WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

35 & 69-93 Hams Road PSA Flood Modelling

Taylors Development Services, on behalf of Waurn Ponds Unit Trust & Echin Pty Ltd

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Document Status

Version	Doc type	Reviewed by	Approved by	Date issued
01	DRAFT	Ben Tate	Ben Tate	28/03/2018
02	FINAL	Ben Hughes	Ben Hughes	11/05/2018
03	FINAL	Warwick Bishop	Warwick Bishop	30/10/2018

Project Details

Project Name	35 & 69-93 Hams Road PSA Flood Modelling
Client	Taylors Development Services, on behalf of Waurn Ponds Unit Trust & Echin Pty Ltd
Client Project Manager	Roger Cooper
Water Technology Project Manager	Lachlan Inglis
Water Technology Project Director	Ben Tate
Authors	Lachlan Inglis
Document Number	R01_V03_5581_01b.docx

PO Box 436
Geelong VIC 3220
Telephone 0458 015 664
ACN 093 377 283
ABN 60 093 377 283

Quality ISO 9001 SAI GLOBAL
Environment ISO 14001 SAI GLOBAL
Health & Safety AS 4801 SAI GLOBAL

R01_V03_5581_01b.docx

CONTENTS

LIST OF FIGURES

R01_V03_5581_01b.docx

LIST OF TABLES

1 INTRODUCTION

Water Technology was engaged to undertake flood modelling of the site at 35 & 65-93 Hams Road, Waurn Ponds. The site covers 23.7 ha and is proposed to be a residential development. A previous Stormwater Management Strategy (SWMS) was undertaken by Neil Craigie in June 2014[1]. The flood modelling undertaken is in response to a request for additional information from the City of Greater Geelong (CoGG) and the Corangamite Catchment Management Authority (CCMA). Initial modelling was provided to the City of Greater Geelong in May 2018. Feedback from CoGG identified the several changes to the overall layout were required. Discussions with the Wathaurung Aboriginal Corporation also identified the need to ensure the existing waterway alignment was maintained, whilst aiming to undertake minimal excavation to provide the required storage volume and maintain a natural "look and feel" of the waterway.

The report outlines the hydrologic and hydraulic investigation developed for the site and associated catchment and builds on the previous SWMS work undertaken in 2014.

[1] 35 Hams Road and 151-229 Anglesea Road, Waurn Ponds – Surface Water Management Strategy (v4), prepared for SMEC Urban P/L, June 2014.

2 SUBJECT SITE

The site is located at Waurn Ponds and is bound by the Geelong Ring Road (formerly Anglesea Road) to the west, Hams Road to the north, Ghazeepore Road and the Powercor Waurn Ponds Terminal Station to the east and the Warrnambool-Melbourne Railway line to the south.

The site slopes from the north-west to the south-east with external catchments draining via culverts under the Geelong Ring Road and Hams Road. The site itself is drained via two main depressions/open drains into the upper reaches of Armstrong Creek with three onstream dams located within the site.

Upstream catchments to the site include around 28 ha of residential development to the north of Hams Road and 85 ha to the west of the site (much of which is internally drained) consisting of rural residential and a quarry site.

The 28 ha of residential land to the north of Hams Road (Grange Park Estate) drains via an existing retarding basin which discharges across Hams Road into the subject site. The previous SWMS (Neil Craigie, 2014) notes the basin outlet is undersized:

> *"This basin was constructed as part of Stage 3 of Grange Park Estate and currently has a temporary 300 mm pipe outfall across Hams Road with a bubble-up pit outlet to the open drain in the subject land. As part of future drainage works within the subject land this 300 mm pipe will need to be replaced with a 600 mm diameter pipe extending across to the pit on the north side of Hams Road."*

The catchment area to the west of the site includes 39 ha of rural residential land located north of McPherson Road and 46 ha to the south of McPherson Road which has been excavated for quarrying. Based on a site inspection and a Digital Elevation Model (DEM) built from LiDAR[2] survey, there is a 750 mm culvert crossing beneath the Geelong Ring Road and into the site. A secondary box culvert (3 m x 3 m) which is used as a bike trail crossing also conveys flows from the west across to Hams Road and into the site.

A smaller catchment that includes part of the Geelong Ring Road and Baanip intersection also drains into the site via a 450 mm culvert.

The downstream water levels at the site are controlled by a 1500 mm diameter outlet culvert situated underneath the railway embankment (VicTrack asset). During the site inspection (Figure 2-2) it was noted that this culvert was filled with sediment and not likely to operate at full capacity unless cleaned out. Works to upgrade the railway are likely to be required prior to any development, this would involve a significant clean out of the culvert and the drainage path upstream and downstream of the railway embankment. The modelling undertaken for this report utilises the railway culvert with no blockage.

Downstream of the railway, a 900 mm culvert is located on Ghazeepore Road which drains to a culvert beneath Baanip Boulevard and further on into the Armstrong Creek West Precinct. The culverts identified above are shown in Figure 2-1.

[2] LiDAR (Light Detection and Ranging) is an aerial survey technique that uses a laser instrument to take accurate measurements of the ground surface over large areas. It is the predominant method for large-scale surface data capture and has typical vertical accuracies of +/- 100 mm to one standard deviation.

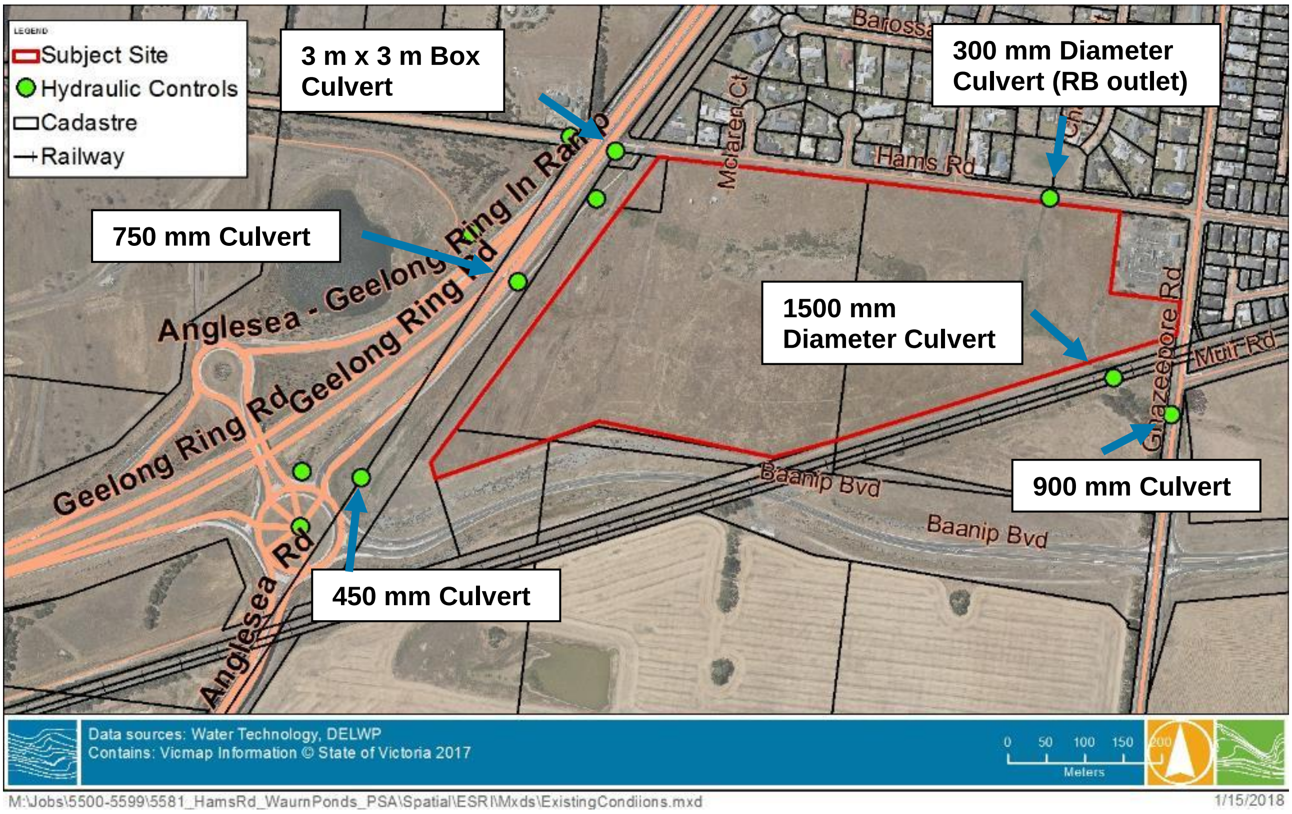


FIGURE 2-1 KEY HYDRAULIC CONTROLS INTO AND OUT OF THE SITE

FIGURE 2-2 RAILWAY CULVERT LOCATED DOWNSTREAM OF THE SITE

3 HYDROLOGY

The site drains a large area (around 136 ha) of mixed-use land including grazing, residential, rural residential and roads. An initial rainfall-runoff hydrology model was constructed in RORB and is shown in Figure 3-1.

M:\Jobs\5500-5599\5581_HamsRd_WaurnPonds_PSA\Spatial\ESRI\Mxds\RORB_Storages.mxd 1/29/2018

FIGURE 3-1 HAMS RD RORB CATCHMENT LAYOUT

The catchment was delineated based on the DEM developed from the latest LiDAR (flown in 2017) and separated into appropriate sub-catchments to allow for flow hydrograph locations to be extracted where required. The catchment delineation differs slightly from the previous RORB models used in the area for Armstrong Creek and the earlier SWMS as a result of the construction of the Geelong Ring Road and Baanip Boulevard. Furthermore, results of the updated RORB model differ from the previous RORB model due to the changes in best practice approaches since the development of the previous model. The hydrology assessment undertaken for this study used the latest guidelines from ARR 2016 as well as updated Bureau of Meteorology rainfall Intensity-Frequency-Duration (IFD) information. As a result, it is not possible to directly compare flows from the current modelling with flows developed as part of the previous investigations.

3.1 Storages

Seven “special storage” nodes were placed in the model to account for significant storage areas within the catchment. This included two large dams within the quarry to the west of the Geelong Ring Road, two retarding basins (Hams Road & Sugar Gum Drive), the railway embankment at the south of the subject site, Ghazeepore Road embankment and the Geelong Ring Road. The storage locations are shown in Figure 3-2. Stage-storage relationships were developed using LiDAR and culvert information.

R01_V03_5581_01b.docx

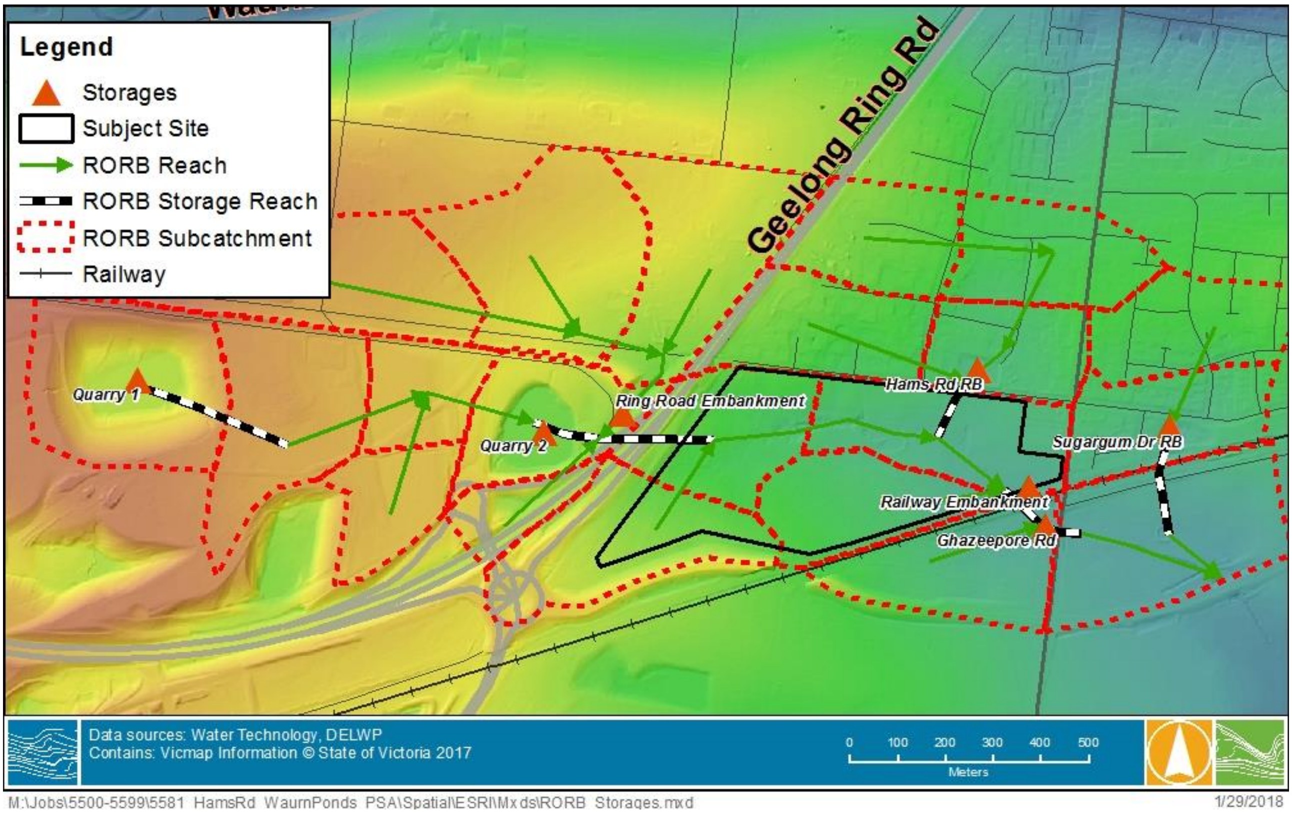


FIGURE 3-2 SPECIAL STORAGES INCLUDED IN THE HYDROLOGY MODEL

The initial hydrology model had all storages starting as empty and results showed minimal outflow from the two quarry dams. Both dams are significant storages within the quarry and it is unlikely they would spill. This was verified using rain-on-grid hydraulic modelling which showed several metres of airspace above the maximum water level modelled. Discussions with the CCMA suggested running the model again with the storages full to provide a sensitivity analysis and a conservative representation of catchment conditions for the design event. The results showed both dams overflowing in this scenario. A summary of the peak volume when the dams are empty is shown in Table 3-1, with the adopted design flows assuming the dams are full at the start of the storm. These flows are much lower than the catchment flows generated north of McPhersons Road.

TABLE 3-1 STORAGE VOLUMES FOR THE QUARRY SITE AND FLOW RATES WITH DAMS STARTING FULL

Storage	Estimated Storage Volume (m^3)	Inflow volume in 12-hour storm (m^3)	Peak Flow in (m^3/s)	Peak Flow out (m^3/s)
Quarry 1 Dam	974,000	6,620	0.66	0.15
Quarry 2 Dam	66,500	16,100	1.34	0.48

3.2 RORB Model Parameters

Rainfall depths for the catchment were determined using the Australian Rainfall and Runoff (2016) rainfall Intensity-Frequency-Duration (IFD) data from the Bureau of Meteorology. Areal Reduction Factors and temporal patterns were sourced from the ARR Data Hub. ARR datahub losses were found to be lower compared with the ARR Region equation and median loss value from the ARR loss maps (Table 3-2).

TABLE 3-2 STORM LOSSES COMPARISON TABLE

Loss Type	ARR Data Hub	ARR Region Equation	Median from ARR Map
Initial Loss	16 mm	27.5 mm	20 mm
Continuing Loss	3 mm/hr	3.1 mm/hr	4 mm/hr

The initial losses and continuing losses represented in Table 3-2 give a relatively close match between values. The differential between losses can be attributed to the regionalisation of the various approaches. The adopted the losses (ARR datahub) are shown in Table 3-3, these are lower and therefore a conservative approach was taken.

TABLE 3-3 ADOPTED LOSSES

Loss Type	Loss
Initial Loss	16 mm
Continuing Loss	3.0 mm/hr

3.2.1.1 RORB Kc and m

Kc is a RORB model routing parameter estimated using empirical equations that generally represent a wide range of fitted data for Australian catchments and dictates the attenuation along model reaches. In gauged catchments, the kc value is one of the major parameters used to calibrate the RORB model, varying peak flow and timing. In ungauged catchments (such as Hams Road), the kc can be estimated using empirical equations.

With RORB there are multiple equation-based Kc estimates available for Victoria, these are outlined in Table 3-4. The equations vary in dependence on the catchment area (A) and the average reach distance (D_{av}). Generally, those associated with the D_{av} are preferred.

TABLE 3-4 EQUATION BASED KC ESTIMATES

Description	Equation	kc estimate
Victoria (Mean Annual Rainfall <800mm)	$kc = 0.49 * A^{0.65}$	0.67
Victorian based data (Pearse et al, 2002)	$kc = 1.25 * \mathrm{D_{av}}$	1.46
Australian based data (Dyer, 1994)	$kc = 1.14 * \mathrm{D_{av}}$	1.33
Australian based data (Yu, 1989)	$kc = 0.96 * \mathrm{D_{av}}$	1.12

Sensitivity testing of the kc values was completed using the RORB Monte Carlo analysis, comparing RORB model peak flows on the model outlet (downstream of Ghazeepore Rd). The modelled peak flows and critical durations are shown in Table 3-5. These are the median flows from the Monte-Carlo analysis critical duration, and all show close agreeance on the resulting peak 1% AEP design flow.

TABLE 3-5 1% AEP PEAK FLOW AND CRITICAL DURATIONS WITH VARYING KC (FULL QUARRY PITS)

kc calculation method	Critical Duration	Peak 1% AEP Design Flow (m^3/s)
MAR	1hr	6.38
Pearce et. al.	2hr	5.62
Dyer et. al.	1hr	5.47
Yu et. al.	2hr	5.89

As a method of verification for the most appropriate kc, the ARR Regional Flood Frequency Estimation Model[3], the VicRoads Modified Rational Method, the Rational Method (using Adams Method for time of concentration), and Hydrological Recipes[4] approximation methods were used to calculate an estimated peak flow for the catchment. Peak flow estimates for the ARR RFF and VicRoads Rational Method are shown in Table 3-6 and Table 3-7. Equations 2-3 and 2-4 show peak 1% AEP design flow estimates for the Rational Method (Adams) and Hydrologic Recipes approach.

TABLE 3-6 ARR REGIONAL FLOOD FREQUENCY ESTIMATION MODEL RESULTS

AEP (%)	Discharge (m^3/s)	Lower Confidence Limit (5%) (m^3/s)	Upper Confidence Limit (95%) (m^3/s)
50	**0.45**	0.16	1.28
20	**0.84**	0.31	2.26
10	**1.17**	0.43	3.2
5	**1.56**	0.56	4.37
2	**2.15**	0.74	6.33
1	**2.67**	0.88	8.20

TABLE 3-7 VICROADS RATIONAL METHOD RESULTS

AEP (%)	Iy (mm/h)	Py	Discharge (m^3/s)
50	18.54	0.15	**1.24**
20	24.04	0.18	**1.92**
10	27.68	0.20	**2.46**
5	32.65	0.22	**3.19**
2	39.70	0.24	**4.23**
1	45.47	0.26	**5.25**

EQUATION 2-3 RATIONAL (ADAMS METHOD)

The probabilistic Rational Method utilises IFD charts to determine the peak flow of the centroid of the catchment.

[3] http://rffe.arr-software.org/ - Accessed 20/01/2018
[4] Grayson, R.B., Argent, R.M., Nathan, R.J., McMahon, T.A. and Mein, R. (1996) Hydrological Recipes: Estimation Techniques in Australian Hydrology. Cooperative Research Centre for Catchment Hydrology, Australia, p 108-125.

$$Q_{100} = (C \times I \times A)/360$$

$$Q_{100} = 2.63\ m^3/s$$

EQUATION 2-4 HYDROLOGICAL RECIPES URBAN AND RURAL ESTIMATES

This method utilises a regional equation for the calculation of the 1% AEP event in rural catchments.

$$\boldsymbol{Rural\ Catchment}:$$

$$Q_{100} = 4.67 \times area^{0.763}$$

$$Q_{100} = 4.67 \times 1.6^{0.763}$$

$$Q_{100} = 6.68\ m^3/s$$

Table 3-8 below summarises the various approaches to estimating the 1% AEP peak flow. RORB flows quoted are from Monte-Carlo assessment with the quarry basins full at the beginning of the model.

TABLE 3-8 1% AEP PEAK FLOW SUMMARY OF APPROACHES

Approximation Method	1% AEP Peak Flow (m^3/s)
RORB (MAR Kc)	6.38
RORB (Pearce et. al. Kc)	5.62
RORB (Dyer et. al. Kc)	5.47
RORB (Yu et. al. Kc)	5.89
ARR Regional FFA	8.20
VICROADS Rational Method	5.25
Rational Method (Adams)	2.63
Hydrological Recipes (Rural)	6.68

Water Technology has found the Pearce et. al. (2002) kc prediction equation works well in many Victorian catchments. Given the equation determined by Pearce et. al. (2002) is based on Victorian data and gives a 1% AEP peak design flow well within the range of values predicted by the various methods, it was adopted for this study. This flow is used as a comparison for the model outlet flow rates and in the final inputs to the hydraulic model inflow locations upstream of the subject site. This flow also matches well with the VICROADS Rational Method peak flow generated for the catchment.

3.2.2 RORB Temporal Pattern Selection

To select an appropriate temporal pattern for the 1% AEP flood event, the temporal pattern from the ensemble of ARR temporal patterns that most closely matched the Monte-Carlo peak flow was selected. Temporal pattern 25 was selected as it most closely matched the peak flow rates from the Monte Carlo analysis. The rainfall temporal pattern selected is shown in Figure 3-3. The peak flow rates for downstream of Ghazeepore Road are shown in Table 3-9.

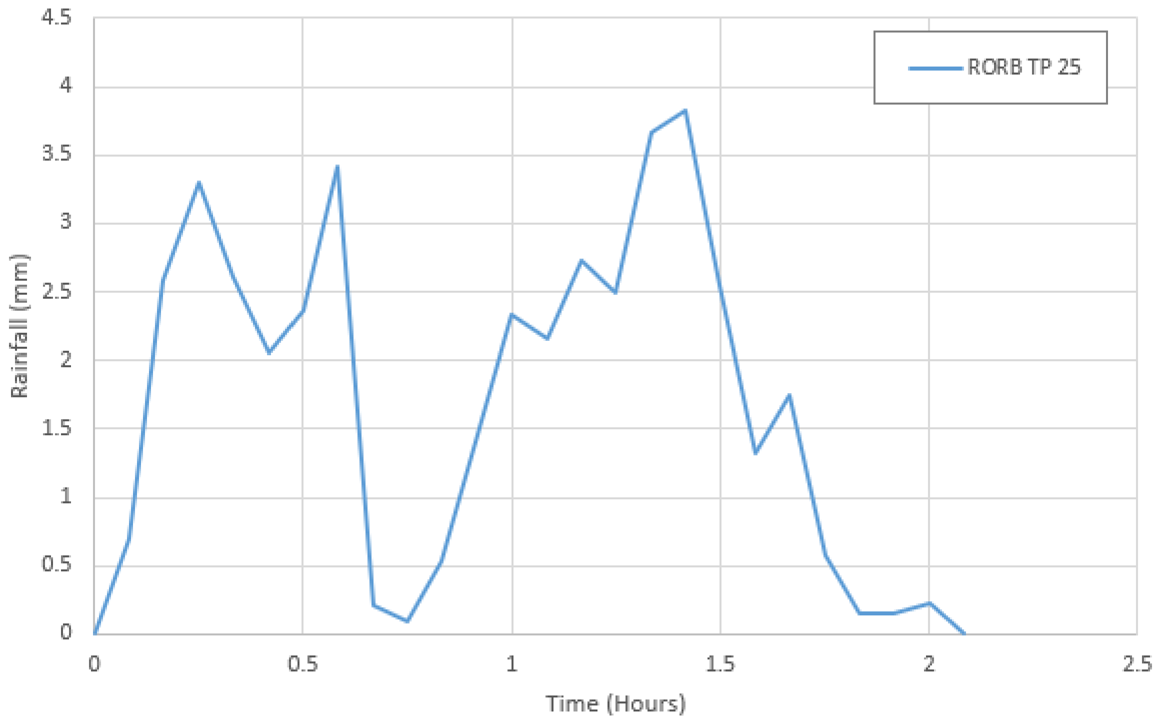


FIGURE 3-3 SELECTED TEMPORAL PATTERN FOR DESIGN MODELLING

TABLE 3-9 TEMPORAL PATTERN SELECTION FOR 1% AEP EVENTS

Model Duration	Flow Rate Downstream of Ghazeepore Road (m^3/s)
15 Minute	2.56
30 Minute	4.08
45 Minute	5.12
1 Hour	5.62
2 Hour	5.70
3 Hour	4.41
6 Hour	4.08
12 Hour	2.56

3.3 Rain-on-Grid IFD Parameters

Rain-on-grid modelling of the site was completed as well as the RORB modelling. The BOM IFD 2016 rainfall data was used and individual storm file hyetographs for each AEP, duration and temporal patterns from the "rare" temporal pattern bins were developed. These were generated for use in the TUFLOW rain-on-grid model using the "ARR_to_TUFLOW" python script. The same temporal pattern as adopted in the RORB modelling was used in the rain-on-grid modelling.

The rainfall was then applied to each grid cell within the "Rain on Grid" area shown in Figure 3-4 and losses applied to each cell based on an initial loss and continuing loss applicable to the land use type and loss values used in the RORB modelling. The initial loss and continuing loss values applied to each area are shown in Figure 3-5 and Figure 3-6 respectively. Areas upstream of the rain-on-grid model area used inflows generated from the RORB modelling as a conservative approach.

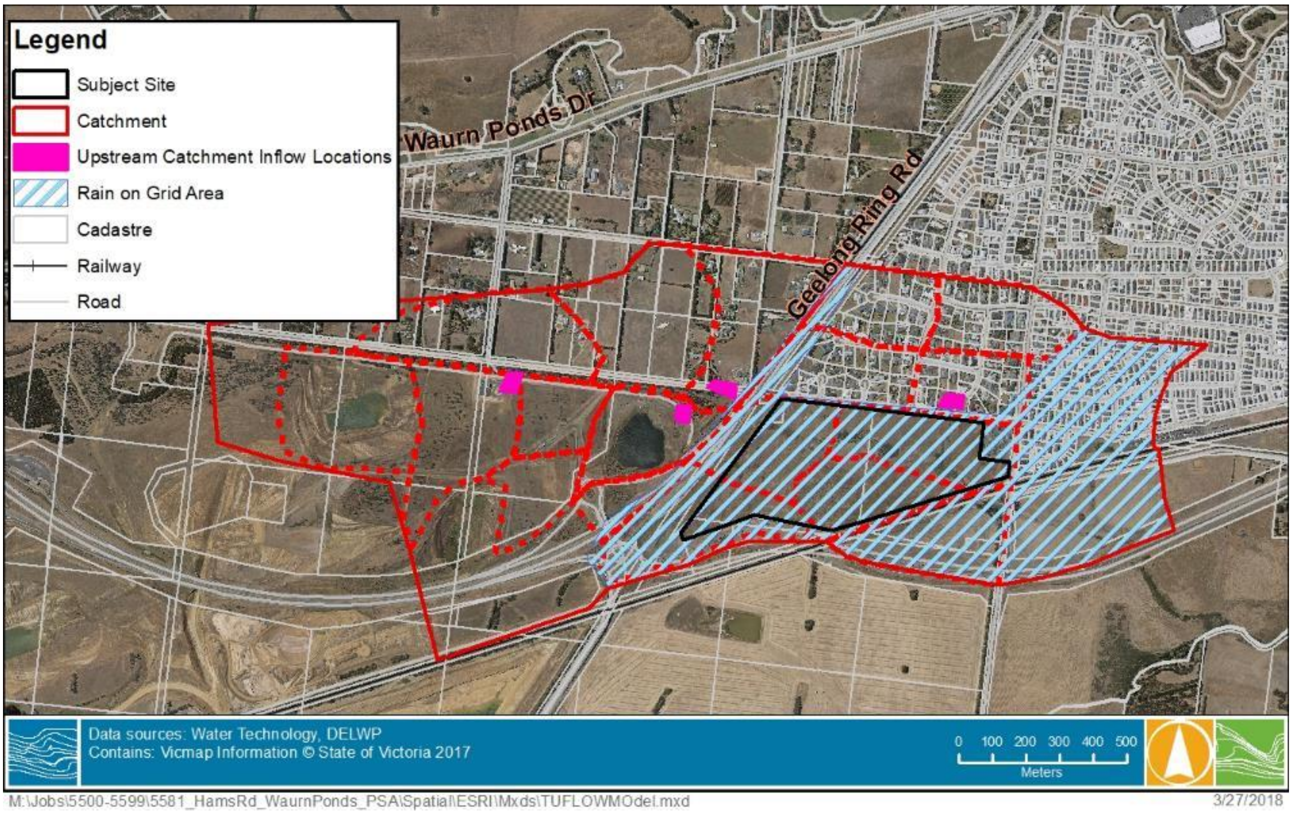


FIGURE 3-4 RAIN ON GRID AREA AND UPSTREAM CATCHMENT INFLOW LOCATIONS

To generate the impacts of runoff on urban areas, the following initial loss and continuing loss values (summarised in Table 3-10) were adopted. This used the ARR datahub loss values as a starting point.

TABLE 3-10 RAINFALL ON GRID LOSS VALUES ADOPTED

Area	Initial Loss (mm)	Continuing Loss (mm/hr)	Notes
Open Space	16	3	ARR Loss Values
Retarding Basins	16	3	ARR Loss Values
Unsealed Road	10	1.2	Includes road reserve
Sealed Road	5	0.6	Includes road reserve (FI typically 0.7-0.8)
Rural Living Zone	16	3	ARR Loss Values
Urban Growth Zone	16	3	ARR Loss Values
Waterway/Dam	0	0	Water body (no loss applied)

R01_V03_5581_01b.docx

FIGURE 3-5 INITIAL LOSS VALUES USED IN THE HYDRAULIC MODEL (DEVELOPED CONDITIONS)

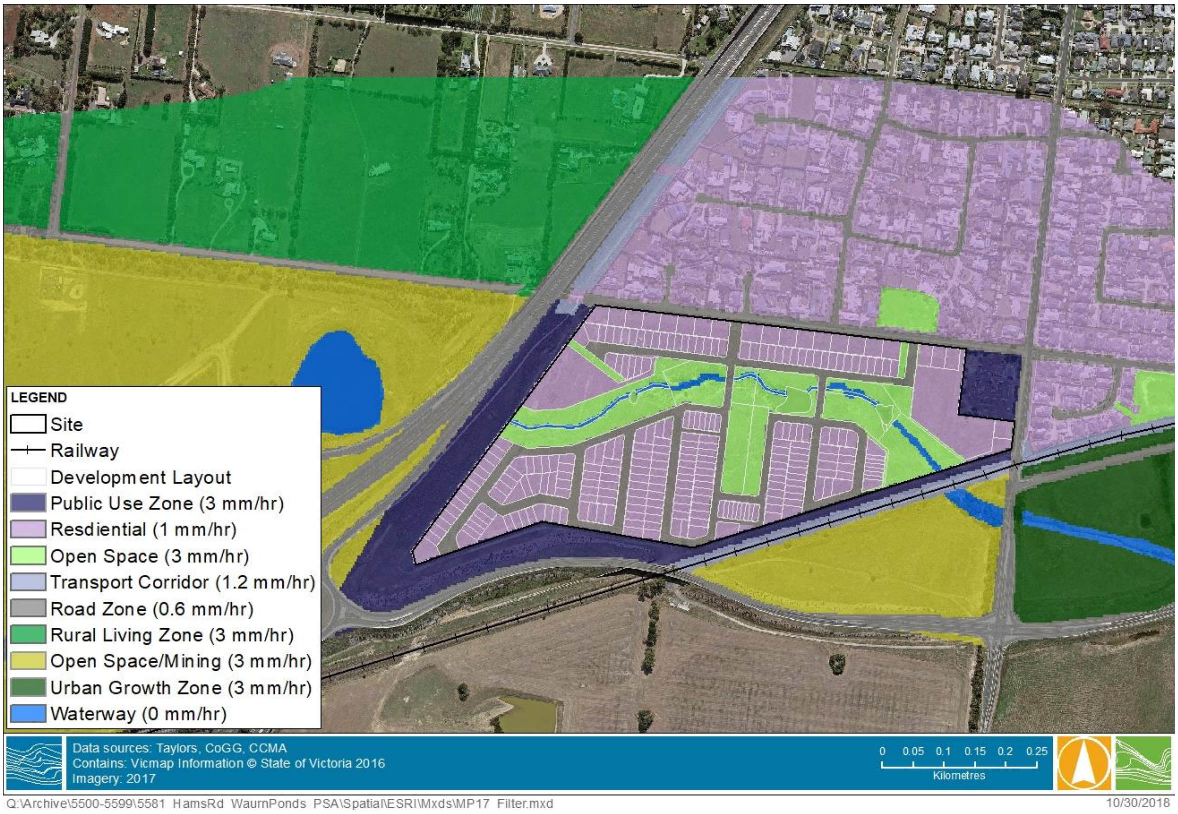


FIGURE 3-6 CONTINUING LOSS VALUES USED IN THE HYDRAULIC MODEL (DEVELOPED CONDITIONS)

4 HYDRAULIC MODELLING

4.1 Methodology

To ensure all flows are accounted for across the site, a joint source-point-inflow and rain-on-grid methodology has been used to ensure the impact of the proposed development can be assessed. This approach adopted flows developed from the RORB modelling for the upstream catchments of the site given drainage infrastructure (pit and pipe) information is limited. Across the site, rain-on-grid was used to get the direct impact of increased impervious areas as a result of the proposed development. This adopts a conservative approach for flows across and into the site.

4.2 Rain-on-Grid Verification

The flows at critical locations were compared across all hydrological methods trialled in this project. The results are shown in Table 4-1, this shows the adopted approach (combined TUFLOW/RORB inflows) is generally conservative, the adopted flows for Hams Road are based on the RORB model which ensures the residential catchment flows make their way to the existing retarding basin, whereas the rain-on-grid modelling which does not include the pits and pipes in the upstream catchment does not deliver all flow to the basin.

TABLE 4-1 TUFLOW FLOW POINT COMPARISON 1% AEP EVENT (MAX DURATION)

	RORB Model (empty dam) (m^3/s)	RORB Model (full dam) (m^3/s)	TUFLOW Rain on Grid (m^3/s)	TUFLOW Source Point (m^3/s)	Combined TUFLOW/RORB Source Points (m^3/s)
Ring Road Culvert #1 (750 mm culvert)	0.46	0.48	0.76	0.87	0.93
Geelong Ring Road Culvert #2 (McPhersons Rd)	3.00	3.54	0.98	0.80	4.84
Hams Road (Piped)	2.45 (pipe and overland)	2.45 (pipe and overland)	0.14	0.40	0.21
Hams Road (Overland)			0.49	1.24	2.62
DS Railway Culvert	5.49	5.64	3.25	2.89	4.02
DS Ghazeepore Rd	5.55	5.70	3.58	3.10	4.21

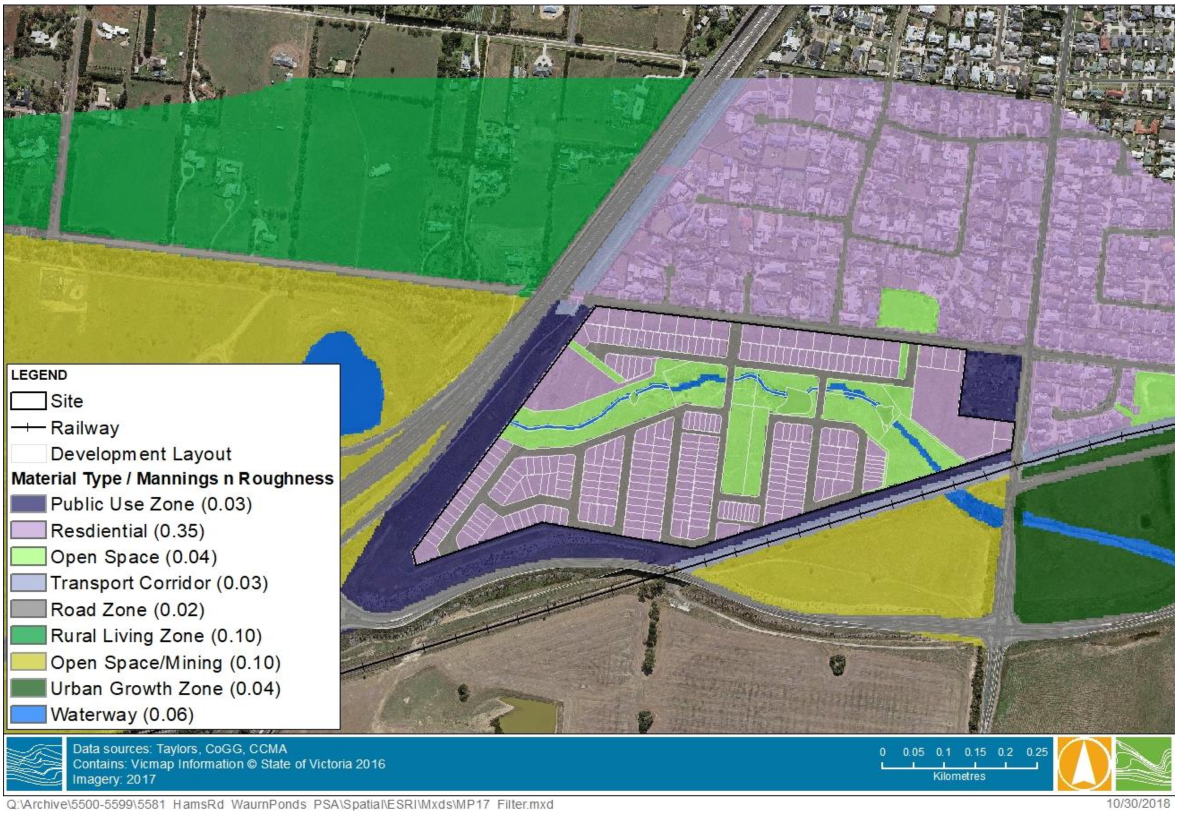


FIGURE 4-1 HYDRAULIC MODEL MANNINGS N ROUGHNESS VALUES (DEVELOPED CONDITIONS)

5 RESULTS

The TUFLOW hydraulic model was simulated for the 1% AEP design flood for durations between 30 minutes and 6 hours. The results for each duration were then combined to produce the maximum flood depth and velocity envelope across the site.

5.1 Existing Conditions

The filtered maximum results show two main flow paths that join in the west of the site (Figure 5-1). The results shown are filtered, with depths less than 2 cm and puddles less than 100 m^2 not shown. The northern flow path results from flows upstream of the Princes Freeway out falling into the site via two culverts. The southern branch flow path conveys flows from within the site along a drainage path. The flow path then travels through two small ponds and a small farm dam within the centre of the site. The existing retarding basin north of Hams Road is overtopped in the 1% AEP event with flow travelling over Hams Road into the site at depths up to 0.2 m. Flows accumulate and pond against the railway embankment to a depth of just over 2 m. Ghazeepore Road is also overtopped in a 1% AEP event at depths up to 0.2 m. Maximum velocities in a 1% AEP event were found to be around 1.8 m/s located at the outfall of the existing pond in the east of the site as well as water overtopping Ghazeepore Road as shown in Figure 5-2.

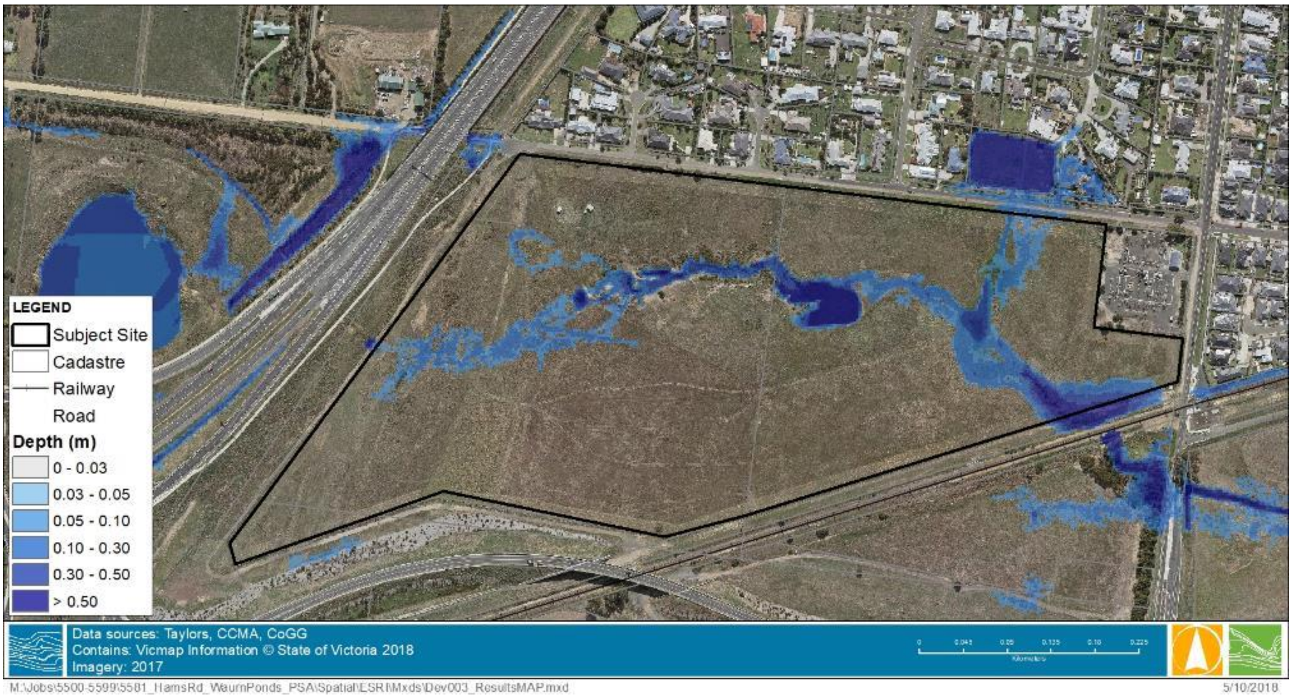


FIGURE 5-1 1% AEP FILTERED DEPTH PLOT – EXISTING CONDITIONS

R01_V03_5581_01b.docx

FIGURE 5-2 1% AEP FILTERED MAXIMUM VELOCITY – EXISTING CONDITIONS

5.2 Developed Conditions

A revised layout that incorporated several changes to the initial design layout was provided to Water Technology (Figure 5-4). The changes were incorporated to include a buffer along the existing waterway and maintain the existing ponds and dams. Several drainage links have been incorporated into the plan to convey overland flow into the waterway area.

A final masterplan layout (MP17) was provided to Water Technology following discussions with the Registered Aboriginal Party, Practical Ecology group and Taylors Development Services. The discussions also reflected comments received from the City of Greater Geelong drainage and environmental departments following the submission of an earlier version of the report in May 2018.

Water Technology was engaged to ensure the final masterplan was able to maintain the existing alignment of the waterway, minimise excavation within the waterway corridor and provide efficient and safe drainage of the site. Other objectives related to the flood modelling and drainage layout are aimed to provide best practice water quality treatment and protect the waterway against erosion as a result of the development. The strategy and concepts of the management of stormwater were identified in the previous SWMP and have been maintained within the flood modelling. This allows the incorporation of online treatment due to the proximity of the development to the upper reaches of the waterway.

The developed conditions model incorporated the same setup parameters as the “existing conditions” model as well as an updated DEM within the subject site based on the site layout provided to Water Technology by Taylors (Figure 5-3). This updated DEM raised residential parcels above the natural surface to ensure the parcels were around 300 mm above the internal roadways to allow for 1% AEP stormwater flow to flow back to the waterway. Embankments were located at the two retarding basin locations with RB1 having a 900 mm wide x 1500 mm high single box culvert with a low flow weir level (dimensions as per previous SWMS) located below the roadway at a level of 62.5 m AHD. RB2 located at the eastern end of the development also used a low flow weir structure 300 mm wide x 500 mm high, an initial spillway (1100 mm Wide x 1750 mm High) and a secondary Spillway 20 m Wide at a height of 1.8 m. The embankment level of RB 2 is 58.5 m AHD and is located upstream of the railway reserve. No internal (within the development site) stormwater pipes were included in the model. Initial and continuing losses were adjusted to reflect the “developed conditions” of the residential parcels and roadways (Section 3.3). The retarding basins are placed online with a 5 m wide waterway channel that follows the existing waterway alignment. The depth plot results were provided to the client to undertake further analysis.

FIGURE 5-3 INITIAL PROPOSED LAYOUT

Figure 5-5 shows a filtered depth plot (with shallow depths less than 3 cm removed) for the 1% AEP design flood under proposed developed conditions. The majority of flow is confined to within the waterway reserve and drainage links. Figure 5-6 shows the filtered 1% AEP design flood peak velocity plot, which highlights high-flow areas. The results show significant overland flow coming from the Hams Road retarding basin, which highlights the need to upgrade the existing outlet pipe from a 300 mm diameter culvert. This also shows two parcels adjacent to the waterway (either side of the drainage link) are not entirely flood free. Further refinement (including a piped link to the waterway reserve) are recommended for the next stage of design. There is also a high flow path along the internal road in the west of the site as flows from upstream enter the site and travel along the roadway into the waterway reserve. It is recommended that these are piped to convey flows directly to the waterway reserve. There is also a small overland flow path from the south of the site that travels across four parcels before reaching the internal roadway. An additional drainage link that pipes flows into the site and internal drainage network may be required in this area to ensure the residential parcels are flood free.

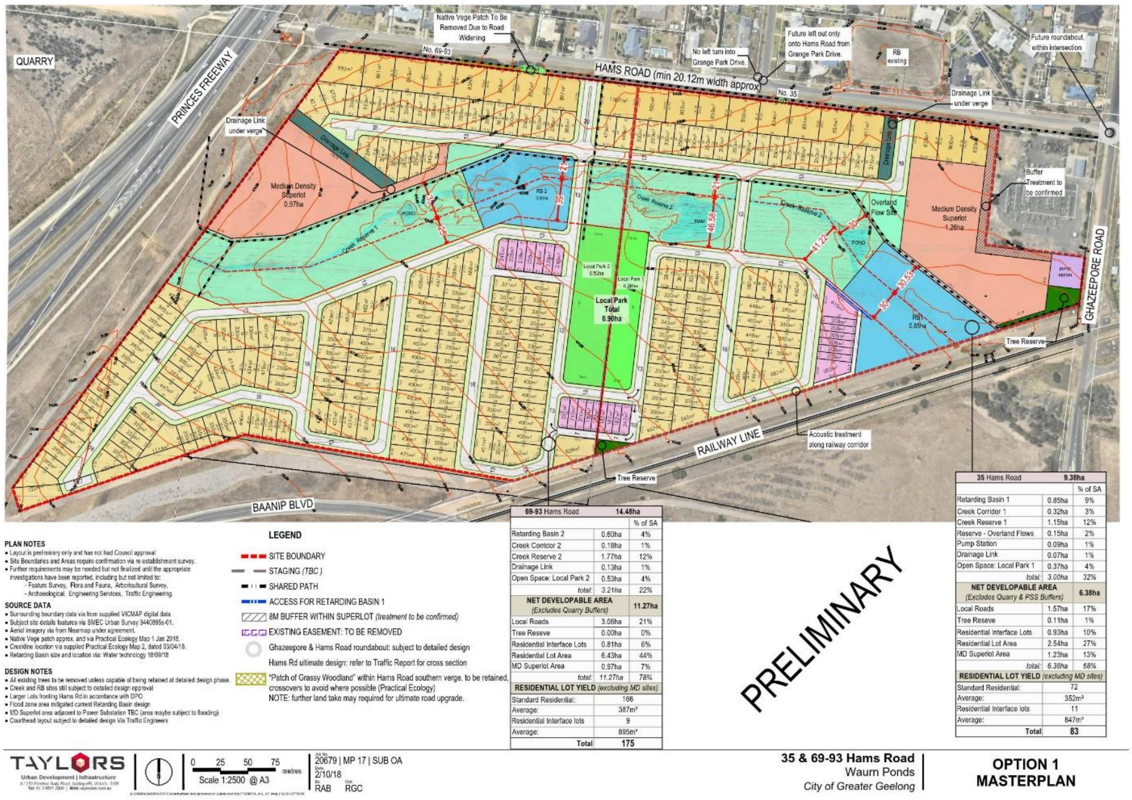


FIGURE 5-4 REVISED DEVELOPMENT LAYOUT (TAYLORS, OCTOBER 2018)

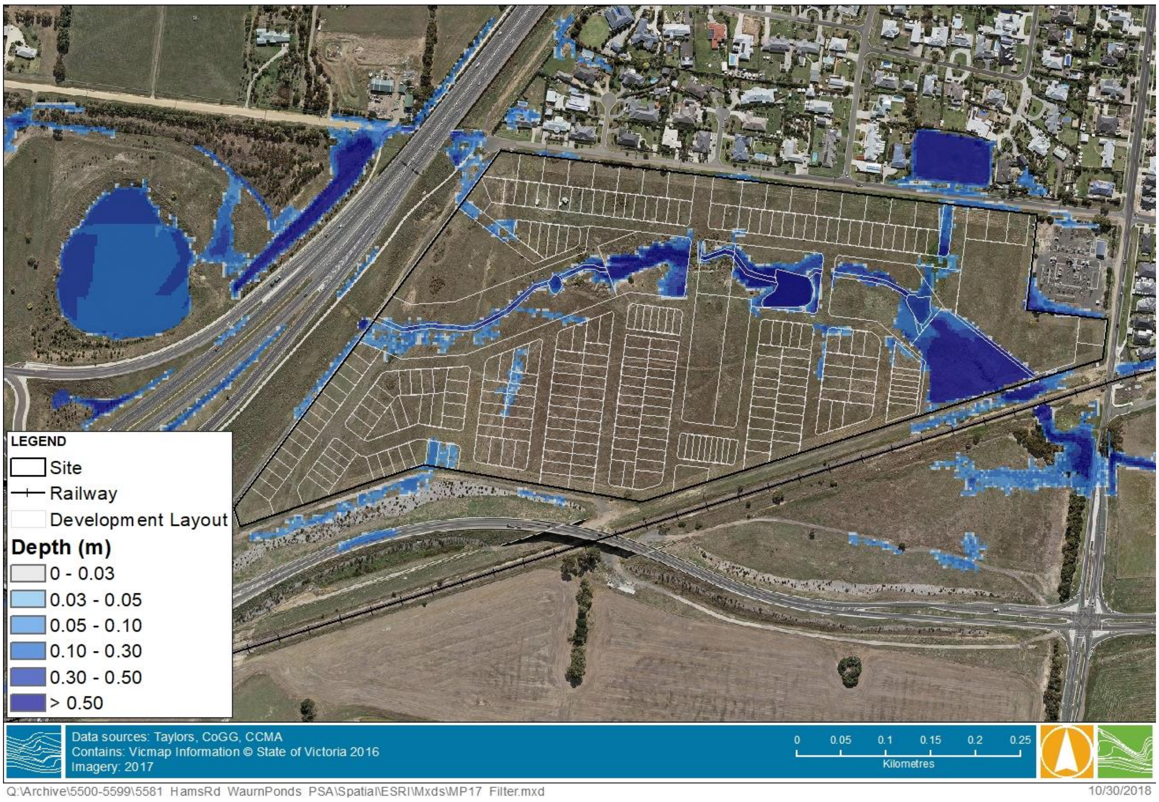


FIGURE 5-5 DEVELOPED SCENARIO FILTERED DEPTH PLOT 1% AEP

R01_V03_5581_01b.docx

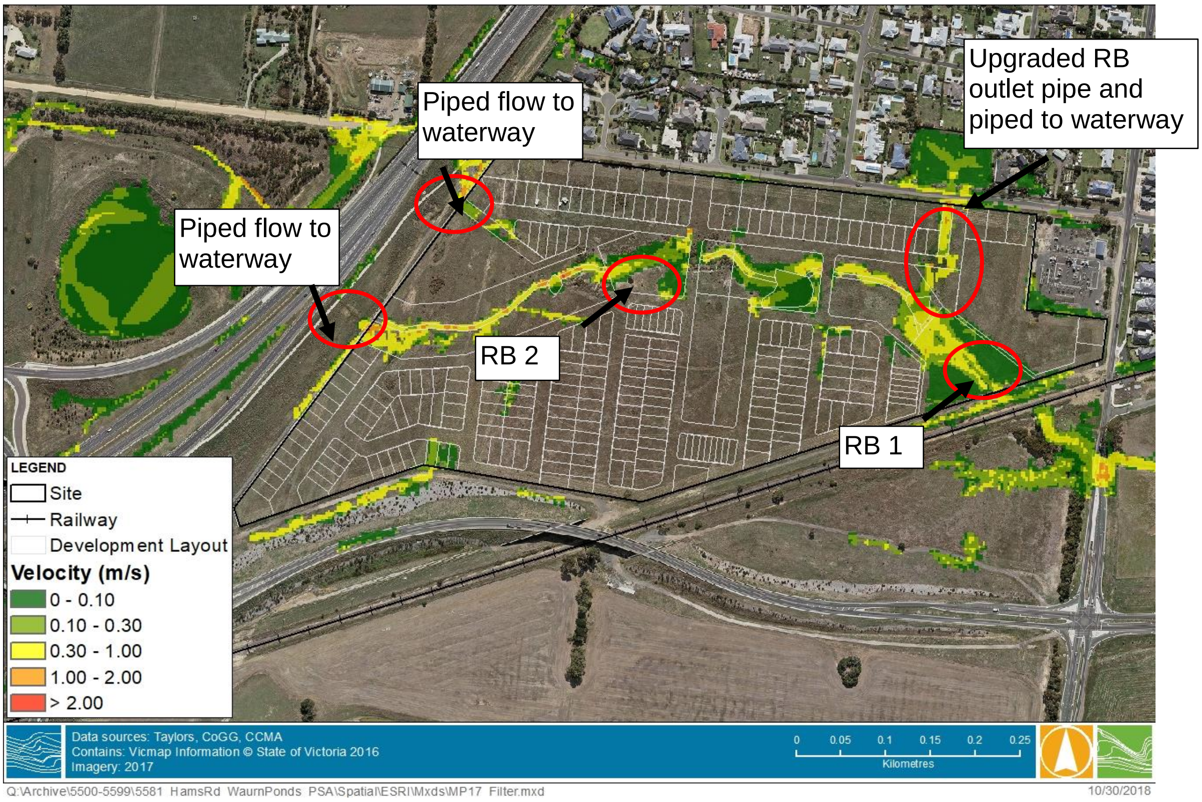


FIGURE 5-6 DEVELOPED SCENARIO FILTERED VELOCITY PLOT 1% AEP

5.2.1 Flow Rates from the site

The second retarding basin located at the south east of the site controls flow rates exiting the site via the railway culvert immediately downstream. The flow rate is controlled by a spillway 300 mm wide with a height of around 0.50 m (above the invert of the basin), and a 1.1 m wide secondary spillway at a depth of 1.25 m. An embankment then separates the retarding basin from the railway reserve. Table 5-1 shows a comparison of peak flow rates from the site through the railway culvert in a 1% AEP event. The results show a slight reduction in peak flow rates. The embankment is not overtopped in the 1% AEP design flood (maximum water surface elevation 57.96 m AHD) and should provide 300 mm freeboard. The 2-hour and 3-hour design storm flow rates from the site (at the railway culvert) are plotted for both the existing and developed conditions in Figure 5-7.

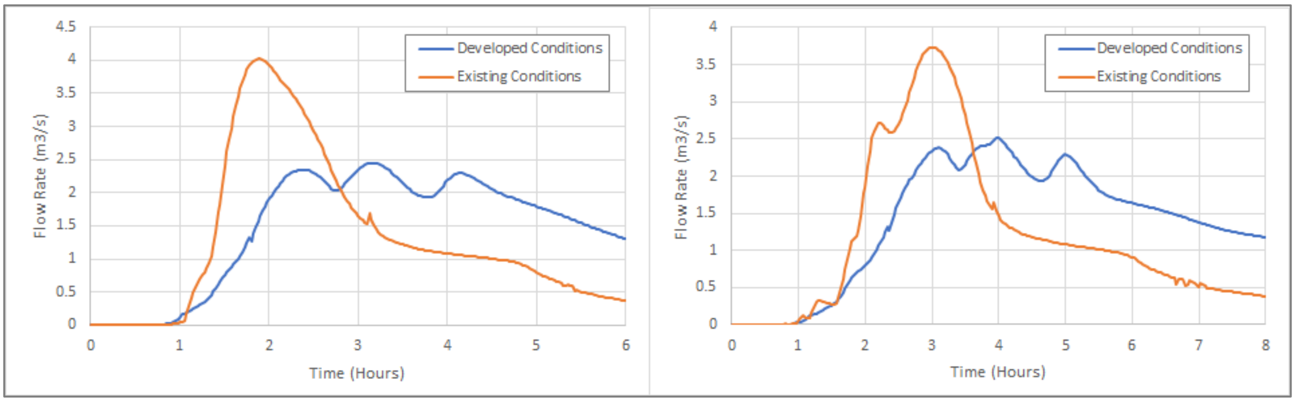


FIGURE 5-7 FLOW RATES FROM THE SITE (LEFT: 2-HOUR EVENT, RIGHT: 3-HOUR EVENT)

R01_V03_5581_01b.docx

TABLE 5-1 PEAK FLOW RATES FROM SITE (RAILWAY CULVERT – EXISTING & RB OUTLET FOR DEVELOPED) FOR 1% AEP EVENTS

	30 Minute	60 Minute	120 Minute	180 Minute	360 Minute	Maximum
Existing Conditions (m^3/s)	1.86	3.11	4.02	3.73	3.02	4.02
Proposed Development (m^3/s)	1.41	1.96	2.46	2.51	2.28	2.51

5.2.2 Storage

The two retarding basins incorporated into the flood model were initially sized (in the previous SWMS report[1]) to provide 6,000 m^3 (RB2) and 8,500 m^3 (RB1) of flood storage, using a RORB model (storage sizing is generally conservative). The maximum flood level and volume stored within each retarding basin was obtained from the hydraulic flood modelling results. This showed that RB1 contains less volume in a 1% AEP design flood due to its location upstream of the first waterway crossing. Immediately upstream of the waterway crossing there is around 3,000 m^3 of volume stored within the waterway reserve at a maximum depth of 2.20 m and a water surface elevation of 62.40 m AHD. Further efficiency (with regards to providing more storage at this location) may be gained by reducing the size of the culvert structure at RB2 and retarding more flow. The final road levels developed at the detailed design stage will need to work with the maximum water levels to incorporate appropriate freeboard. RB1 (from the outlet structure/embankment at the downstream end of the property back to the outlet of the existing pond) contains around 9,000 m^3 of storage (within the RB and the waterway reserve) at a maximum depth of 2.4 m and a water surface elevation of 58.20 m AHD. This RB currently contains around 3,000 m^3 of "cut area" below the existing natural surface outside of the waterway alignment.

Further refinement of finished fill levels, roadways and retarding basin design can be achieved at the next stage of design. The inclusion of a full earthworks model and sub-surface (minor) drainage assets are also likely to impact the storage requirements and levels, however there appears sufficient capacity within the layout to accommodate additional storage if required. No allocation for the inclusion of a wetland/sedimentation basin within the retarding basins has been undertaken for this report. The treatment of stormwater runoff from the development to meet best practice was previously identified within the 2013 SWMS. The basic principles of online water quality treatment appear to fit within the current layout, however a review of the previous MUSIC modelling has not been undertaken.

5.2.3 Waterway Cross-Section

The existing drainage alignment has an approximate slope of 1.5% with a fall of 11.7 m across a 760 m flow path (from the top to the bottom of the site). Several reaches of the alignment are not well defined and result in shallow wide spread flow. To maintain the structural integrity of the waterway and maintain the general alignment of the waterway, it is recommended the waterway be lowered and widened along the length of the site (excluding existing pond areas) with the inclusion of rock lining and vegetation. This is to ensure the 1% AEP flow is contained within the waterway and to ensure the flows from the development do not generate erosion issues within the waterway. Five typical cross-sections of the waterway were extracted from locations within the site to show the existing conditions and the channel capacity used within the modelling. It should be noted that due to the resolution of the model grid, appropriate batter slopes were not represented within the hydraulic modelling (Figure 5-8). A Manning's calculation was also undertaken to ensure the average cross-section was able to convey the flow rates (Table 3-1). This showed that for the 1% AEP design flow (2.90 m^3/s) water can be conveyed within the 5 m waterway channel, at a depth of around 0.80 m. Given the extensive waterway corridor (60 m buffer), the flood risk if the channel capacity is exceeded is not significant.

R01_V03_5581_01b.docx

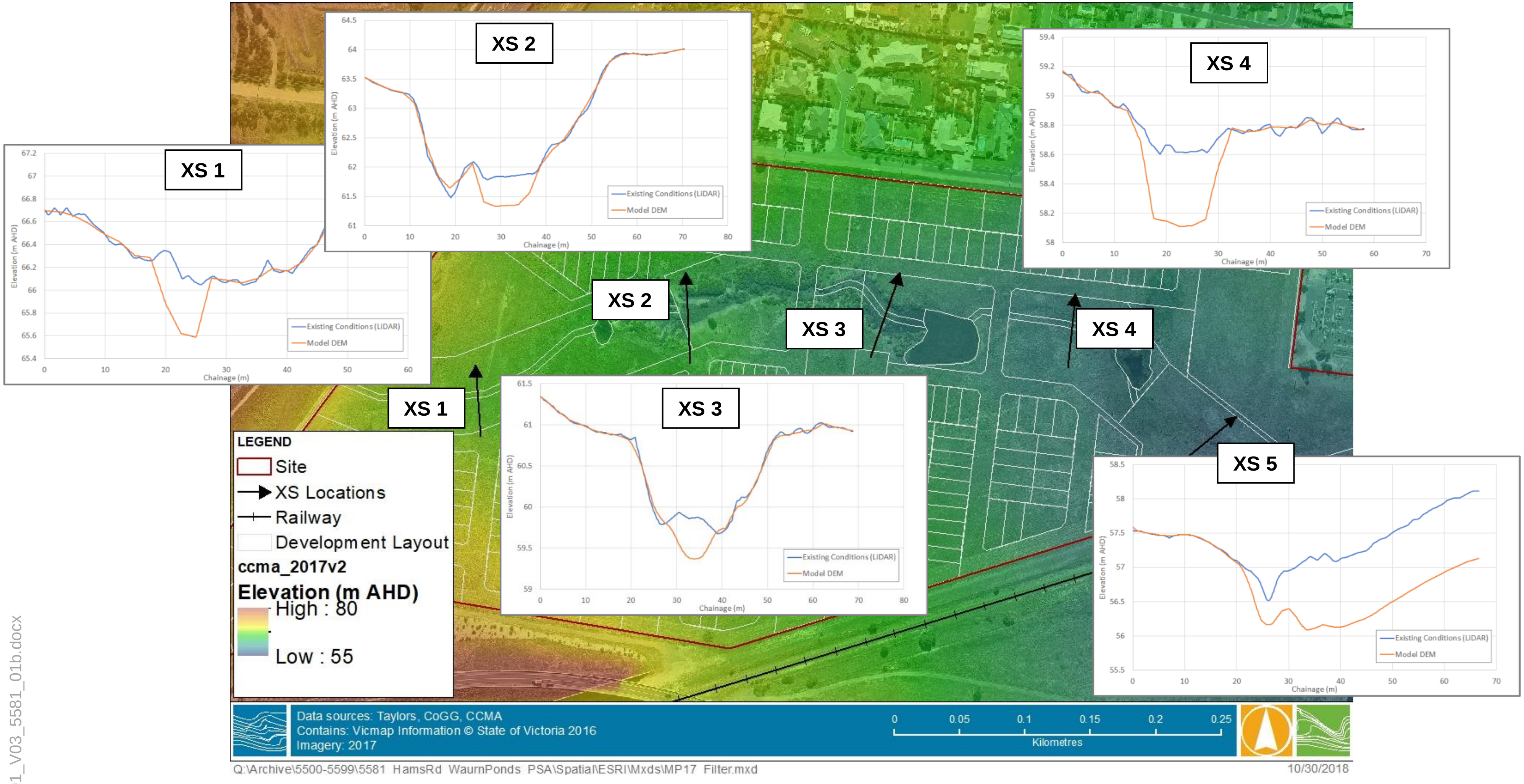


FIGURE 5-8 WATERWAY ALIGNMENT CROSS-SECTIONS EXISTING AND DEVELOPED (HYDRAULIC MODEL)

TABLE 5-2 MANNINGS CALCULATION CHECKS

Mannings Calculations (Specify all channel dimensions - Variable Dimensions)

Turn On Iterations

Turn Off Iterations

Comments:

Input Parameters		
Depth at	0.8	m
Material/Type	5.1.1.1 Minor stream: surface width at flood stage < 30m, fairly regular section (some grass and weeds, little or r	
Channel Condition	Good	
Channel Slope	0.015	***Slope of the channel***

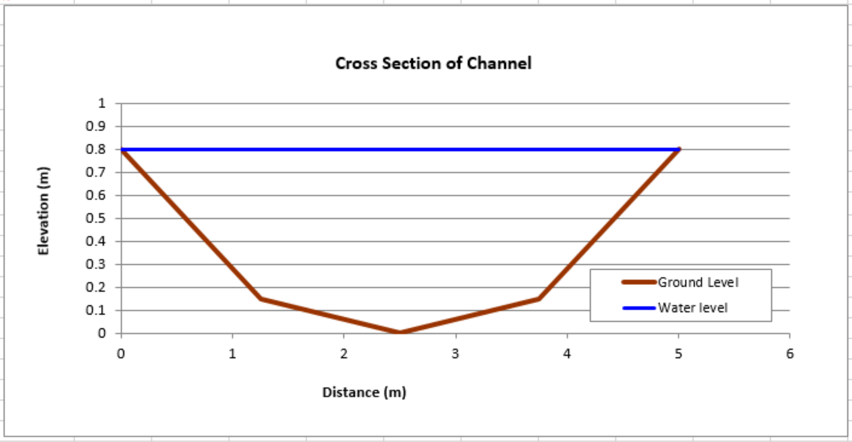
Check: OK, Input parameters are fine, calculations are alright

Output Parameter		
Cross sectional area	1.63	m^2
Wetter perimeter	5.64	m
Manning's n	0.03	
Hydraulic radius R	0.3	m
Velocity	1.78	m/s
Flow rate Q	2.90	m^3/s
Froude Number	0.64	

Click here if x-coordinates are not in ascending order

Insert Cross Section Coordinates here (in m):

x coordinates (must be ascending)	z coordinates
0	0.8
1.25	0.15
2.5	0
3.75	0.15
5	0.8



R01_V03_5581_01b.docx

6 RECOMMENDATIONS

The next stage of design will produce a more detailed finished surface from an earthworks model. Once this is completed, an analysis should incorporate a full cut and fill balance. This will not only provide detail on the quantum of earth works provided but will also ensure that no floodplain storage is removed from the site as a result of the development. Final road levels and minor stormwater drainage infrastructure can then be incorporated into the design and final wetland/retarding basins can be designed. The current development layout results in no increase in modelled peak flow rates leaving the site at the Railway culvert for a 1% AEP design flood. This design has maintained the existing ponds and dam located within the waterway reserve and would likely provide suitable opportunities for water quality treatment from the local stormwater runoff. The waterway alignment has been maintained and it is recommended that a cross-section similar to that presented in Section 5.2.3 be adopted to incorporate rock lining and vegetation to provide erosion control within an efficient drainage alignment.

The next stage will also likely incorporate internal drainage layouts to effectively drain stormwater runoff from the site into the waterway, basins and out of the site. The inclusion of the internal drainage may slightly impact on flow rates out of the site with a pit and pipe network draining the site quicker compared to purely overland flow as modelled within this investigation, however there appears to be sufficient scope and room within the development to allow for additional flow up to the 1% AEP existing conditions peak flow rate (an additional 60% of developed peak flow rate). Three locations use pipes to convey water entering the site from outside to the waterway reserve. These locations are shown in Figure 6-1 along with a small catch drain that conveys overland flow from the south west of the site to the culvert beneath the railway.

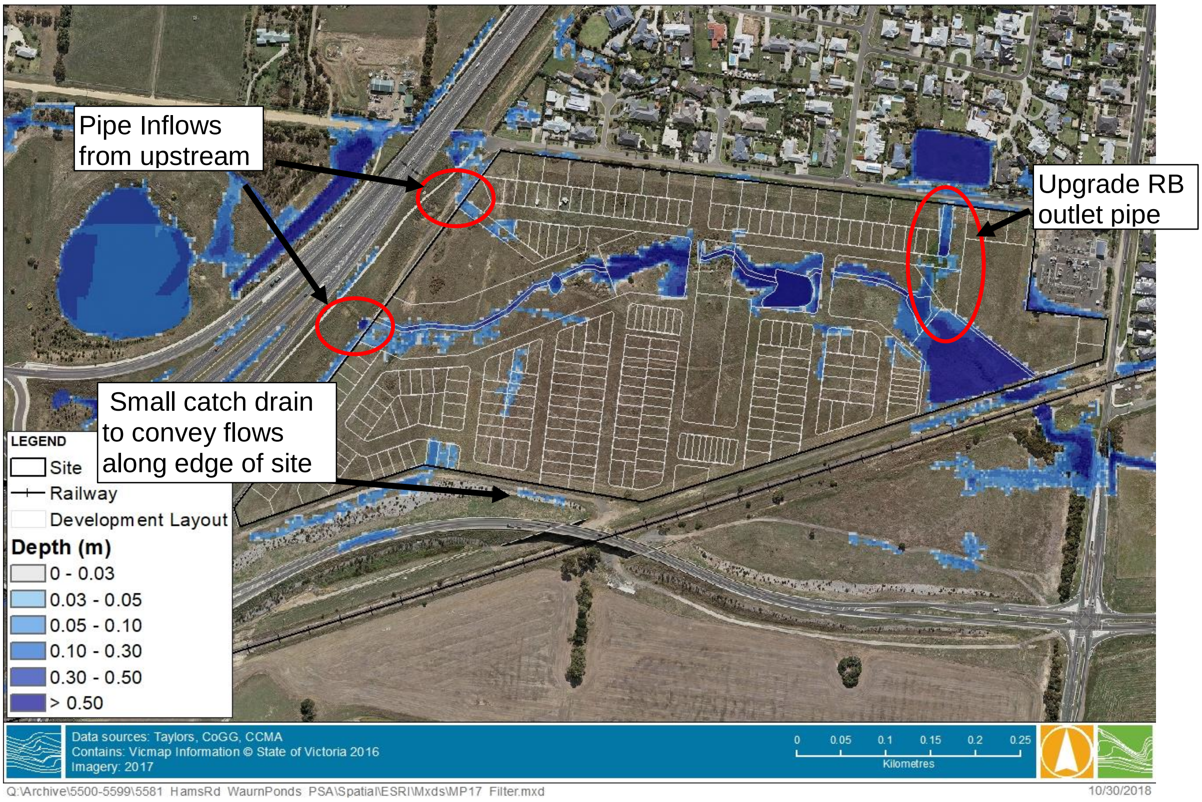


FIGURE 6-1 RECOMMENDED UPGRADES AT NEXT DESIGN PHASE

R01_V03_5581_01b.docx

Previous investigations identified a need to upgrade the Hams Road outlet structure to a 600 mm diameter culvert. It is recommended this be undertaken as per the modelling. This includes an extension of the piped outlet to the waterway reserve and drainage easement identified in the current layout. Any overland flow not contained within the pipe network would then travel across the designated drainage link into the waterway reserve.

7 SUMMARY

The hydrology assessment and hydraulic modelling undertaken for this investigation has found that the proposed development site is controlled at the downstream end of the site (the railway and Ghazeepore Road) as well as much of the upstream catchment (Geelong Ring Road & Hams Road retarding basin).

Initial development plans were used in the flood modelling and results were assessed and presented to several referral authorities that provided comments and recommended changes. Discussions with CoGG and Wathawurrung Aboriginal Corporation were undertaken with the aim to ensure the existing alignment of the waterway is maintained, the creek corridor widened and minimal excavation within the creek corridor undertaken. This aims to ensure a natural looking and functioning waterway whilst providing a safe interface between the drainage functionality and aesthetic open space within the development. The proposed layout was modified to provide increased conveyance along the existing waterway alignment and allocate storage for the additional stormwater runoff generated within the site, without compromising the integrity of downstream drainage infrastructure by maintaining 1% AEP flow rates to pre-developed conditions.

The development does not appear likely to have any adverse effects on downstream property owners as the peak flow rate exiting the site in a 1% AEP design flood is reduced with the current design layout. The principles presented within the 2013 SWMS for online detention of the stormwater runoff are not expected to increase the peak flow rate from the site.

Melbourne

15 Business Park Drive
Notting Hill VIC 3168
Telephone (03) 8526 0800
Fax (03) 9558 9365

Brisbane

Level 3, 43 Peel Street
South Brisbane QLD 4101
Telephone (07) 3105 1460
Fax (07) 3846 5144

Adelaide

1/198 Greenhill Road
Eastwood SA 5063
Telephone (08) 8378 8000
Fax (08) 8357 8988

Perth

Ground Floor
430 Roberts Road
Subiaco WA 6008
Telephone 0438 347 968

Geelong

PO Box 436
Geelong VIC 3220
Telephone 0458 015 664

Gippsland

154 Macleod Street
Bairnsdale VIC 3875
Telephone (03) 5152 5833

Wangaratta

First Floor, 40 Rowan Street
Wangaratta VIC 3677
Telephone (03) 5721 2650

Wimmera

PO Box 584
Stawell VIC 3380
Telephone 0438 510 240

www.watertech.com.au

info@watertech.com.au