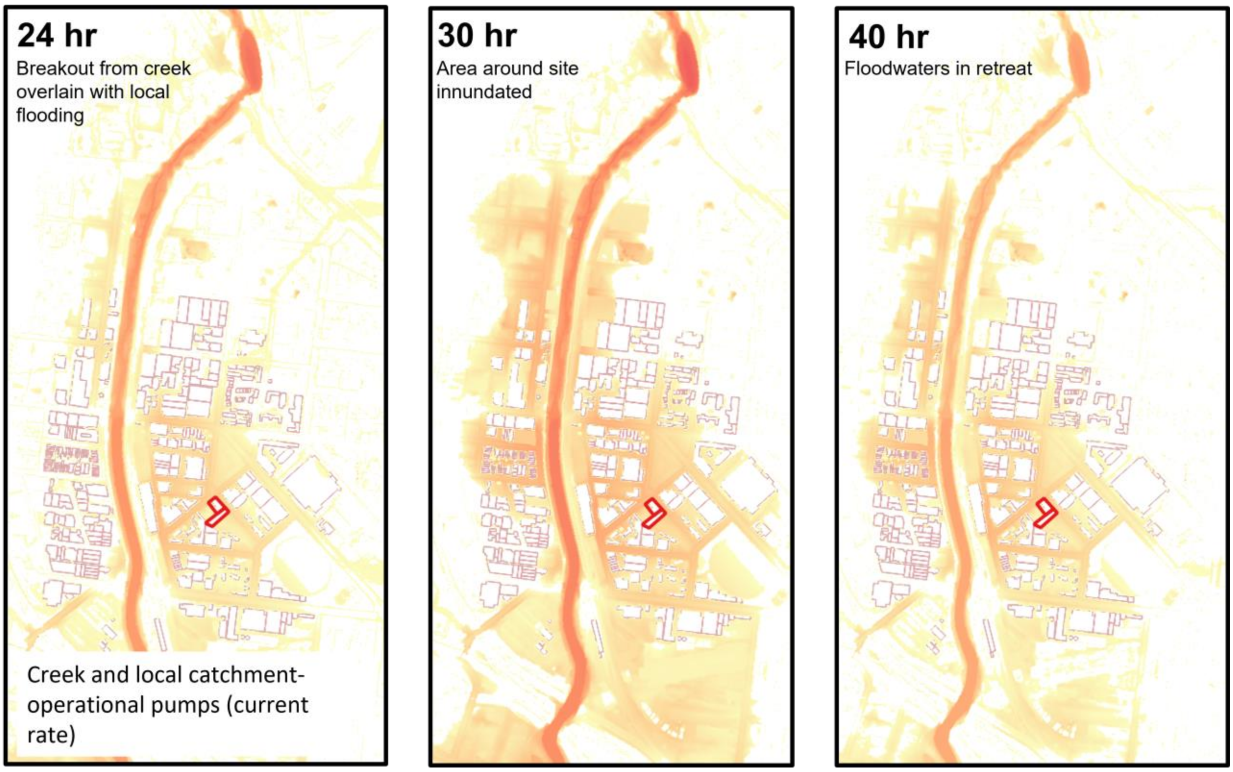
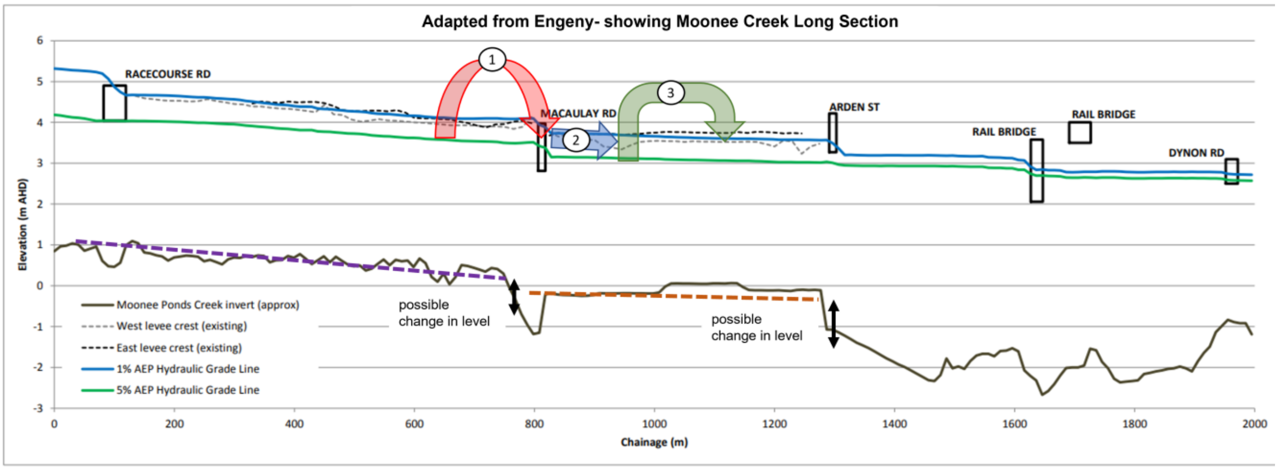


Figure 25. Flood advance and retreat with pumps

Figure 26. Flood advance and retreat with pumps Mitigation scenario-

It is considered likely that the Moonee Ponds Creek is subject to a change in level in the vicinity of Macaulay Road and again at Arden Street. Figure 27 adapts a creek long section that has been obtained from a flood mitigation report prepared by Engeny and shows step changes in grade along the watercourse. It is possible that there may have been a section of rapids or a small waterfall in historic times. Development of the area and building has likely obscured some of this underlying topography, but the mechanism of water being 'pumped' back into the creek below the waterfall after breaching the upper section appears a consistent explanation of what is occurring.

The same figure also illustrates this mechanism of levee breach and pumping back into the creek at a lower elevation.



1. Water breeching levee flows into low lying land adjacent to creek
2. Water flows along streets outside levee toward low point
3. Pumps operate to lift water into creek channel

Indicative slope of creek invert (Racecourse Road to Macauley)

Indicative slope of creek invert (Macauley Road to Arden Street)

Source: Adapted from Engeny

Figure 27. Moonee Ponds Creek- Long section and grade changes

From our preliminary assessment it also appears that the sizing of the pumps and levees is ‘underdone’ to cater for the 185m³/s event used in our model and prevent widespread flooding through the area (i.e. largely contained in roads). This observation this remains consistent with the increased climate change flow and rainfall scenario which extend the peak flow to above 200m³/s.

The operation of the pumps does appear to have an impact on the level of flooding that occurs in scenarios that are characterised by local catchment flooding, however this will be sensitive to the exact location of the pumps in relation to terrain and drainage infrastructure responsible for delivering local runoff to the umping locations. This is a consideration that has been included in the 2021 mitigation options presented by Engeny, however it has not been possible to replicate this detail in this study due to limitations.

The difference in flood depth between the local catchment scenarios with and without pumps is in the order of 100mm. It is speculated that the volume delivered by water from the creek is a dominating factor. As indicated by the ability of pumps to reduce flooding in various scenarios where they are operational these are likely a viable mitigation factor if the water from the creek is able to be exclude (i.e. by raised levees).

It is apparent from our results that the nature of flooding is not consistent throughout the study area, and that there are areas where water is deeper than others, and this is influenced by operational pump conditions.

The reporting of flood depth is an important consideration, and we have deliberately chosen a palette developed by the City of Melbourne to show flooding. The reality (and perception) of risk is vastly different if water is only centimetres deep as opposed to when it deep enough for people to lose their footing or cause vehicles to float.

In previous work we are familiar with the ‘blanket’ approach used in the preparation of flood related overlays- in providing an extent of flooding they obscure additional information such as depth, hazard or direction of flow. It is acknowledged that overlays are intended as a mechanism for additional information to be provided and where appropriate, further advice be sought.

The purpose of the overlay and the supporting information needs to be interpreted with the correct context. It would be misleading to contend an entire area was subject to significant flooding depth if infrastructure designed to provide protection was installed and operating correctly, the overlay should still allow development in the area to avoid excavation adjacent to flooded areas (e.g. where there is a risk of basements becoming flooded).

The difference between approaches in shown in Figure 28, while the overlay covers a wide area, the flooding itself is largely contained within road areas.

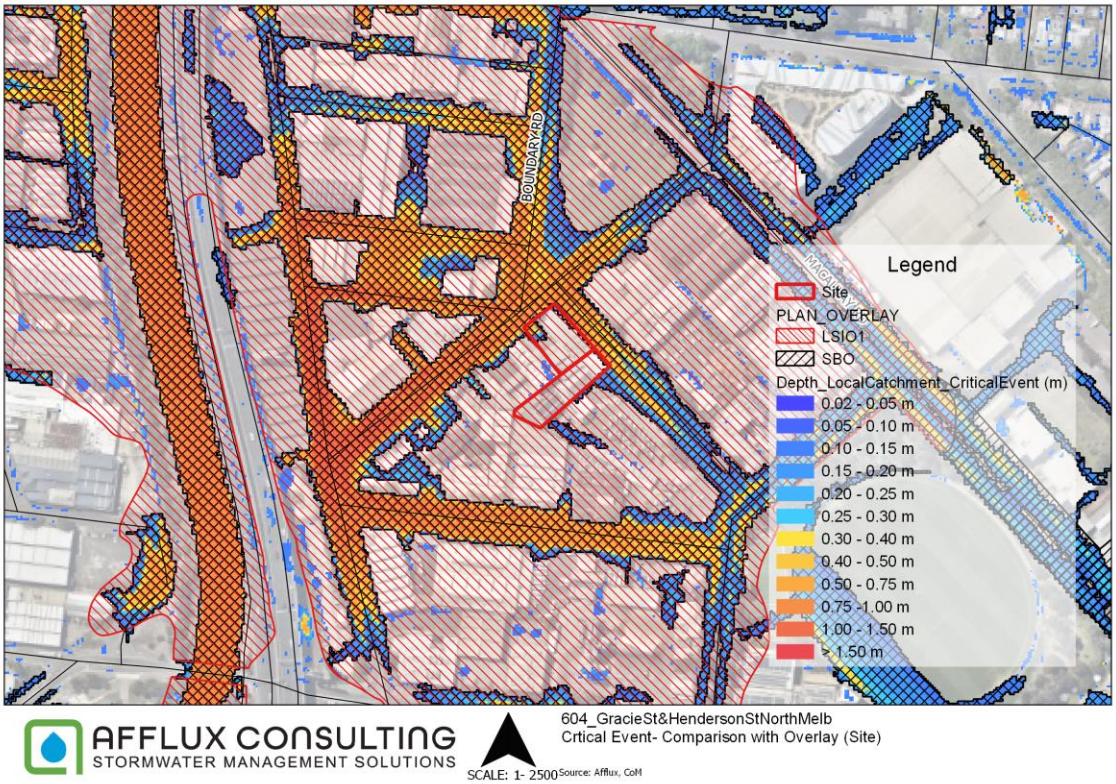


Figure 28. Comparison of modelling results with existing Planning Scheme (LSIO)

Finally, the dynamic and practical nature of flooding needs to be considered along with the likelihood of events that will lead to an occurrence. Creek flooding is likely to lead to deep inundation if it is not able to be contained behind levees. If pumps required to relieve this type of flooding (which is dominant, even if rare), they are likely to be successful in relieving local flooding which is of a greatly smaller nature.

If local floods occur but can be managed to limit duration or magnitude of impact, and are unlikely to affect individual landholdings and key egress routs can be maintained this should be acknowledged in any response.

Flood Management and Response Plans can be developed to further manage risk. Reinforcing of levees can be employed to reduce the occurrence of flooding from the creek. Pump operations can be optimised to lower and relieve flood impacts. In combination these are viable measures to manage flooding and preserve the viability and value of land in the area that would otherwise be relatively flood free.

9. Quality Assurance

The modelling undertaken in this exercise is based on the best information available, and in some instances extracted from reports or provided as inputs under instruction.

A model is a tool that can be used to represent a real world situation. While it is possible to use simple checklists to sign off on model health and suitability, we also expect that a QA process should have some narrative appeal that allows the professional expertise and judgement of the modeller to be used.

As such we believe the following steps are pertinent as part of a QA process

- Walked catchment to gain an appreciation of terrain and likely issues to be encountered
- Review of historic reports of flooding and photographic evidence
- Calibration of model runs as follows:
- One run against historical data (see below)
- No pump scenario broadly agrees with extents determined by others
- Detailed modelling underpinned by independent, 'first principles' analysis provided by UWCS
- Choice of model grid resolution and timesteps appropriate to characterise terrain
- High quality terrain inputs obtained from latest commercial sources
- Additional layers included to represent real world influences (pipes, pumps, buildings)

Calibration to an observed event

Perhaps the strongest evidence of a model suitability is when it is able to predict or replicate real world events. With UWCS we have been able to obtain relevant information of a significant storm that hit Melbourne in March 2010 and for which there were contemporaneous newspaper reports of flooding in the vicinity of the site. Photographs taken show flooding at the corner of Gracie Street and Langford Street and we have reviewed key site features to ascertain that the location is correctly specified.

We have undertaken a scenario to replicate a real event that is known to have caused flooding in the local area and cross referenced our results against the reports of flooding behaviour as reported in the CoM Flood plan

Figure 29 shows vehicles trapped in flood waters and are compared with the levels reported in the model run for the actual event. Modelling where the maroon car is located appears to align reasonably well with the modelled flood depth- water is deeper toward the front as the car dips down, and the flood waters become shallower toward the western edge of the road. Absolute depth reported (~1.2m) is slightly higher than the flood level shown on the impacted vehicle (which is estimated to be around 0.6- 0.7m in depth). Flooding extends some distance along the road at moderate depths as evidenced by the vehicle driving through the floodwaters.

The slightly higher levels reported in the model are possible explained by several factors based on the scenarios which identified that local catchment runoff is a significant factor when levees are not breached.

- The simplified approach to introduce catchment runoff with no loss factors will lead to an overestimation of flood depth. Based on the nature of the study and available information, we have not sought to verify catchment surfaces and assign loss parameters.
- The location of pumps has not been verified through survey or accurate positioning. They have been assigned 'close' to where they are expected to be based on location description and assessment of terrain.
- Similarly, the operation of the pipe network has not been verified, and the extent of pipes is limited.

- It is not clear at what stage in the flood event the photo was taken. Presumably the reports were obtained after the flood event was notified, and delays in obtaining the pictures may have meant that the flood was abating.

Figure 29. Observed Event, flood comparison along Langford Street

Comparison with other modelling results

The modelling results for the No Pump scenario have been compared with the flood extent determined through as part of the Planning scheme amendment process.

Error! Reference source not found. shows the flooding extent in the vicinity of the site as determined by o ur modelling for the no pump scenario compared with the new proposed overlay while **Error! Reference source not found.** shows the underlying results from the accompanying flood modelling report.

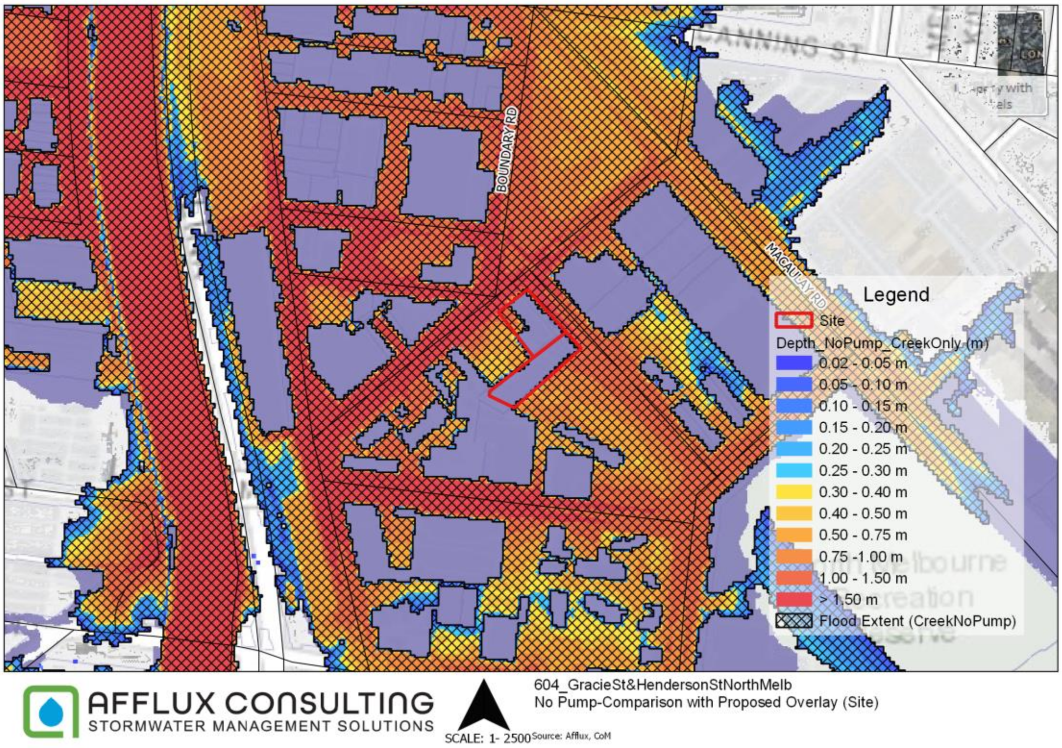
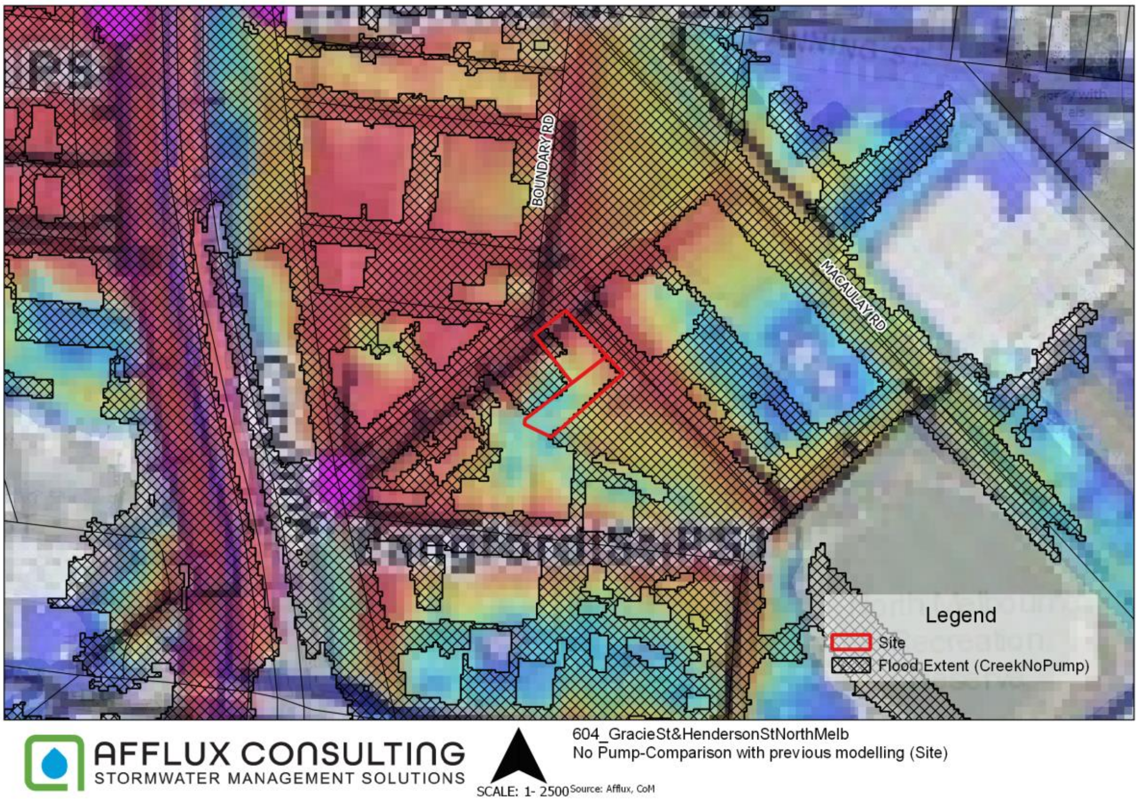


Figure 30. Modelled flood extent compared with proposed LSIO



Source: Adapted from Engeny

Figure 31. Modelled flood extent compared with previous modelling results

In drawing comparisons it is worth noting that the planning scheme mapping includes a couple of additional attributes that would increase flood depth and extent, namely rainfall on the local catchment and a higher flow applied to the creek channel.

Nonetheless it is fairly evident that the results developed through this report are ‘comparable’ with previous efforts in terms of total extent and gives confidence that the model is behaving as required.

Use of HPC modelling engine

Finally the HPC was chosen as the preferred modelling engine due to its faster runtimes and computational stability to address issues that are often encountered with rainfall on gird and stream discharges where water levels can change suddenly. There are recommended processes to address these concerns and involve re-running the model with slightly altered parameters to determine if this makes a change.

Table 5 shows various parameters reported for selected models. In general volume and mass errors are small with no time switching behaviour which provides confidence in the models. By extension this could be applied to all modelled scenarios contemplated in this study.

Table 5. HPC model Quality Assurance parameters

Scenario	Vol Error	CME	Warnings (prior to simulation)	Time step changes (No.)	Time steps used (No.)	TS Ratio (%)
1	0	0.4	2	6	240839	0
2	-0.34	0.34	135	3	346856	0
3	-0.31	0.31	135	1	377476	0
4	0	0	133	1	66102	0
6	-0.01	-0.01	135	0	8953	0
7	0.00	0	135	0	9303	0
QA run	0.00	0	135	0	81971	0

The QA was undertaken for the observed event as this is a known point where water levels are calibrated as described above. The recommended process is to determine if the change on sensitivity factors used within the model to trigger a change in computational timestep (i.e. Control Number Factor) actually lead to a significant difference in levels. A Quality Assurance model run was undertaken with this factor reduced from its default of 1 down to 0.8 as suggested in the TUFLOW manual and Melbourne Water technical specifications.

Figure 32 shows the difference as negligible (I.e. 1mm for the majority). Figure 33 shows these results in close up in the vicinity of the site and showing the operation of the pumps. The results provide further confidence that the chosen computational engine is providing robust results.

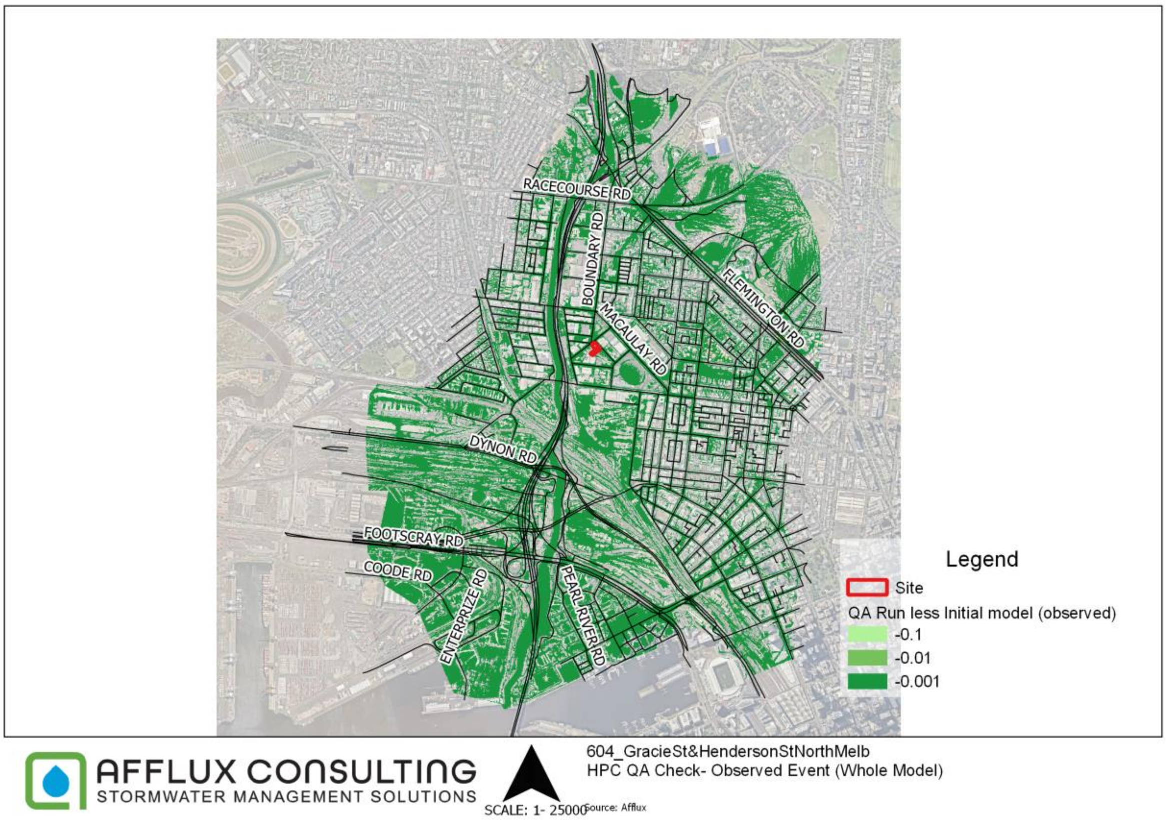


Figure 32. HPC check Level difference (Control number reduced to 0.8)

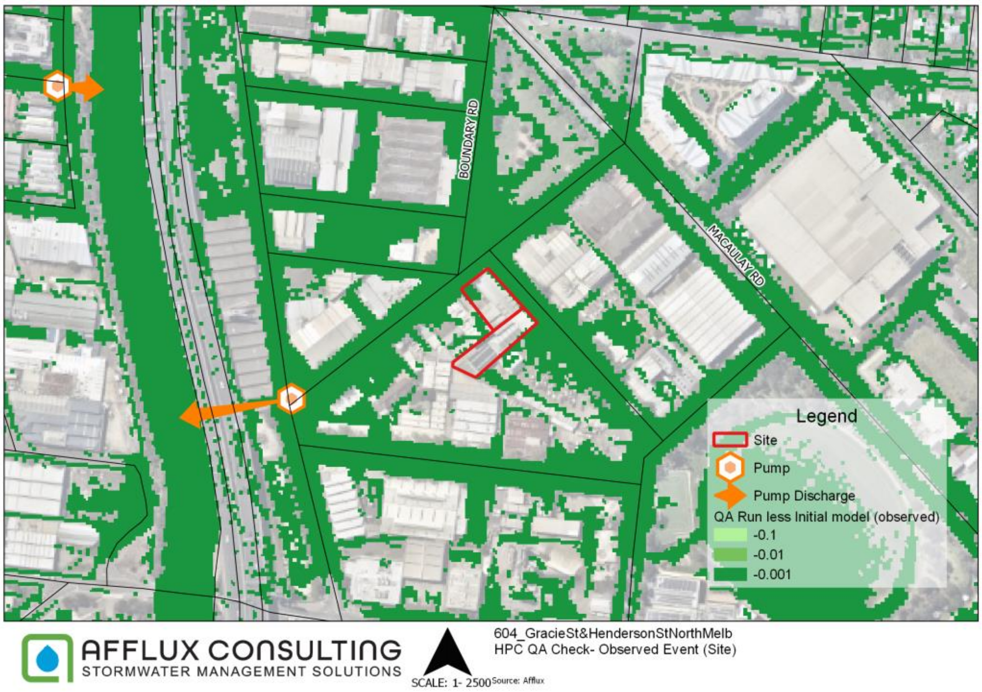


Figure 33. HPC check Level difference (Control number reduced to 0.8)- SIte

10. Conclusions

The flooding characteristics of the North Melbourne area have been represented in this repot for a range of scenarios that enable the impact on private property to be assessed for a number of scenarios based on real world and theoretical conditions (i.e. synthetic rainfall applied on a local catchment).

Model assumptions differ from those used in previous Authority assessments for the area and are based on independent analysis undertaken by UWCS and used in our modelling ‘under instruction’.

In broad terms the TUFLOW model represents a fit for purpose tool capable of approximating flooding. While there have been some simplifications in schematisation due to nature of our engagement, the key findings of flooding in the area in terms of depth and extent are likely to be of a similar scale and magnitude with previous modelling efforts when ‘like for like’ assumptions are made.

In general, flooding depths are reduced due to lower flows applied direct to the Moonee Ponds Creek as a result of upstream analysis of catchment response times and assumptions made regarding flood retarding behaviour of interventions applied over numerous years. We have not chosen to replicate the flows used in other models, but are reasonably comfortable that the flood results would be reproduced in order of magnitude terms. On this basis, the model developed for this study is considered fit for purpose.

We have assessed multiple scenarios to determine a mechanism that likely describes flooding. When high flows within the creek breech levees along the eastern and western confines there is resultant flooding of adjacent low-lying areas. Pumps have been previously used to relieve these areas of flooding, by taking advantage of reducing grades along the longitudinal alignment of the creek. These are fundamental principles in applying drainage theory to real world situations that should still apply.

The presentation of flood results in static maps that show maximum flood results obscures the dynamic nature of flooding. Conservative assumptions that result in increased flows within the creek channel can exasperate the nature of flooding. These assumptions include applied flows and the operation of various mitigation infrastructure items that exist or could be improved.

In addition to structural options, management options may be appropriate based on the differing nature or (upstream) catchment flooding and localised flooding caused by rain on the local catchment. These mechanisms of flooding vary both in scale and timing; application of probability theory can be used to assess the true nature of risk which can be mitigated by infrastructure and management options.

In addition to static maps, we have presented results which show how the nature of flooding varied over time. Flood waters build to a maximum, and then abate over time. The true nature of risk and impact is best understood when these factors are considered along with an appreciation of the probability of occurrence. In short, flooding may be tolerable if it is infrequent and its occurrence does not unduly impact on fundamental principles such as safety, access or egress. Dynamic assessments provide a useful tool to assess these.

The general area is undoubtedly flood prone. The City of Melbourne’s proposed Planning Scheme Amendment would be remiss to not address this reality head on. The application of overlays should rightly be conservative in order to set appropriate floor levels, and to avoid undesirable outcomes that could include excavations compromising the integrity of future land use adjacent to areas expected to flood. However, the analysis undertaken suggests that for the most part mitigation options in the form of upgraded pumps and levees have the potential to limit these impacts to road area. Private land is in the vicinity of the sites of interest is largely exempt from the worst excesses of flooding if these are operational.

While no evaluation of the cost or viability of providing upgraded levees and pump capacity has been undertaken, it is considered meritorious to evaluate these against the potential to continue the productive use of the sites, especially given the proximity to a large population base and connectivity with major transportation and freight networks in the area.

For information on this report:

Andrew Allan
Senior Engineer

0417 516 756

andrew-allan-3964b118

andrew@afflux.com.au

www.afflux.com.au

Afflux Consulting Pty Ltd
PO Box 457 Emerald VIC 3782

03 9036 2530

info@afflux.com.au

afflux.com.au