REPORT:

PMP Printing – Stormwater drainage assessment

February 2019

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Abbreviations

Alluvium	Alluvium Consulting Australia Pty Ltd
AHD	Australian Height Datum
BPEM	Best Practice Environmental Management (for Urban Stormwater)
kL	kilolitre
MUSIC	Model for Urban Stormwater Improvement Conceptualisation
NWL	Normal Water Level
SB	Sediment basin
Tcrit	Critical time
TN	Total Nitrogen
TP	Total Phosphorus
TSS	Total Suspended Solids
WL	Wetland
VPA	Victorian Planning Authority

1 Introduction

The Victorian Planning Authority (VPA) is working with Monash City Council to prepare a precinct structure plan (PSP) for the PMP Printing (PMP) site at 37-49 Browns Road Clayton. The proposal is to develop the s from the existing industrial use to a mixed commercial and residential site. The planning for the PSP includes the preparation of a Stormwater Drainage Assessment.

This Stormwater Drainage Assessment has been prepared to understand water management requirements on the site under both pre and post-development conditions. Specifically, this requires an understanding of the existing drainage network, the extent and impact of flooding and the assets (and footprint) required to meet stormwater treatment and quality requirements.

The assessment required the following tasks:

1. A situational analysis that establishes current conditions
2. An analysis of existing surface water conditions and constraints
3. Assessment of drainage, flooding and stormwater quality requirements under developed conditions.

1.1 The site

The PMP Printing PSP is Located 18km from Melbourne's CBD on the corner of Browns and Carinish Roads, Clayton. It covers 10 hectares including the PMP Printing factory (8Ha) and several properties on Bendix Drive (2Ha).

Acknowledging the previous industrial uses of the precinct, the vision for the site's redevelopment is: *"The PMP Printing site will be a contemporary mixed-use place that incorporates diverse housing opportunities, local employment, community facilities and high-quality public places. This precinct will support a growing local community, complement the Clayton Road shopping strip and enhance connections to local institutions and open spaces."*

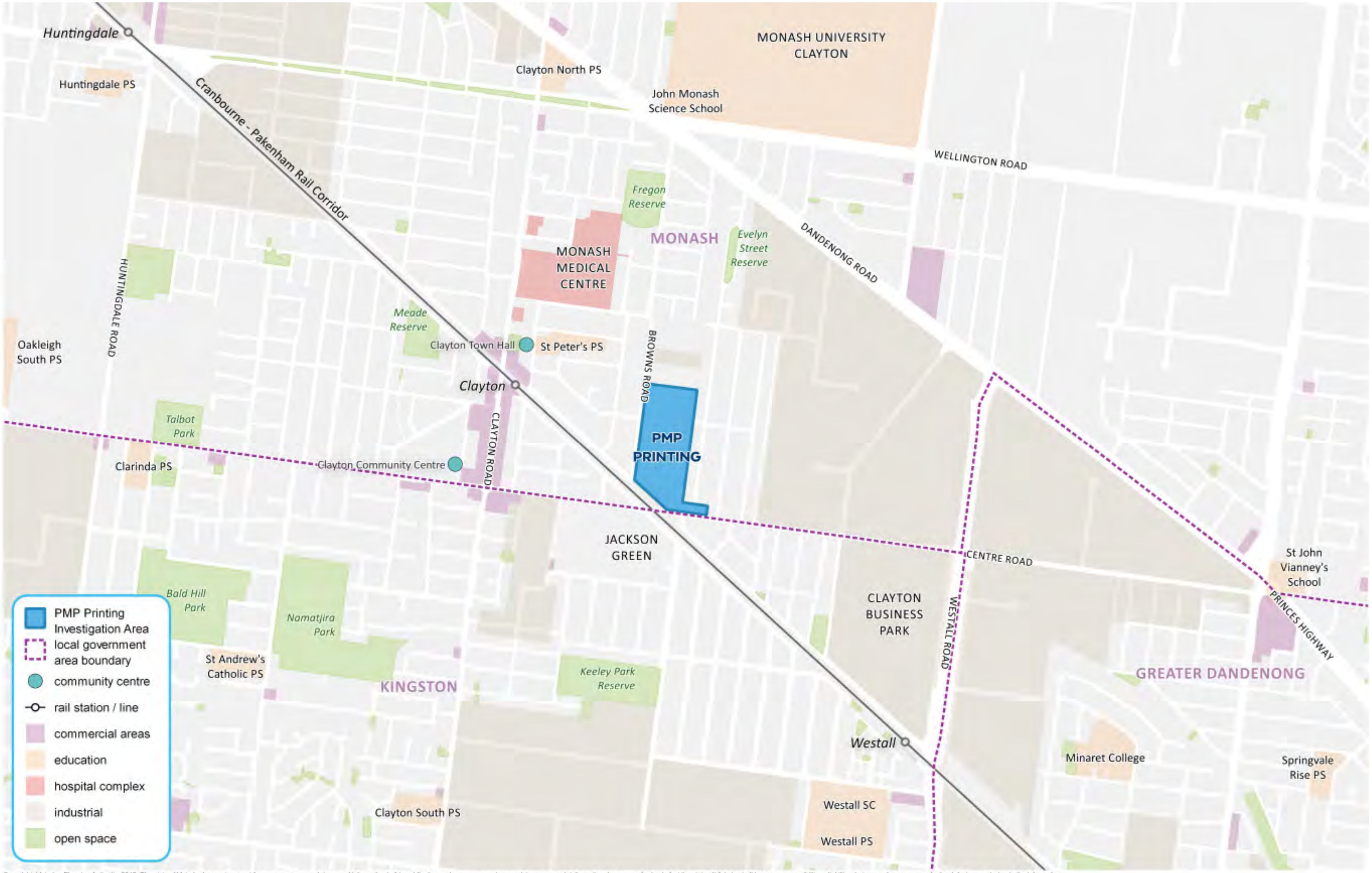


Figure 1. *PMP printing site context map*

2 Existing Conditions

The existing conditions of the site have been compiled from a range of information and data gathered from the VPA, City of Monash and Melbourne Water, including the size and alignment of existing drainage infrastructure. This report also draws on previous site assessments including an Environmental site assessment (Atma Environmental, 2012) and a Feature and level survey (Charter Keck Cramer, 2017).

2.1 Current conditions

A site visit was undertaken in May of 2017. The site is characterised by various impermeable surfaces including as car parks, driveways and factory roofs.

There is a 3.6 m wide drainage and sewer easement down the eastern boundary of the site, this is shown in the photo under Figure 3. This easement contains two to three stormwater drains (and a sewerage main), with stormwater runoff from the site draining to pits along that eastern boundary. The easement also contains a number of large, mature trees.

The staff conducting the site tour highlighted two flooding 'hotspots' (Figure 4 and Figure 5). The flooding in these locations has been managed through augmentations to the internal drainage network. It is envisaged that redevelopment of the site will remove the flooding risk currently associated with these locations.

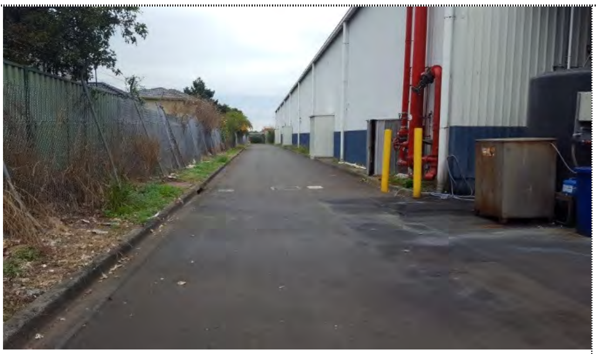


Figure 2. *The site is characterised by industrial buildings and impermeable surfaces (Image facing south along the eastern boundary)*

Figure 3. *Easement down the eastern boundary of the site (facing north on the eastern boundary)*

Figure 4. *A localised flooding location within the site occurred as a result of water running down this concrete ramp. This was rectified by increasing the strip drain size at the bottom of the ramp.*

Figure 5. *A flooding location highlighted by the PMP staff member at the south-western corner of the site (a low point)*

2.2 Existing services

A summary of the site's existing services was compiled based on:

- City of Monash stormwater asset information (pipe and pit network) and pipe construction drawings where available to verify this data
- Melbourne Water drainage infrastructure data and information
- A services plan for the site prepared by Taylors
- A Dial-Before-You-Dig (DBYD) search to verify services.

Catchment
The 13.4 Ha internal catchment and 37 Ha upstream catchment (external to the PMP site) that drains to those pipes in the easement is shown in Figure 6.

Drainage infrastructure
The site drains to the south east with stormwater conveyed via pipes within the eastern easement. Two drains run the length of the easement, with a third introduced at or near the south-east corner of Sub Precinct A. The pipes change diameter as they extend south, both increasing and decreasing in size from 225mm to 1050mm (according to the stormwater network data and the construction drawings). The construction drawings for these pipes verified pipe diameters and grades to estimate pipe capacity. The location and alignment of these pipes and other services is provided in Figure 7 (see Appendix A for detail versions).

Upon exiting the site, stormwater is either carried beneath Centre Road via a City of Monash's 1050 mm diameter drain to Melbourne Water's Westall Drain. Alternatively, water is diverted to drains at the south of Bendix Drive which cross Centre Road and join the Westall Drain via a 750mm diameter drain.

Melbourne Water's Westall Drain diameter is 1060 mm diameter for a short section before increasing to a 1750 mm diameter. The drain travels in a south-easterly direction along the rail corridor before crossing the Clayton Business Park site and Westall Road. Stormwater then discharges to Mile Creek on the east side of Westall Road.

Mile Creek is a major tributary of Dandenong Creek with a largely urbanised catchment area of approximately 39 km^2. Near the PMP Printing site, Mile Creek is a highly modified, concrete lined channel with limited environmental and ecological values.

Sewer
A 375mm diameter sewer pipe runs in a north-south direction within the eastern easement between stormwater drains. The service plan shows this sewer main on a very similar alignment as one of council's stormwater pipes.

Drainage requirements
The City of Monash's drainage guidelines require the sub-surface or minor drainage network to convey the 1 in 5-year average recurrence interval (ARI) flowrate at full development. The surface or major drainage network (including roadways), is to be designed to convey up to the 1 in 100-year average recurrence interval (ARI) flowrate at full development.

The Melbourne Water sub-surface drains are similarly designed to take the 1 in 5-year flowrate. Melbourne Water's requirements will generally be to ensure that greater flows are not generated as a result of the proposed development so that existing drainage infrastructure can continue to deliver the existing level of service. Additionally, Melbourne Water would prefer that as a consequence of development there are not additional connections made to larger drains.

Figure 6. *Stormwater catchment plan*

Figure 7. *Existing services*

2.3 Future development layout

The future development layout is yet to be defined in detail, however a draft layout has been prepared by VPA in collaboration with Monash City Council to inform analysis. The latest development layout is included in Figure 8 below. It should be noted that the analysis undertaken is based on an earlier iteration of that plan. The in-principle results are applicable to the new layout, however they may need to be reviewed as a more detailed understanding of the layout is gained. The layout is also used to estimate stormwater volumes and pollutant loads that are likely to be generated.

Figure 8. *Development layout (updated)*

3 Catchment analysis

With an understanding of existing site conditions, drainage infrastructure and the proposed development layout, analysis was undertaken to define the sites internal catchments and land uses. The site will include a mix of residential, mixed use and commercial activities as well as open space are also proposed (See Figure 8 above).

For the purposes of surface water modelling each land use type assumes a 'fraction impervious'. The fraction impervious reflects the assumed proportion of that land use that is likely to be impervious or paved. This impacts the volume of stormwater that land will generate in a rainfall event.

The adopted fraction impervious (as a proportion between 0 and 1.0) was sourced from Melbourne Water's MUSIC guidelines (2016) and summarised in Table 1.

Table 1. *Adopted fraction effective imperviousness for proposed land use type*

PSP proposed Land use	Adopted zone description	Adopted zone code	Fraction imperviousness
Medium Density Residential	General Residential Zone –Standard densities (Allotment size 300-600 m^2)	GRZ	0.75
High Density Residential	General Residential Zone –High densities (allotment size <300 m^2)	GRZ	0.85
Business	Commercial zone	C1Z	0.9
Local Park	Public Park and Recreation Zone	PPRZ	0.1
Landscape Values	Road Zone – Secondary and local roads	RDZ2	0.6

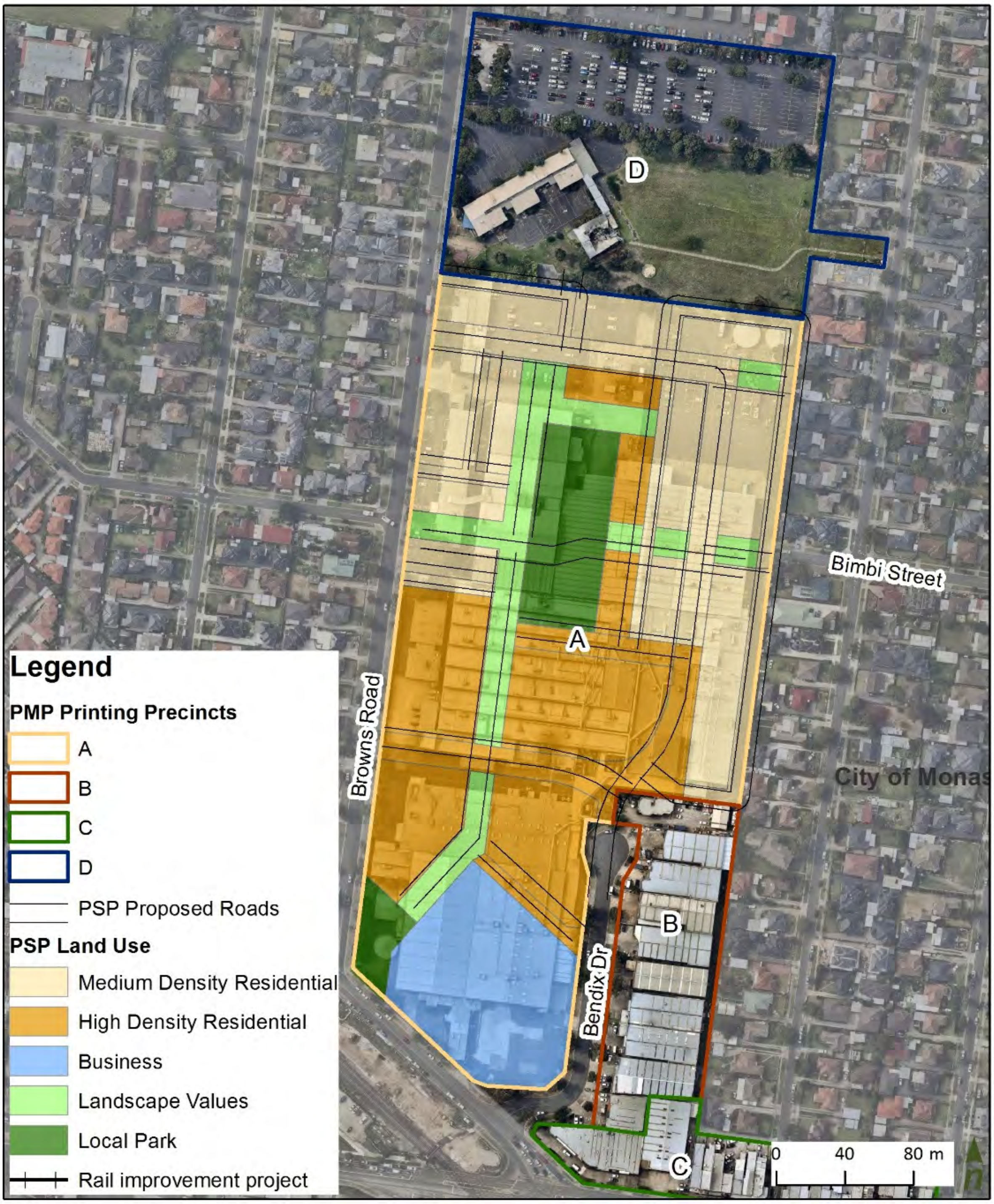


Figure 9. *Proposed post development land (based on VPA draft urban layout)*

3.1 Sub-catchments

The precinct was divided into four sub-catchments (1 through 4) based on the development layout, road network and site topography (Figure 10). The area of each sub-catchment and the imperviousness fraction are summarised in Table 2.

Figure 10. *Sub-catchment mapping*

The role of open space
The location and area of open space is an important factor in planning the nature and location of stormwater treatment assets. In brief, larger open space areas can accommodate larger stormwater treatment assets like wetlands, whereas in more confined or built up areas the WSUD assets are more likely to be integrated into the built form or streetscape in the form of raingardens or biofilters.

There are two public open spaces proposed for the Precinct. The first is situated downstream of sub-catchments 1 and 3 that make up approximately 55 % of the site's total impervious area. Sub-catchments 2 and 4 make up the remaining 45 % of impervious, however there are no open spaces downstream of those catchments, requiring a more distributed response.

These constraints and opportunities are discussed further in section 5.1 below.

Fraction impervious
The calculated fraction impervious for each catchment is provided in Table 2. The 'Effective impervious area' is a product of the area of that particular land use and the fraction impervious for that land use, as drawn from Table 1 above.

The summary estimates that the post development PM site has an effective impervious area of 6 Ha.

Table 2. *Effective imperviousness area by sub-catchment*

Sub-Catchment	PSP proposed land use	Area (ha)	Fraction impervious	Effective impervious area (ha) (Area x Fraction impervious)
1	Local Park	0.35	0.10	0.03
	Landscape Values	0.54	0.60	0.32
	Medium Density Residential	2.19	0.75	1.64
	High Density Residential	0.23	0.85	0.20
	Sub total	**3.31**		**2.20**
2	Local Park	0.19	0.10	0.02
	Medium Density Residential	0.62	0.75	0.46
	High Density Residential	1.55	0.85	1.31
	Sub total	**2.35**		**1.80**
3	Local Park	0.14	0.10	0.01
	Landscape Values	0.33	0.60	0.20
	Medium Density Residential	0.15	0.75	0.12
	High Density Residential	0.93	0.85	0.79
	Sub total	**1.56**		**1.12**
4	Business	**0.99**	**0.90**	**0.89**
	Total	**8.22**		**6.01**

4 Hydrologic modelling

This section details the hydrological analysis undertaken to estimate pre and post development peak stormwater flowrates (in m^3/s). The policies guiding this work include:

- Monash City Council's Stormwater Management Policy that stipulates: "Stormwater flows generated from increased imperviousness area be managed by on-site retention systems"
- The Legal Point of Discharge (LPD) approval stipulates that stormwater detention is required for developments to balance the 10-year ARI post development peak flows with the existing 5-year ARI peak flow rate.

Hydrological modelling was undertaken to understand how these requirements are managed within the context of existing drainage capacity and considering the potential role of on-site detention.

4.1 Hydrologic modelling detail

Hydrologic modelling was undertaken using RORB (v6.31), a runoff-routing model that simulates attenuation and delay of a hydrograph to produce flood estimates at specified catchment locations. A RORB model was built using the existing stormwater network for both pre and post developed conditions to understand:

- the impact of development on peak flows
- the reductions in peak flows that were possible through on-site storage (i.e. tanks).

The RORB model was built by delineating the major stormwater network catchments into sub-areas based on existing drains. The catchments, reach lengths and nodes used to build the RORB model are shown in Figure 11. The main discharge point of interest was at Centre Road near the Westall main drain.

The fraction impervious values adopted for the existing and developed conditions models are shown in Table 3 below. The same fraction impervious values were adopted for the stormwater treatment modelling (in MUSIC) and were based on the draft urban layout. For the site of the former primary school, the fraction impervious values adopted in the stormwater management plan created for the site were used for consistency (David Lock Associates, 2016).

Table 3. Fraction impervious values adopted for existing and developed conditions RORB model

Land type	FI (existing)	FI (developed)
Existing residential	0.6	0.6
PMP subcatchment 1	0.9	0.66
PMP subcatchment 2	0.9	0.77
PMP subcatchment 3	0.9	0.71
PMP subcatchment 4	0.9	0.9
Remaining PMP subcatchments*	0.9	0.9

* FI kept the same as no information on future development of these areas

The existing conditions model was calibrated against urban rational calculations. Several developed conditions models were run with different storage scenarios to understand the reduction in peak flows that they could provide. Further detail on RORB modelling method is provided in Appendix C.

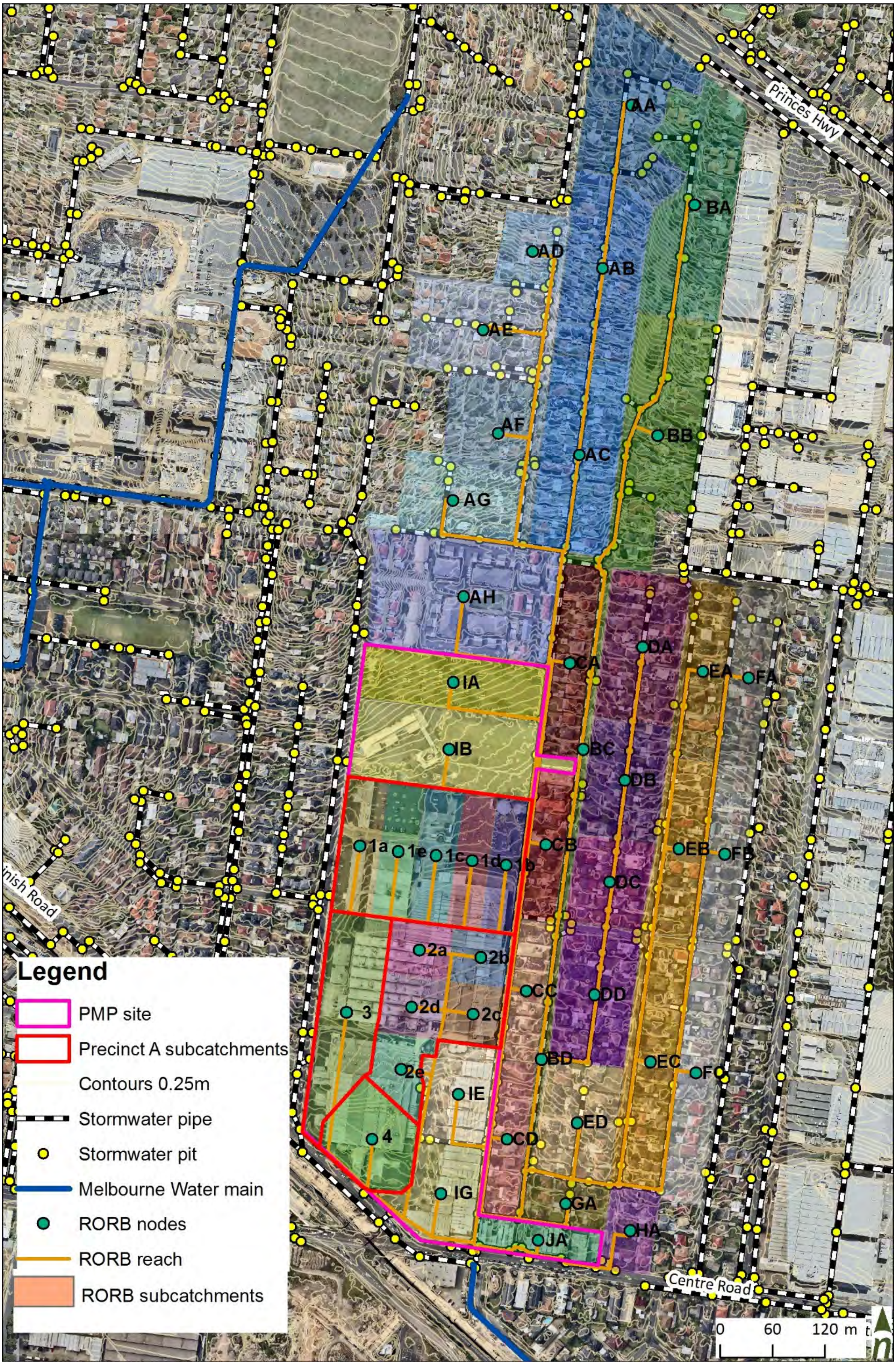


Figure 11. *RORB sub catchments, nodes and reaches*

4.2 Hydrologic modelling results

The pipe capacity of the 1050mm stormwater pipe located across Centre Rd is estimated to be 2.71m^3/s. This flowrate is the limiting constraint downstream of the PSP.

Hydrologic modelling was undertaken to simulate flow rates for pre and post development conditions. The method undertaken was a Monte-Carlo simulation to determine peak design flows based on varying temporal patterns. In reviewing the results in Table 4 below it should be noted that the stochastic treatment of temporal distribution of rainfall can cause variabilities in results. That is, the same scenario can produce slightly varying results.

Four scenarios were investigated to understand the pre and post development flowrates.

Table 4. Peak flow results at Centre Rd under existing conditions and various developed condition scenarios

Scenario	Q5 ARI peak flow	Tcrit
1. Pre-development conditions	4.9 m^3/s	15 mins
2. Post-development conditions (no on-site storage)	5.0 m^3/s	15 mins
3. Post-development conditions (with 80kL storage)	4.9 m^3/s	15 mins
4. Post-development conditions (with 1200kL)	4.6 m^3/s	15 mins

What can be observed is:

- Pre-development conditions exceed the capacity of downstream drainage (i.e. 4.9 m^3/s) is greater than 2.71 m^3/s). This is consistent with anecdotal evidence of localised flooding.
- Post-development flowrates are similar to pre-development flowrates. This is because the change in land uses within the PMP site is relatively insignificant in the context of the larger upstream catchment
- The installation of storage volumes has some impact on the peak flowrates (reducing them from 5 to 4.6 m^3/s), but nowhere near enough to reduce the flowrates back to the capacity of the existing system.
- The relatively large storage of 1,200 kL exhibits ‘diminishing marginal returns. That is, ever larger storage volumes achieve a decreasing proportion of peak flow reduction.

In conclusion, the existing drainage network is (and has been) undersized. The cost associated with upsizing the downstream network is likely to be prohibitive and is not considered as an option here. On site storage will not be sufficient to reduce peak flows to the downstream capacity of the network.

Internal storage

Despite the results above, there may still be value in maintaining storage within the PSP so that the internal and sub-surface drainage network can meet the drainage requirements of containing up to the 20% AEP event (or the 1 in 5 year ARI event).

The reason for this is that if the Centre Road main acts as a throttle, then flows from the whole catchment may start to back up and inundate the development. This may require storing stormwater from up to the 20% AEP event (for example) on the site and releasing it when there is downstream capacity. In this way the development of the PMP site will not exacerbate existing downstream flooding issues.

Initial calculations suggest storing the 20% AEP event for the critical duration of 15 minutes equates to a volume of approximately 860kL for sub catchment A. This is a relatively innovative approach that should be further investigated in the next stages of the design and development in consultation with the site's developers. For example, who would manage the storage is unclear, however as it could be considered part of the minor drainage network it such that maintenance may become Council's responsibility.

The presence of a storage like this also opens up the opportunity for the treatment and reuse of the stored water, potential for irrigation of open space within (and potentially beyond) the PSP.

Therefore, additional storage within the PSP may be required to enable the internal drainage network to meet the 20% AEP or 1 in 5 years ARI level of service typically required by the minor drainage network.

5 Stormwater treatment

Stormwater treatment is driven by the requirement to meet the pollutant removal targets set out in the Best Practice Environmental Management Guidelines (BPEMG) (CSIRO, 1999). These targets are necessary to meet the Victorian Planning Provisions and State Environment Protection Policy (Waters of Victoria) objectives. This SEPP is a statutory policy under section 16 of the Environment Protection Act 1970 that identifies the beneficial uses of Victoria's waterways.

The pollutant removal requirements are:

- Total Suspended Solids (TSS): 80 % removal of the typical urban annual load
- Total Nitrogen (TN): 45 % removal of the typical urban annual load
- Total Phosphorus (TP): 45 % removal of the typical urban annual load
- Litter: 70 % reduction of the typical urban annual load

Water Sensitive Urban Design assets (WSUD) including wetlands and biofiltration assets are sized and located within the catchment to meet these stormwater pollution reduction requirements. In the context of this development layout, wetlands are not practicable given the area they would require here to meet BPEM. For that reason, biofilters, that are more effective treatment mechanisms per unit of area, have been modelled in the scenarios discussed below.

Based on the development layout provided three scenarios were investigated to respond to the catchment areas, land uses and imperviousness described above. The pros and cons of each scenario are discussed below with a view to proposing a preferred option.

A MUSIC (Model for Urban Stormwater Improvement Conceptualisation) model has been developed for each scenario with the following input parameters:

- Rainfall data from Koo Wee Rup rainfall gauge (1971-1980) at a 6 min time-step
- Catchment fraction imperviousness based on values in Table 1.

5.1 Stormwater treatment Scenario 1

Title: End of catchment WSUD assets

Description: This option proposes installing WSUD assets within each catchment with the aim of treating as much of that individual catchment as possible. What can be observed is:

- For sub-catchments 1 and 3 biofitration assets have been notionally located within the open spaces that have been designated within the plan.
- For sub-catchments 2 and 4 the required bioretention area is illustrated. The approach in these built up area would be to integrate the biofilter within the streetscape. At this stage the approach would be to integrate them into the public realm so that no developable land is lost.

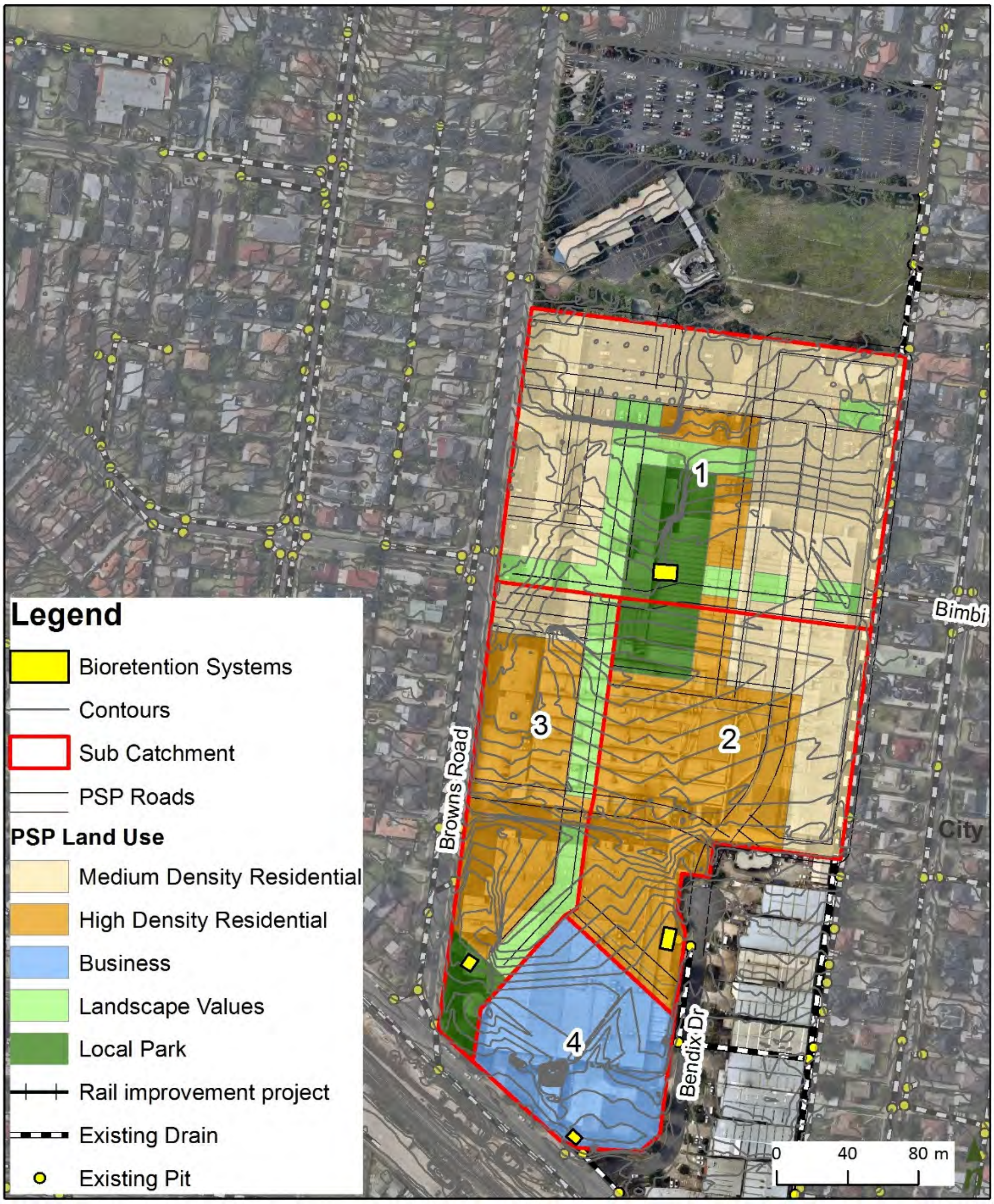


Figure 12. *Scenario 1*

5.2 Stormwater treatment Scenario 2

Title: Oversized WSUD in open space

Description: WSUD assets have been 'oversized' within the open spaces in sub-catchments 1 and 3 to take maximum advantage of that available space and in turn, reduce the space requirements in sub-catchments 2 and 4.

- Sub-catchment 1 and 3: WSUD assets (bioretention systems) are installed within public open spaces and oversized to treat stormwater beyond best practice. In sizing the WSUD for open spaces, the treatment area has been limited to 10% of the total open space area.
- Modelling indicates that WSUD in sub-catchments 1 and 3 cannot be sized to an extent that avoids the need for WSUD in sub-catchments 2 and 4
- Sub-catchments 2 and 4: Bioretention systems will ideally be installed within the residential / commercial streetscapes and public realm to avoid land take associated with those assets.

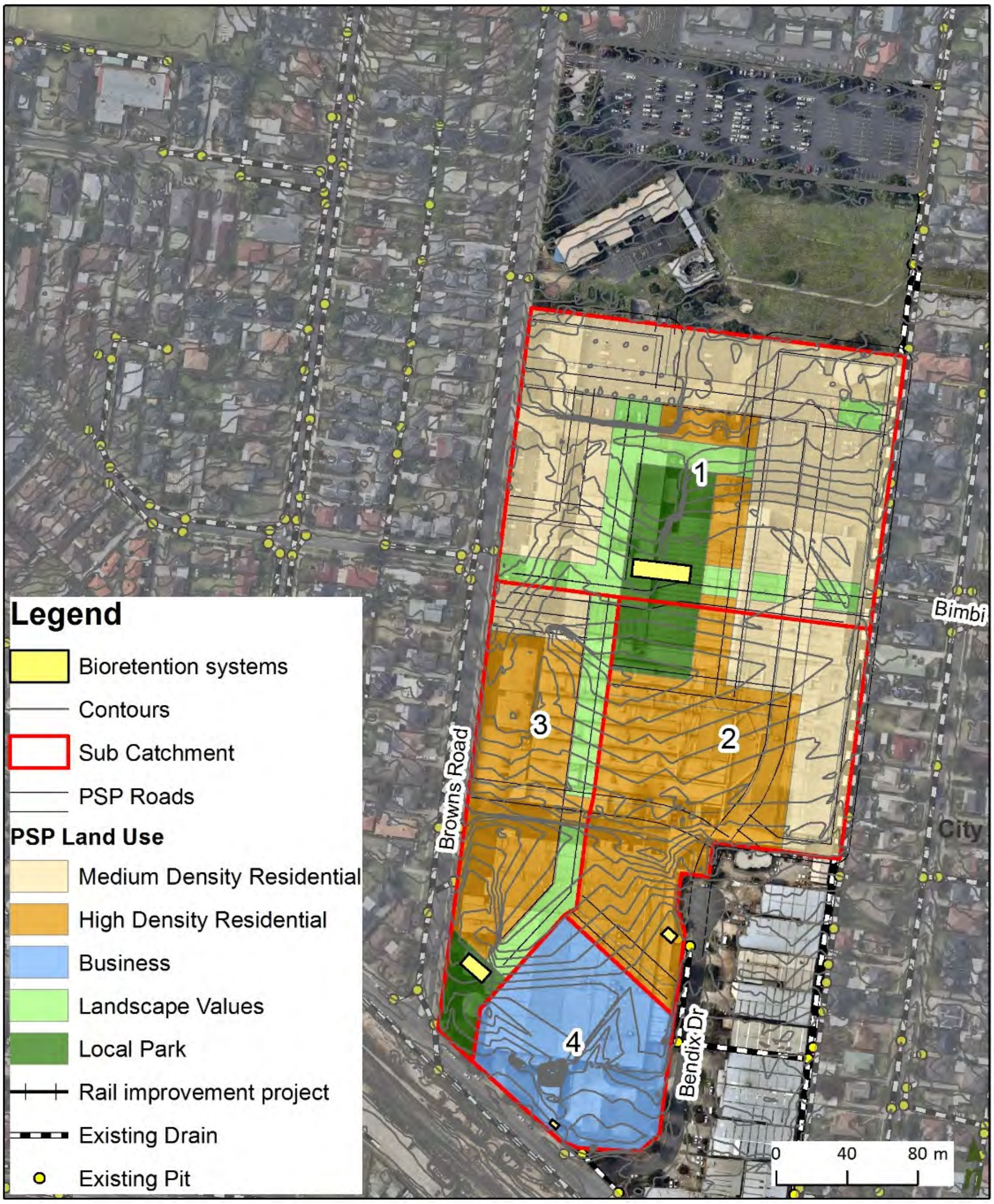


Figure 13. *Scenario 2*

5.3 Stormwater treatment Scenario 3

Title: Oversized WSUD in open space with rainwater reuse and distributed WSUD in sub-catchments 2 and 4.

Description: This scenario adopts the oversizing of WSUD assets in open space (up to a total of 10% of the total area of the open space) as per scenario 2. It also investigates the impact of rainwater reuse on WSUD asset requirements.

In sub-catchment 3:

- Distributed rainwater tanks for residential developments to collect roof runoff for use in toilet flushing. Modelling assumes a medium density of 50 dwellings/ha and high density of 100 dwellings/ha. Each dwelling has 2 people and a water demand for toilet flushing of 20 litre/person/day. Tanks harvest from 50 % of roof area catchment with tanks sized to achieve 80 % toilet flushing demand reliability.
- Street scale raingardens are installed within the streetscape and public realm
- A GPT is installed at the catchment outfall and is assumed to have a treatment removal efficiency of 50 % for TSS and 70 % for Gross Pollutants.

In sub-catchment 4 a GPT is installed at the catchment outfall with an assumed treatment removal efficiency of 50 % for TSS and 70 % for Gross Pollutants.

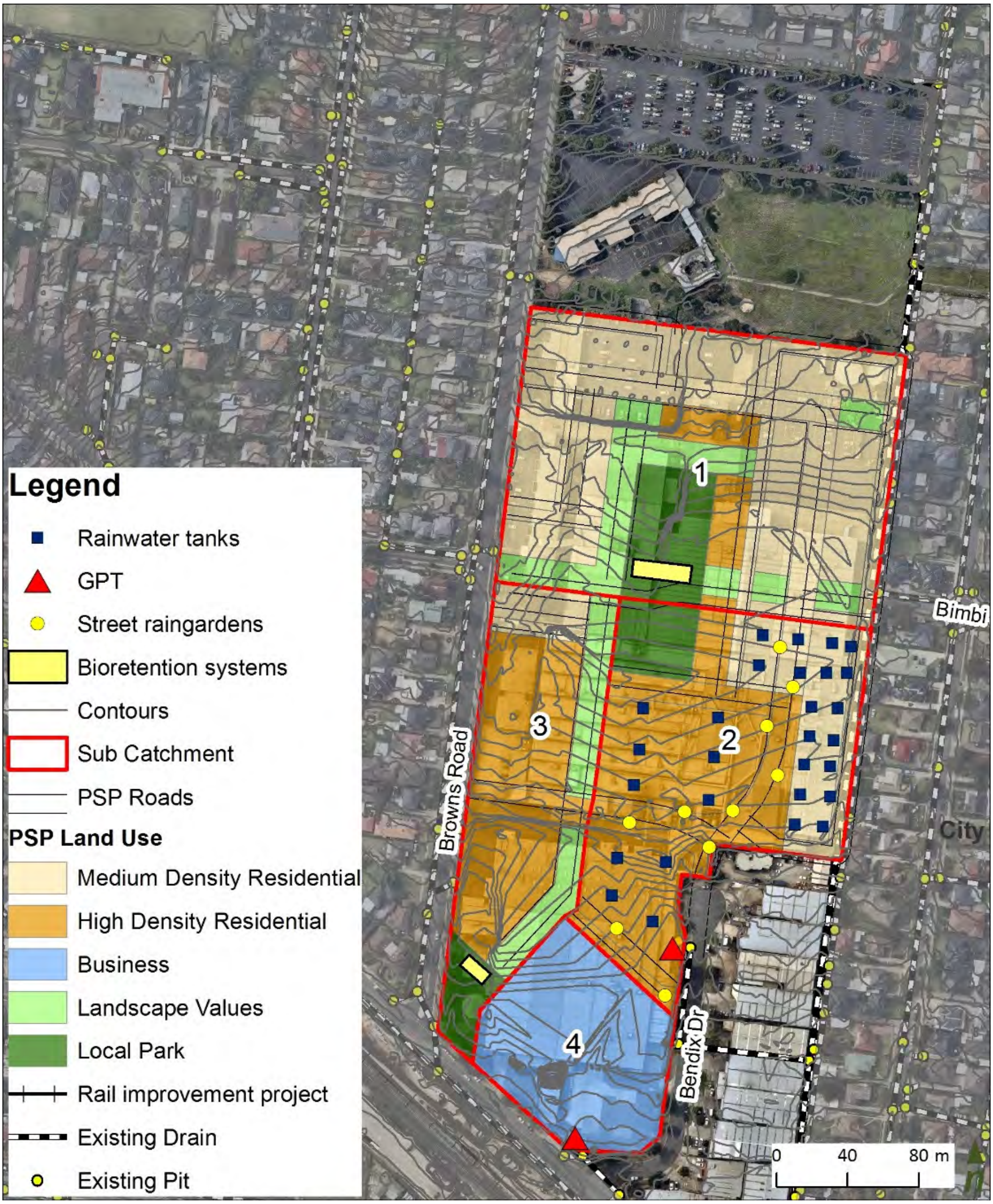


Figure 14. *Scenario 3*

5.4 Scenario analysis summary

MUSIC modelling was undertaken for each scenario to understand the footprint of each asset with a high-level costs estimate associated with each. At this stage it is anticipated that smaller biofiltration assets should be able to be integrated within the streetscape without impact on developable land budget, however this will need to be investigated in greater detail.

Table 5 provides a summary of the modelling outcomes, with more detailed analysis provided in Appendix B. While an operational cost has not been calculated, it can be assumed that operational costs will be proportional to capital cost, and therefore higher for Scenario 3 than 1.

Table 5. *Scenario results summary*

Scenario	Assets	Estimated capital cost
1	270 m^2 of bioretention asset area • 150 m^2 in sub-catchments 1 and 3 • 120 m^2 for sub-catchments 2 and 4	$270,000
2	544 m^2 of bioretention asset area • 489 m^2 in sub-catchments 1 and 3 • 55 m^2 for sub-catchments 2 and 4	$377,000
3	Sub-catchment 1 and 3: 489 m^2 of bioretention area Sub-catchment 2: 10 street raingardens and 1 GPT Sub-catchment4: 1 GPT 80 kL of distributed rainwater storage: Notionally 30 × 1kL rainwater tanks in the medium density residential area and 10 × 5kL in the high-density residential area (yielding a potable water saving of 2.4 ML/year)	$552,000

5.5 Discussion and recommendation

The 'pros and cons' of each of these scenarios are summarised in Table 6 below. In summary Scenario 1 is recommended and will depend upon sufficient space being allocated toward WSUD in the streetscapes and public realm of catchments 2 and 4.

Table 6. *Pros and cons*

Scenario	Pros	Cons
1	• Fewer assets with less complex implementation and maintenance • Lowest capital and operating cost • Amenity improvements in open space	• 120 m^2 of space required within sub-catchment 2 and 4 • If this area can't be found in the streetscape it may require otherwise developable land
2	• Fewer assets with less complex implementation and maintenance • Less area required in sub-catchment 2 and 4 compared to Scenario 1	• Greater capital cost than Scenario 1 • More area for WSUD taken from open space (but a reasonable percentage)
3	• No loss of developable land in sub-catchment 2 and 4 • Potable water use reduction (2.4ML/year)	• Greatest capital costs • Issues getting developers to install tanks • Responsibility for maintenance of tanks • More area for WSUD taken from open space (but a reasonable percentage)

6 Flood analysis

The following chapter summarises the outcomes of the flood modelling undertaken in partnership with Water Technology. A summary is provided here with a Summary Report provided in Appendix D.

In summary, the post development flooding that can be expected is not considered hazardous or high risk taking into account both the depth and velocity of flood waters. There is a reduction in flood levels in and around open space and no significant change in flood conditions external to the development.

6.1 The site

The site is situated towards the top of the catchment, with a ridge line to the west of the site. The total catchment extends north as far as Princes Highway and there are no existing flood overlays on the site. Anecdotal information gathered during the site visit (Communications: Kane Flynn) suggests that intense storm events can result in localised flooding on Centre Road with "one to two feet" of floodwater.

6.2 Model

A 2-Dimensional TUFLOW model was built to assess site flooding for post pre and post development conditions. The model was run for the 1% AEP 2-hour storm event which was identified as the critical storm. The aim of the modelling was to understand the difference in Flood depth, Water surface elevation and maximum flood velocities for both the pre and post development conditions.

An additional climate change scenario was modelled that incorporated a 19% increase in rainfall intensity as per Melbourne Water's requirements. Sea level rise is not considered a factor here.

6.3 Results

Figure 15 shows the main flow path near the subject site under pre-development conditions. During the 1% AEP flood event, a maximum of approximately 120 mm flood depth can be expected on the access road to the site while the flood water ponds at the downstream end of the site at Carinish Road.

Figure 16 illustrates the same pattern under the proposed developed conditions. In this case the maximum flood depth at the access road has increased by 40 mm to a total maximum depth of 160 mm. The increases are contained within the site and do not cause any external flooding. Appendix D contains figures that illustrate the difference between pre and post development.

Overall, it is considered that the 1% AEP flooding within the site will be relatively safe and will meet all requirements of Melbourne Water as outlined below:

- **Water Surface Elevation (WSE)**: post development conditions result in a reduction in WSE in the areas that became less impervious under the proposed development. That is, those areas identified as open spaces. An increase of up to 40 mm is identified at the access road to the site. No adverse effects are observed outside the boundary of the subject site.
- **Maximum flood velocities**: there is a maximum 0.03 m/s increase in the velocity at the access road to the site, however, there is no significant change in the velocity at the downstream of the site.
- **Flood safety**: this is measured as the product of depth and velocity. Flood waters are considered hazardous where the product of these two numbers is above 1.5 m^2/s. Using a conservative approach in calculating this across the entire PMP site results in 0.02 m^2/s. It is therefore considered that the 1% AEP flooding within the site will be safe.

A summary of the results, comparing the existing and proposed development at the access road (within the site) and Carinish Road (just outside the site) is presented in below:

Table 7. Peak flow results at Centre Rd under existing conditions and various developed condition scenarios

Results	Existing		Developed	
	Access Road	Carinish Road	Access Road	Carinish Road
Flood Depth (m)	0.12	0.47	0.16	0.48
WSE (m AHD)	56.16	54.02	56.20	54.03
Velocity (m/s)	0.17	0.17	0.20	0.17

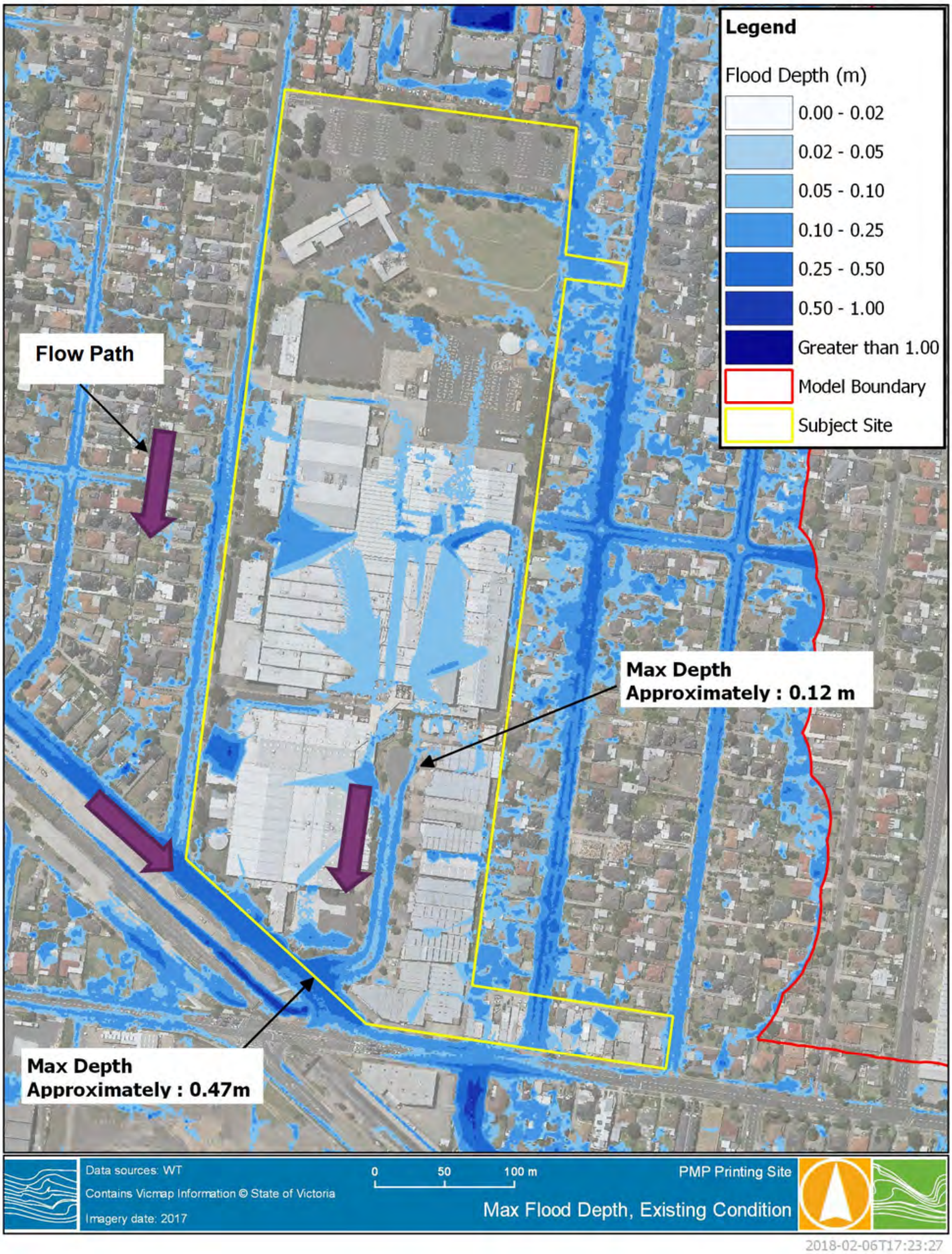


Figure 15. *Flood depths under existing conditions*

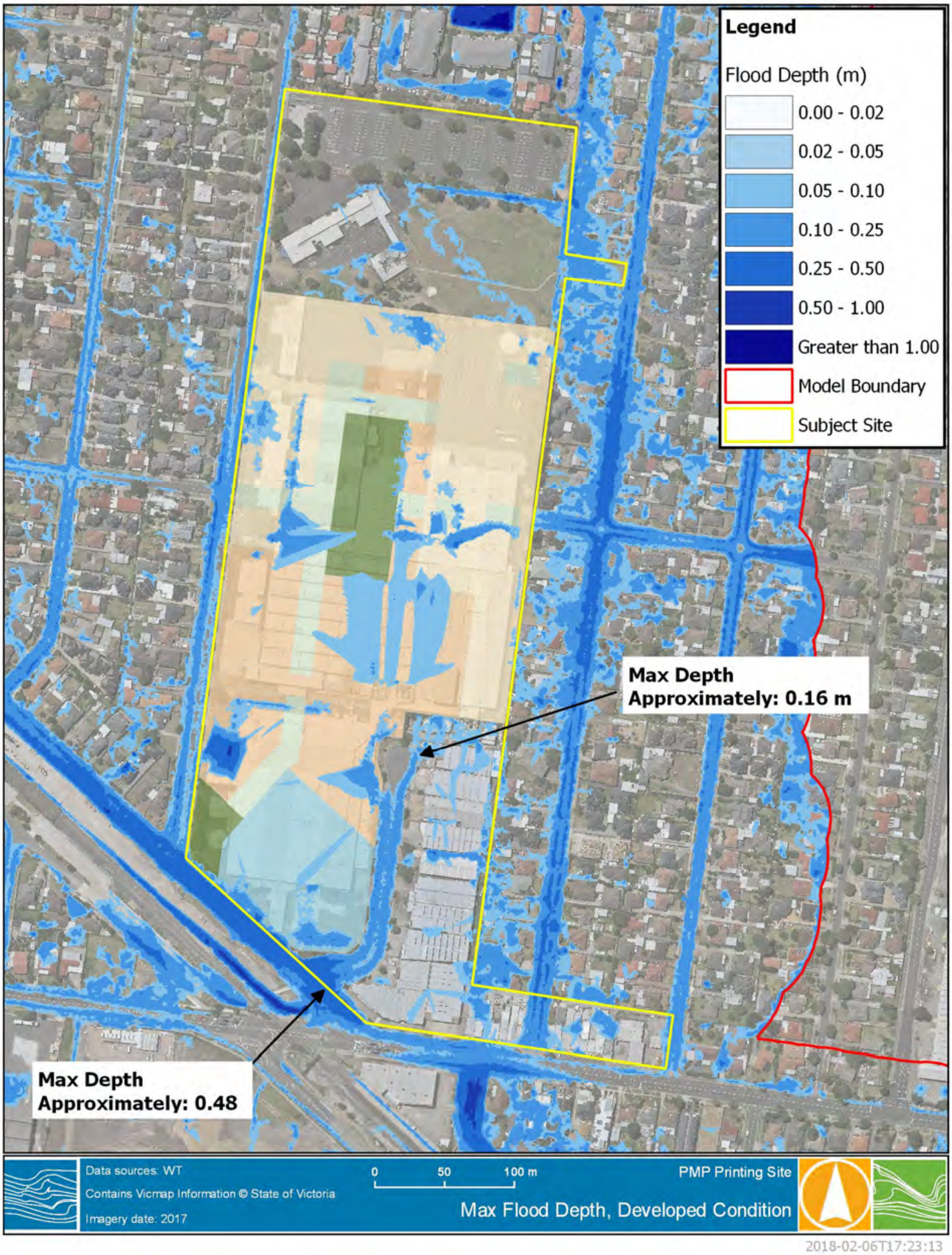


Figure 16. *Flood depths under developed conditions*

6.4 Climate change analysis

A climate change scenario was modelled in accordance with Melbourne Water's directive that requires the inclusion of a 19% increase in rainfall intensity. It was assumed that the sea level rise requirements would not be applicable in Clayton.

The impact of the increase in rainfall intensity due to climate change on flood depth, water surface elevation and flood velocity is summarised in Table 8 that compares existing conditions, developed conditions and a climate change scenario.

Table 8. Climate change scenario results

Results	Existing		Developed		Climate change	
	Access Road	Carinish Road	Access Road	Carinish Road	Access Road	Carinish Road
Flood Depth (m)	0.12	0.47	0.16	0.48	0.20	0.50
WSE (m AHD)	56.16	54.02	56.20	54.03	56.27	54.26
Velocity (m/s)	0.17	0.17	0.20	0.17	0.19	0.18

Results summary

In summary there is no significant change to the level of flood safety across the site under a climate change scenario. There is an increase in flood depth of approximately 40 mm at Access Road and 20 mm at Carinish Road when compared to the developed scenario.

The maximum depth times x velocity across the site is approximately 0.5 m^2/s, remaining comfortably below the 'hazardous' threshold of 1.5 m^2/s.

A figure illustrating these results is provided in Figure 17.

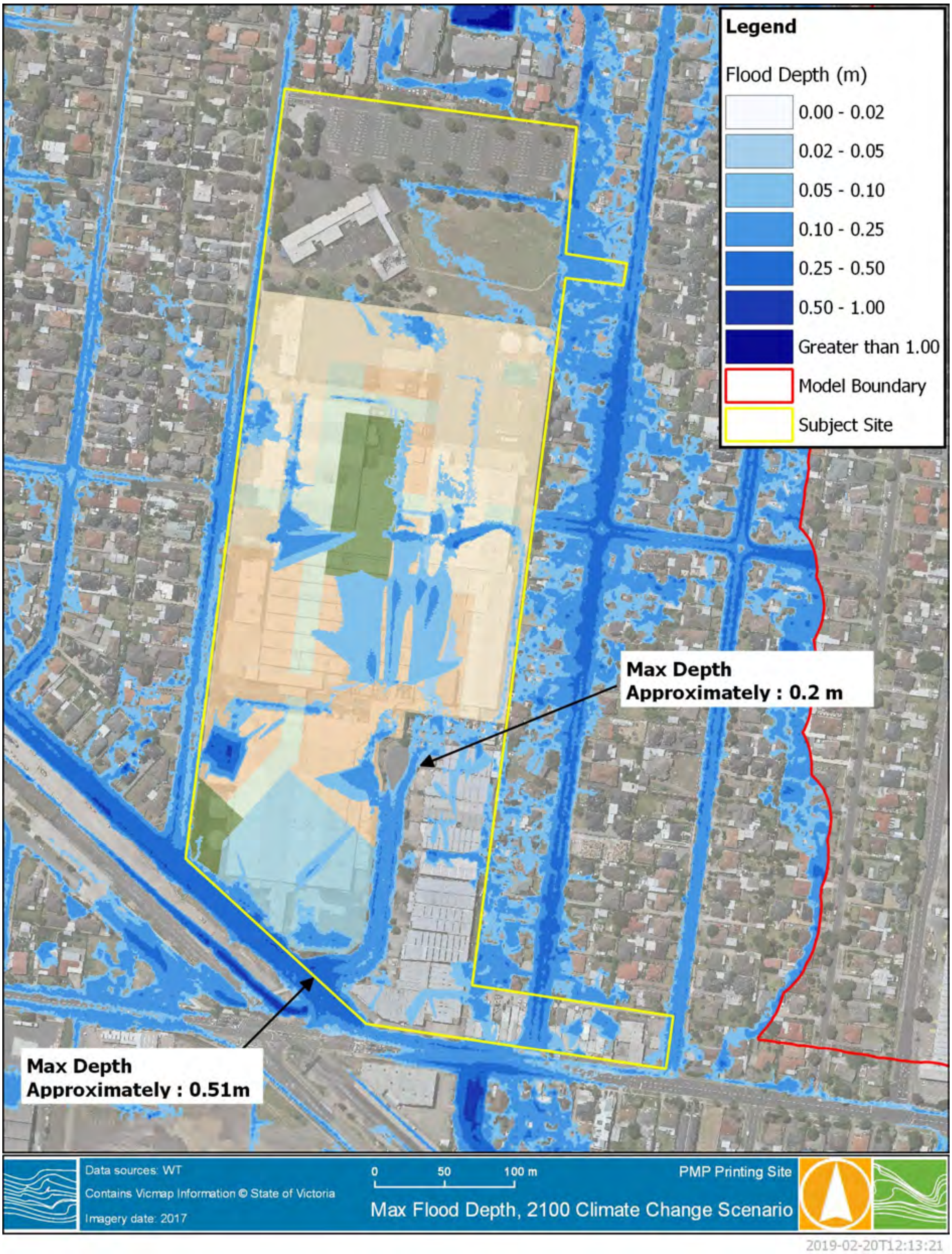


Figure 17. *Climate change 2100: Flood depths under developed conditions*

7 Conclusion and recommendation

Analysis of the PMP site has focussed on:

- The capacity of drainage infrastructure downstream of the site to manage pre and post development stormwater flows
- Stormwater treatment scenarios to treat post development stormwater volumes to meet pollution reduction targets
- An assessment of the safety and risk of post development flooding

Important takeaways from the analysis includes:

- The existing drainage network is (and has been) undersized. The cost associated with upsizing the downstream network is likely to be prohibitive and is not considered as an option here.
- Analysis suggests that the deficit between peak flowrates (about 5.0 m^3/s) and downstream capacity (2.7 m^3/s) is too great to be bridged by on site storage
- The change in imperviousness within the PMP Printing site, due to the addition of open space, is not significant enough in the context of the upstream catchment to significantly reduce peak flowrates.
- Additional storage may be required within the PSP to enable the internal drainage network to meet acceptable drainage service levels (i.e. the 20% AEP or 1 in 5 years ARI level of service). Without additional storage it is likely that stormwater will back up the network into the PMP Printing site. Preliminary assessment suggests this volume is in the order of 860kL. This storage will act to improve the level of protection against nuisance flooding by isolating the site from the existing downstream stormwater network. It is Alluvium's recommendation therefore that further analysis on internal storage options within the PMP site be undertaken to refine the volume required and the potential ownership and maintenance of that storage.
- Scenario 1 is the recommended stormwater treatment scenario that includes WSUD assets within open spaces and streetscape or public realm WSUD within catchments where there is no assigned open space. It will be for the land owners and their proponents to agree upon an approach for adoption and further design.
- The flood analysis undertaken that post development conditions will have very little impact beyond the PMP Printing boundary, with the velocity and depth of those flows not posing a risk for the community.
- Under a climate change scenario, while there was a modest increase in depth, the scenario remained below what is considered hazardous.

8 References

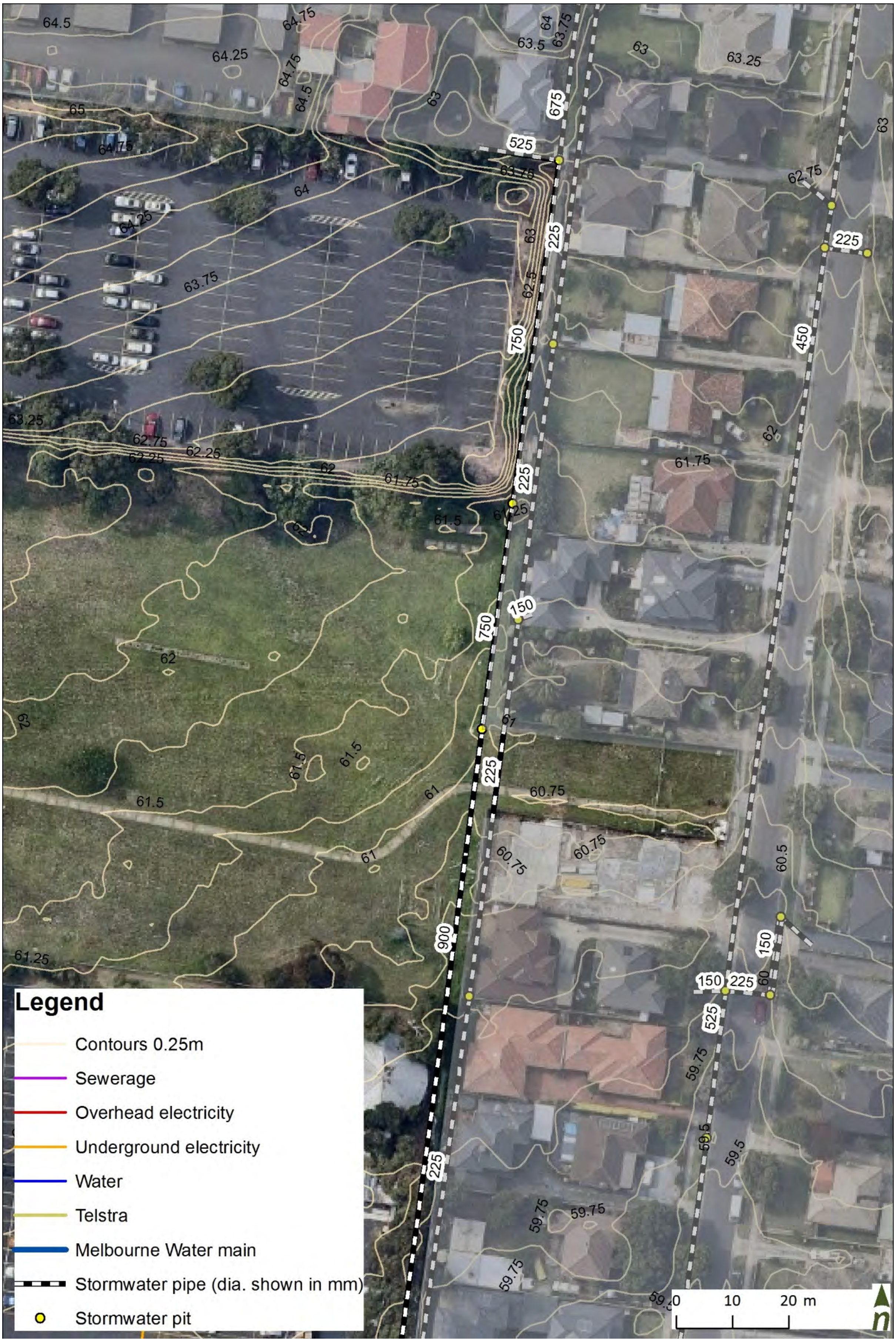
City of Monash. Monash Planning Scheme, Stormwater Management Policy.

CSIRO (1999), Urban stormwater best practice environmental management guidelines

Melbourne Water (2016), MUSIC modelling guidelines

David Lock Associates, 2016. 29 Browns Road, Clayton Development Plan.

Appendix A
Stormwater drainage asset maps


Legend
Contours 0.25m
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Overhead electricity
Underground electricity
Water
Telstra
Melbourne Water main
Stormwater pipe (dia. shown in mm)
Stormwater pit
0
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Legend
Contours 0.25m
Sewerage
Overhead electricity
Underground electricity
Water
Telstra
Melbourne Water main
Stormwater pipe
Stormwater pit
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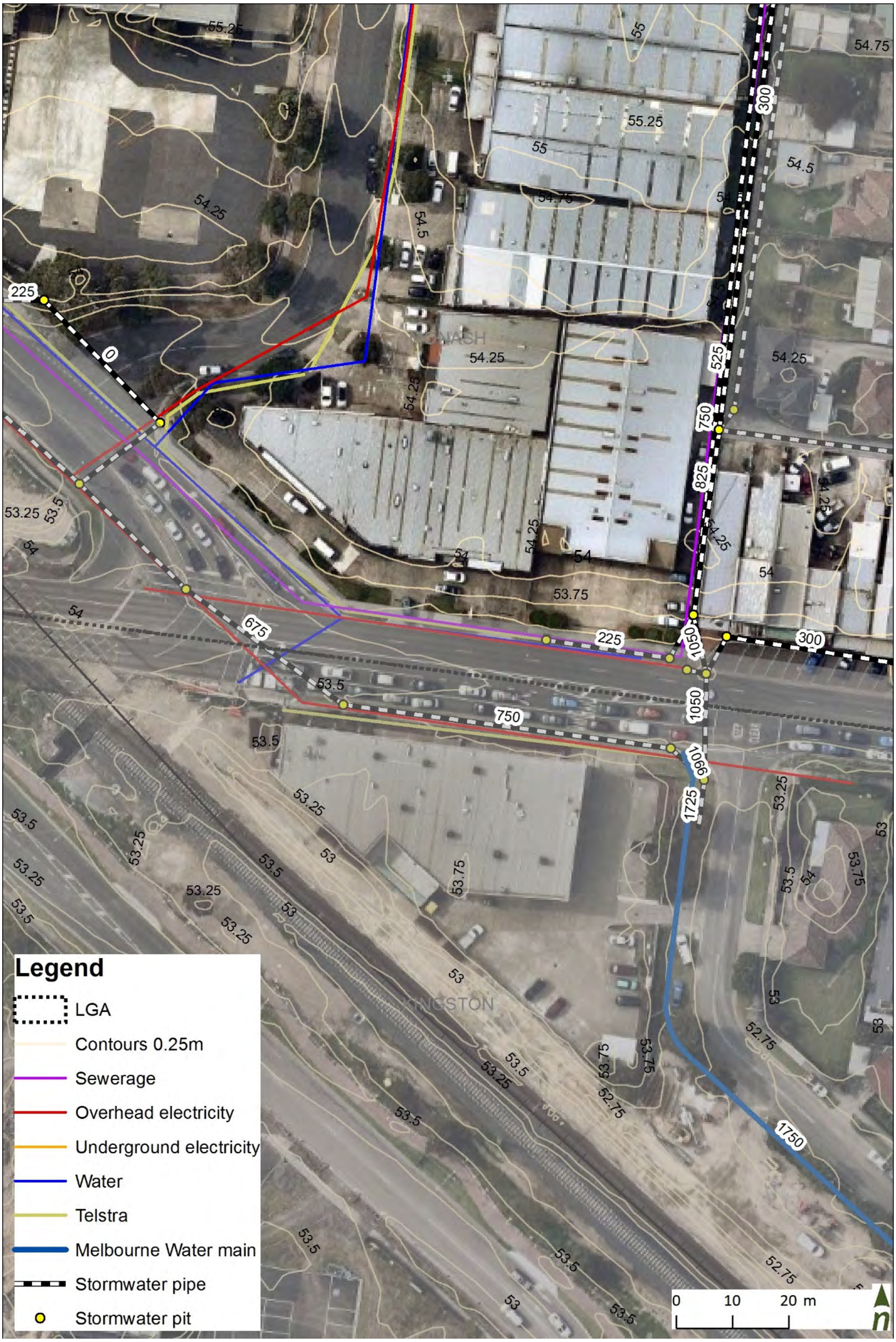
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Melbourne Water main
Stormwater pipe
Stormwater pit
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NASH
KINGSTON

Appendix B
WSUD asset area and cost detail

Table B1. *Scenario results*

Scenario	Catchment	Asset	Area	Pollution reduction performance (% removal)			
				TSS	TP	TN	GP
1	1	1 Bioretention	100 m²	80.2%	56.1%	45.7%	100.0%
	2	1 Bioretention	80 m²	79.7%	56.0%	44.4%	100.0%
	3	1 Bioretention	50 m²	79.6%	55.9%	44.7%	100.0%
	4	1 Bioretention	40 m²	79.6%	56.1%	43.8%	100.0%
				79.8%	**56.0%**	**44.6%**	**100.0%**
2	1	1 Bioretention	345 m²	96.3%	68.9%	71.8%	100.0%
	2	1 Bioretention	40 m²	64.9%	44.8%	29.2%	100.0%
	3	1 Bioretention	144 m²	94.5%	67.6%	68.2%	100.0%
	4	1 Bioretention	15 m²	47.6%	32.5%	17.0%	100.0%
				79.3%	**56.2%**	**50.6%**	**100.0%**
3	1	1 Bioretention	345 m²	96.3%	68.9%	71.8%	100.0%
	2	Tanks*	80 KL	68.1%	35.6%	28.8%	86.6%
		1 GPT	-				
		10 raingardens	10 m²				
	3	1 Bioretention	144 m²	94.5%	67.6%	68.2%	100.0%
	4	1 GPT	-	50.0%	0.0%	0.0%	70.0%
				80.9%	**48.8%**	**48.3%**	**91.9%**

*80 KL is combined tank storage (30 × 1KL tanks in medium density residential and 10 × 5KL in high density residential)

Table B2. *Construction and annual maintenance cost estimate*

Scenario	Catchment	Asset type	Total size	Construction rate ($)	Construction cost ($)	Maintenance rate ($/m²/ yr)	Maintenance cost ($/yr)
1	1	Bioretention	100 m²	$1,000	$100,000	$5	$500
	2	Bioretention	80 m²	$1,000	$80,000	$5	$400
	3	Bioretention	50 m²	$1,000	$50,000	$5	$250
	4	Bioretention	40 m²	$1,000	$40,000	$5	$200
				Total	**$270,000**		**$1,350**
2	1	Bioretention	345 m²	$600	$207,000	$5	$1,725
	2	Bioretention	40 m²	$1,000	$40,000	$5	$200
	3	Bioretention	144 m²	$800	$115,200	$5	$720
	4	Bioretention	15 m²	$1,000	$15,000	$5	$75
				Total	**$377,200**		**$2,720**
3	1	Bioretention	345 m²	$600	$207,000	$5	$1,725
	2	Tank*	80 KL		$30,000		$3,000
		GPT#	1	$75,000	$75,000	$2,000	$2,000
		Raingarden	10 m²	$5,000	$50,000	$30	$300
	3	Bioretention	144 m²	$800	$115,200	$5	$720
	4	GPT	1	$75,000	$75,000	$2,000	$2,000
				Total	**$552,200**		**$9,745**

* Tank construction cost is supply only and does not include plumbing cost.

GPT maintenance rate is based on two clean-outs per year each costing $ 1,000

Appendix C
Hydrological modelling detail

Modelling inputs

Rainfall inputs in the form of Bureau of Meteorology (BoM) 2016 Intensity Frequency Durations (IFDs) were used for all flood estimation, with temporal patterns and aerial reduction factors from ARR 2016.

The runoff coefficient model (RoC) model was used. An initial storm loss of 15mm and a runoff coefficient of 0.6 were adopted.

The catchment routing parameter Kc was determined by calibrating the flood estimates under different kc scenarios against urban rational method flow calculations. A Monte-Carlo simulation was used to determine peak design flows based on varying temporal patterns. This is a much more robust process than a simple deterministic estimate as it reduces uncertainty associated with antecedent catchment conditions and provides a more realistic simulation of actual rainfall events.

Table 9 summarises the adopted RORB model parameters.

Table 9. Adopted RORB model parameters

Parameter	Value
Kc	1
m	0.8
IL*	15 mm
RoC	0.6

The RORB model structure is shown below.

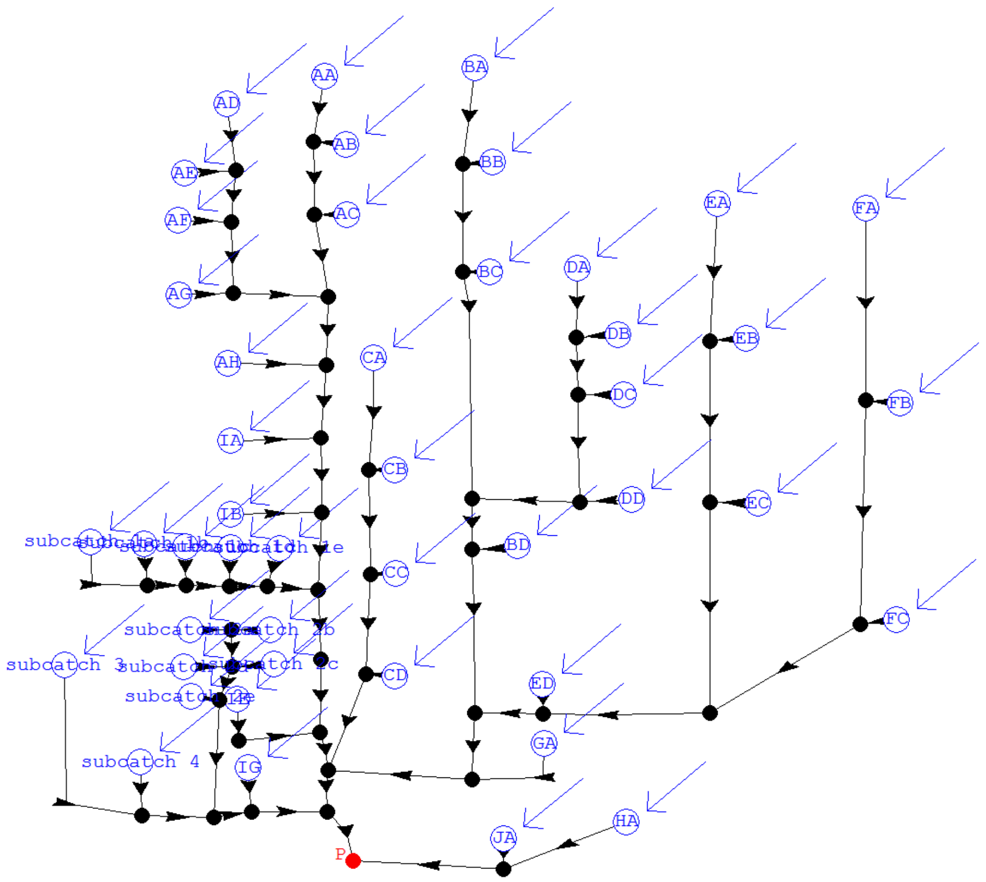

Figure 18. *RORB model structure*

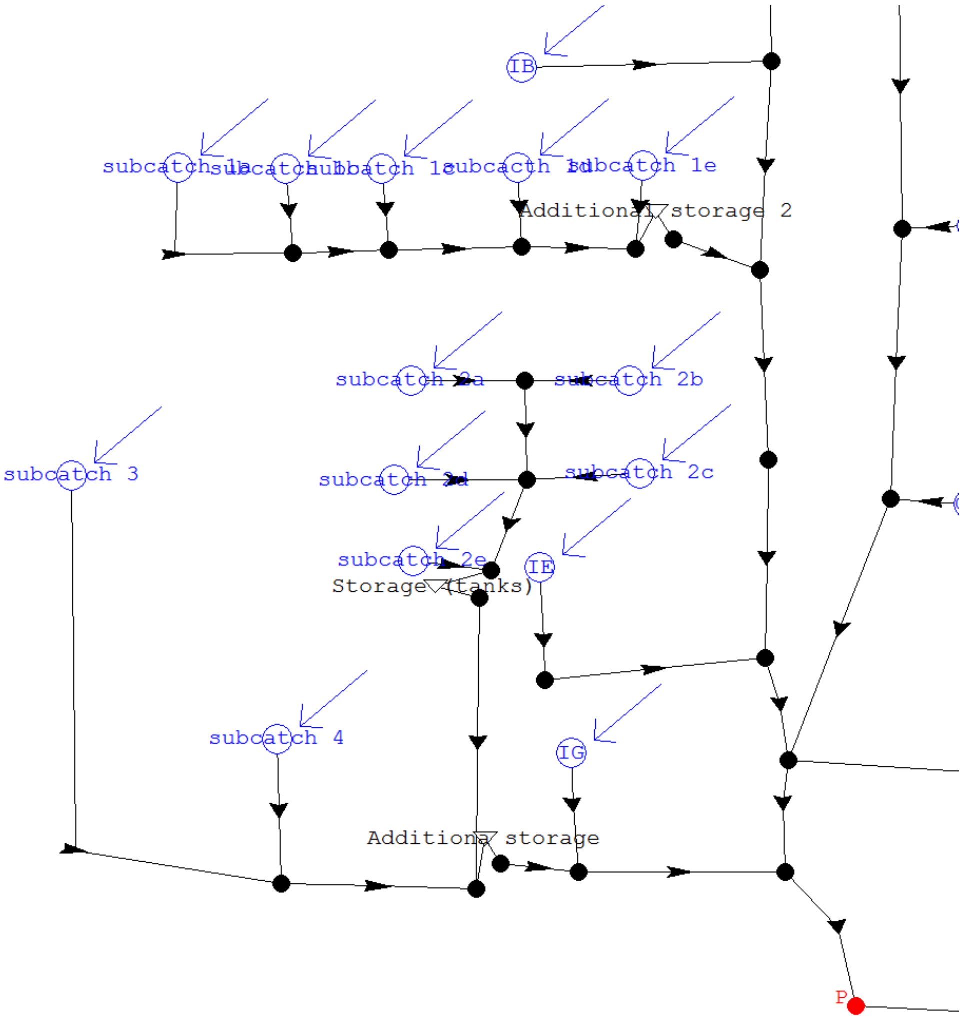

Figure 19. *Example of developed conditions RORB model showing storage options within the PMP site (zoomed in)*

Appendix D
Flood summary report (Water Technology)

Summary Report

PMP Printing Site – Hydraulic Modelling

Victorian Planning Authority

07 February 2018

Document Status

Version	Doc type	Reviewed by	Approved by	Date issued
V01	Memo	LJC	LJC	07/02/17

Project Details

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5285-01_R01v02b

07 February 2018

Dear Dan,

PMP Printing Site – Hydraulic Modelling

This brief memo describes work completed for the PMP Printing Site to assess the hydraulic impacts of proposed development and determine the 1% AEP flood regime under existing and proposed development conditions.

Please don't hesitate to contact Luke Cunningham or myself if you have any questions.

Yours sincerely

B. Esfandiar

Bahman Esfandiar
Project Engineer
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WATER TECHNOLOGY PTY LTD

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1 HYDRAULIC MODEL

1.1 Key Modelling Information Summary

A 1 m grid 2-Dimensional TUFLOW model was built to assess the flood impact of the proposed works. This model used the 2012 LiDAR dataset as the base layer.

Manning's n values were as follows:

- Residential areas surrounding the subject site - 0.35
- Commercial building- 0.3
- Car park/pavement/wide driveways/roads - 0.02

In developed conditions, the proposed layout of the buildings and surrounding area was used to alter the material roughness and represent the development. The developed layout is presented in Figure 1-1. The model was run for the 1% AEP 2-hour storm event which was identified as the critical duration. The hydraulic model directly applied the rainfall onto the 2D domain and then routed the flow both overland across the 2D domain. HQ boundaries were used around the outer boundary of each model to convey flow out of the model in a steady manner. These boundaries were set to a blanket value slope of 1% based on average terrain slopes. A 2d_rf rainfall file was produced for the 1% AEP event which consisted of rainfall polygons.

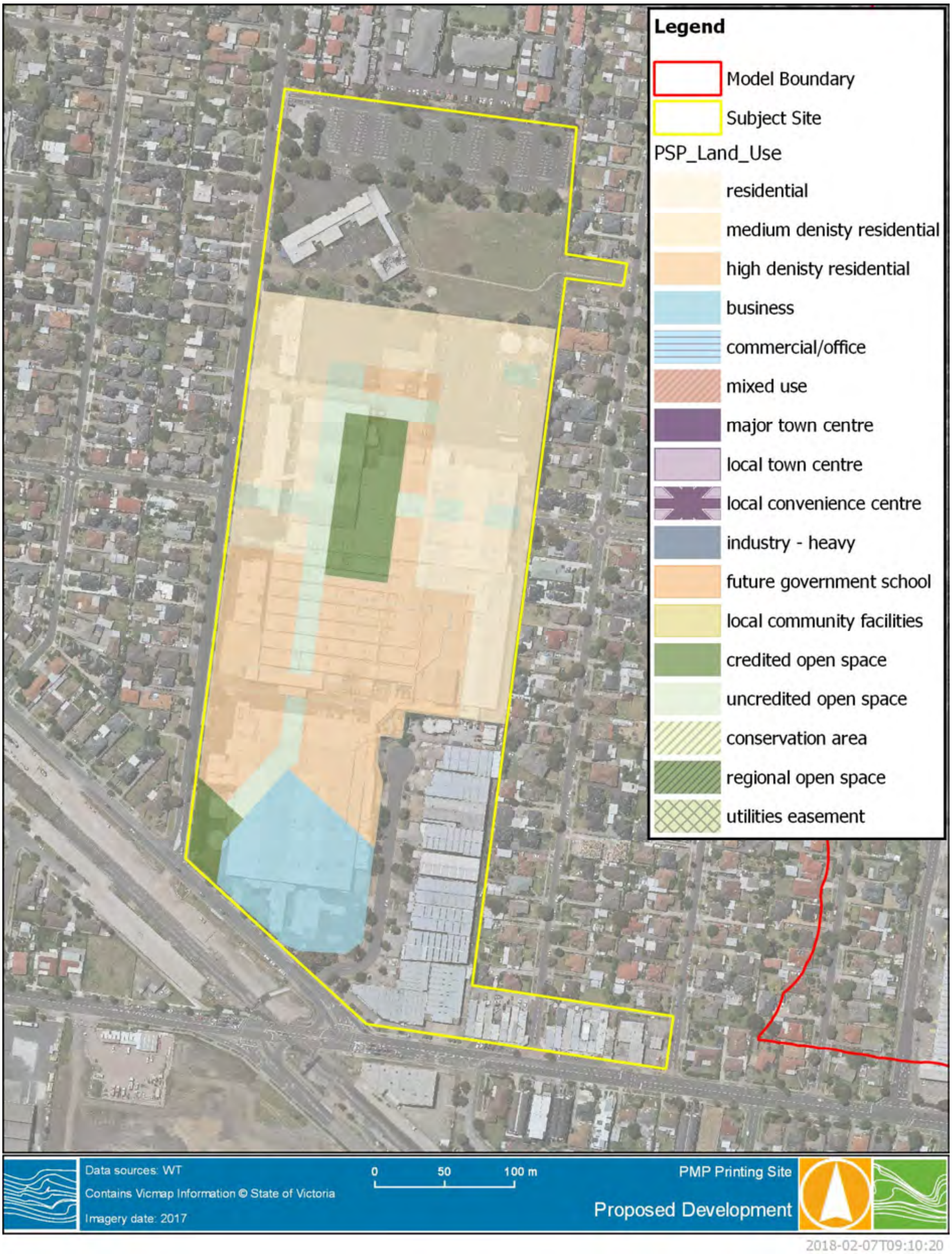

FIGURE 1-1 PROPOSED DEVELOPMENT (SUPPLIED)

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2 RESULTS

Flood depth plots for existing site conditions are shown in Figure 2-1. Figure 2-2, illustrates the 1% AEP flood depths under proposed development conditions.

In general, there is no significant changes in the flood regime. Figure 2-1 shows the main flow path in the vicinity of the subject site. During the 1% AEP flood event, a maximum of approximately 120 mm flood depth can be expected on the access road to the site while the flood water ponds at the downstream end of the site at Carinish Road. Figure 2-2, illustrates the same pattern under the proposed developed conditions. The maximum flood depth at the access road has **increased by 40 mm** to a total maximum depth of 160 mm. The increases are contained within the site and do not cause any external flooding. A flood depth of 160 mm would be considered low risk.

The Water Surface Elevation (WSE) in existing and proposed developed conditions are represented in Figure 2-3 and Figure 2-4 respectively. The difference in Water Surface Elevation (WSE) is illustrated in Figure 2-5 and Figure 2-6. The proposed development results in reduction in WSE in the areas which became less impervious under the proposed development (areas identified as open spaces). An increase of up to 40 mm is identified mainly in the access road to the site as also shown in the depth plots. Again, no adverse effect can be identified outside the boundary of the subject site.

The maximum flood velocities under the existing and proposed developed conditions are illustrated in Figure 2-7 and Figure 2-8 respectively. A maximum of 0.03 m/s increase in the velocity can be expected at the access road to the site due to the proposed development, however, there is no significant change in the velocity at the downstream of the site.

Flood safety is measured as the product of depth and velocity. Using a conservative approach of the maximum depth and velocity across the entire site (0.12 m and 0.17 m/s) results in 0.02 m^2/s. Flood waters are considered hazardous above 1.5 m^2/s. It is hence considered that the 1% AEP flooding within the site will be safe in both existing and proposed development scenarios.

A summary of the results, comparing the existing and proposed development at the access road (within the site) and Carinish Road (just outside the site) is presented in Table 2-1.

Table 2-1 Summary of Results

Results	Existing		Developed	
	Access Rd	**Carinish Rd**	**Access Rd**	**Carinish Rd**
Flood Depth (m)	0.12	0.47	0.16 ⬆	0.48 ⬆
WSE (m AHD)	56.16	54.02	56.20 ⬆	54.03 ⬆
Velocity (m/s)	0.17	0.17	0.20 ⬆	0.17 ▬

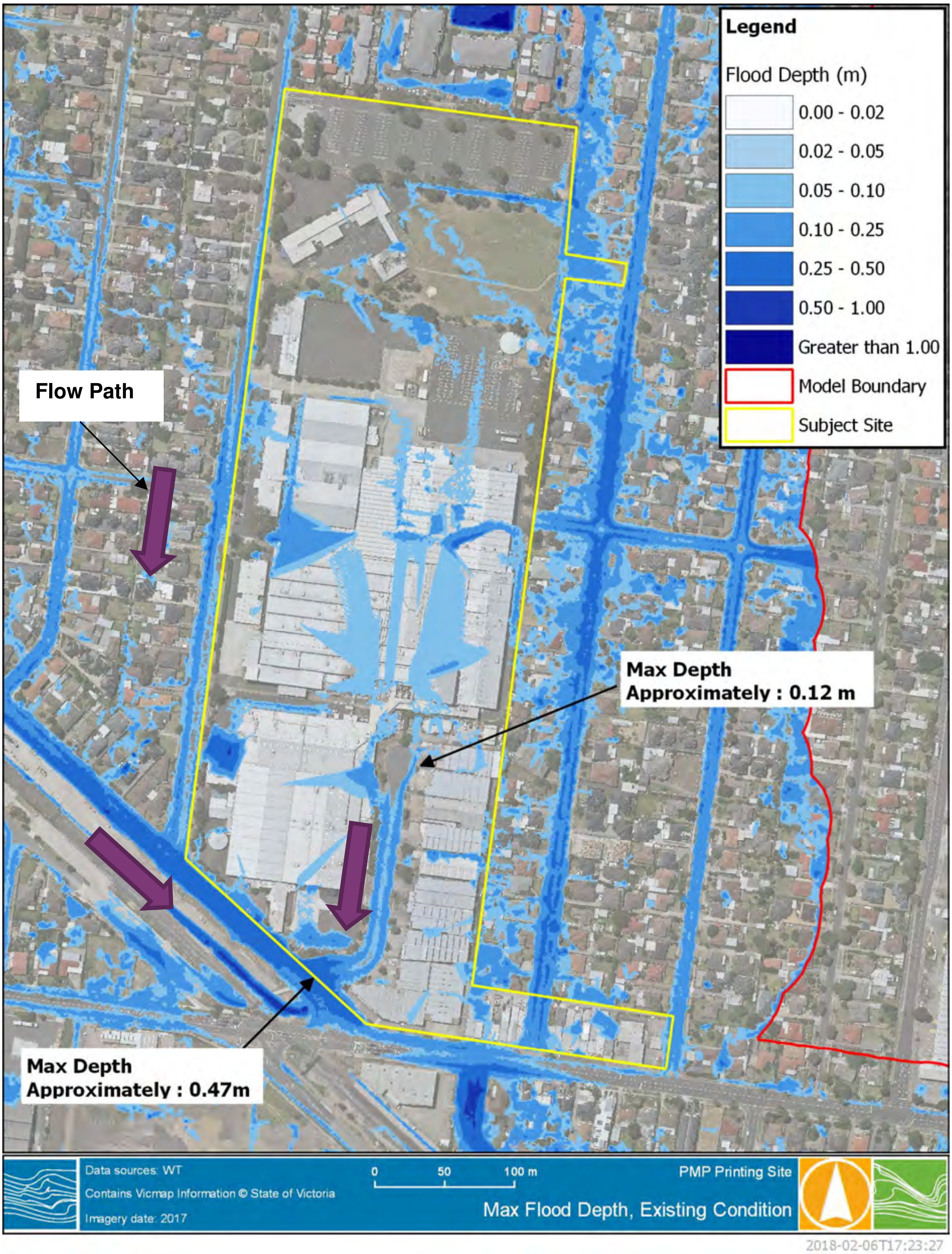

FIGURE 2-1 FLOOD DEPTH IN EXISTING CONDITIONS

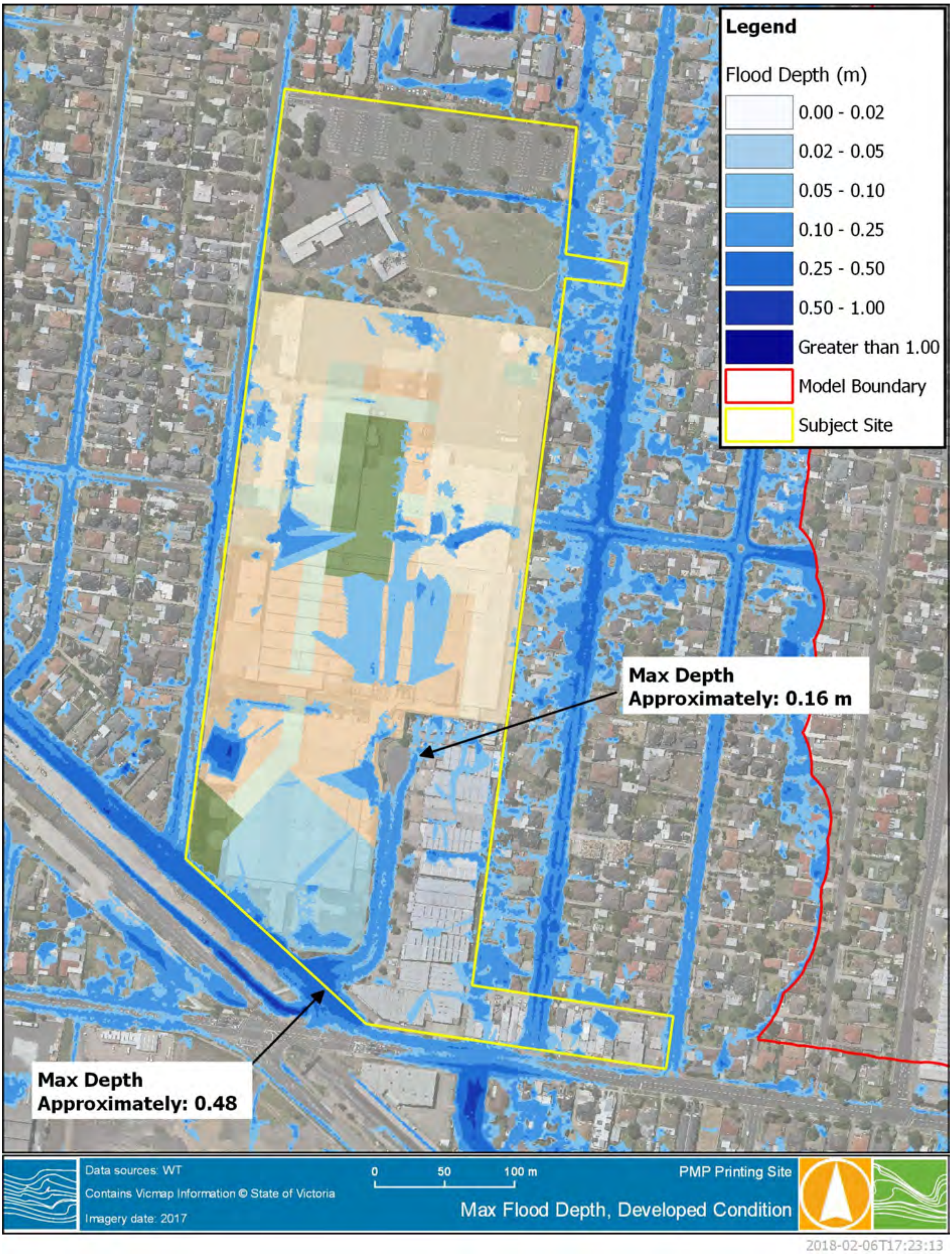

FIGURE 2-2 FLOOD DEPTH IN PROPOSED DEVELOPED CONDITIONS

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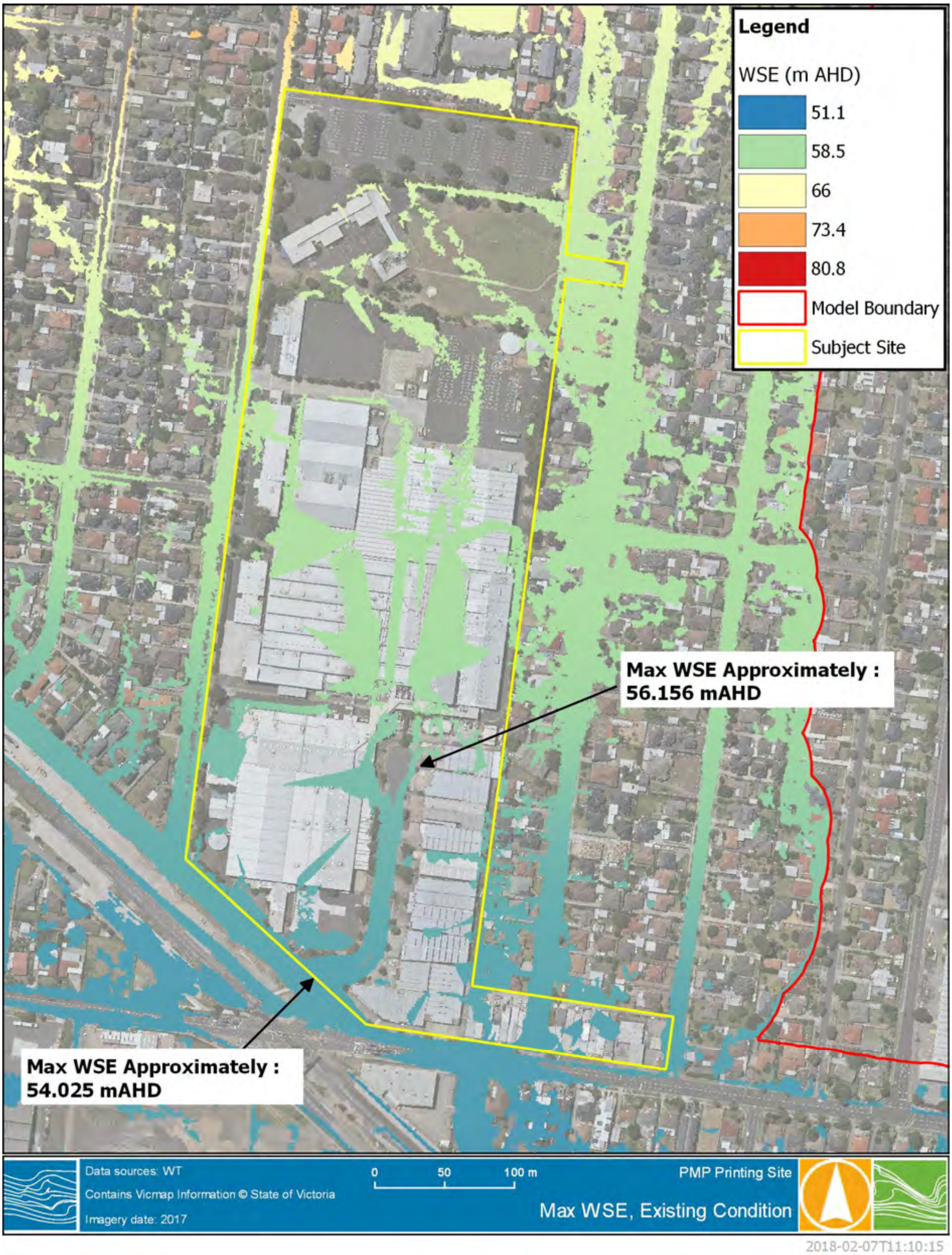

FIGURE 2-3 WSE IN EXISTING CONDITIONS

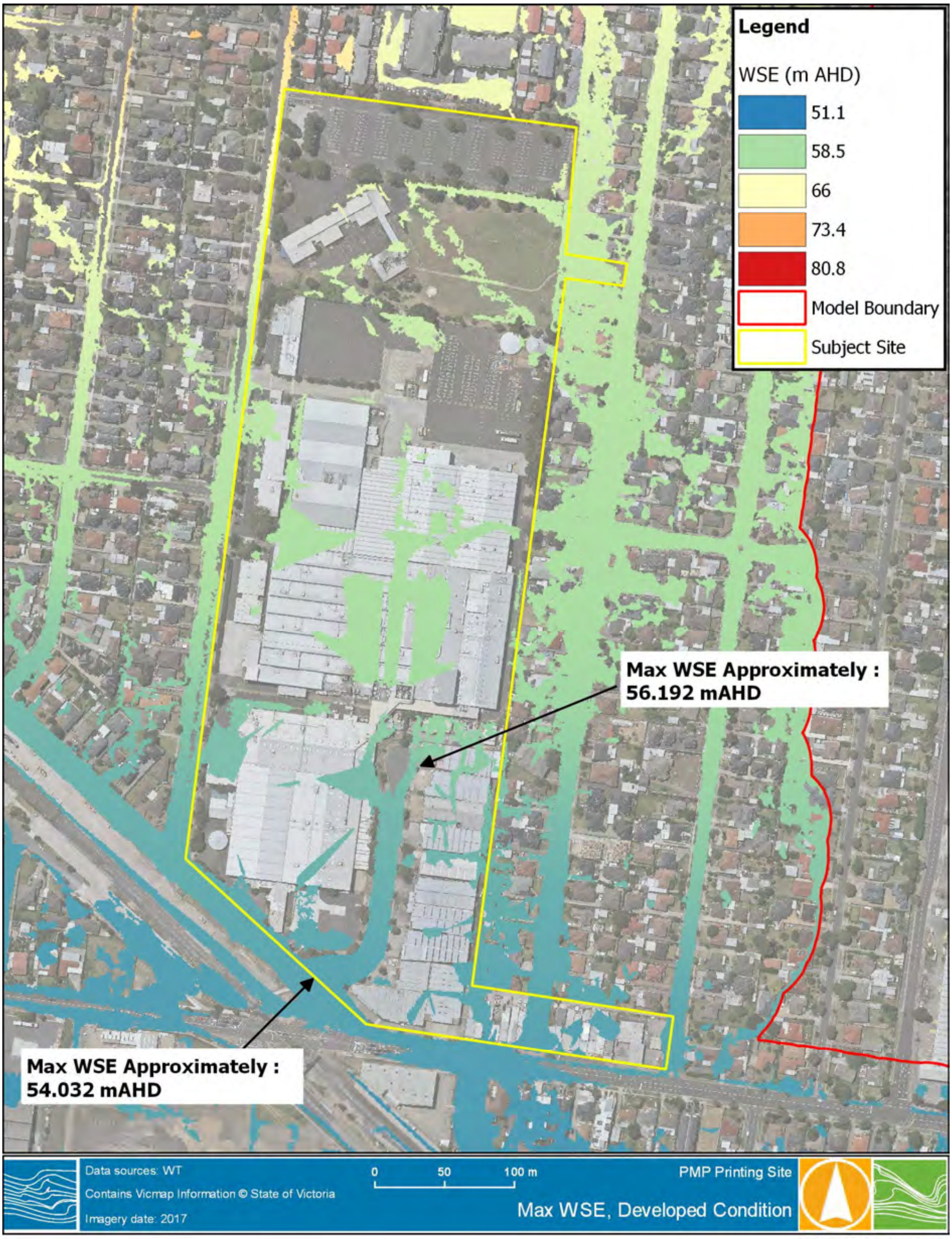

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FIGURE 2-4 WSE IN PROPOSED DEVELOPED CONDITIONS

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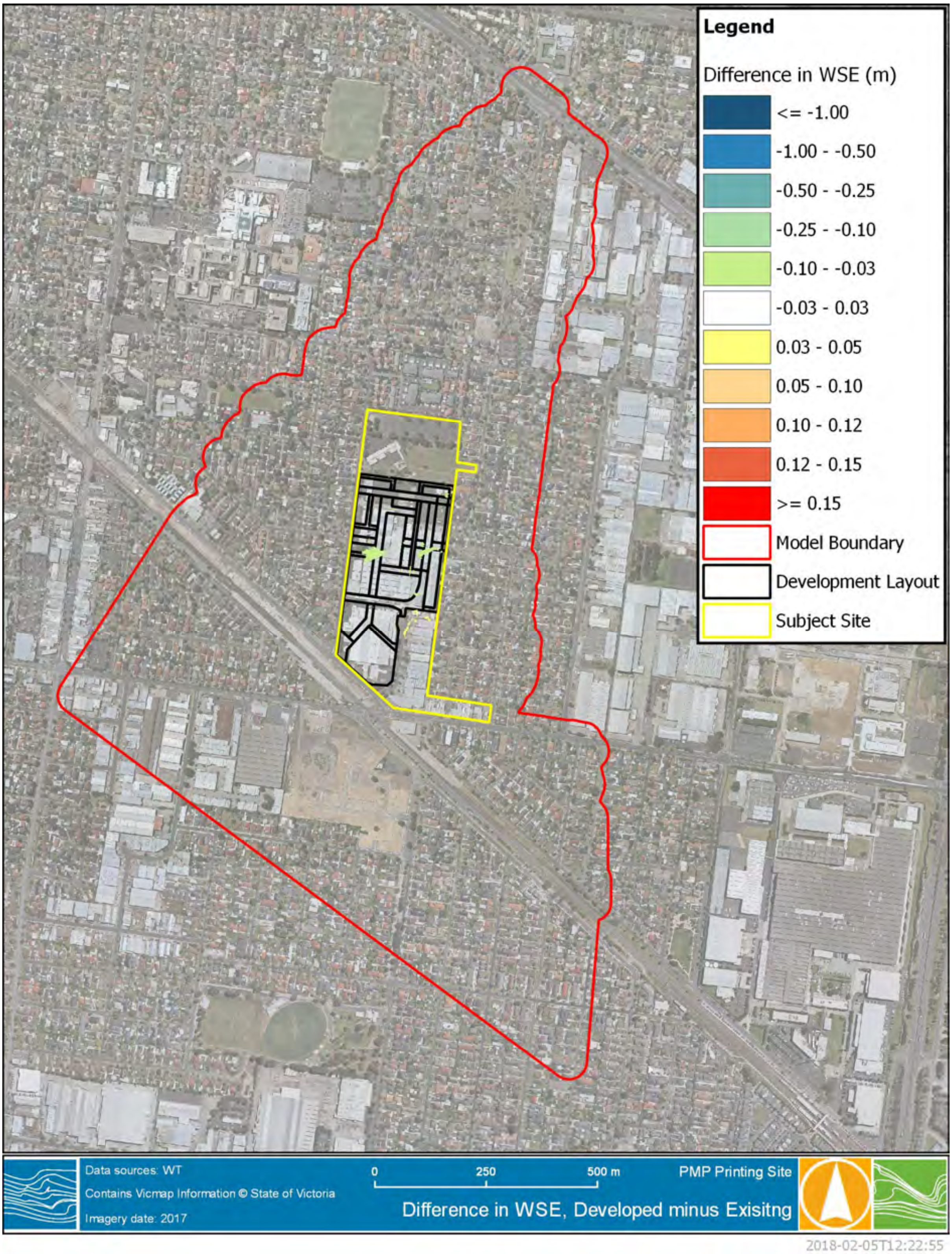

FIGURE 2-5 DIFFERENCE IN WSE, PROPOSED DEVELOPED MINUS EXISTING CONDITIONS

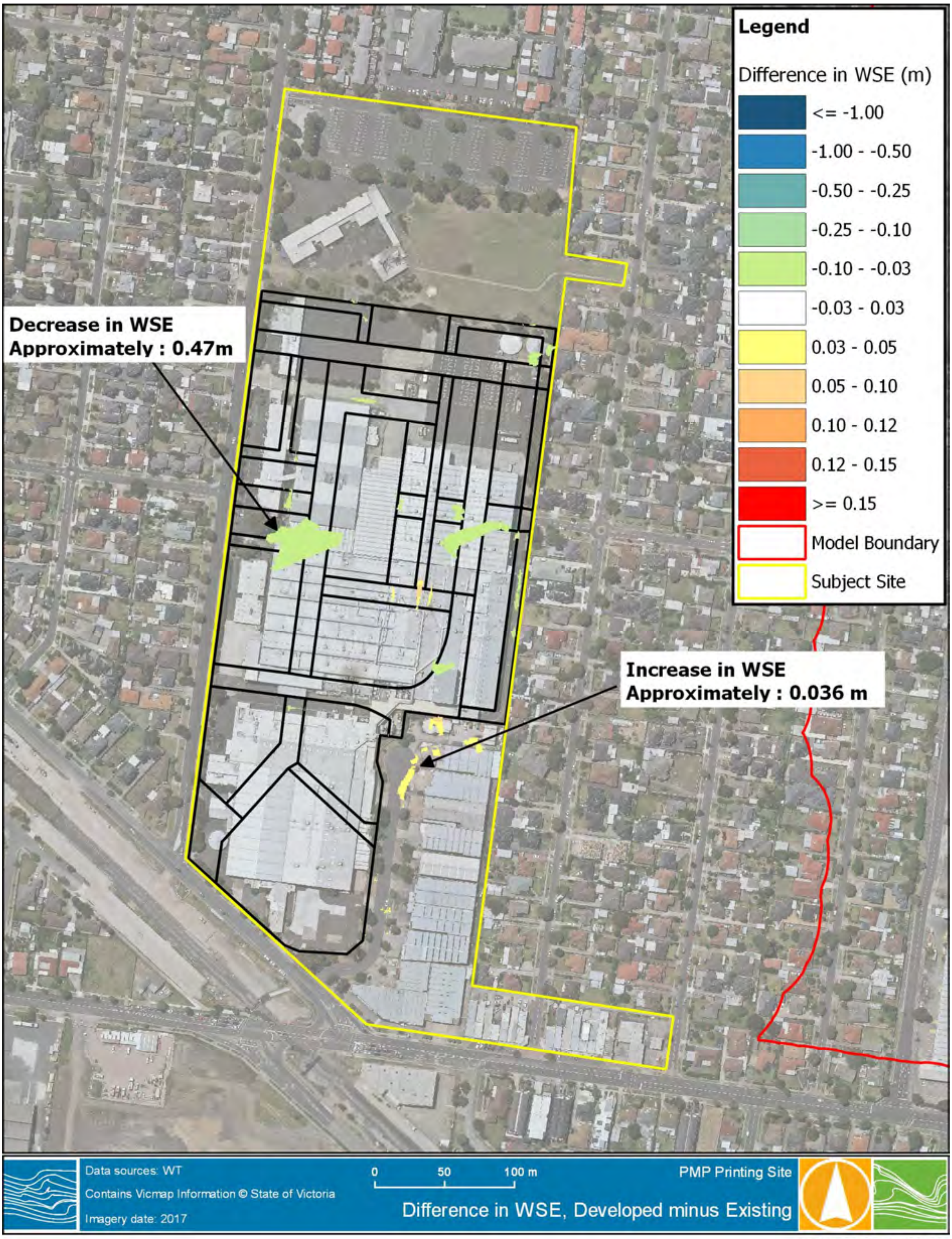

FIGURE 2-6 DIFFERENCE IN WSE, FOCUSED ON THE SITE - PROPOSED DEVELOPED MINUS EXISTING CONDITIONS

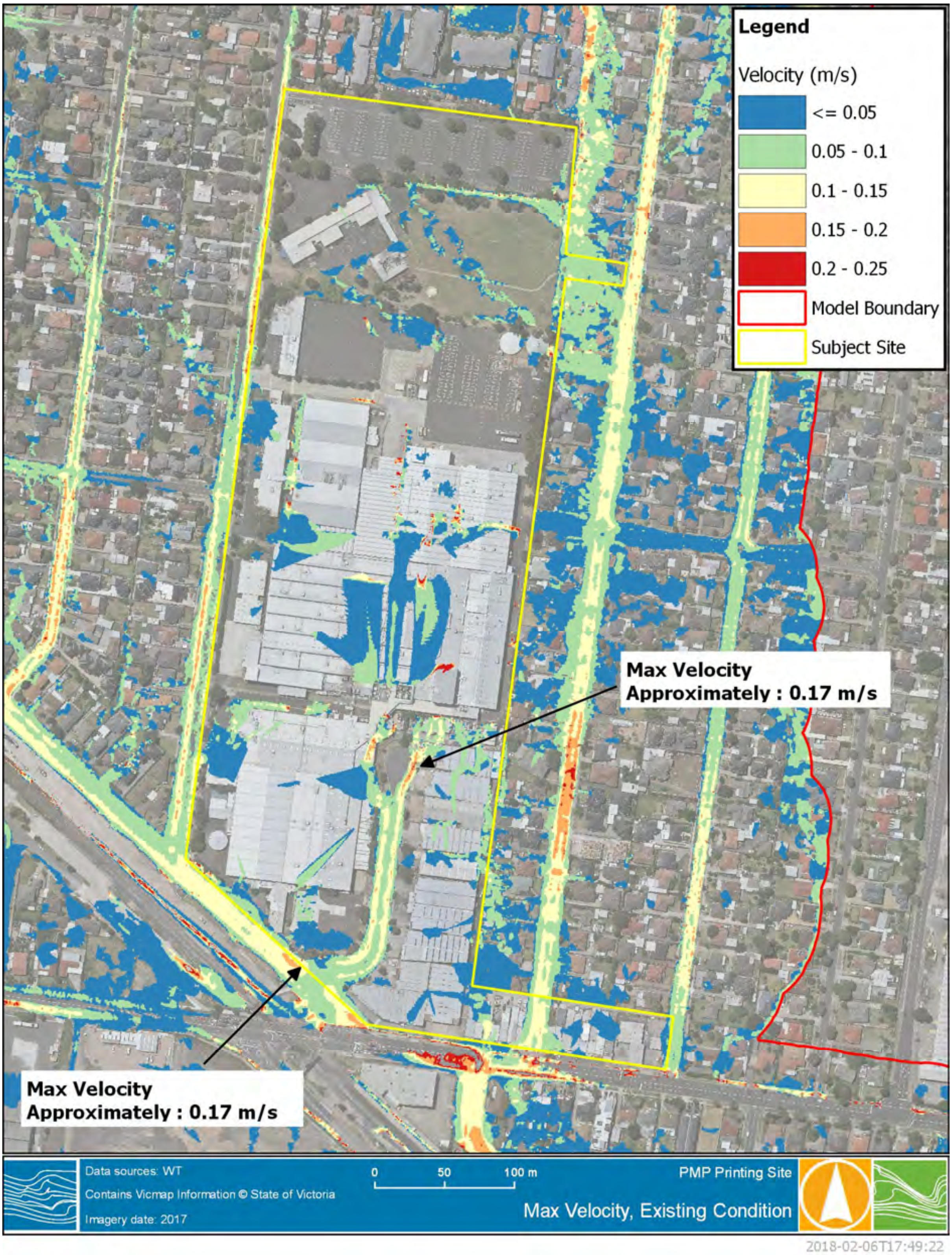

FIGURE 2-7 MAXIMUM VELOCITY IN EXISTING CONDITIONS

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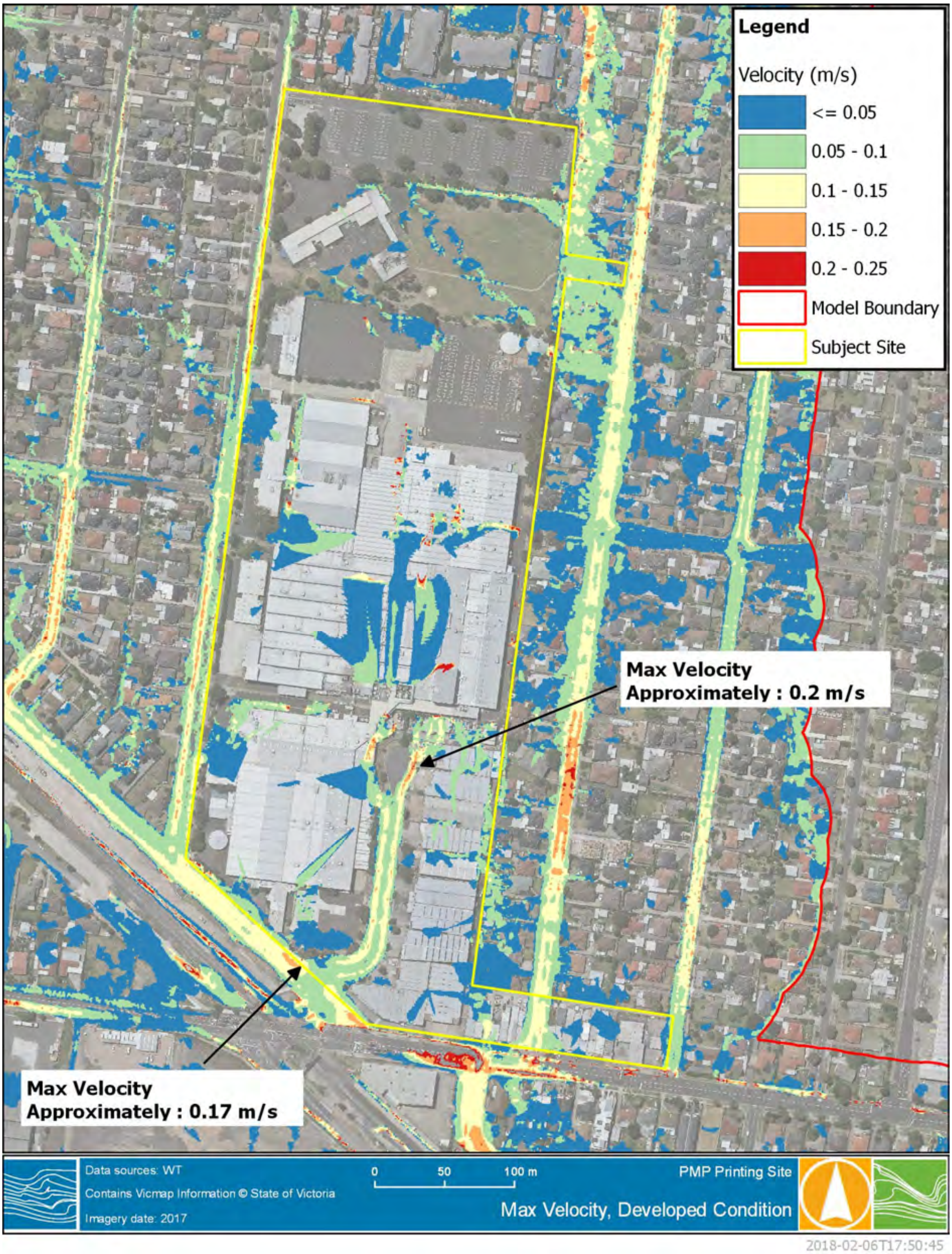

FIGURE 2-8 MAXIMUM VELOCITY IN PROPOSED DEVELOPED CONDITION

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3 CONCLUSIONS AND RECOMMENDATIONS

The hydraulic modelling completed has shown that there are **very minor** differences in flood conditions between the current site conditions and the proposed developed layout. Small increases in both flood depth and flood velocity are seen in the infrequent 1% AEP storm event, which at their maximums, **do not exceed any safety criteria thresholds**. Generally, as the impervious fraction of the site will be lowered, less runoff volume will be generated from the site also. Any additional re-use and/or harvesting will also reduce this volume.

The results show that there is no increase in flood depths or velocities outside of the subject site. It is therefore concluded that **no specific flood mitigation works would be required for the site**. Works could be completed within the site to manage nuisance flooding if desired, although the opportunity for this may be during the site drainage design. Although results do not worsen outside of the site, flooding is shown to be problematic outside of the site and may impact operations of the future site. Flooding would not be expected to pond for significant time periods with large underground drainage eventually dealing with the volumes.

The modelling uses the existing terrain of the site and any wholesale changes to the terrain as part of the development has the potential to change flow behaviour through the site and alter flood conditions. This should be kept as a consideration and modelled at a later date if relevant. The results shown are also expected to be conservative as the model does not include underground assets. Including these assets would likely reduce flood levels at the southern end of the site. If any large drainage works are considered, it would be recommended to update the model with these assets in place before commencing any further drainage design work.

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