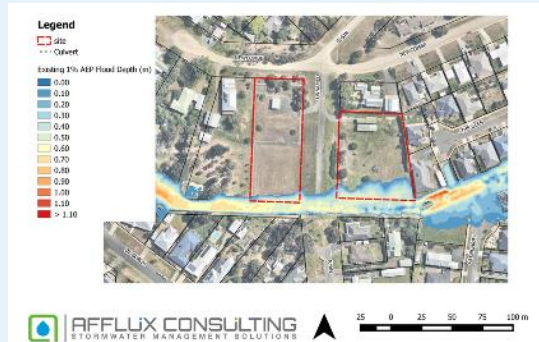




40 – 42 Newcombe St and 41 – 47 Elgin St, Drysdale

Flood investigation



May 2016



AFFLUX CONSULTING
STORMWATER MANAGEMENT SOLUTIONS

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Client	St Quentin
Reference Number	J250
Author	TP
Review	CMB
Report Version	V01
Document Name	Drysdale_R01a

1. Introduction

Afflux Consulting were engaged by St Quentin to investigate the impact of development on an adjacent floodway at 40 – 42 Newcombe St & 41- 47 Elgin St, Drysdale. This report addresses risks identified by Corangamite CMA and ensures that future development does not breach their standards. In particular, this report provides advice on:

- Appropriate development - ensuring the safety of people and vehicles during a flood event
- Depth and flow of floodwater on properties
- Provisions needed to protect people from entering or leaving their property in case of storm event
- Minimum floor levels to provide flood protection

The proposed development (shown in Figure 1) has an area of 8,200m² and is situated at the intersection of Newcombe and Elgin St, Drysdale. It is bounded by Newcombe St to the north, residential properties to the east and west, and existing floodway to the south. The area is to be developed into several residential lots, with an expected increase to imperviousness of 60% from existing 10%. The proposed development itself drains from north to south however the primary focus of this flood investigation are the characteristics of the floodway, such as its external catchment hydrology, flood extents, flow capacity, downstream flow constraints and its interaction with the development site.

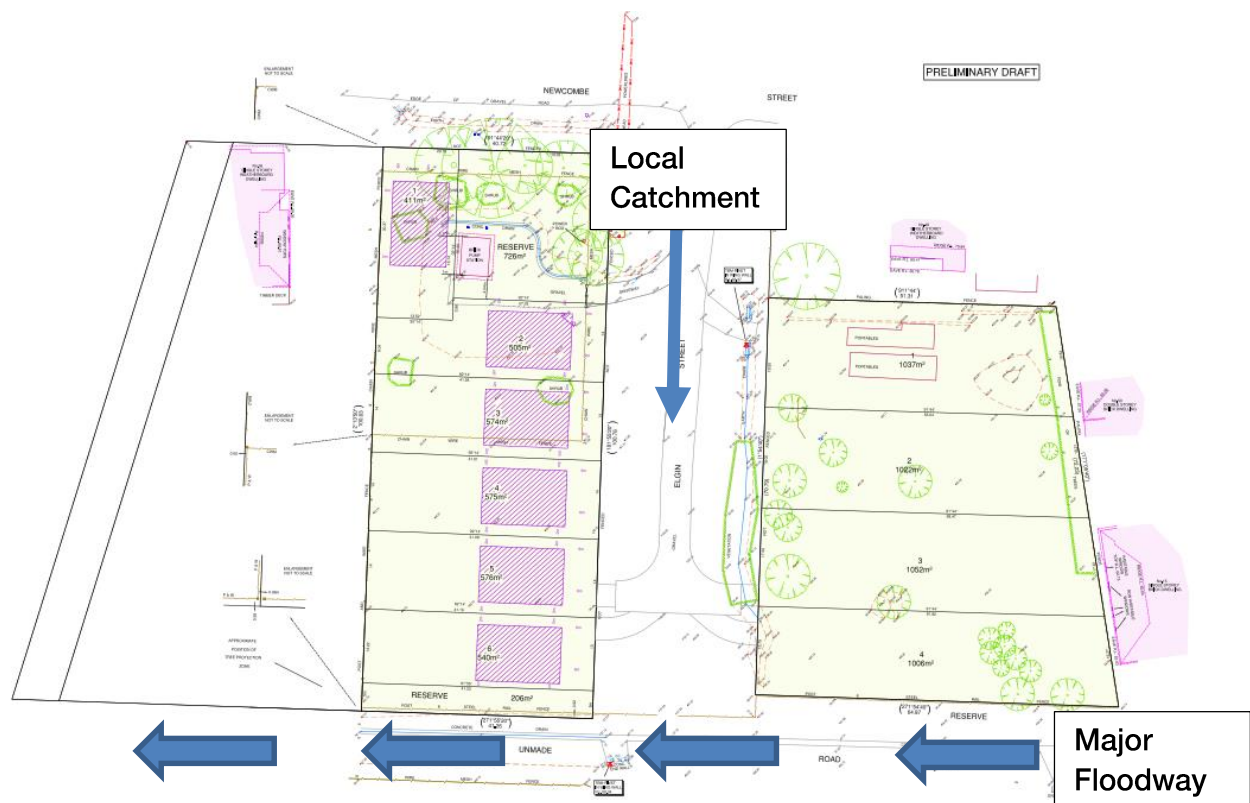


Figure 1: Proposed development



Figure 2: Image of existing development

2. Flood Modelling

Various modelling techniques were used to understand stormwater and flood characteristics of the catchment and to demonstrate that the stormwater strategy for the site as proposed by Afflux Consulting is supported by widely accepted engineering science. This section explains the hydrological and hydraulic investigations for the site and associated assumptions.

2.1 Hydrology

The hydrology for this site is based on an extension of the 2012 Woodville Street Functional Design Report (Central Walk Drysdale). This set the hydrological parameters and storage for the area.

The primary model for flow evaluations in this study is Monash Universities RORB model. RORB was produced by Laurenson and Mein as a runoff routing model for the production of flood hydrographs. It is considered the industry standard model for Victorian Flood studies.

An extract from this study (Water Technology, 2012) can be seen below:

Design flows for the site under existing and developed conditions were estimated using the runoff routing program, RORB. The following inputs were included in the RORB model:

- Subcatchment delineation based on existing Millar and Merrigan drainage strategy plan and 1m LiDAR topographic data.
- Fraction impervious values based on an assessment of land zoning and manual assessment from aerial photography
- Initial loss and runoff coefficient values based on the Melbourne Water Guidelines and Technical Specifications for Flood Mapping Projects (2010a)
- IFD data for Drysdale sourced from Bureau of Meteorology (BOM) to provide site specific design rainfall data
- Filtered temporal patterns filtered and Areal Reduction Factors based on ARR87 Book II
- m value of 0.8
- k_c value reconciled to the rational method existing 100 year ARI peak flow estimate

The routing and loss parameters adopted for the two areas are summarised in Table 2-1 below.

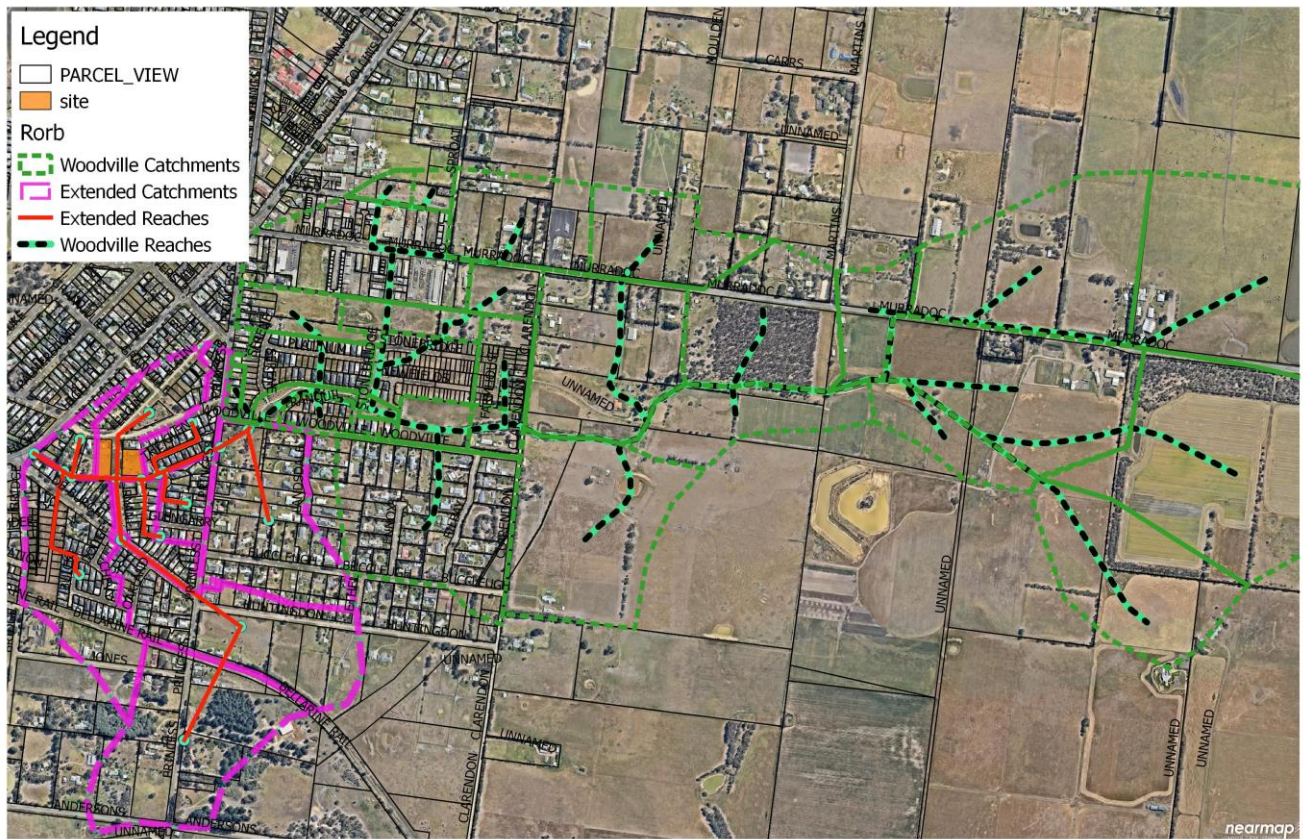
Table 2-1 Adopted RORB parameters

Parameter	East of Clarendon Road	West of Clarendon Road
K_c	3.36	0.42
m	0.8	0.8
IL	20mm	20mm (existing) 10mm (developed)
Runoff Coefficient	0.6 (100y), 0.25 (5y)	0.6 (100y), 0.25 (5y)

This model has been extended as part of this investigation. The existing model catchments and extension catchments can be seen in Figure 3 below.

The major features of this revised Rorb model were:

- Additional 10 catchments were added roughly with similar catchment sizes and based on the terrain and discharge points as identified on site inspection
- Reach lengths were defined as Type 3 “lined channels” and based loosely on the road and pipe networks
- Pipe diversions were added where pipe outlets differed from the major road network. This was particularly relevant at the De Burgh Road and Ryan Court intersection with the pipe network heading to the creek and the overland flow continuing down De Burgh Road.
- The catchment K_c value was updated based on maintaining the K_c/D_{av} ratio.
 - The existing conditions K_c from the Water Technology Study was 0.42. This equated to a K_c/D_{av} ratio of 0.627 for this model
 - The updated model with revised D_{av} results in a K_c of 0.66
- Based on the updated model, and revised K_c the updated flows can be seen in Table 2-2



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Drysdale Catchment RORB models
Upstream of Woodville and Extended Model

Figure 3 Catchment Rorb model

Table 2-2 Catchment Flows

Storm Duration	ARI	Catchment Flows			
		Woodville St	US Site	DS Site	High Street*
10m	100y	3.7671	4.0253	7.8103	12.7128
15m	100y	4.2584	4.9595	8.5543	15.0343
20m	100y	4.9076	5.6709	9.0149	14.8089
25m	100y	5.4183	5.7656	9.4342	15.1301
30m	100y	6.1647	6.1636	9.0153	13.9454
45m	100y	6.7068	7.1046	9.7956	12.1994
1h	100y	6.8702	7.251	9.7595	14.274
1.5h	100y	6.9237	6.9926	10.4822	14.9658
2h	100y	6.601	6.9817	10.3301	15.0257
3h	100y	5.0649	5.3588	7.943	8.9405
4.5h	100y	5.5747	5.9734	9.2	10.4754
6h	100y	4.891	5.3208	7.4064	8.1662
9h	100y	5.9066	6.2337	8.2469	8.9728
12h	100y	6.1945	6.4511	7.581	7.999
18h	100y	5.1904	5.3645	6.2857	6.6212
24h	100y	5.4619	5.6657	6.7908	7.1926
30h	100y	5.1207	5.2774	6.242	6.6372
36h	100y	5.5754	5.7505	6.6154	6.925
48h	100y	5.0884	5.2746	6.4149	6.8202
72h	100y	3.4544	3.5493	3.8674	3.9833

* Assumes all flows De Burgh and Bridge Street enter drain US of High Street. Realistically some overland flows will continue across High Street and as such the culvert flow at High Street would be closer to 11-12m³/s.

Based on this the 1 hour flows (peak flow upstream of site) have been selected for the 1% and 20% AEP flows and input into the hydraulic model. These flows can be seen below.

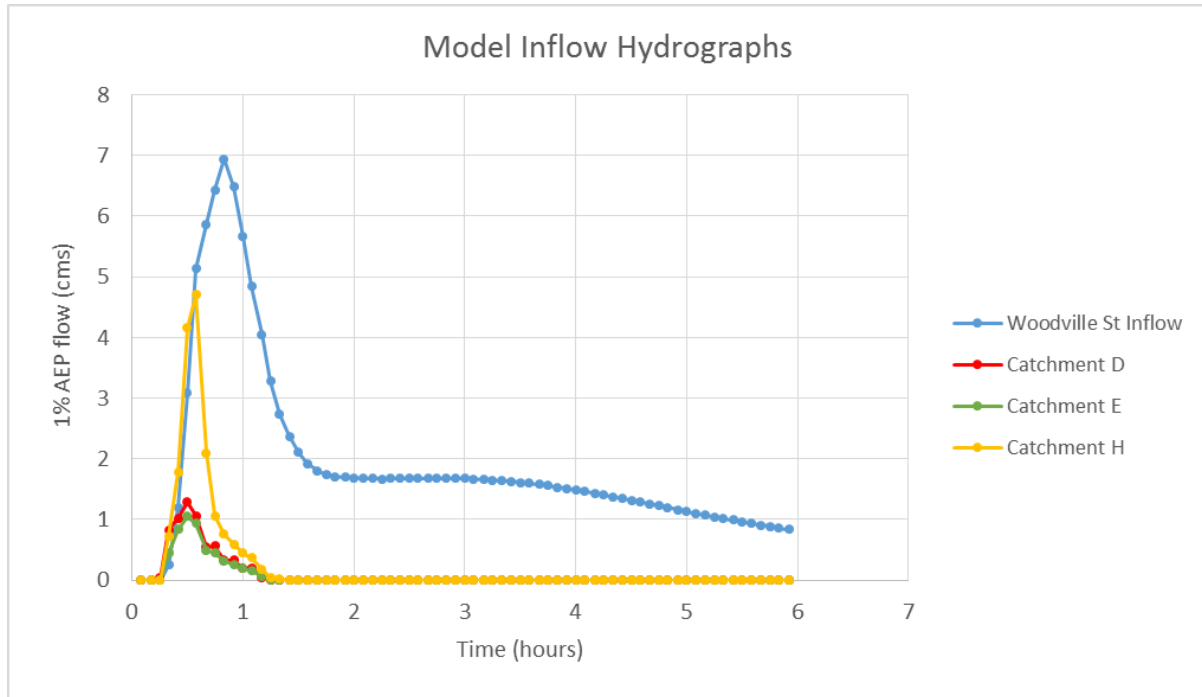


Figure 4: 1% AEP inflow hydrographs

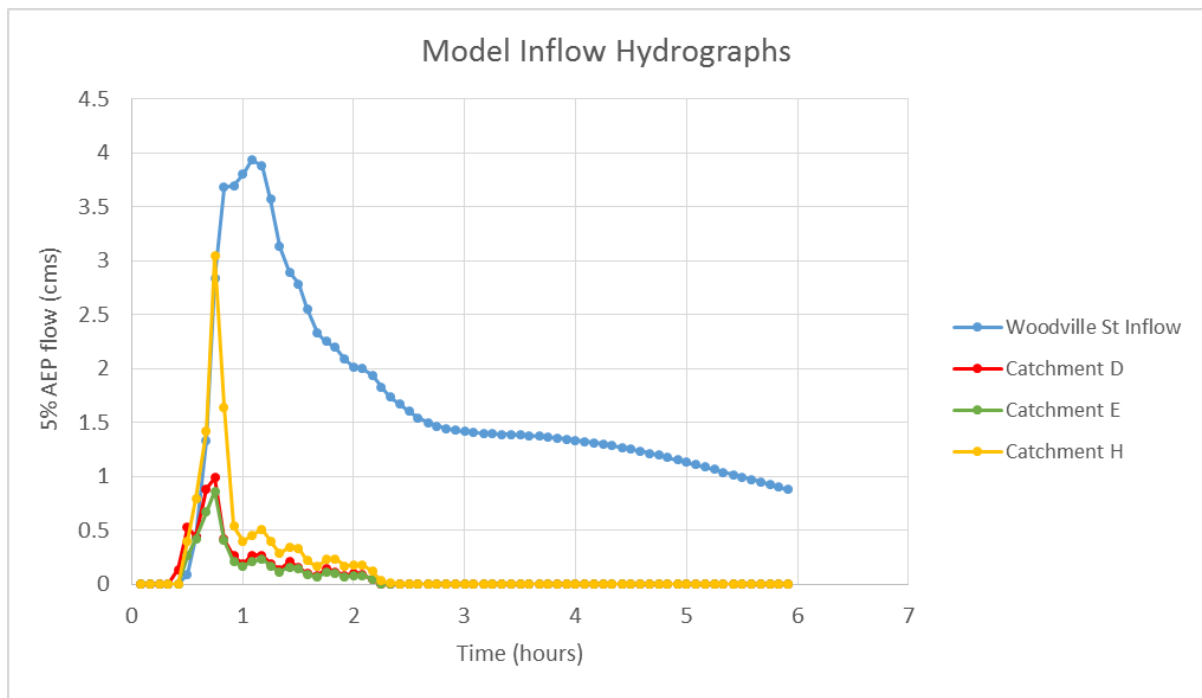


Figure 5: 5% AEP inflow hydrograph

2.2 Hydraulics

A two dimensional flood model was built to define the existing flow and flow extent for the site in a 1% Annual Exceedance Probability (AEP) storm. This information will provide guidance on developable area, environmental protection buffers and flood hazard prone areas. The model was built in Tuflow using LiDAR data as the DEM (Digital Elevation Model).

The Figure 6 below shows the model setup with annotations, with the following assumptions:

- A 1m grid resolution was used for all models.
- The inflow boundaries applies the hydrographs determined in the hydrologic components of the project discussed earlier.
- The 2100 x 2100mm box culvert at High St controls the flow though the floodway, as such this was set as the model downstream boundary.
- In light of proposed High St culvert upgrades, the downstream boundary tail water was set at 300mm below the adjacent residential floor levels, as agreed upon with Geelong City Council. (Appendix A Correspondence)
- Consequently, a tailwater level of 56.7m AHD was adopted at the High St culvert (Figure 7 shows a floor level of roughly 57m AHD for the adjacent property).
- The culvert at Princess St was measured and model as 1625mm diameter, however it was significantly blocked during a site inspection of the area. As such, a 50% blockage factor was run for one scenario in developed conditions.
- A 5% AEP scenario was also run to help understand the flood effect of higher probability storms.

A summary of model scenarios are below:

- Existing conditions – 1% AEP storm event
- Developed conditions – 1% AEP storm event
- Developed conditions – 5% AEP storm event
- Developed conditions – 1% AEP storm event with 50% blockage on the Princess St culvert

The model was run for 5 hours, at a 0.25 second timestep. Several materials and corresponding Manning's Roughness (n) were used:

- Lawn areas - n = 0.04 (set as default)
- Waterway - n = 0.06
- Road and driveways - n = 0.02

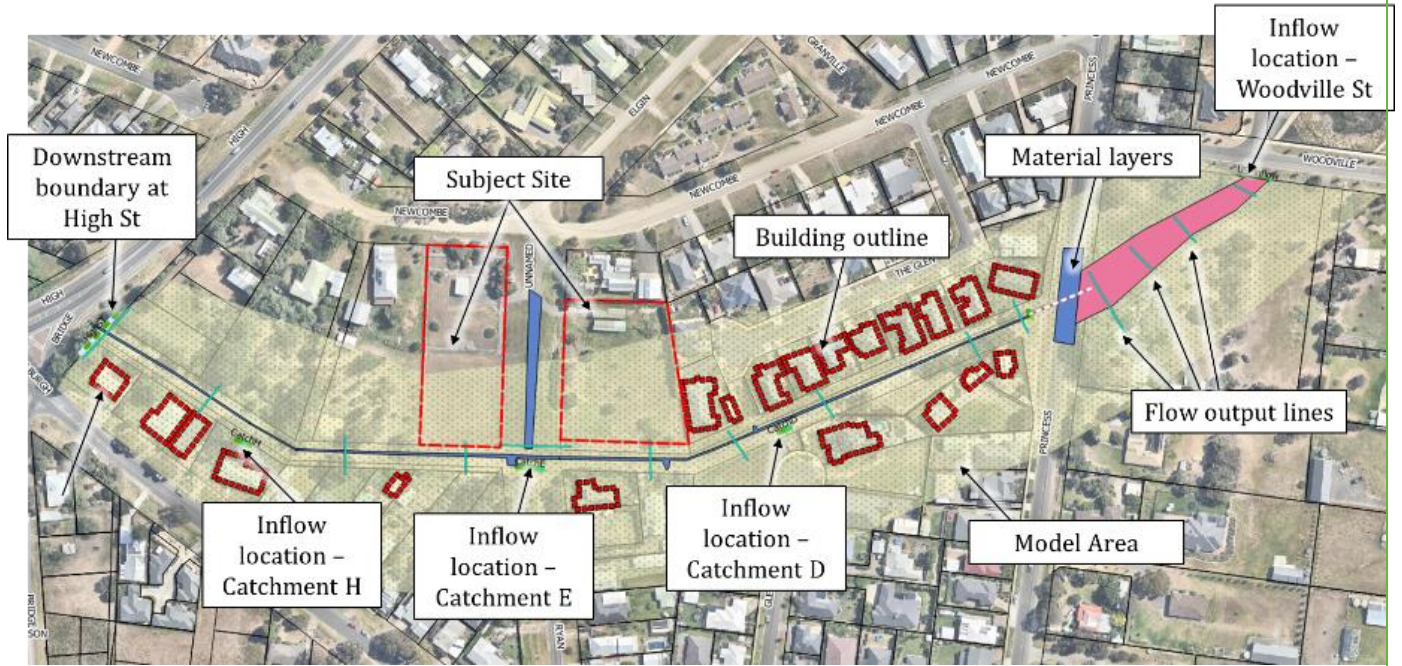


Figure 6: Model setup, boundary locations and material layers. Areas not covered by the material layer have a default Manning's roughness of 0.04.



Figure 7: Topography near High St Culvert

2.2.1 Existing Conditions

Figure 8 through to Figure 15 below map the 1% AEP flood extents, water surface elevation, velocity and hazard through the site. Most importantly, the results show significant flood flow through the southern sections of the site. A maximum flood depth of 0.5 – 0.6m is expected, along with significant flow velocity (maximum of 1.7m/s) and hazard (maximum of 0.6m²/s) near the southern border. This emphasises the importance of recognising flood risks within the development and providing well thought-out engineering solutions.

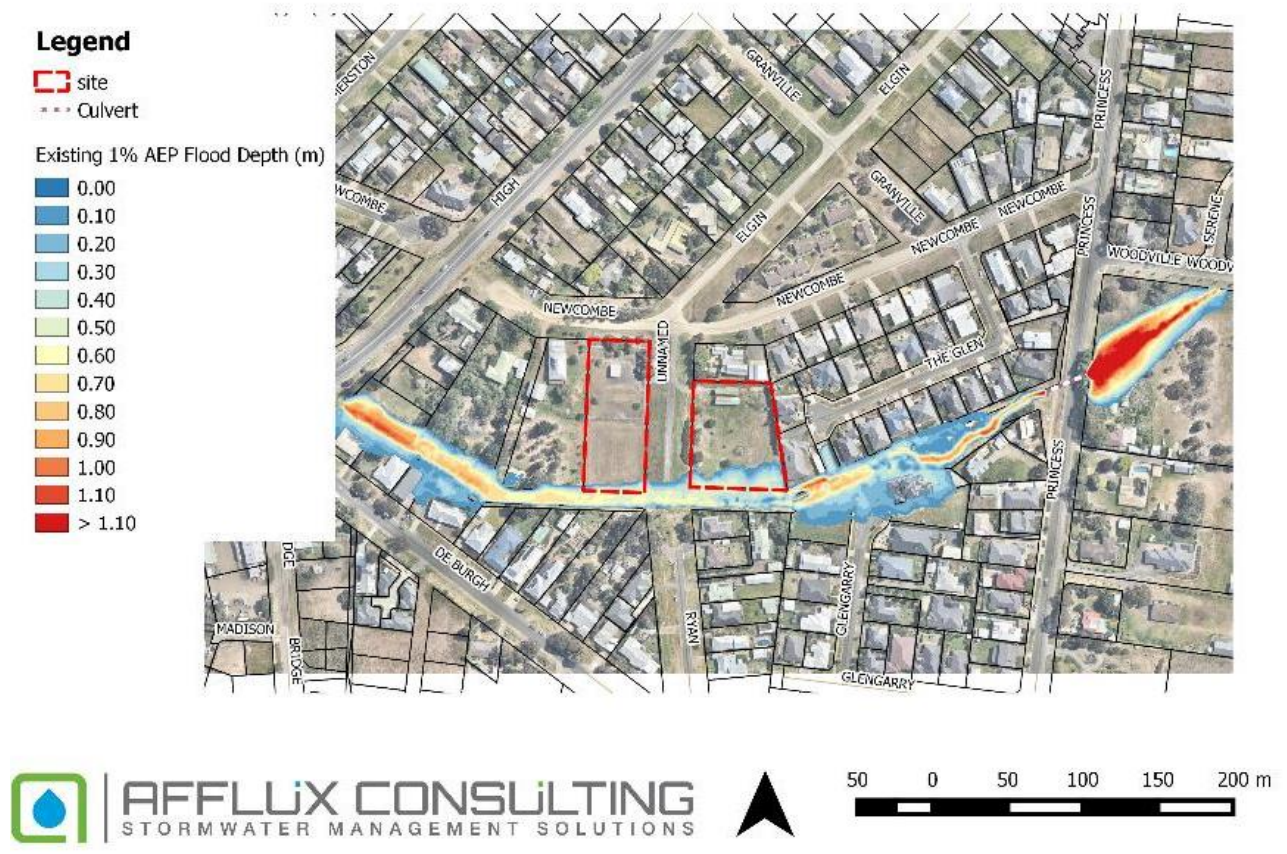


Figure 8: Existing 1% AEP Flood depth (m) – Broad view

Legend

-  site
-  Culvert

Existing 1% AEP Flood Depth (m)














-  0.00
-  0.10
-  0.20
-  0.30
-  0.40
-  0.50
-  0.60
-  0.70
-  0.80
-  0.90
-  1.00
-  1.10
-  > 1.10




















Figure 9: Existing 1% AEP Flood depth (m) – Close up of site

Legend

-  Site
-  Culvert

Existing 1% AEP WSE

-  56.000
-  56.500
-  57.000
-  57.500
-  58.000
-  58.500
-  59.000
-  59.500
-  60.000
-  60.500
-  61.000
-  61.500
-  62.000
-  62.500
-  63.000
-  63.500
-  64.000
-  64.500
-  65.000

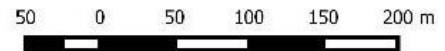


Figure 10: Existing 1% AEP Water Surface Elevation (WSE) – Broad view

Legend





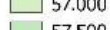
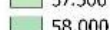
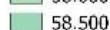
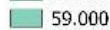


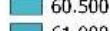
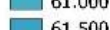
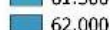










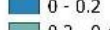
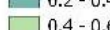
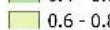
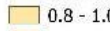
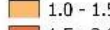
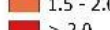


-  Site
-  Culvert
- Existing 1% AEP WSE
-  56.000
-  56.500
-  57.000
-  57.500
-  58.000
-  58.500
-  59.000
-  59.500
-  60.000
-  60.500
-  61.000
-  61.500
-  62.000
-  62.500
-  63.000
-  63.500
-  64.000
-  64.500
-  65.000



Figure 11: Existing 1% AEP Water Surface Elevation (WSE) – Close up of site

Legend

-  Site
-  Culvert
- Existing 1% AEP Flood Velocity (m/s)
-  0 - 0.2
-  0.2 - 0.4
-  0.4 - 0.6
-  0.6 - 0.8
-  0.8 - 1.0
-  1.0 - 1.5
-  1.5 - 2.0
-  > 2.0

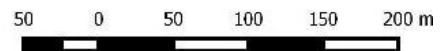


Figure 12: Existing 1% AEP flow velocity (m/s) – Broad view

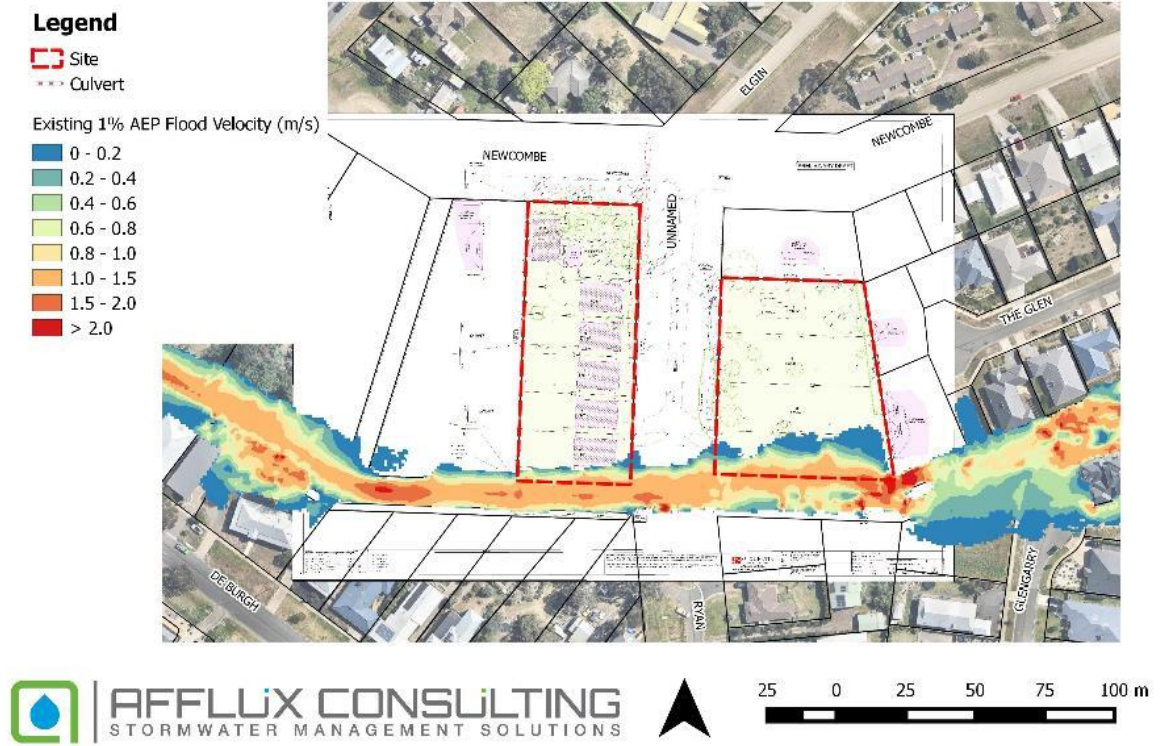


Figure 13: Existing 1% AEP flow velocity (m/s) – Close up

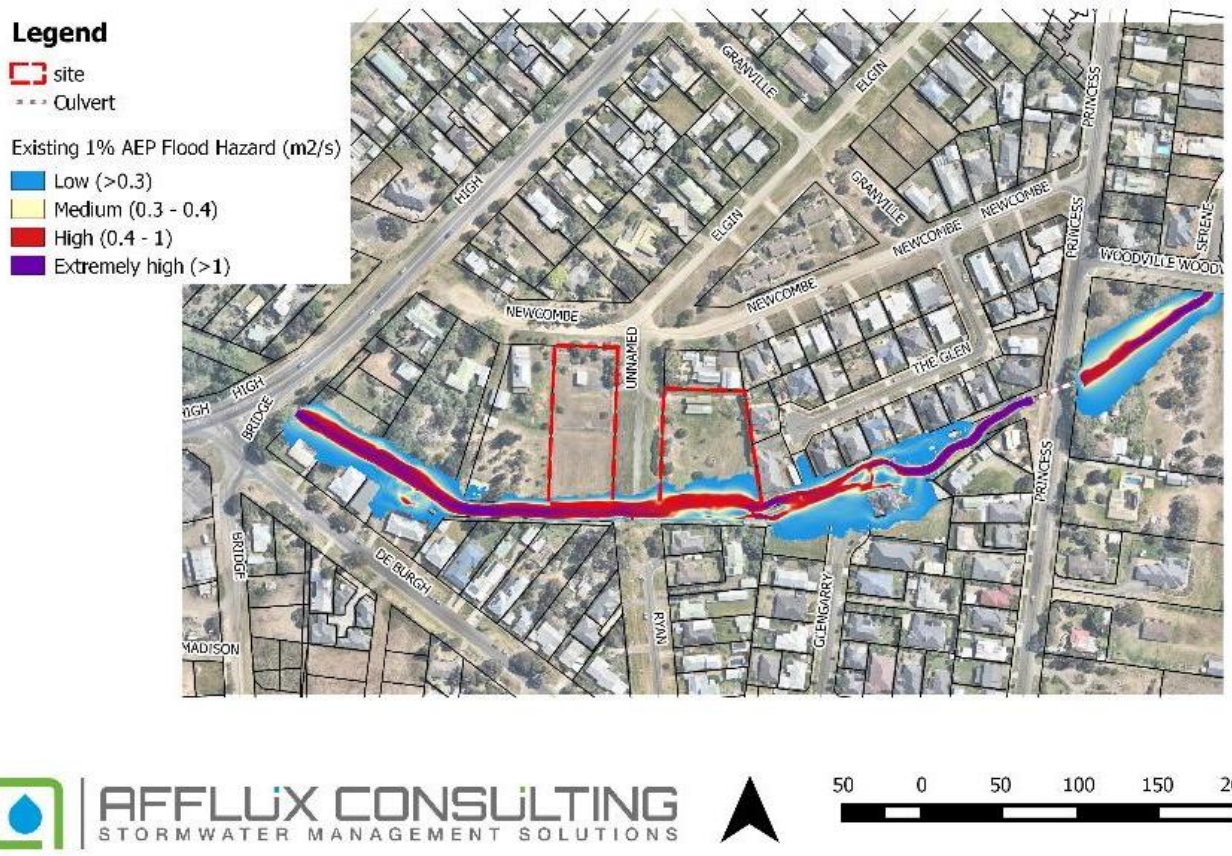






Figure 14: Existing 1% AEP Flood Hazard (v x d) (m²/s) – Broad view

Legend

-  site
-  Culvert

Existing 1% AEP Flood Hazard (m²/s)

-  Low (>0.3)
-  Medium (0.3 - 0.4)
-  High (0.4 - 1)
-  Extremely high (>1)

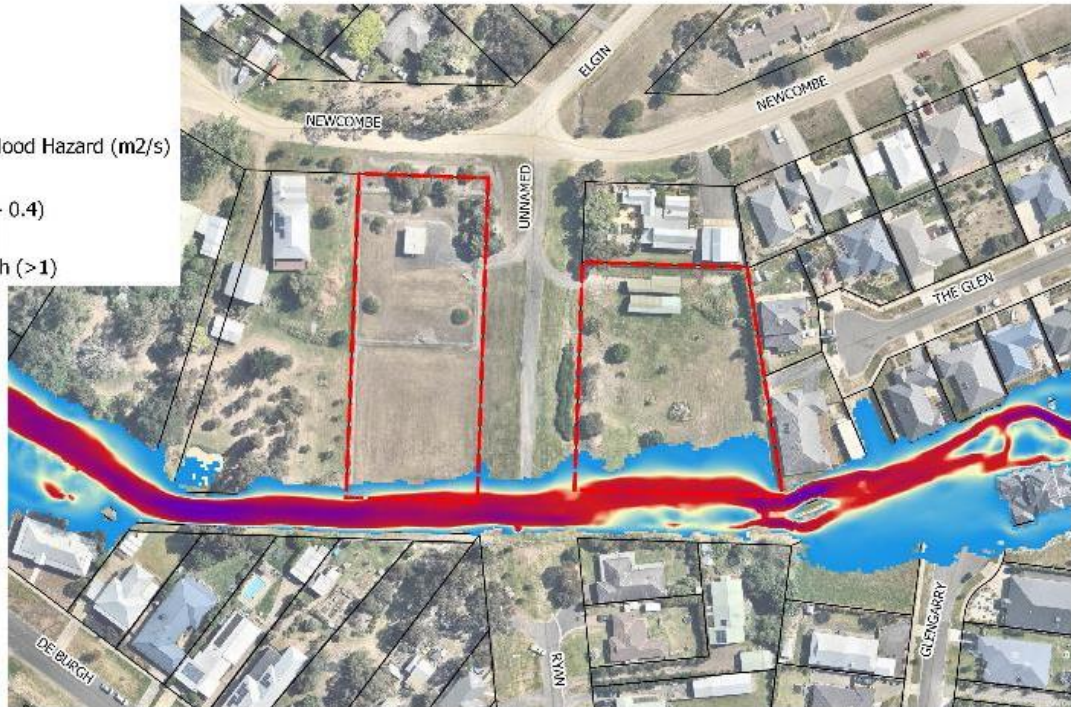


Figure 15: Existing 1% AEP Flood Hazard (v x d) (m²/s) – Close up of site

2.2.2 Developed Conditions - 1% AEP results

The objective of the developed conditions modelling is to offer practical solutions which manage on site flooding and pose no increased flood risk to the development nor surrounding properties. In all of the developed scenarios, a number of elements were added to the model to minimise flooding impacts. These can be seen as development requirements and can be seen in Figure 16.

















Various scenarios were run in the developed case to understand how sensitive the flood extents and hazards are, especially to storm event and culvert blockage at Princess St.



Figure 16: Elements added in the Developed Conditions modelling to curb flow

The Figure 17 through Figure 24 present the results of the developed conditions hydraulic modelling. The maximum flood depth on the developed site primarily occurs on the fringes of the lots, to a maximum of 0.51m. The maximum velocity through the subject site is 2.5m/s, occurring in a small section in the south-eastern corner of the development. Floor hazard through the site is quite high, however areas of high flood hazards are kept toward the boundary of the site. The flood hazard is low (less than 0.3m²/s) in front of the proposed residential buildings.

Legend

-  Site
 -  Culvert
 -  Developed scenario items
- Developed 1% AEP Flood Depth (m)
-  0.00
 -  0.10
 -  0.20
 -  0.30
 -  0.40
 -  0.50
 -  0.60
 -  0.70
 -  0.80
 -  0.90
 -  1.00
 -  1.10
 -  > 1.10

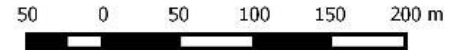


















Figure 17: Developed 1% AEP Flood depth (m) – Broad view

Legend

-  Site
 -  Culvert
 -  Developed scenario items
- Developed 1% AEP Flood Depth (m)
-  0.00
 -  0.10
 -  0.20
 -  0.30
 -  0.40
 -  0.50
 -  0.60
 -  0.70
 -  0.80
 -  0.90
 -  1.00
 -  1.10
 -  > 1.10

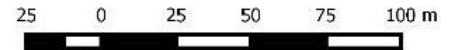
























Figure 18: Developed 1% AEP Flood depth (m) – Close up

Legend

-  Site
 -  Culvert
 -  Developed scenario items
- Developed 1% AEP WSE
-  56.000
 -  56.500
 -  57.000
 -  57.500
 -  58.000
 -  58.500
 -  59.000
 -  59.500
 -  60.000
 -  60.500
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 -  65.000

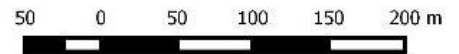


Figure 19: Developed 1% AEP WSE– Broad view

Legend
























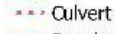
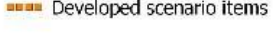








-  Site
 -  Culvert
 -  Developed scenario items
- Developed 1% AEP WSE
-  56.000
 -  56.500
 -  57.000
 -  57.500
 -  58.000
 -  58.500
 -  59.000
 -  59.500
 -  60.000
 -  60.500
 -  61.000
 -  61.500
 -  62.000
 -  62.500
 -  63.000
 -  63.500
 -  64.000
 -  64.500
 -  65.000



Figure 20: Developed 1% AEP WSE– Close Up

Legend

-  Site
 -  Culvert
 -  Developed scenario items
- Developed 1% AEP Flood Velocity (m/s)
-  0 - 0.2
 -  0.2 - 0.4
 -  0.4 - 0.6
 -  0.6 - 0.8
 -  0.8 - 1.0
 -  1.0 - 1.5
 -  1.5 - 2.0
 -  > 2.0

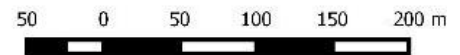

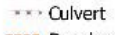
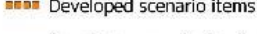










Figure 21: Developed 1% AEP Flood velocity (m/s) – Broad view

Legend

-  Site
 -  Culvert
 -  Developed scenario items
- Developed 1% AEP Flood Velocity (m/s)
-  0 - 0.2
 -  0.2 - 0.4
 -  0.4 - 0.6
 -  0.6 - 0.8
 -  0.8 - 1.0
 -  1.0 - 1.5
 -  1.5 - 2.0
 -  > 2.0

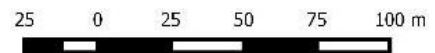


Figure 22: Developed 1% AEP Flood velocity (m/s) – Close up

Legend

- Site
 - Culvert
 - Developed scenario items
- Developed 1% AEP Flood Hazard (m²/s)
- Low (<0.3)
 - Medium (0.3 - 0.4)
 - High (0.4 - 1)
 - Extremely high (>1)

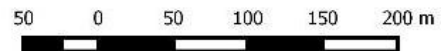
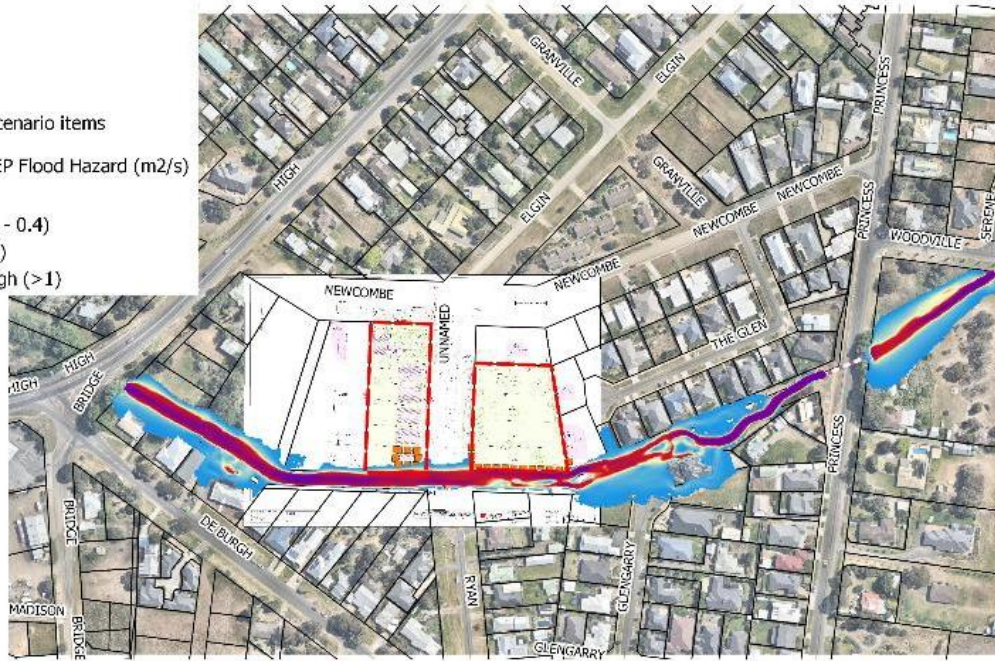


Figure 23: Developed 1% AEP Flood Hazard (v x d) (m²/s) – Broad view

Legend

- Site
 - Culvert
 - Developed scenario items
- Developed 1% AEP Flood Hazard (m²/s)
- Low (<0.3)
 - Medium (0.3 - 0.4)
 - High (0.4 - 1)
 - Extremely high (>1)

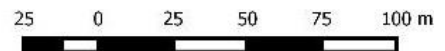


Figure 24: Developed 1% AEP Flood Hazard (v x d) (m²/s) – Close up

2.2.3 Developed Conditions - 5% AEP results

The flood results of a 5% AEP storm event are shown in Figure 25 - Figure 32. As expected, the flood is less hazardous than during a 1% AEP storm, however the flood extents continue to encroach on the subject site. This means that the development’s flood safety needs to be considered for the higher AEP storms, not only for the lower ones.

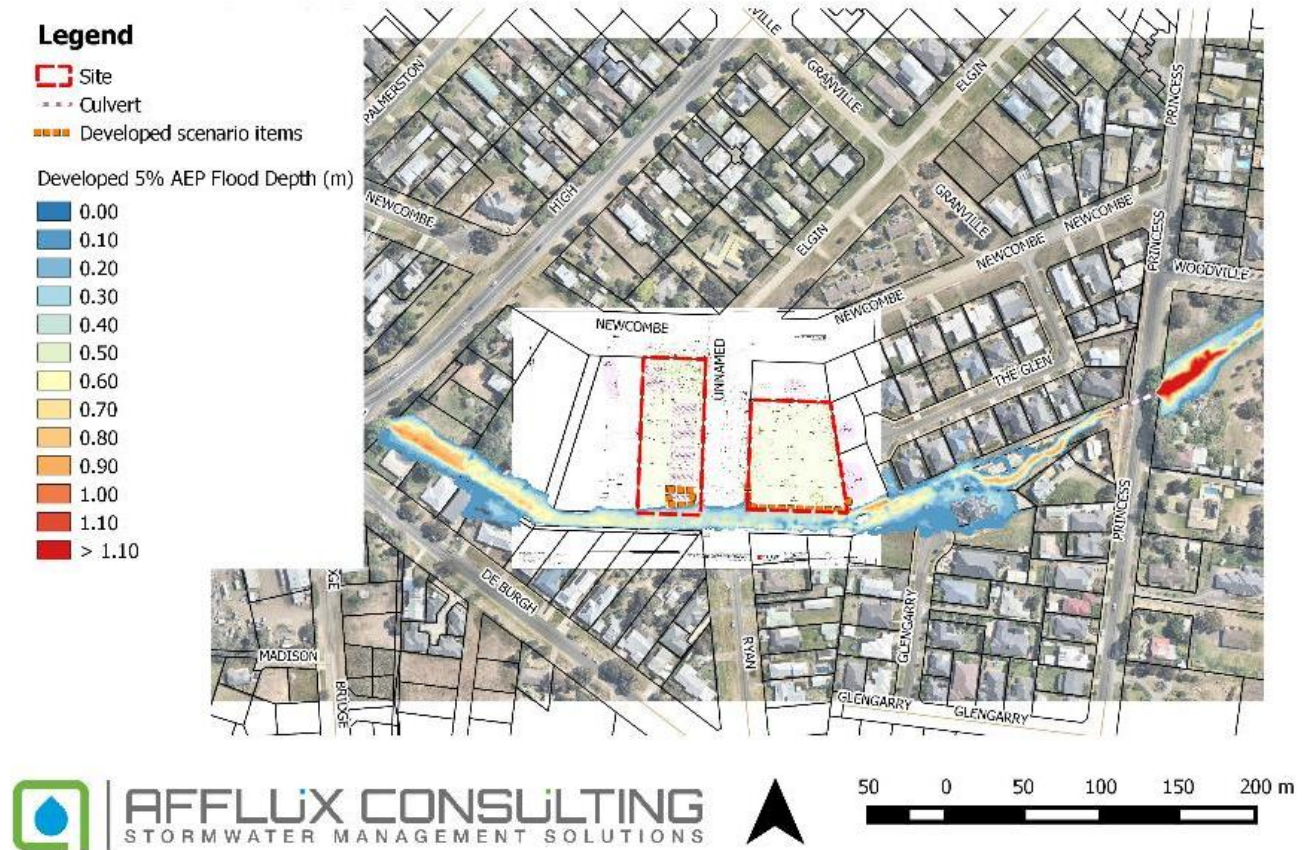


Figure 25: Developed 1% AEP Flood depth (m) – Broad view

Legend







































-  Site
 -  Culvert
 -  Developed scenario items
- Developed 5% AEP Flood Depth (m)
-  0.00
 -  0.10
 -  0.20
 -  0.30
 -  0.40
 -  0.50
 -  0.60
 -  0.70
 -  0.80
 -  0.90
 -  1.00
 -  1.10
 -  > 1.10



Figure 26: Developed 1% AEP Flood depth (m) – Close up

Legend

-  Site
 -  Culvert
 -  Developed scenario items
- Developed 5% AEP WSE
-  56.000
 -  56.500
 -  57.000
 -  57.500
 -  58.000
 -  58.500
 -  59.000
 -  59.500
 -  60.000
 -  60.500
 -  61.000
 -  61.500
 -  62.000
 -  62.500
 -  63.000
 -  63.500
 -  64.000
 -  64.500
 -  65.000

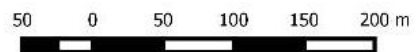




Figure 27: Developed 5% AEP WSE – Broad view

Legend

-  Site
-  Culvert
-  Developed scenario items

Developed 5% AEP WSE























-  56.000
-  56.500
-  57.000
-  57.500
-  58.000
-  58.500
-  59.000
-  59.500
-  60.000
-  60.500
-  61.000
-  61.500
-  62.000
-  62.500
-  63.000
-  63.500
-  64.000
-  64.500
-  65.000











Figure 28: Developed 5% AEP WSE – Broad view

Legend

-  Site
-  Culvert
-  Developed scenario items

Developed 5% AEP Flood Velocity (m/s)

-  0 - 0.2
-  0.2 - 0.4
-  0.4 - 0.6
-  0.6 - 0.8
-  0.8 - 1.0
-  1.0 - 1.5
-  1.5 - 2.0
-  > 2.0

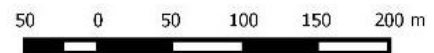


Figure 29: Developed 5% AEP Flood Velocity (m/s) - Broad view



Figure 30: Developed 5% AEP Flood Velocity (m/s) - Close up

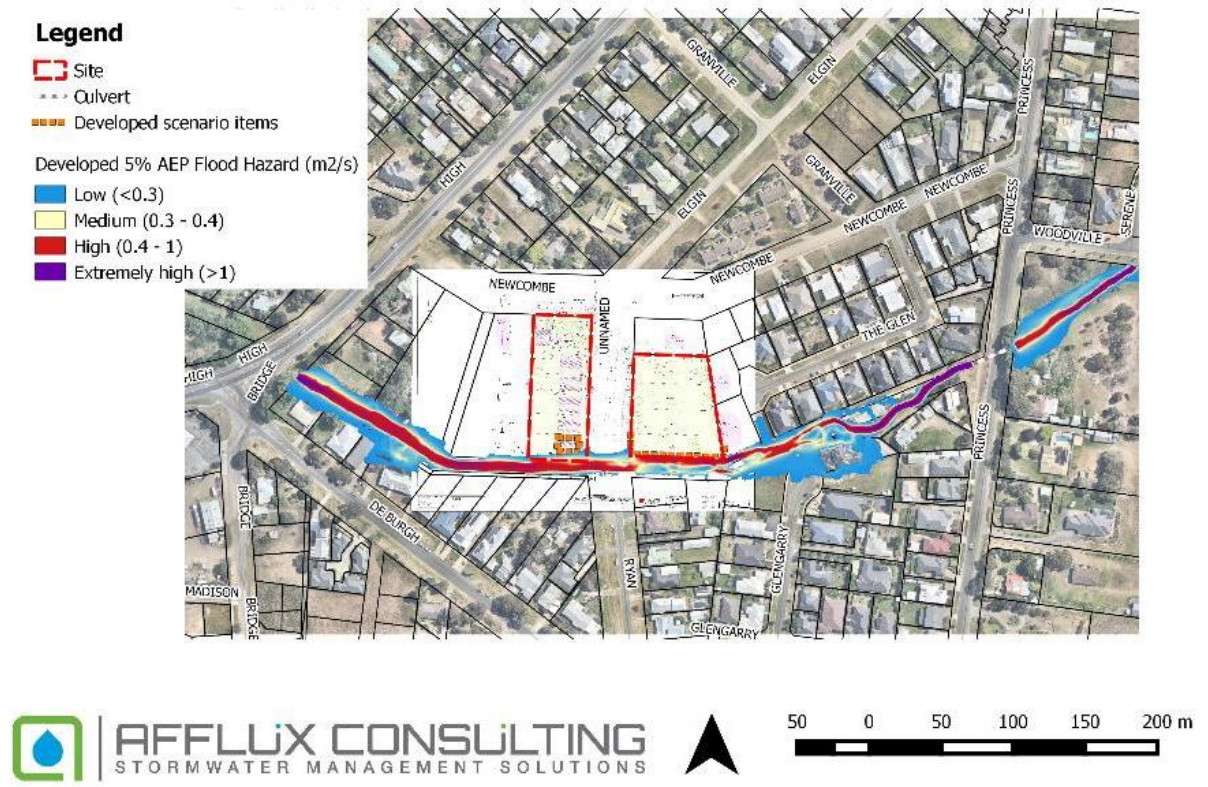


Figure 31: Developed 5% AEP Flood Hazard – Broad view

Legend

-  Site
 -  Culvert
 -  Developed scenario items
- Developed 5% AEP Flood Hazard (m²/s)
-  Low (<0.3)
 -  Medium (0.3 - 0.4)
 -  High (0.4 - 1)
 -  Extremely high (>1)

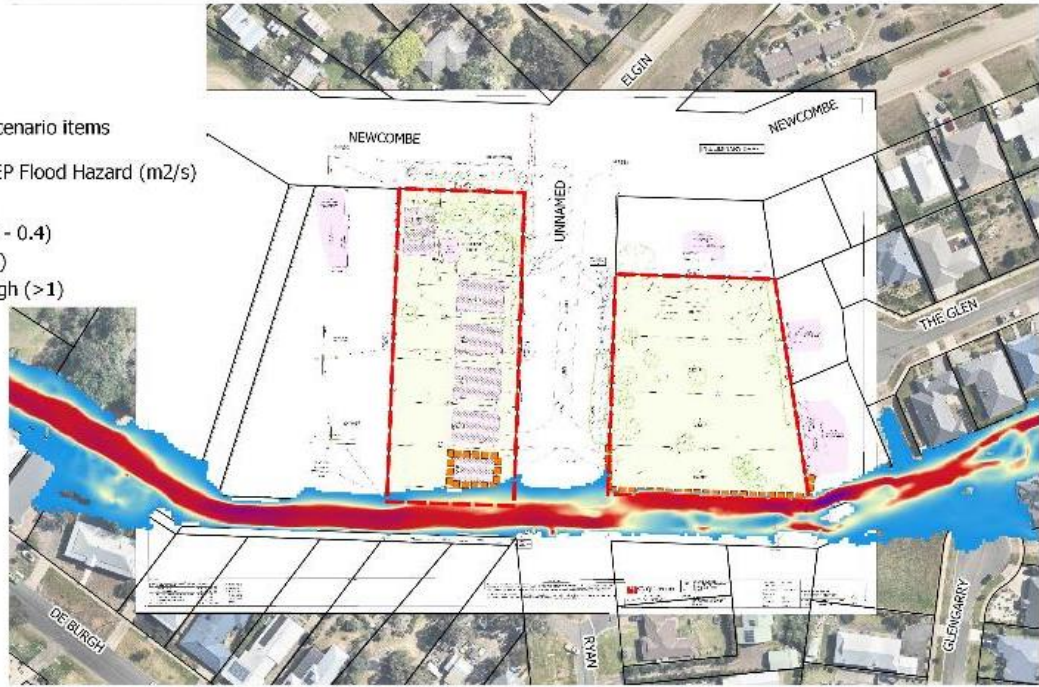


Figure 32: Developed 5% AEP Flood Hazard – Close up

2.2.4 Developed Conditions - 1% AEP results with 50% culvert blockage at Princess St

During a site inspection of the site and surrounding area, it was found that the upstream end of the culvert crossing Princess St was severely blocked by tree branches. A 50% blockage factor was assumed. The modelling results below (Figure 34 - Figure 37) show that blockages of the Princess St crossing has a significant impact on flood characteristics in the area, especially flooding upstream of the culvert. In this scenario, stormwater fills up the floodway upstream before overtopping across Princess St. Flows continue to pass through the subject site, however the flow depth is expected to be up to 100mm lower.

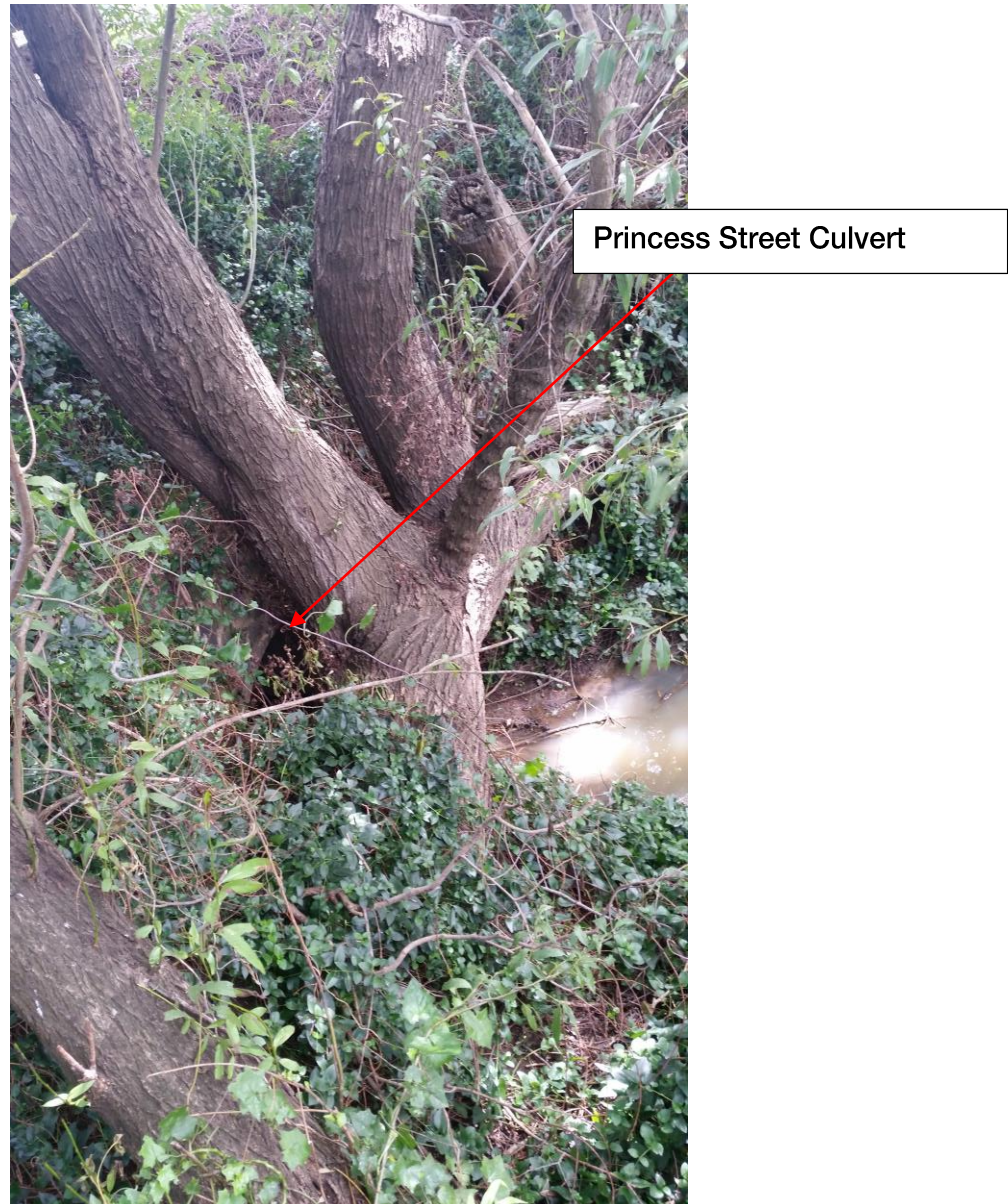


















Figure 33 Princess Street Culvert – Upstream Blockage

Legend

-  Site
 -  Culvert
 -  Developed scenario items
- Developed 1% AEP Flood Depth (m)
-  0.00
 -  0.10
 -  0.20
 -  0.30
 -  0.40
 -  0.50
 -  0.60
 -  0.70
 -  0.80
 -  0.90
 -  1.00
 -  1.10
 -  > 1.10

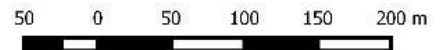
























Figure 34: Developed 1% AEP Flood depth (m) when the Princess St is 50% blocked

Legend

-  Site
 -  Culvert
 -  Developed scenario items
- Developed 1% AEP WSE
-  56.000
 -  56.500
 -  57.000
 -  57.500
 -  58.000
 -  58.500
 -  59.000
 -  59.500
 -  60.000
 -  60.500
 -  61.000
 -  61.500
 -  62.000
 -  62.500
 -  63.000
 -  63.500
 -  64.000
 -  64.500
 -  65.000

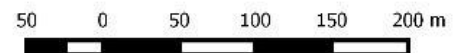


Figure 35: Developed 1% AEP WSE when the Princess St is 50% blocked



Figure 36: Developed 1% AEP Flow velocity when the Princess St is 50% blocked

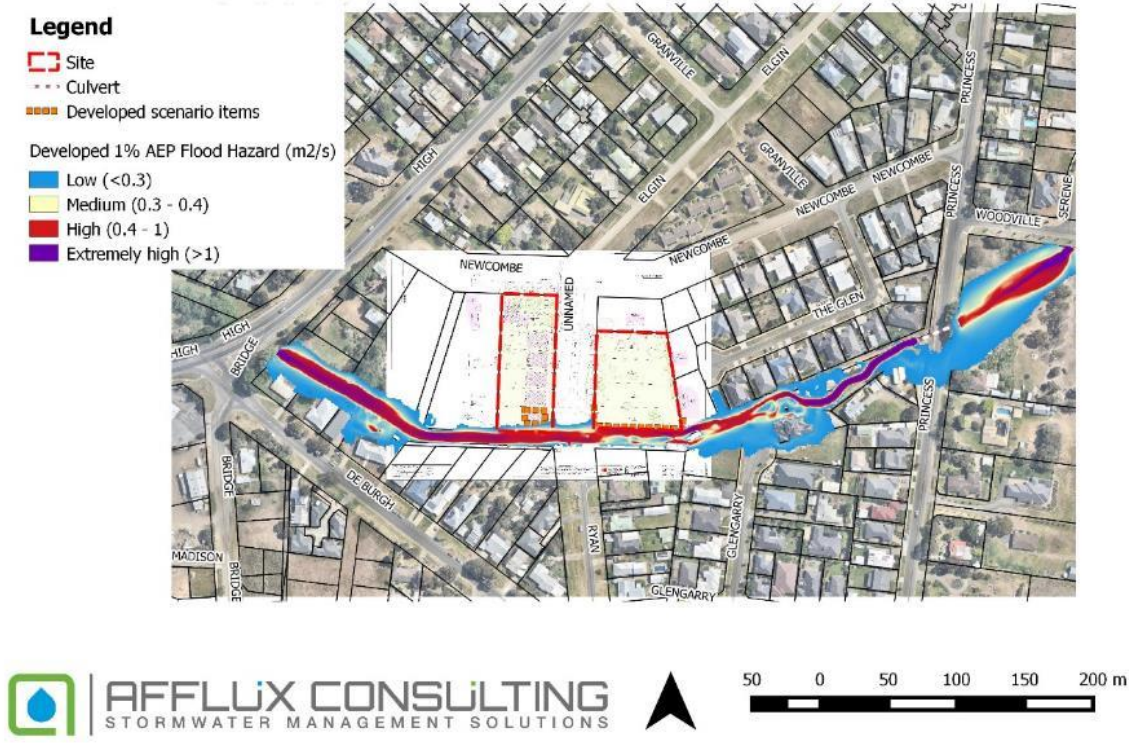


Figure 37: Developed 1% AEP Flood Hazard (m²/s) when the Princess St is 50% blocked

3. Discussion

The flood modelling has shown that despite the relatively large flows through the area the subject site is largely flood free expect for the lower 5-10 meters adjacent to the creek. It has also been shown that much of this flow area is not active flooding, and as such some minor filling to provide a consistent stream frontage with upstream developments can be undertaken.

A Difference Plot between the existing and developed 1% AEP flood depths is shown in Figure 38. The proposed flood solution will direct and contain flows in the floodway rather than allowing them to spread over the lots, as found in the existing flood model. As a result, flow through the floodway is expected to increase to a maximum of 240mm locally, however there are no significant impacts to neighbouring properties. The velocity through the developed site is within acceptable limits of 3.0m/s.

The results showed that the flood extent was sensitive to blockages at the Princess St culvert. Increased flood depths upstream of the culvert and flow crossing Princess St are expected. The primary concerns are the increased flood hazards to properties near Princess St and those who may be using the road at the time. The maximum flood depth in the area is 0.3m and the flood hazard is low (< 0.3m²/s). However, some maintenance and clearing back of trees to the culvert entrance should be considered so that the culvert can carry its maximum capacity.



Figure 38: Flood difference between developed and existing 1% AEP flood results

3.1 Recommendations

For the proposed development to meet safely and risk mitigation requirements for the site, the development must adopt the following measures:

- Earthworks to the eastern development in order to contain flows in the flood way. These works should be set a minimum of 1.2m back from the site boundary.
- Finished floor levels (FFL) at least 300mm above the WSE. Namely, the FFL for the affected lots should be:
 - Lot 6 on the western side of Elgin St = 58.05m AHD
 - Unit 4 on the eastern side of Elgin St = 58.55m AHD (if proposed unit is to be built in the middle of the lot, otherwise, see Figure 39)
- Ensure that access to driveways and proposed buildings are above the expected flood level, and located as north of the problematic lots as possible
- Buildings and highly trafficked areas (sheds/outdoor entertainment areas etc.) should also be located as north as possible on the lot however this is flexible if other requirements are met

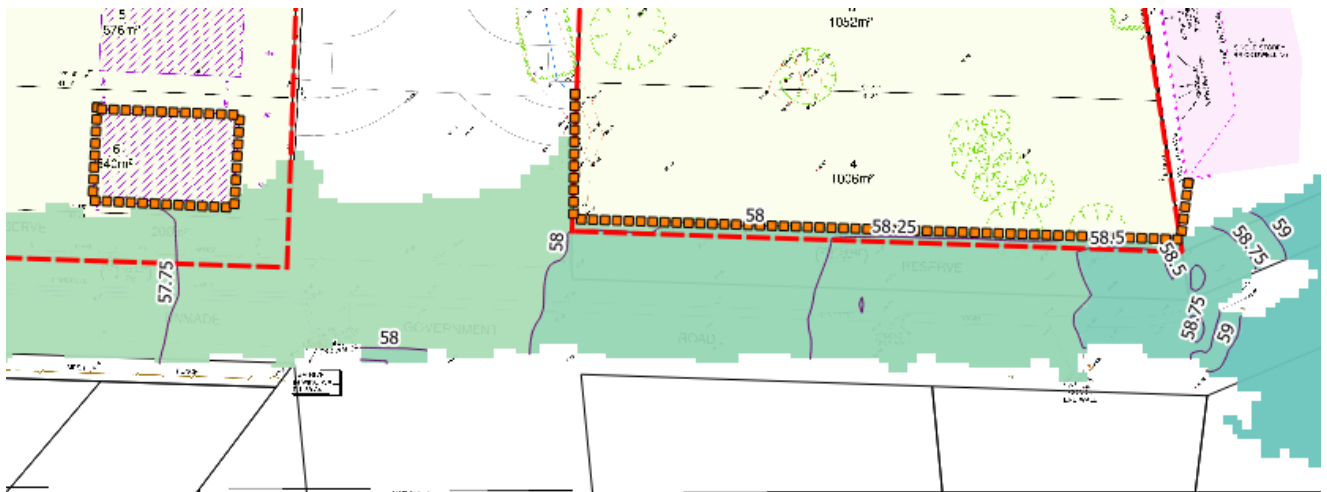


Figure 39: 1% AEP Water surface elevation near development boundaries

4. Conclusions

This report presents the findings of a flood investigation for 40 – 42 Newcombe and 41 – 47 Elgin St. The detailed investigation covers various hydrologic and hydraulic aspects of the floodway adjacent to the site to support future development of the lots. It demonstrates that the proposed development can meet its safety and flood risk mitigation objectives by follow the key elements outlined. In particular, to ensure that driveways and access to proposed buildings are not located within flood affected areas and as north as possible on the lots, earthworks or other solutions which protects the eastern lot (set back a minimum of 1.2m) and finished floor levels 300mm above the developed 1% AEP WSE. Given that the proposed mitigation technique is related to permanent infrastructure (buildings, driveways and surface levels), Corangamite CMA and Geelong City Council can have confidence that proposed solution will provide permanent protection from flood risk and meet its long-term safety objectives, even with the change-over in owners.

5. References

Water Technology, (2012), *“Woodville Street Drysdale WSUD and Drainage Functional Design Technical Report”*, Water Technology, Notting Hill

Appendix A

Geelong City Correspondence

Hi Chris

I'm happy with your suggested approach.

Regards

Matthew Wilson
Planning Engineer

P: 03 5272 4510
F: 03 5272 4374
E: mcwilson@geelongcity.vic.gov.au

100 BROUGHAM STREET, GEELONG
PO BOX 104 GEELONG 3220 AUSTRALIA
www.geelongaustralia.com.au

From: Chris Beardshaw [<mailto:chris@afflux.com.au>]
Sent: Monday, 18 April 2016 8:50 AM
To: Matthew Wilson
Cc: "Tracey Pham"
Subject: RE: High Street culvert

Thanks Mathew,

The below makes sense. I think the rebuilt RORB model results in a flow of about 11.5 cumecs for the peak 1 hour storm.

To move forward for this development, can I assume a tail water on the model boundary (at High Street) around 300mm below the floor level of the closest properties?

Cheers

Chris Beardshaw
BEnvEng, MEngSci, Grad Cert River Health

From: Matthew Wilson [<mailto:mcwilson@geelongcity.vic.gov.au>]
Sent: Friday, April 15, 2016 1:16 PM
To: 'chris@afflux.com.au'
Subject: High Street culvert

Hi Chris

I have been advised that funds have been allocated to upgrade the culvert at High Street, Drysdale in the 2016-17 budget. I suspect works will not occur until late in the cycle (ie. early-mid 2017) as design has not yet commenced. The brief requires upgrading of the culvert to allow a Q100 capacity of 11 cumecs under High Street, and a headwater level allowing appropriate freeboard to the lowest floor level to properties fronting DeBurgh Rd/Newcombe St.

Regards

Matthew Wilson
Planning Engineer
