Hydrological and Environmental Engineering

Pakenham East Precinct Structure Plan

Proposed Drainage Strategy

REVISION D

20 December 2017

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Contents

1. Introduction

The Pakenham East Precinct Structure Plan (PSP) draft drainage strategy has been prepared for the Cardinia Shire Council (Council) by Stormy Water Solutions.

The PSP area is shown in Figure 1 (bold red line). It is generally bounded to the east by Mount Ararat Road and Mount Ararat Road North, to the west by Deep Creek, to the north by the electricity easement (approximately) and to the south by the Pakenham Bypass.

As detailed, much of the area south of the PSP (between the railway and the Pakenham bypass) is encumbered by the Department of Transport. This land may be utilised for a train stabling yard in the future.

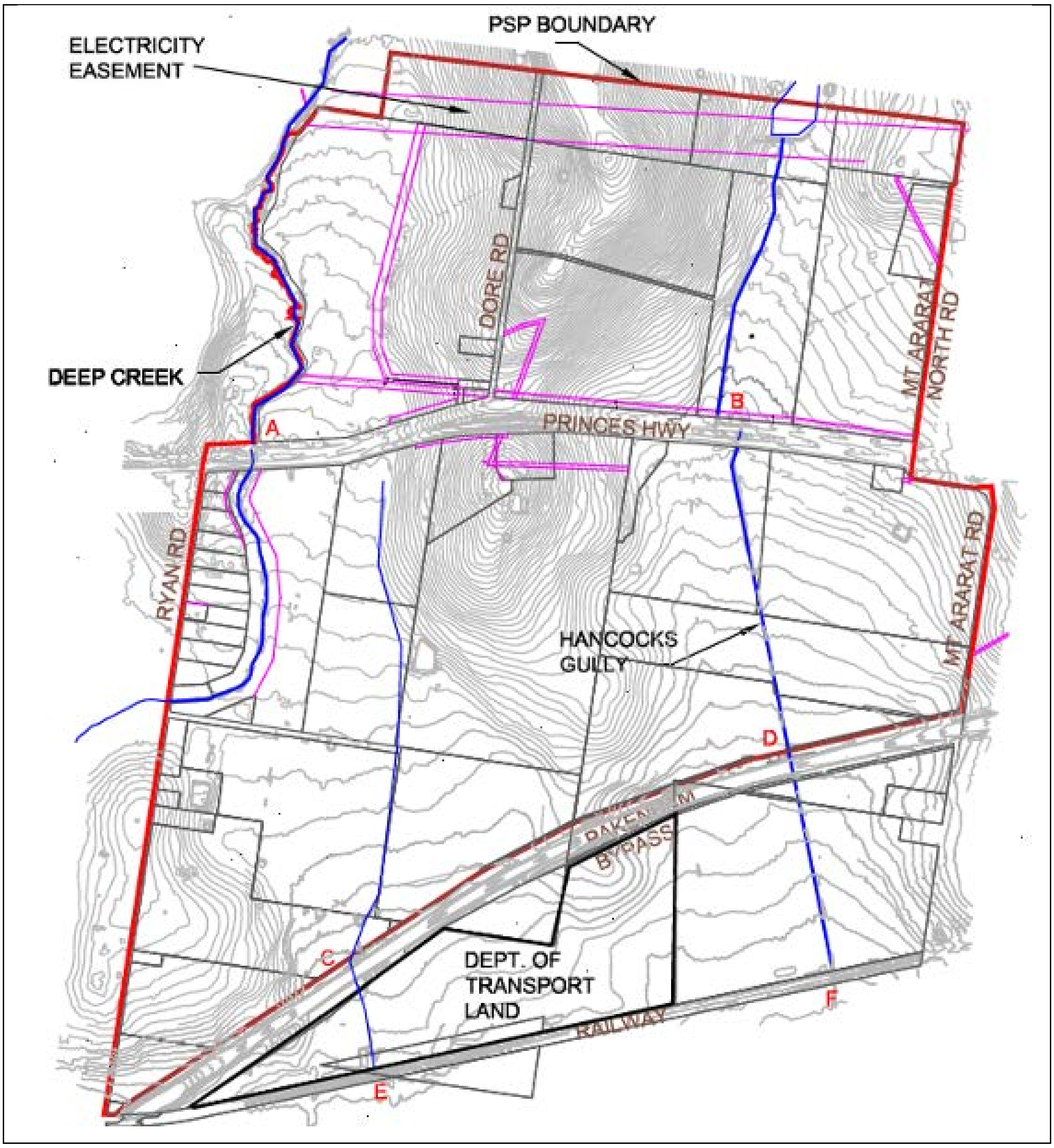

Figure 1 **Pakenham East PSP Site Area**

This report considers the major drainage, flooding and water quality management issues within (and immediately downstream of) the PSP area. The aim of the current investigation is to clearly define the potential land take requirements of major drainage assets.

Stormwater harvesting, potable water reduction and waste water generation reduction issues and strategies are important water management considerations. However, they are not considered in detail in this report. However, any future initiatives should complement the drainage strategies advocated in this report. Stormwater harvesting is considered in regard to the potential regional stormwater harvesting schemes could have on drainage asset land take requirements. A Whole of Water Cycle Management Assessment (WOWCA) is being undertaken as part of the preparation of the PSP in conjunction with Melbourne Water, South East Water, the Department of Environment, Land, Water and Planning (DELWP) and the Metropolitan Planning Authority (MPA).

An Ecological Investigations Report has been undertaken for the precinct including targeted surveys for species of national environmental significance.

A Native Vegetation Precinct Plan (NVPP) will be prepared for the precinct to identify the native vegetation to be retained and native vegetation that can be removed, including specifying offset requirements.

Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, 5 October 2014 should also be referred to.

A Cultural Heritage Management Plan (CHMP) will be prepared for the precinct which will identify any areas of cultural heritage significance to be retained within the constructed waterway corridor and include management requirements.

The entire PSP area is assumed to be developed for urban purposes at this stage. Land uses have yet to be formally developed. Some high density and low density development is expected, however, generally lots in the order of 650 m^2 over the developable area are expected. Apart from drainage reserve requirements, development potential may be limited by steep topography slopes, existing gas easements and the existing electricity easement locations. As such, the “average” assumption of 650 m^2 lots over the entire area is considered a conservative assumption in regard to drainage requirements at this stage.

Main drainage, floodplain and waterway management in this area are the responsibility of Melbourne Water Corporation (MWC) with minor drainage from urban catchments of less than 60 ha generally remaining with Cardinia Shire Council.

It is likely that a Development Services Scheme (DSS) will be prepared and implemented by MWC to guide orderly provision of main drainage services through the PSP area.

Stormy Water Solutions has developed four drainage strategy options for consideration. In this way, council can incorporate various scenarios in relation to drainage requirements and ultimately adopt a PSP formulation which optimises all PSP objectives, not just the drainage requirements.

All assets detailed in this report are at the strategy development/concept design stage. As such, all proposals are subject to change as the planning and design process for the PSP continues.

The earlier draft of this report (Rev C, April 2014) resulted in subsequent development of functional designs of the major drainage infrastructure in the region as recommended (at a concept design stage) in the April 2014 draft report. As such, although this current revision (D) is dated after the following reports, each of the following reports supersede this report in terms of drainage infrastructure design detail in the region.

- Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, 5 October 2014.
- Hancocks Gully Development Services Scheme, Functional Design of Two Wetland/Retarding Basins and Two Vegetated Channels, Rev B, 12 December 2016,
- Dore Road Development Services Scheme, Functional Design of the Dore Road DSS Wetland/Retarding Basin, Rev A, 24 February 2017,
- Ryan Road Development Services Scheme, Wetland Retarding Basin Functional Design, Rev C, 7 April 2017, and
- Dore Road Drainage Scheme (1606), Swale Functional Design, 15 June 2017.

2. Information Sources

In carrying out this review Stormy Water Solutions (SWS) has used:

- Calculated flood levels in Deep Creek as per the Stormy Water Solutions Report "Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, 5 October 2014",

- Valerie Mag's background knowledge and experience with the Deep Creek and Hancocks Gully catchments, which includes the flood plain modelling of Deep Creek in the early 1990's and development of various drainage strategies and schemes in and around Pakenham over 20 years,

- A site inspection in November 2012, used to make rough estimates of waterway cross section roughness, visually assessing flood plain dynamics and waterway and floodplain vegetation and assessing the applicability of the site for the integration of Water Sensitive Urban Design (WSUD) elements such as wetland systems,

- Lidar survey plans for the PSP area obtained from Council and catchment boundary and contour information obtained from MWC,

- Various Vic Road design plans of the Princes Highway and Pakenham bypass culvert systems,

- Gas infrastructure design plans as provided by APA GasNet Australia (Operations) Pty Ltd,

- Site observations and hand measured dimensions (provided by council) of the Hancock gully railway bridge structure,

- A Stormy Water Solutions report entitled "Cardinia Investigation Area T5/T6/T7, Deep Creek / Toomuc Creek Catchment Investigation, FINAL, 28 May 2009",

- A RORB model (an industry-standard Runoff Routing Model originally developed by Monash University (Laurenson EM and Mein RG)) developed for this study by SWS to estimate flood flows and retarding basin requirements,

- Various hydraulic formula (including Mannings formula (PC Convey), Hec Ras and culvert analysis) to estimate culvert capacities, required floodway dimensions and required preliminary pipe sizes, and

- a MUSIC Model V5 (Model for Urban Stormwater Improvement Conceptualisation software developed by the Cooperative Research Centre for Catchment Hydrology) developed for this study by Stormy Water Solutions to simulate runoff and pollutant load regimes and the Water Sensitive Urban Design (WSUD) elements required to mitigate these impacts.

All elements proposed as part of this drainage strategy have been fully considered in regard to their applicability. As much as possible actual invert levels, normal water levels, batter requirements etc have been set at this stage to ensure all elements can be constructed and will not be constrained by outfall invert levels, gas line levels, or ecological constraints (e.g. tree locations).

Notwithstanding the above, all designs are at the concept design stage only and are subject to change during the design process.

3. Existing Catchment and Waterway Description

3.1 Deep Creek

Deep Creek is a relatively large waterway which forms the western boundary of much of the PSP area. As its name suggests, the waterway has been prone to erosion. Existing robust instream and riparian vegetation is prevalent. Figure 2 shows a typical form of the waterway.

Figure 2 Deep Creek - Looking Upstream from Princes Highway

MWC have declared flood levels adjacent to Deep Creek. In addition, easement lines 50 metres either side of the creek downstream of Princes Highway have been set as a direct result of the approval for the 1990's low density subdivision located on the western side of the creek downstream of Princes Highway.

Stormwater runoff originating within the PSP area directly adjacent to the creek and north of the Princes Highway is largely diverted towards Deep Creek via the large table drain located along the northern edge of the highway. The headwall on the right side of the creek in Figure 2 is the outfall from this table drain to the creek.

Stormwater runoff originating within the PSP area directly adjacent to the creek and south of the Princes Highway is directed away from the creek and outfalls, via a small rural drain, to Point C (Figure 1).

3.2 Hancocks Gully

Hancocks Gully is an artificial drainage line constructed by the farmers of the land over time. North of Princes Highway, the drain is relatively straight, with some trees located adjacent to the waterway. However, preliminary advice suggests that this vegetation is not indigenous. South of Princes Highway, the drain is again relatively straight, but not affected by any significant vegetation. At this point it has been constructed away from the natural valley.

Figure 3 Hancocks Gully - Looking downstream from Princes Highway

3.3 Western Tributary - Outfall at Point C

Small local drainage lines from the existing paddocks outfall at Point C, Figure 1. This drainage line is referred to as the western tributary in this report.

There are very little ecological attributes to the drainage system here. It is noted Growling Grass Frogs have been found in local dams in the past. Preliminary advice suggests that relocation of these habitats closer to more ecological functioning watercourses and water bodies (eg wetlands) would be advantageous in terms of the final ecological outcomes in the area.

4. Design Objectives and Requirements

4.1 Integration of Landscape Values and Ecological Objectives

The importance of considering the management objectives, landscape values and community aspirations is a fundamental part of developing an integrated design solution. To this end, the Council and Stormy Water Solutions have worked closely to ensure that drainage elements, such as wetland systems, offer the opportunity to complement the landscape amenity and ecological diversity of the final development form. In particular provision of existing and future habitat corridors along existing drainage paths and creeks has been seen as a major objective, particularly in terms of providing future habitat for the Growling Grass Frog, the dwarf galaxias, and the Southern Brown Bandicoot.

4.2 Internal Drainage Design and Staging

It is proposed to separate the PSP area into small distinct sub catchments to enable development to occur much more readily on a local scale.

4.3 Water Quality Requirements

Under current best practice requirements, any drainage strategy must ensure that all stormwater is treated to at least current best practice prior to discharge from the PSP area. Therefore, the strategy must ensure 80% retention of Total Suspended Solids (TSS), 45% retention of Total Phosphorus (TP) and 45% retention of Total Nitrogen (TN). Notwithstanding the above, MWC have indicated that the requirements may increase to 85%/50%/50% in regard to TSS, TP and TN respectively by the time the PSP rollout occurs.

In addition to the above, various authorities are advocating “over treatment” given the RAMSAR wetland system located within Westernport Bay. The EPA have advised, given previous investigations in Westernport, that State Environment Protection Policy (SEPP) Schedule F8 could be interpreted as requiring 93%/66%/63% retention in regard to TSS, TP and TN respectively prior to stormwater discharge to Westernport Bay.

- At this stage the strategy aims to retain stormwater pollutants to current best practice requirements. However, the MUSIC modelling to date indicates this objective can be exceeded, especially if the larger WSUD elements proposed are supplemented by streetscape and site scale assets.

- However, meeting SEPP Schedule F8 objectives has been considered. As such, this report further investigates two options (1 and 4) in regard to meeting this objective within, or very close to, the PSP area. It should be noted that future regional wetland systems located in the Cardinia Creek outfall downstream may be able to supplement and local treatment initiatives, although this regional analysis has not been undertaken at this stage..

4.4 Flood Storage Requirements

In line with current Koo Wee Rup Flood Protection District (KWRFPD) flood protection guidelines, the flood retarding basin objectives are to ensure:

- The peak 100 Year ARI flow from the future development does not exceed the predevelopment flow rate at all outfall points from the PSP,
- The peak 24 hour 100 Year ARI flow from the future development does not exceed the predevelopment flow rate for a storm of this duration at all PSP outfall points, and
- The retarding basins can store at least the difference between the expected post development and predevelopment 24 hour 100 Year ARI flow volume to ensure no increased flood effect within the KWRFPD during a 24 hour 100 Year ARI flood event in the region.

In addition to the above, development of the functional design of these retarding basins must include clearly defining the current low flow regimes to the downstream receiving bodies. Flow regimes will need to be maintained post development in order to protect the existing ecology and channel morphology of the downstream Deep Creek and Hancocks Gully creek systems. The concept design of Option 4 includes this consideration via retarding the 2 Year ARI flow to predevelopment flow rates. In all other options it is assumed that this aspect of the design will be formulated during the functional design process.

4.5 Flood Protection Requirements

4.5.1 Deep Creek

It is anticipated that deep creek will retain its current form and vegetation, although ongoing waterway management will occur in line with current and existing ecological and landscape requirements. A development line will be set as part of the PSP process. Any future development must incorporate filling to required MWC standards adjacent to the creek to ensure adequate flood protection (assumed to be 600 mm above the flood levels determined given ultimate reserve requirements).

In addition, some shaping of the flood plain and augmentation of the Ryan Road culvert system is required as described in Section 5.4 of this report.

4.5.2 Hancocks Gully

Any remodelling of Hancocks Gully must ensure that the new watercourse is capable of conveying the 100 Year ARI flow through the PSP area.

4.6 Ecological Objectives

The preliminary ecological advice indicates that, at this stage, there appears to be very little vegetation issues in regard to drainage infrastructure apart from retention of the Deep Creek riparian zone and remnant roadside vegetation. At the present time, Hancocks Gully is just the straight drain the farmer originally cut with any vegetation present offering little ecological attributes.

Some dams will be removed. However, the strategy allows for new frog ponds to be sited close to wetland systems if required.

In addition to the above all existing frog ponds located within the Pakenham Bypass road reserve will be retained. Wetland/retarding basins located directly upstream of these systems will take into account the requirement of maintaining low flow feeds to these systems in the design of their outlet systems.

The use of vegetated channel systems and wetland systems is expected to greatly enhance current ecological attributes along the local drainage lines, including providing robust ecological corridors.

It is recommended that Council engage ecologist to review this draft drainage strategy to determine the exact ecological impacts and constraints in regard to the preliminary proposals.

5. Drainage Strategy Option Description and Requirements

As the PSP land use breakup and requirements (in addition to the drainage requirements) have yet to be formalised, Stormy Water Solutions has developed four drainage strategy options for consideration. In this way, council can incorporate various scenarios in relation to drainage requirements and ultimately adopt a PSP formulation which optimises all PSP objectives, not just the drainage requirements.

Stormy Water Solutions has produced four Drainage Strategy plans (1304/1-5) which detail Options 1 to 4 and their proposed drainage strategy elements at this stage (see Appendix A).

Option 1 shows all required drainage elements within the PSP boundary. Option 2 details a possible alternative where some elements are located directly downstream of the PSP boundary. Option 2 would require agreement from affected landowners and the Department of Transport (DoT) on the land affected by potential future railway stabling infrastructure. Options 3 and 4 are variation on Option 2 which try to minimise drainage encumbrances on the land located between the Pakenham Bypass and the Railway (including DoT land).

The primary drainage elements proposed (as detailed in Appendix A and Stormy Water Solutions Drawings Set 1304/1-5) are:

- Retarding Basin/Wetland W1 servicing catchments Ext 2, Ext 3, Int. 2 and Int. 3 as defined in 5.2 below,

- Retarding Basin/Wetland W2 (separated into W2A and W2 in Option 2 and W2A and W2W4 in Options 3 and 4) servicing catchments Ext 4, Int. 5 and Int. 6 (with the addition of DS 2 in Options 2, 3 and 4) as defined in 5.2 below,

- Retarding Basin/Wetland W3 servicing catchment Int. 1 as defined in 5.2 below,

- Retarding Basin/Wetland W4 (separated into W4A and W4 in Option 2 and W4A and W2W4 in Option 3 and only W2W4 in Option 4) servicing catchments Int. 4 (with the addition of DS 1 in Options 2 and 3 and DS1 and DS2 in Option 4) as defined in 5.2 below,

- Vegetated Channels V1 and V2 which will be the remodelled drainage reserve encompassing Hancocks Gully, and

- Various vegetated channels located between the Pakenham bypass and the railway in options 2, 3 and 4.

These options and their required drainage elements are discussed below.

5.1 General Considerations

- The strategy allows for development of small catchments individually (upstream of wetland or sediment ponds) without cleanout/enlargement works downstream,

- No upgrading of any highway, bypass or railway culvert system is required (See Appendix B),

- The pipe capacities in the developed areas will be sized to contain flows for the 5 Year ARI (Average Recurrence Interval) flow in residential areas as required by Melbourne Water and Cardinia City Council.

- The strategy allows for piped catchments discharging to regional WSUD elements (sediment ponds and wetlands),

- Some recreation initiatives within the creek/wetland corridors such as paths can be allowed for in the space allocated for drainage reserves,

- Two outlets to Deep Creek are proposed to mimic existing outlets and to minimise catchment sizes and help with strategy implementation, and

- The strategy must ensure local catchment diversion into Deep Creek at PSP northern boundary and at Princes Highway.

5.2 PSP Catchment Delineation

Catchment delineation and definition is described below and detailed in Figure 4.

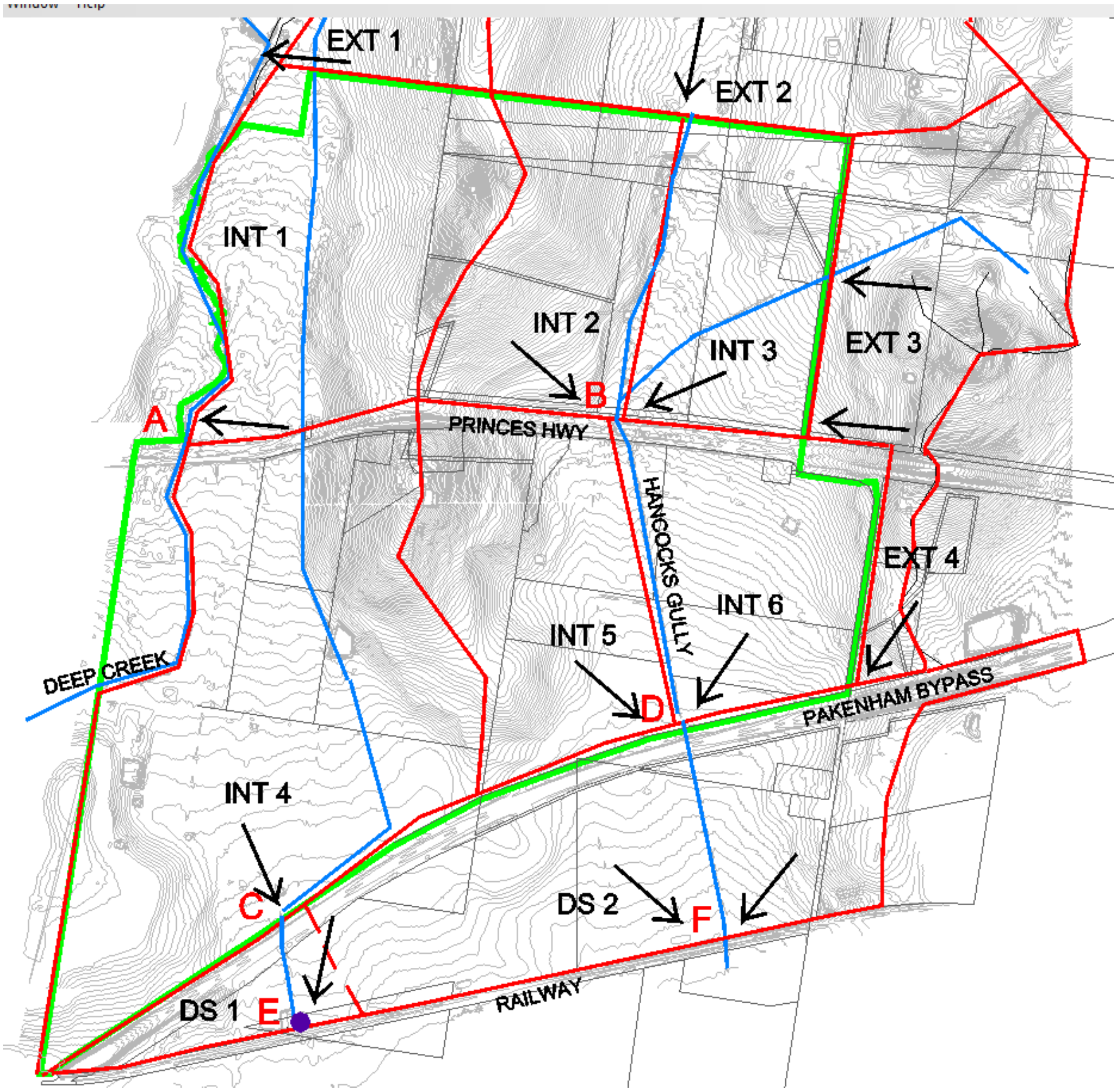

Figure 4 **Site Catchment Delineation**
Catchments are approximate only and subject to change during the design process

5.2.1 External Catchments

DEEP CREEK Deep Creek essentially forms the western catchment boundary with the PSP. The relatively large nature of this creek, together with filling designed to protect the PSP area from flooding due to Deep Creek, will ensure external Deep Creek flows will be conveyed around the PSP.

EXT 1 An external catchment which accounts for any flow which exceeds the capacity of the small Deep Creek tributary north of the Northern PSP boundary. The existing tributary probably catches some flow just north of this point, for direct input into Deep Creek. However, the drainage strategy allows for small levee/filling work along the northern PSP boundary to ensure all flows (up to the 100 Year ARI event flow) are diverted to Deep creek and do not enter the PSP at this point.

EXT 2 The external Hancocks Gully Catchment at the northern PSP boundary. Hancocks Gully will be modified to ensure this the 100 Year ARI flow from this catchment can be conveyed to Point B (Figure 4), in a safe and effective manner.

EXT 3 A small catchment to the east of the PSP, north of Princes Highway. It is proposed to pick up this catchment in pipes/overland flow paths at the eastern PSP boundary for safe conveyance to Point B.

EXT 4 A small catchment to the east of the PSP, south of Princes Highway. It is proposed to pick up this catchment in a pipe/overland flow path located along the interface with the bypass for safe conveyance to Point D.

5.2.2 Internal PSP Catchments

INT 1 An internal catchment bounded by Deep Creek to the west, the local, well defined, ridge line to the east, the northern PSP boundary and Princes Highway to the south. This catchment will be serviced by pipes which will outfall to W3 at Point A, before discharge to Deep Creek at Point A. In the existing situation, a very small flow is conveyed south of Princes Highway to Point C. However most flows are conveyed west to Point A, due to the existing large, northern highway table drain.

INT 2 An internal catchment bounded by the local, well defined, ridge line to the west, the northern PSP boundary, Princes Highway to the south and Hancocks Gully to the east. This catchment will be serviced by pipes which will outfall to W1 at Point B. Flows will only enter Hancocks Gully at Point B, which negates the requirements for many, small treatment facilities along Hancocks gully at multiple outfall points.

INT 3 An internal catchment bounded by Hancocks Gully to the west, the northern PSP boundary, Princes Highway to the south and the eastern PSP boundary. This catchment will be serviced by pipes which will outfall to W1 at Point B. Flows will only enter Hancocks Gully at Point B, which negates the requirements for many, small treatment facilities along Hancocks gully at multiple outfall points.

INT 4 An internal catchment bounded to the west by Deep Creek or the western PSP boundary, Princes Highway to the north, Pakenham Bypass to the south and the local, well defined, ridge line to the east. This catchment will be serviced by pipes which will outfall to W4 at Point C.

INT 5 An internal catchment bounded by Hancocks Gully to the east, Princes Highway to the north, Pakenham Bypass to the south and the local, well defined, ridge line to the west. This catchment will be serviced by pipes which will outfall to W2 at Point D. Flows will only enter Hancocks Gully at Point D, which negates the requirements for many, small treatment facilities along Hancocks gully at multiple outfall points.

INT 6 An internal catchment bounded by Hancocks Gully to the west, Princes Highway to the north, Pakenham Bypass to the south and the PSP eastern boundary. This catchment will be serviced by pipes which will outfall to W2 at Point D. Flows will only enter Hancocks Gully at Point D, which negates the requirements for many, small treatment facilities along Hancocks gully at multiple outfall points.

By separating the PSP area into six distinct, small sub catchments it is anticipated that development can occur much more readily locally, without requiring drainage works to be provided well downstream of an area of interest. Also, outfall works at Points A, B, C or D do not require drain cleanouts downstream, which allows all drainage infrastructure to be retained inside the PSP boundary.

5.2.3 Downstream Catchments

Options 2, 3 and 4 detail possible drainage strategies which consider some drainage elements are located downstream (south) of the PSP boundary. In this way these alternatives allow for stormwater treatment of the existing development of the Pakenham Bypass and the potential development of the Department of Transport (DoT) land located south of the Pakenham Bypass. These options would require agreement from affected landowners and the DoT on the land affected by potential future railway stabling infrastructure.

DS1 This catchment varies in size depending on whether Option 2, 3 or 4 is being considered. However, it could essentially be totally developed in regard to freeway and DoT railway stabling infrastructure. If developed it can be serviced by piped infrastructure out falling to W4 at Point E or via diverting flows in a vegetated channel to W2W4.

DS2 This catchment varies in size depending on whether Option 2, 3 or 4 is being considered. It is partially developed already in regard to freeway infrastructure. However depending on which option is being considered, part of the catchment could also be affected by DoT railway stabling infrastructure. This catchment will be serviced by pipes which will outfall to W2W4 at Point F.

5.3 Gas Line Considerations

5.3.1 Gas Line at Point A (Upstream of Wetland W3)

The strategy drawings (Appendix A) detail the gas main alignments in the vicinity of Wetland/Retarding basin W3. Unfortunately, one gas main is relatively high and one relatively low in this area. Figure 5 below details the concept design of the inlet pipe system to W3 which proposes that the pipe system be constructed under the gas line along Drainage Pipeline A. As detailed, the design is very dependent on the invert level of Deep Creek at Princes Highway, the assumed normal water level of W3 and the sizing and alignment location of the incoming pipe system. However, the investigation does indicate that it is possible to drain Catchment Int. 1 to W3 without requiring modifications to the existing gas infrastructure in the area. Clearance requirements will need to be confirmed with APA GasNet Australia (Operations) Pty Ltd.

5.3.2 Gas Line at Point D (Downstream of Wetland W1)

There are two 750 mm diameter gas lines located in an easement located at Point B, just upstream of Princes Highway. The northern line has an obvert of about 36.4 m AHD and the southern line has an obvert of about 35.8 m AHD. The Retarding Basin/Wetland W1 concept design outlet pipe is proposed to vary between an invert level of 37.5 – 37.0 m AHD. As such, it is proposed that the drainage system be constructed above the gas lines. If additional clearance is required, there is room (given the normal water level of W1 = 39.0 m AHD) to raise increase the outlet invert levels, however, this may increase the site area required for W1.

Clearance requirements will need to be confirmed with APA GasNet Australia (Operations) Pty Ltd.

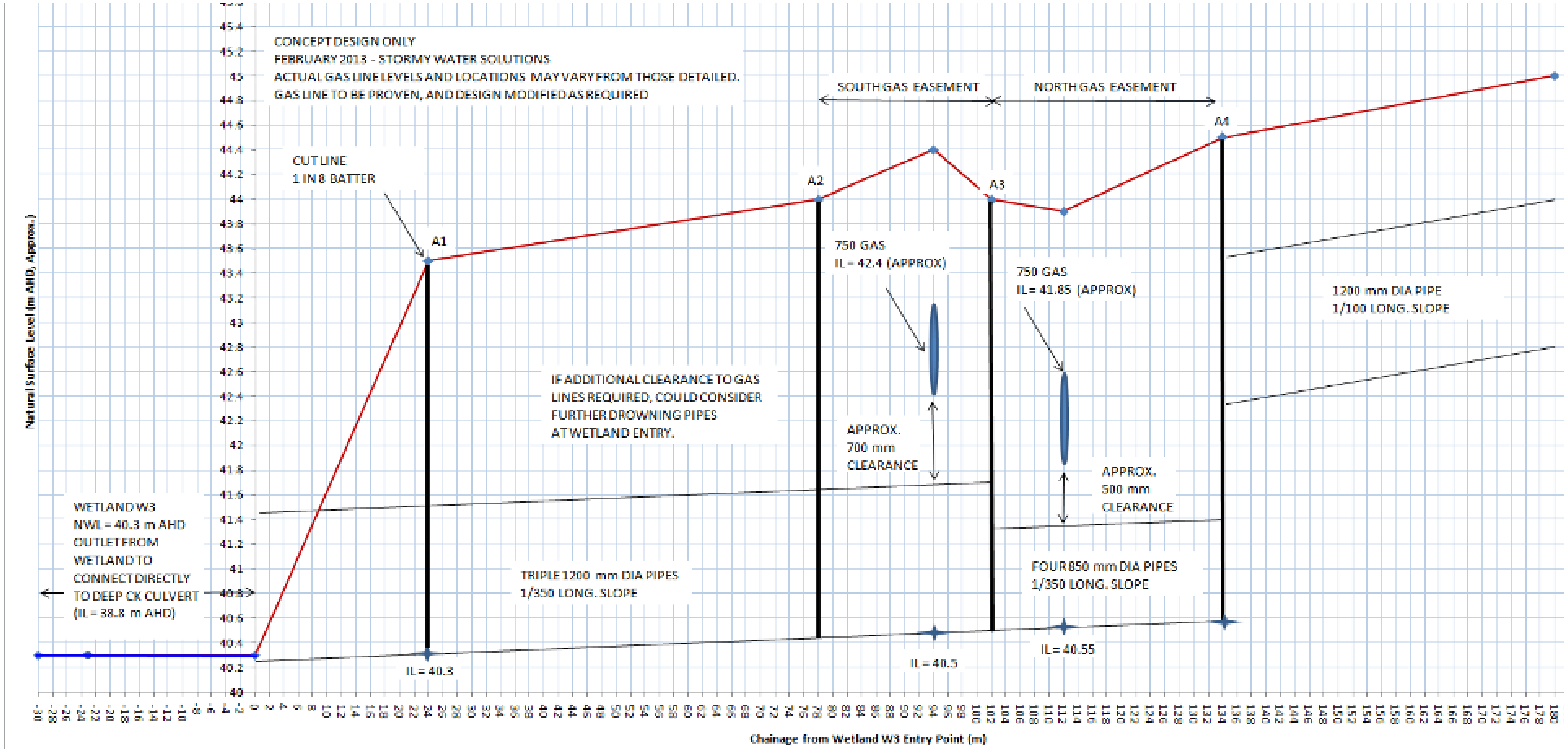

Figure 5 **Required Longitudinal Section of Pipeline A**
Concept Design Only – Subject to Change

5.4 Deep Creek

It is proposed to retain Deep Creek and its riparian zone in their current form. However, ongoing waterway management will occur in line with current and existing ecological and landscape requirements.

In line with current PSP proposals, the Deep Creek corridor is proposed to incorporate:

- A 100 m reserve east of Deep Creek upstream of Princes Highway, while downstream of Princes Highway it will be 100 m meandering down to 50 m near Canty Lane.
- A 50 metre reserve on the west of Deep Creek downstream of Princes Highway. An assumption that any future development west of Deep Creek upstream of Princes Highway would require a 50 m reserve to be consistent with the above. However, in the interim, the UFZ line (which is consistent with the declared flood plain line) could be assumed.

The proposed reserve encompasses the entire Deep Creek riparian vegetation and the existing Deep Creek Road (upstream of the Highway).

Stormy Water Solutions Report “Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, 5 October 2014” shows that the reserve detailed can accommodate future flows in Deep Creek provided:

- some flood plain shaping works are undertaken within the reserve allocation, and
- The Ryan Road culvert and road crossing is augmented.

The above report should be referred to for the full description of the above works.

Any future development must incorporate filling to required MWC standards adjacent to the creek to ensure adequate flood protection. At this stage fill requirements are assumed to be 600 mm above the flood levels determined given ultimate reserve requirements. Fill requirements may vary in the order of between 600 to 1200 mm adjacent to Deep Creek. It is proposed to grade the fill down to natural surface level over (say) 100 metres, creating, in effect, a very wide, flat levee adjacent to the creek. Preliminary flood levels are detailed within the above report.

The Deep Creek Princess Highway Culvert has enough capacity to convey the future and existing 100 Year ARI flow (See Appendix B).

5.5 Hancocks Gully

The existing straight drain will be remodelled as a 40 metre wide vegetated channel meandering within a 65 metre reserve. Figure 6 and Appendix A (SWS Drawing 1304/5) details the concept design of the Hancocks Gully vegetated channel. No significant existing vegetation is affected.

The alignment shown on the plans can change as required by the site development. However, if it is moved away from the “flood plain” valley low point, an alternative “valley floor” must be provided along this alignment (or the valley floor filled to protect the future development). The only fixed points are at the northern PSP boundary, the Princes Highway Culvert, the bypass culvert system and the railway culvert.

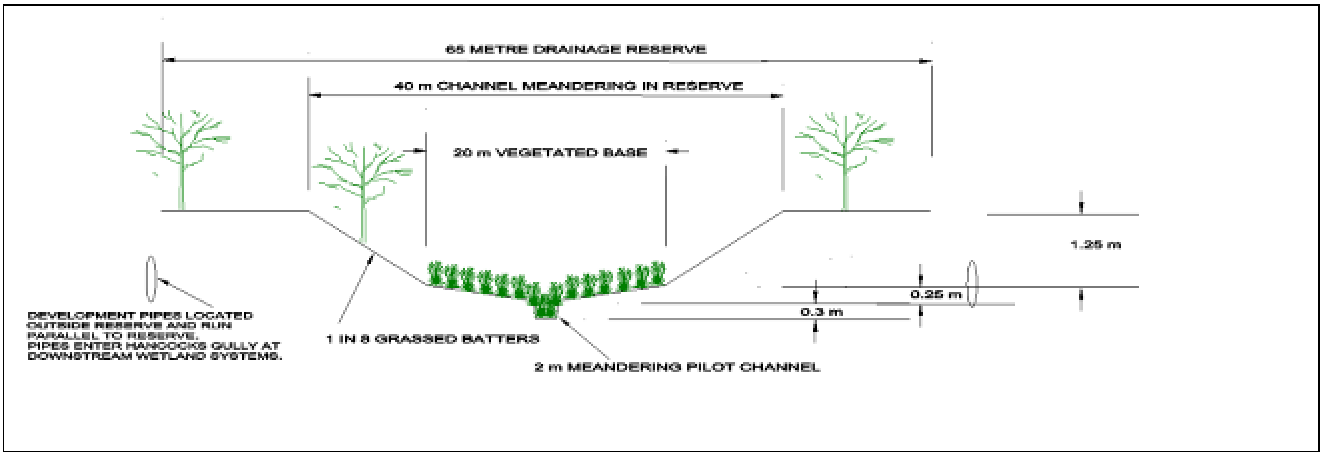

Figure 6 Proposed Hancocks Gully Vegetated Channel

The Princes Highway culvert, the Pakenham Bypass culvert and the railway culvert have enough capacity to convey the future and existing 100 Year ARI flows at these points (See Appendix B). However, the existing invert levels do form constraints in regard to the design of the vegetated upstream and downstream of all crossings.

Hec Ras modelling indicates that some filling may be required upstream of the railway culvert is Option 4 is implemented.

5.6 Proposed Retarding Basins and Wetlands

In line with the requirements set out in Section 3, retarding basins and wetlands are proposed to be combined at various locations in the PSP. These systems are:

- Retarding Basin/Wetland W1,
- Retarding Basin/Wetland W2 (Separated into Wetland W2A and Wetland/Retarding Basin W2 in Option 2, Wetland W2A and Wetland/Retarding Basin W2W4 in Option 3 and various sediment ponds and Wetland/Retarding Basin W2W4 in Option 4),

- Retarding Basin/Wetland W3,
- Retarding Basin/Wetland W4 (Separated in Wetland W4A and Wetland/Retarding Basin W4 in Option 2, Wetland W4A and Wetland/Retarding Basin W2W4 in Option 3 and various sediment ponds and Wetland/Retarding Basin W2W4 in Option 4)

The details of each system are detailed in SWS drawing set 1304 and Appendix A. Preliminary system sizes are detailed in Appendix A.

The drainage strategy allows for all development within the PSP area to be serviced via 5 Year ARI pipes discharging to four retarding basin/wetland systems described above.

All wetlands are off line from Deep Creek and will have no adverse impact of fish passage within the waterway itself.

All other wetlands have been designed at this time as “on line” to the pipe or small waterway (Hancock's Gully) off which they are feeding. This type of design is consistent with MW’s 2010 Wetland Design Guidelines. Also, it is understood, that they have also been allowed for in the draft MW 2014 Wetland Design Manual.

“On line” wetlands have been proposed (as opposed to off line systems incorporating bypass channels) as:

- Most wetlands are in retarding basins which tend to drown out bypass channel very regularly
- The council designs are responding to the site constraints such as:
    - Steep surrounding topography, the gas main location and level and the existing Princes Highway culvert inverts levels for Wetland W1,
    - The very high outlet level requirements for Wetland W2 and W4 required to ensure the existing GGF ponds can still be fed regularly with treated water (this means that a bypass channel would essentially have no slope through the site and be extremely hard to maintain), and
    - The gas main location and level (and it impact on the upstream pipe invert level design), the location of the highway and the existing Princes Highway culvert inverts levels for Wetland W3.

Notwithstanding the above, the functional design process should ensure that MW wetland velocity and vegetation inundation frequency requirements are adhered to. Notwithstanding the above, council has been very careful to slightly oversize the reserves shown and locate them where possibly adjacent to open space, waterway reserves etc. It is considered that there is enough space allocated within the FUS Plans to accommodate this aspect of the design going forward.

It was agreed with MWC at a meeting on the 7 August 2014 that the constraints on the site make the provision of wetlands off line from Hancocks Gully or pipe prohibitive. It was agreed that the sediment ponds will be completely offline. At this stage of the PSP all that is required is confirmation that the encumbered land provided for drainage infrastructure provides space to ensure that this sediment ponds can be provided offline. This has been confirmed. In addition to stormwater pollutant retention and providing provision for flood retention above NWL, the wetlands and sediment ponds will:

- Maximise the flood storage available in each site area as the horizontal wetland base maximizes the flood storage in the site area available.
- Allow for a relatively deep inlet drainage pipe invert level at Wetland W3 (as defined by the wetland normal water level) and thus allow for the upstream drainage pipes to be constructed under the existing gas infrastructure,
- Allow for a relatively high outlet drainage invert level at Wetland W1 (as defined by the wetland normal water level) and thus allow for the outlet culvert to be constructed over the existing gas infrastructure,
- Consist of self sustaining WSUD elements which, by definition, minimise the maintenance requirements of individual WSUD elements ,
- Have elements which maximise ecological diversity in the urban landscape (including the use of open space adjacent to developments), and
- Supplement the social and landscape amenity of any future development.

5.7 Sediment Ponds and Litter Traps

Off line sediment basins (Appendix A) will be required at pipe outfall locations into the retarding basin/wetland systems within the PSP. Depending on the configuration of other options more sediment ponds may be required downstream. Sediment ponds protect the wetland systems and vegetated channel systems from course sediment deposition. These elements are required to be detailed at the functional/detailed design stage of the project. However, the plans do detail the approximate required sizes of these systems given preliminary sizing calculations.

Litter traps will be required at the downstream end of the local drainage system prior to entering the sediment ponds.

It is assumed that the existing dam within the electricity easement may collect some sediment from the external catchment EXT 2. However, this treatment has not been accounted for in the MUSIC modelling and essentially it is proposed to remodel this element as Growling Grass Frog habitat.

5.8 Other WSUD Initiatives

Other WSUD initiatives can be considered by Council or individual developers. These elements would supplement the drainage strategy elements advocated above, but are not strictly required to meet the existing objectives of any site drainage principles. However, given consideration of expanded water quality requirements (see Section 4.3 above) additional elements which could be considered such as:

- Local melaleuca swamps possibly located in the extended base systems of the vegetated channels proposed,
- Lot and site scale rainwater tanks for internal and external lot use, small public open space irrigation etc,
- Incorporation of sustainable landscaping practices to minimise irrigation within the development,
- Grey water reuse within the development, and
- Use of recycled sewage water from the local South East Water sewage treatment plant.

Notwithstanding the above this report does consider (for discussion purposes) the possible benefit of regional stormwater harvesting if applied to Options 1 and 4 (see Section 8.2).

6. Hydrological Modelling

Hydrological Modelling using the RORB model was undertaken for this study by SWS to estimate flood flows and retarding basin requirements. This analysis is documented in Appendix C.

The hydrological modelling was undertaken to provide realistic site delineation of the retarding basin areas required to meet all objectives. In addition, the modelling provides design flows for the vegetated channels proposed.

The hydrological analysis is considered preliminary only at this stage. The final RORB model developed for the chosen strategy may differ from those models detailed in Appendix C.

The RORB model for Options 1, 2 and 3 were developed in February 2013. The RORB model for Option 4 was developed in February 2014, and given a slight review of the catchment, the RORB parameters and flow results may differ slightly from the 2013 analysis results detailed.

It should be noted that wetlands are proposed in each retarding basin modeled. This is a crucial aspect to the strategy design, as the horizontal wetland base maximizes the flood storage in the site area available.

As such, the concept designs have been formulated given iterations between the pollutant model (MUSIC) and the hydrological model (RORB). The retarding basin areas/volumes required to meet the flood retardation requirements result in wetland areas are able to achieve (and sometimes exceed) the 80%/45%/45% retention objectives of TSS, TP and TN respectively.

The retarding basins modeled were:

- Option 1 - four retarding basins/wetland systems (W1, W2, W3, and W4) incorporating Stage/Storage/Discharge relationships as detailed in Appendix D,
- Option 2 allows for four retarding basins/wetland systems (W1, W2, W3, and W4) incorporating Stage/Storage/Discharge relationships as detailed in Appendix E,
- Option 3 - three retarding basins/wetland systems (W1, W3, and W2W4) incorporating Stage/Storage/Discharge relationships as detailed in Appendix F,
- Option 4 - three retarding basins/wetland systems (W1, W3, and W2W4) incorporating Stage/Storage/Discharge relationships as detailed in Appendix G.

All retarding basin models consider the actual site contours and outlet conditions as predicted by design drawings or the 1 metre Lidar information available. They are considered concept designs only at this stage.

As detailed in Appendix C all options can ensure in:

- The peak 100 Year flow from the future development does not exceed the predevelopment flow rate at all catchment outlet points to receiving bodies,
- The 24 hour 100 Year flow from the future development does not exceed the predevelopment flow rate for a storm of this duration at outlet points to receiving bodies, and
- Each retarding basin can store the difference between the expected post development and predevelopment 24 hour 100 Year flow volume.

As such, the analysis suggests the retarding basin/wetland initiatives detailed in all options can ensure no increase in downstream 100 Year ARI flood flows and no increased flood effect within the KWRFPD during a 24 hour 100 Year ARI flood event.

7. Vegetated Channel Design

In the original 2013 work, the proposed vegetated channel cross section as detailed in Section 5.5 was modelled using PC Convey to estimate the capacity of the vegetated channel. This previous analysis suggested that a 40 metre channel meandering in a 65 metre reserve was capable of containing the 100 Year ARI flow for all vegetated channels proposed.

In addition, a Hec Ras model was undertaken (in March 2014) of the combined flow at the railway (the location where the maximum flow can occur in the channel system in Option 4). This included modelling the vegetated channel upstream and downstream of the railway. This analysis confirmed the section detailed should be capable of conveying the 100 year ARI flow (See Appendix B). However, some filling of land may be required (in option 4) adjacent to this channel upstream of the railway due to the crossing backwater effects. Largely however, the 100 year flow can be contained within the channel.

The proposed drainage reserve is 65 metres. Slight shaping of the reserve could provide additional freeboard or changes to design proposals going forward.

A detailed Hec Ras model of the final vegetated channel/reserve form should be completed at the functional design stage of the project to ensure adequate fill/floor level requirements are set on adjacent development (if required). A decrease in drainage reserve requirements may also be considered, if applicable, in this future work.

It should be noted that development pipes will run parallel to the drainage reserve until discharge into the off line sediment pond upstream of the wetland systems. In this way multiple outfall to all vegetated channels, including the remodelled Hancocks Gully, can be avoided.

8. Stormwater Pollutant Modelling

8.1 Meeting current Best Practice Requirements – All Options

The performance of the stormwater quality management system outlined in Section 5 and detailed in Appendix A was analysed using the MUSIC model, Version 5. This analysis is detailed in Appendix H.

The analysis shows that the pollutant reductions exceed the current best practice requirements of 80% TSS, 45% TP and 45% TN for all options. In fact, in many cases, the size of the wetland (often determined to meet retarding basin requirements) results in the pollutant requirement of 85%/50%/50% retention of TSS, TP and TN being met, or very nearly met.

8.2 Meeting F8 Requirements – Option 1 and 4

- Meeting the SEPP Schedule F8 objectives (93% TSS, 66% TP and 63% TN) is a consideration at this stage. As such, further investigation of Options 1 and 4 in regard to meeting this objective within, or very close to, the PSP area has been undertaken.
- Two scenarios have been investigated for each option above. These are:
- Meeting F8 via increasing stormwater treatment element size (i.e. wetland size), and
- Meeting F8 requirements by incorporating a regional stormwater harvesting scheme at the all or some of the major retarding basins. This scenario assumes that a large demand for the water (in the order of 2000 ML/yr) is required, probably via irrigation within the KWRFPD to the south. Also, a scheme of this sort must ensure adequate downstream environmental flow rates, although preliminary calculations indicate this can be achieved.
- Both scenarios result in additional land take and asset size requirements. Although the harvesting scenario results in less land take, costs such as providing reticulation schemes to those who want/need the water may be a considerable cost consideration. The results of the analysis are detailed below.

8.2.1 Option 1

Figures 7 and 8 detail the approximate land take requirements (under the above two scenarios) to meet F8 requirements under Option 1. Table 1 summarises the results

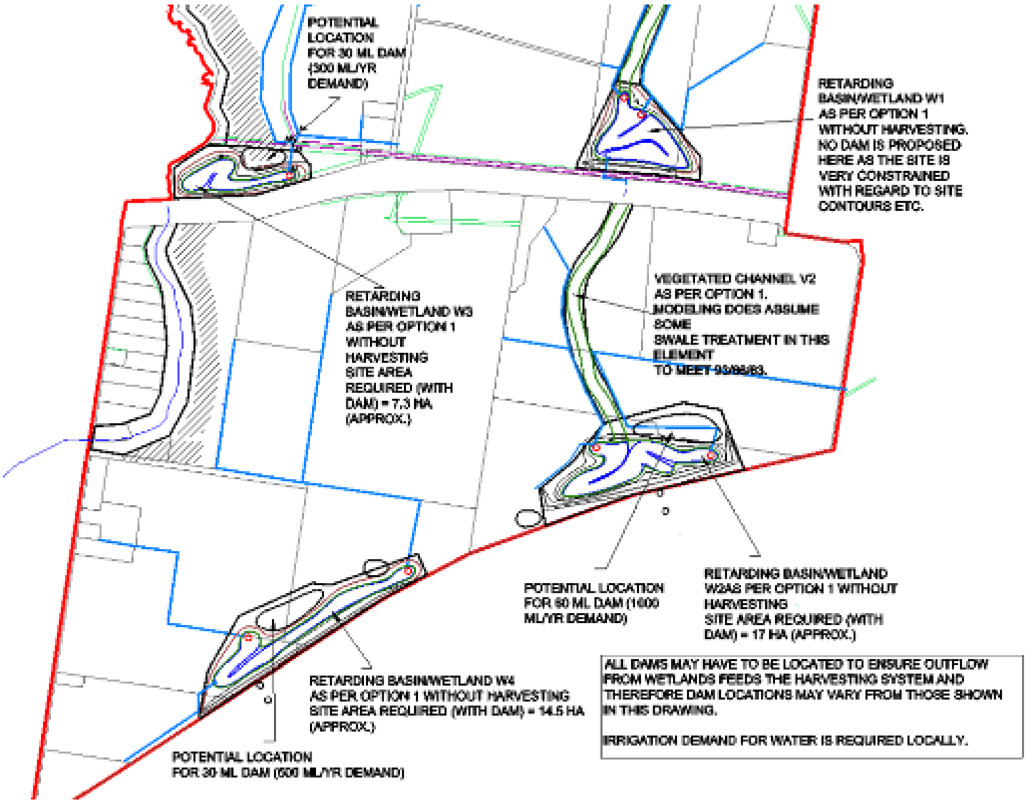

Figure 7 Approximate land take required to meet F8 requirements with regional stormwater harvesting

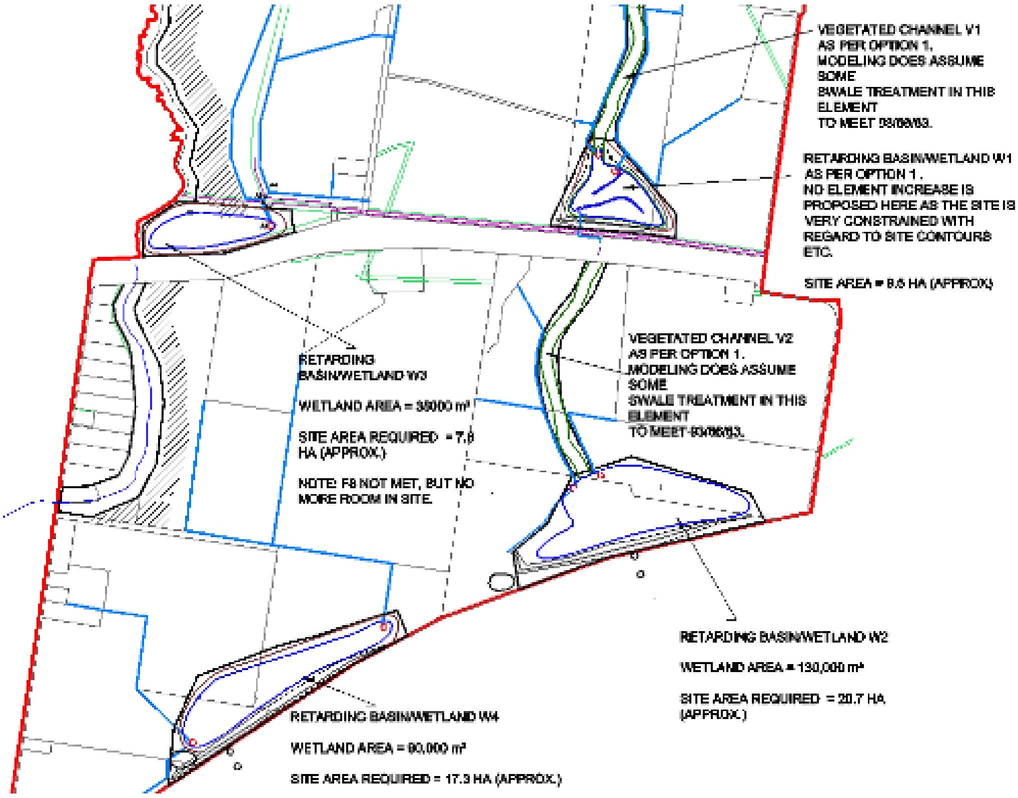

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Figure 8 Approximate land take required to meet F8 requirements with increased wetland size

Table 1 Land Take Comparisons Given Different Stormwater Water Quality requirements

Retarding basin/Wetland system	Wetland area as designated in Appendix A[1]		System area required to meet F8 with stormwater Harvesting[2,6]			System area required to meet F8 with increased wetland areas[6]	
	Wetland area (m^2)	Site Area (ha)	Wetland area (m^2)	Dam area[3] (m^2)	Site Area (ha)	Wetland area (m^2)	Site Area (ha)
W1[4]	41,800	9.5	41,800	0	9.5	41,800	9.5
W2	45,400	15	45,400	20,000	17	130,000	20.7
W3[5]	19,600	6	19,600	10,000	7.3	38,000	7.8
W4	39,150	12.5	39,150	10,000	14.5	90,000	17.3
TOTAL	145,950	43	145,950	40,000	48.3	299,800	55.3

1 - Wetland area has duel flood retardation and water quality benefits (current best practice in regard to stormwater pollutant retention met)
2 - Assumed regional KWRFPD irrigation demands (scaled by PET) -W2 = 1000 ML/yr, W3 = 300 ML/yr, W4 = 500 ML/yr,
3 - Assumes dam 3 metres deep to achieve enough volume for 75 - 80 % reliability of supply
4 - No harvesting is assumed to occur from W1 as the site area is constrained given the local contours etc.
5 - Only 59% TN retention can be achieved with increased wetland size scenario in site area available (given contours and gas line constraints)
6 - Assumes that vegetated channels can be modelled as swales in regard to stormwater treatment for 93/66/63 requirement to be met

8.2.2 Option 4

Figure 9 details the approximate land take required to meet current best practice and the SEPP F8 requirements under the above two scenarios for Option 4. The stormwater harvesting scenario assumes an annual irrigation demand of in the order of 2000 ML/yr (scaled by PET over the year).

Figure 9 **Approximate landtake required to meet 93%TSS, 66%TP and 63% TN at W2W4 (Option 4) compared to the landtake required to meet current best practice**

Note: *W3 treatment included as per Figure 7 and 8 land take for Option 4*
W1 treatment included as per Option 1 land take due to site constraints limiting any increase in system size.

8.3 Conclusions

It should be noted that the above analysis in regard to increased land take required to meet F8 requirements is considered very high level at this stage. However, the following conclusions can be drawn in regard to stormwater pollutant retention:

- All options can meet current best practice,
- Meeting SEPP F8 requirements using stormwater harvesting would require:
  - An increased land take between about 5 – 15% depending on how the irrigation dam can be located within the affected retarding basin/wetland sites,
  - Major dam and water reticulation infrastructure, and
  - A “real” irrigation demand in the order of 2000 ML within the KWRFPD,
- Meeting SEPP Schedule F8 requirements using larger wetland area results in:
  - wetland sizes generally increasing in the order of 250% within affected sites, and
  - land take requirements (considering all retarding basins and wetlands) generally increasing in the order of 25 - 30%. Note that the increase in required wetland size does not result in a similar increase in overall site size due to the fact that, in the base case, some site area required for flood storage is not affected by the wetland footprint. In meeting SEPP Schedule F8 requirements these areas can now have a duel use as wetland treatment.

Of course, subdivisional, road and site scale WSUD and stormwater harvesting initiatives would supplement any DSS scale WSUD elements as detailed above.

In addition, future regional wetland systems located in the Cardinia Creek outfall downstream may be able to supplement and local treatment initiatives, although this regional analysis has not been undertaken at this stage.

As such, the above land take estimates in regard to meeting SEPP F8 requirements within, or close to the PSP, are probably a worst case scenario in regard to the land take implications for the major drainage assets.

The analysis is presented at this time to provide some guidance to Council and other interested parties in regard to this issue going forward.

9. Conclusions and Further Work Required

The stormwater drainage system proposed for the Pakenham East PSP represents a strategy development covering all requirements of best practice floodplain and catchment management.

It is recommended that Council review the four options detailed in this report in the context of other planning implications and constraints. It is anticipated that one of the four options will be chosen for further development into the final PSP drainage strategy plan. Consultation with MWC, the VPA, affected landowners and other stakeholders should be ongoing during this process.

Further work required going forward in the design process includes, but is not limited to:

- Fully understanding the soil and groundwater constraints in the area with regard to drainage asset design and construction issues etc,

- Confirming that the ecological and cultural heritage constraints and opportunities have been fully captured by the drainage strategy and engaging appropriate professionals to review this draft drainage strategy to determine the exact ecological/heritage impacts and constraints in regard to the preliminary proposals,

- Confirming the Department of Transport requirements south of the PSP area,

- Liaising with downstream affected landowners,

- Obtaining agreement with MWC in regard to the reserve requirements adjacent to Deep Creek as per the recommendations within the Stormy Water Solutions Report “Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, 5 October 2014”,

- Determination of future flood levels adjacent to Deep Creek (given the ultimate reserve requirements and floodplain augmentation design) to finalise fill requirements of adjacent land,

- Obtaining agreement with MWC in regard to the drainage reserve requirements and remodelling of Hancocks Gully,

- Producing a detailed Hec Ras model of the final vegetated channel/reserves forming the remodelled Hancocks Gully (and other vegetated channels proposed), at the functional design stage of the project) to ensure adequate fill requirements are set on adjacent development (if required).

- Obtaining clearance confirmation of the concept design for the two gas line crossings is required to be obtained from APA GasNet Australia (Operations) Pty Ltd.

- Conducting a full water balance of all wetland systems (at the functional design stage of the project) to ensure water body turnover periods are sufficient to result in self-sustaining systems over time,

- Ensuring a full design of all sediment pond systems (given final development catchment configurations) to ensure full consideration of MWC cleanout regimes and safety requirements, and

- Possibly assessing other ways to meet SEPP Schedule F8 requirements in regard to the impact of the PSP development at Westernport Bay (e.g. using other regional WSUD elements located well downstream of the PSP area treating not only this PSP area but other developed catchments as well).

It should be noted that the assumptions in regard to sediment pond, wetland and vegetated channel design may change over time, especially given MWC's soon to be released updated waterway design guidelines and wetland design guidelines. However, it is considered at this stage, that the work presented has defined realistic and adequate potential land take requirements required by major drainage assets for each option proposed.

As discussed above, Rev C (April 2014) of this report resulted in the formulation of functional designs for stormwater assets generally in line with Option 1. The functional designs of these elements, although dated earlier than this revision of this report, supersede the details of the designs proposed in this report.

10. Abbreviations and Definitions

The following table lists some common abbreviations and drainage system descriptions and their definitions which are referred to in this report.

Abbreviation Descriptions	Definition
AHD - Australian Height Datum	Common base for all survey levels in Australia. Height in metres above mean sea level.
ARI - Average Recurrence Interval.	The average length of time in years between two floods of a given size or larger
BoM	Bureau of Meteorology
DoT	Department of Transport
Evapotranspiration	The loss of water to the atmosphere by means of evaporation from free water surfaces (e.g. wetlands) or by transpiration by plants
Groundwater	All water stored or flowing below the ground surface level
Inlet Pond	See Sediment Pond
Hectare (ha)	10,000 square metres
Hec Ras	A one dimensional, steady state hydraulic model which uses the Standard Step Method to calculate flood levels and flood extents
Kilometre (km)	1000 metres
KWRFPD	Koo Wee Rup Flood Protection District
m^3/s -cubic metre/second	Unit of discharge usually referring to a design flood flow along a stormwater conveyance system
Megalitre (ML) (1000 cubic metres)	1,000,000 litres = 1000 cubic metres Often a unit of water body (eg pond) size
MUSIC	Hydrologic computer program used to calculate stormwater pollutant generation in a catchment and the amount of treatment which can be attributed to the WSUD elements placed in that catchment. Can also be used to calculate water body turnover period and wetland draw downs etc
NWL	Normal Water Level – invert level of lowest outflow control from a wetland or pond.
PET	Potential Evapotranspiration – potential loss of water to the atmosphere by means of evaporation or transpiration from wetland or pond systems.
Retarding Basin	Drainage element used to retard flood flows to limit flood impacts downstream of a development. Can include complementary WSUD and ecological site benefits if wetland incorporated within the site.
RORB	Hydrologic computer program used to calculate flood flows (m^3/s) and size retarding basins
Sedimentation basin (Sediment pond)	A pond that is used to remove coarse sediments from inflowing water mainly by settlement processes.
SEPP	State Environment Protection Policy
Surface water	All water stored or flowing above the ground surface level
TED	Top of Extended Detention – Level to which stormwater is temporarily stored for treatment in a wetland or pond (above NWL).
TSS	Total Suspended Solids – a term for a particular stormwater pollutant parameter
TP	Total Phosphorus – a term for a particular stormwater pollutant parameter
TN	Total Nitrogen – a term for a particular stormwater pollutant parameter
Wetland	WSUD elements which is used to collect TSS, TP and TN. Either permanently or periodically inundated with shallow water and either permanently or periodically supports the growth of aquatic macrophytes

APPENDIX A Proposed Drainage Strategy Options

The details of the options and element sizes detailed below are preliminary only and subject to change.

The aim of the drawings and calculations undertaken to date is to ensure that there is enough space allocated within (and possibly downstream) of the PSP to ensure all drainage requirements are met going forward.

The location, size, alignments and shapes of all elements are subject to change during the design process.

Notwithstanding the above it should be noted that:

- All options allow for development of small catchments individually (upstream of W1, W2, W3 or W4) without cleanout/enlargement works downstream,
- Enough work has occurred to ensure that upgrading of any highway or Pakenham Bypass culvert system is **not** required, although existing sizes and invert levels do set constraints on upstream drainage infrastructure and must be fully understood during the design process,
- The drainage design can accommodate existing gas line alignments and levels (clearance confirmation required by approval authority),
- All options allow for piped catchments discharging to regional WSUD elements, however additional stormwater harvesting or site/street scale WSUD initiatives will improve system performance.
- Two outlets to Deep Creek are required to minimise catchment sizes and help with strategy implementation,
- All options must ensure local catchment diversion into Deep Creek at PSP northern boundary and at Princes Highway, and
- All options allows for new frog ponds to be sited close to wetland systems.

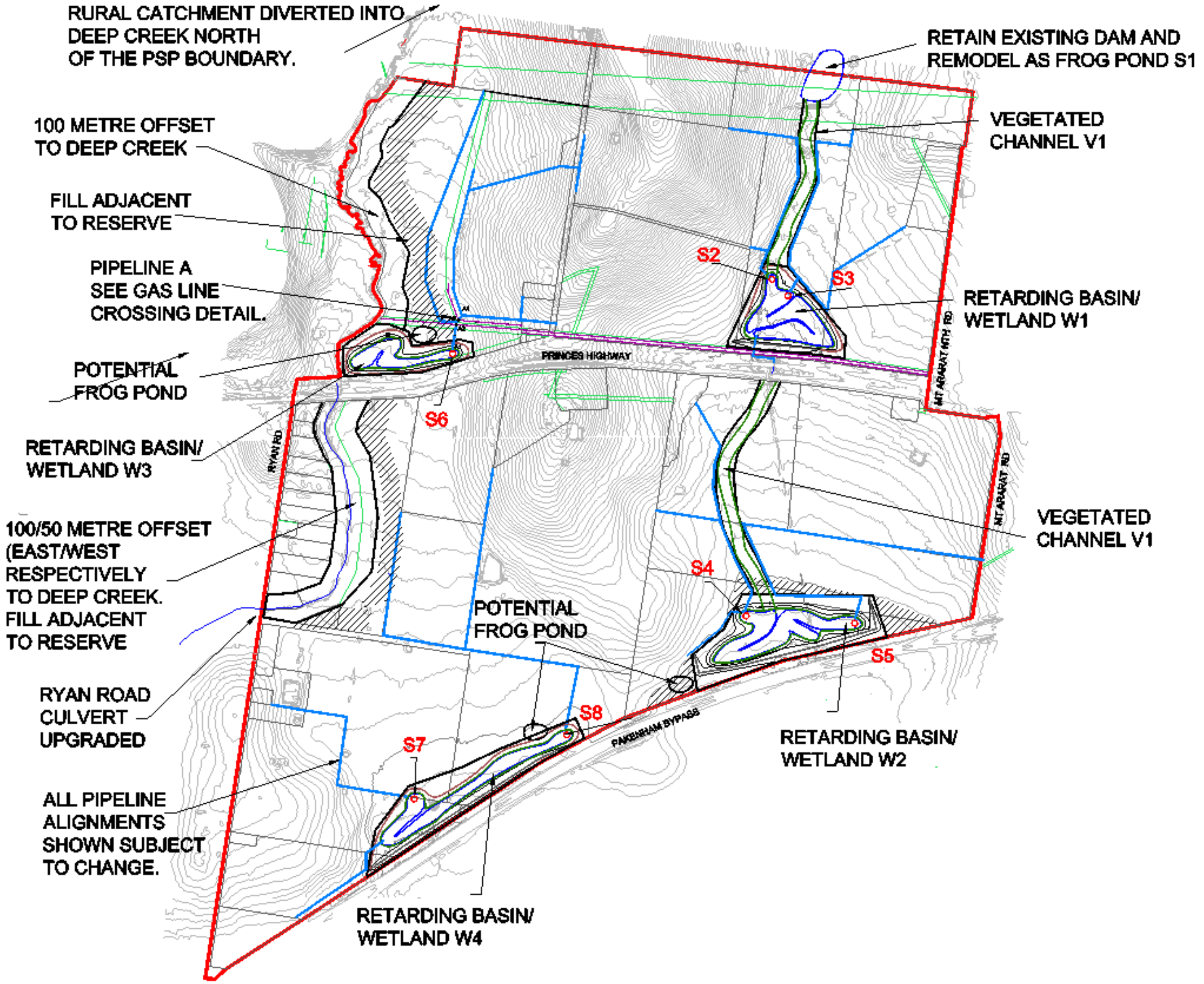


Figure A.1 Option 1 Drainage Strategy – See SWS Drawing 1304/1 for more detail

This drawing shows the land take required to meet 80%/45%/45% pollutant retention and KWRFPD flood retardation requirements within the PSP boundary

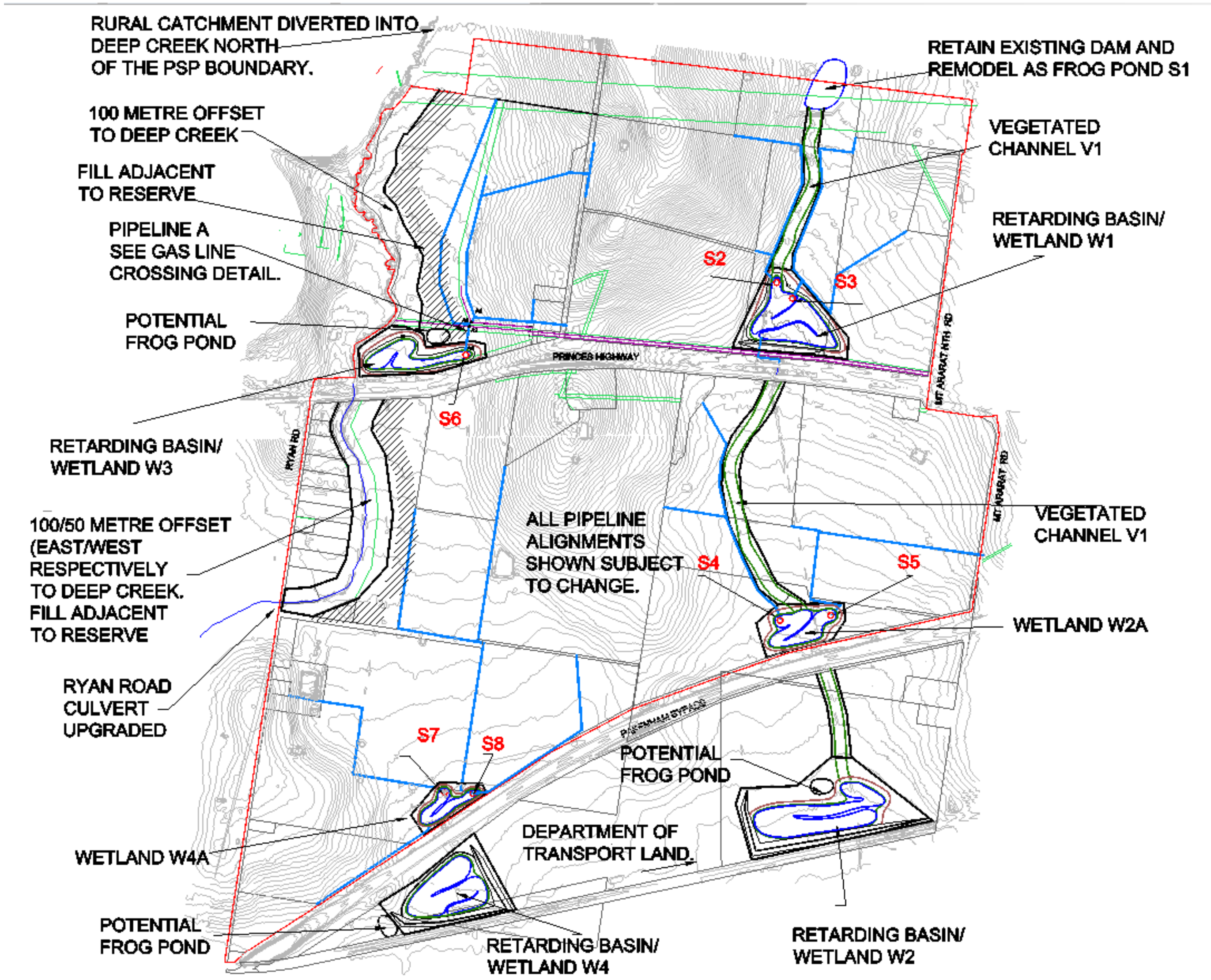


Figure A.2 **Option 2 Drainage Strategy – See SWS Drawing 1304/2 for more detail**

This drawing shows the land take required to meet 80%/45%/45% pollutant retention and KWRFPD flood retardation requirements upstream of the railway
Land around W2 (excluding DoT land) is assumed to be retained as rural.

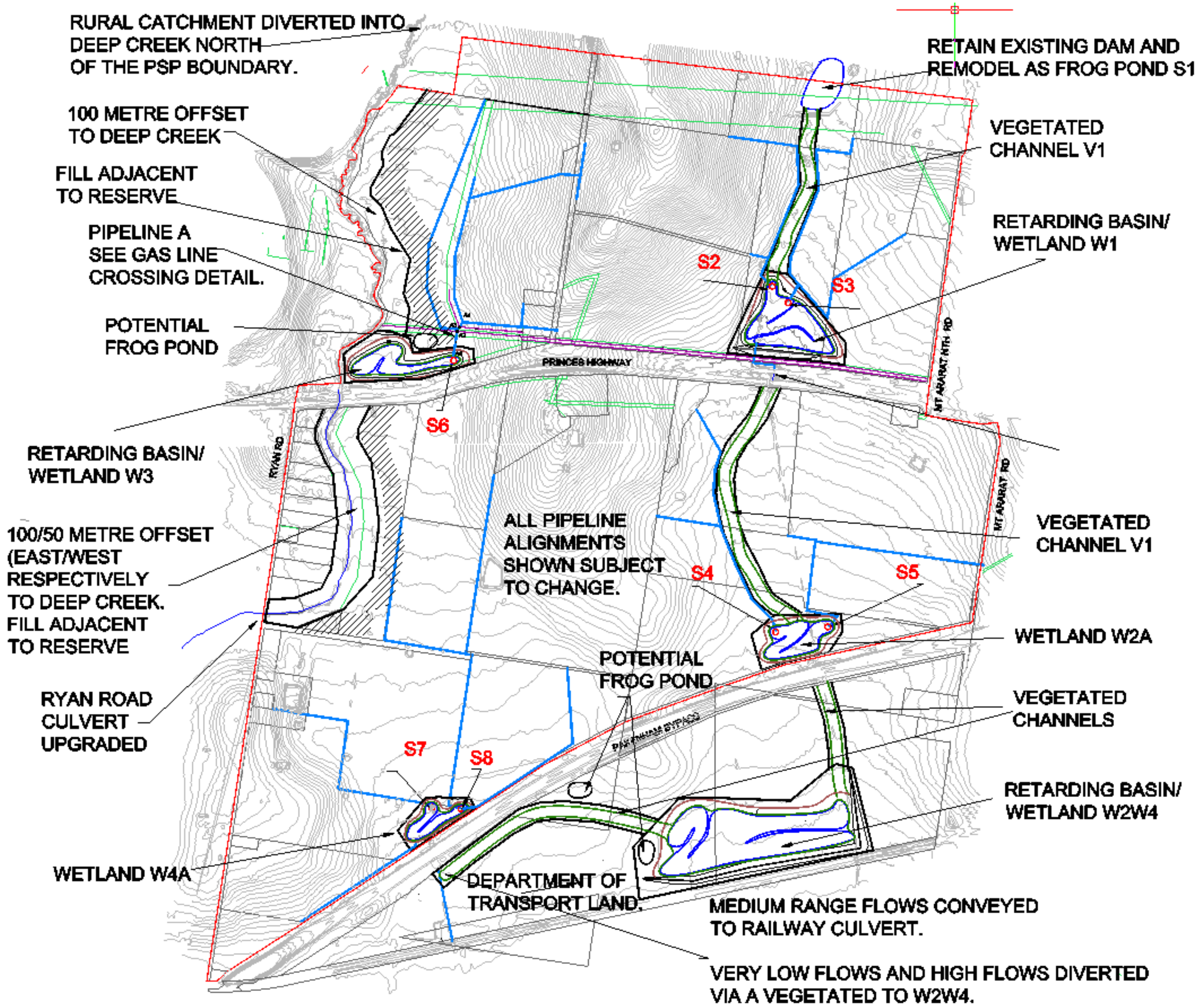


Figure A.3 **Option 3 Drainage Strategy – See SWS Drawing 1304/3 for more detail**
This drawing shows the land take required to meet 80%/45%/45% pollutant retention and KWRFPD flood retardation requirements upstream of the railway
Land around W2 (excluding DoT land) is assumed to be retained as rural.

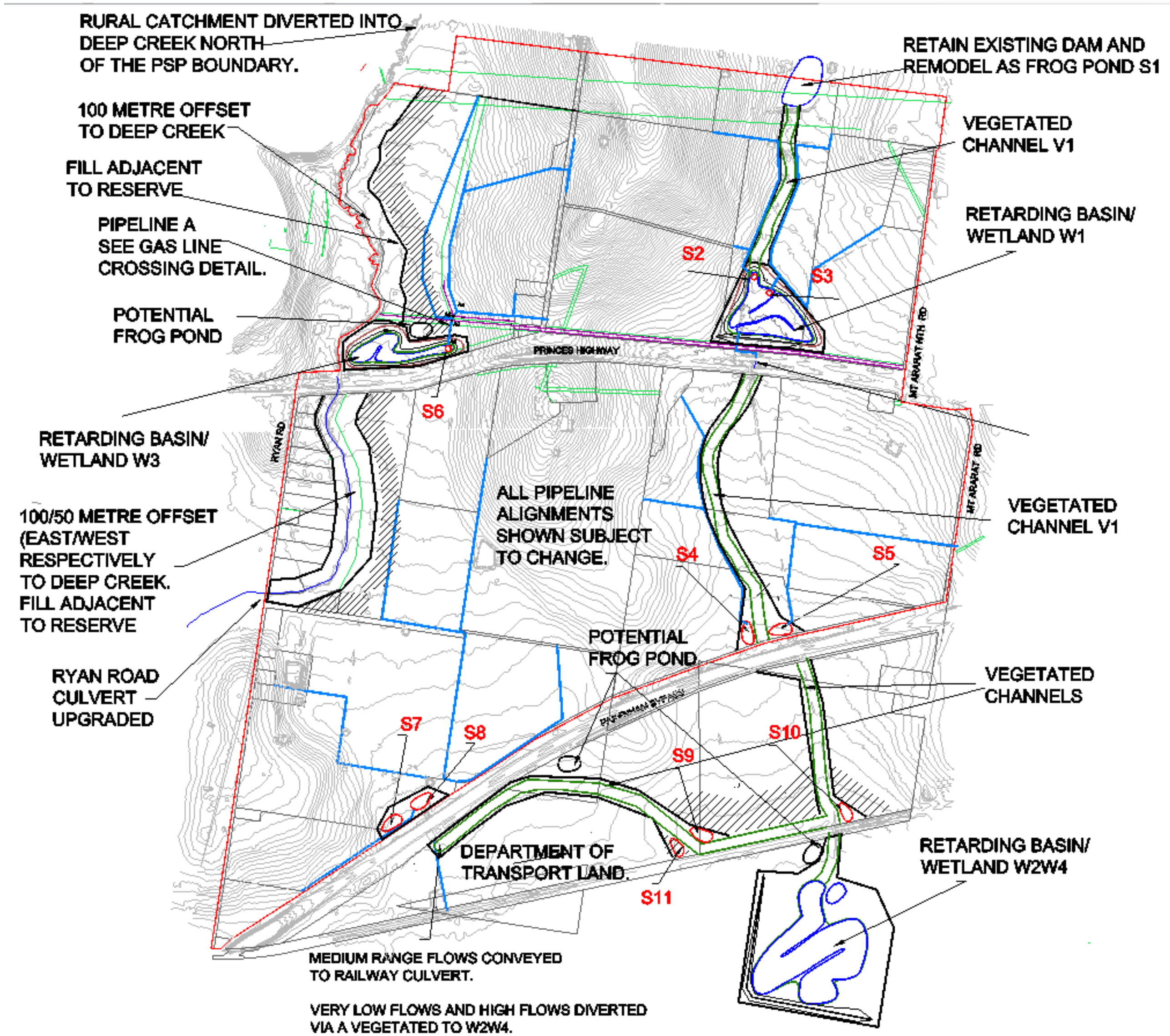
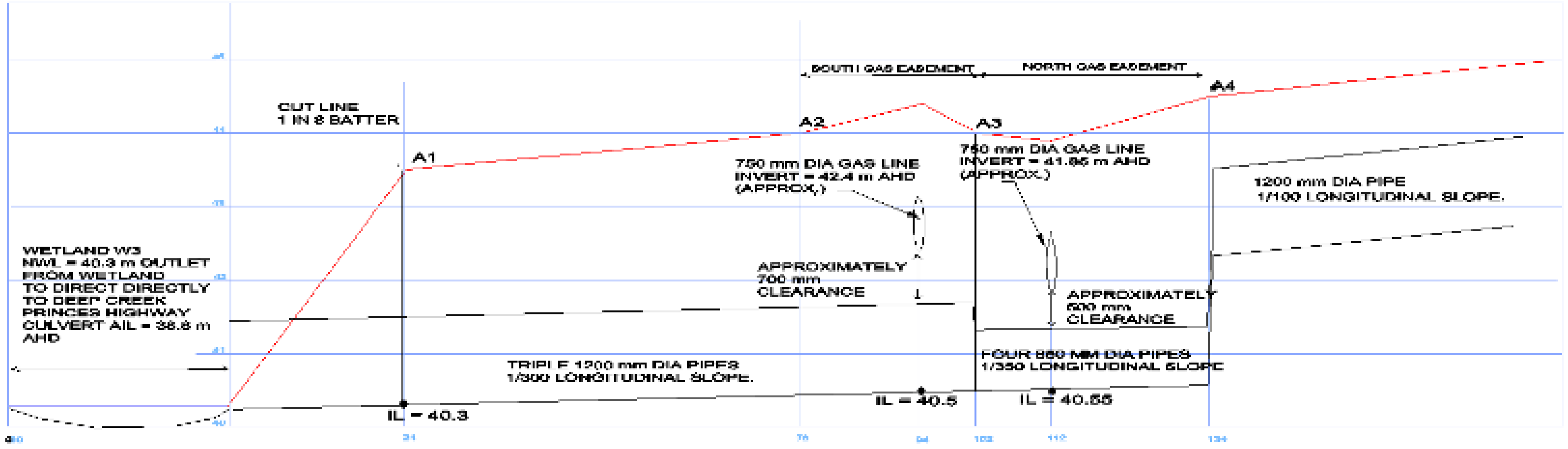


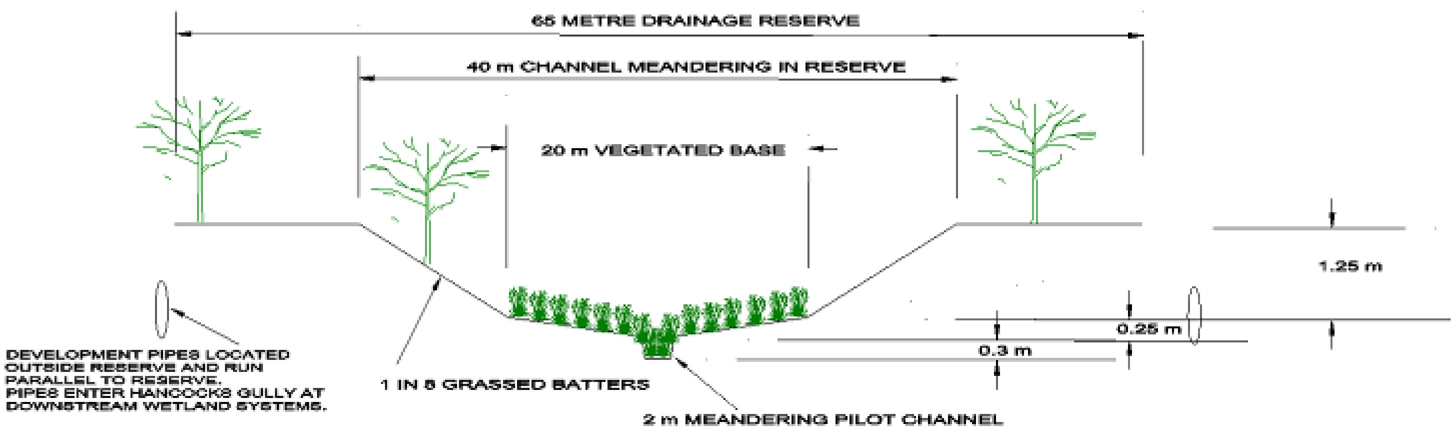
Figure A.4 Option 4 Drainage Strategy – See SWS Drawing 1304/4 for more detail

This drawing shows the land take required to meet 80%/45%/45% pollutant retention and KWRFPD flood retardation requirements to just downstream of the railway
All land between the railway and the Pakenham bypass (including DoT land) is assumed to be developed in the future.

CONCEPT DESIGN ONLY.
ACTUAL GAS LINE LEVELS AND LOCATIONS MAY VARY FROM THOSE DETAILED. GAS LINE TO BE PROVEN AND MODIFICATION MADE AS REQUIRED.



PIPELINE A CONCEPT DESIGN



HANCOCKS GULLY VEGETATED CHANNEL CONCEPT DESIGN

Figure A.5 Inlet Pipe Requirements to W3 given Gas Line Constraints – See SWS Drawing 1304/5 for more detail

Drainage Element	Description	Primary Objectives	Characteristics	Topography considerations	Environmental/Cultural Heritage Considerations
Frog Pond S1	Remodelled dam located within electricity easement on Hancocks Gully	Retain for potential ecological benefits			No frogs identified to date.
Vegetated Channel V1	Remodelled Hancocks Gully.	Ensure 100 Year capacity within drainage channel. Provide ecological corridor and passive recreation opportunities (bicycle paths etc).	40 metre wide vegetated channel meandering within a 60 metre reserve. See 1304/5 for typical cross section	Strategy proposes west boundary of channel reserve on toe of hill. Channel splits two catchments east and west of the reserve. Spine pipelines from subdivision to run parallel with reserve before being discharged separately into off line sediment ponds.	Existing vegetation along drainage reserve is of little ecological value and can be removed.
Retarding Basin/Wetland W1	Major drainage element located on Hancocks Gully directly north of Princes Highway	Retard peak flows to predevelopment flow rates (1 yr, 100 year (peak) and 100 year (24hr)) and treat stormwater pollutants to at least best practice (W1 and W2 combined).	OPTIONS 1 - 3 NWL = 39.2 m AHD (41,800m^2) TED = 39.5 m AHD (48,650 m^2) 100 yr flood level = 40.7 m AHD 100 yr flood volume = 86,700 m^3 .... OPTION 4 NWL = 39.0 m AHD (33,300m2) TED = 39.5 m AHD (41,850 m2) 100 yr flood level = 40.5 m AHD 100 yr flood volume = 59,200 m3 Modified Option 4 to maximise outflow given downstream road culvert capacity and to allow sediment ponds S2 and S3 to be isolated from wetland for maintenance.	Site relatively steep. Outlet system and NWL set by Princes Highway culvert invert level constraints and gas line constraints. Outlet pipe invert varies between 37.5 and 37.0 m AHD. North Gas line - 36.4 m AHD (top). South gas line = 35.8 m AHD (top). Outlet required to go over gas line.	Existing vegetation along drainage reserve is of little ecological value and can be removed.

Drainage Element	Description	Primary Objectives	Characteristics	Topography considerations	Environmental/Cultural Heritage Considerations
Sediment Pond S2	Off line sediment pond treating course sediment developed in local catchment to the west of Vegetated Channel V1	Local catchment course sediment collection to current MWC standards prior to stormwater entry into W1	2500 m^3. WL and TED as per W1	See W1.	See W1.
Sediment Pond S3	Off line sediment pond treating course sediment developed in local catchment to the east of Vegetated Channel V1	Local catchment course sediment collection to current MWC standards prior to stormwater entry into W1	2500 m^3. NWL and TED as per W1	See W1	See W1
Vegetated Channel V2	Remodelled Hancocks Gully.	Ensure 100 Year capacity within drainage channel. Provide ecological corridor and passive recreation opportunities (bicycle paths etc).	40 metre wide vegetated channel meandering within a 60 metre reserve. See 1304/5 for typical cross section	Strategy proposes channel reserve along natural valley. Channel splits two catchments east and west of the reserve. Spine pipelines from subdivision run parallel with reserve before being discharged separately into off line sediment ponds S4 and S5.	Very little existing vegetation along drainage reserve.
Retarding Basin/Wetland W2 - Option1 .... Wetland W2A - Options 2 and 3 .... Sediment Ponds S4 and S5 - Option 4	Major drainage elements located on Hancocks Gully directly north of Pakenham Bypass	Option 1 - Retard peak flows to predevelopment flow rates (2 yr, 100 year (peak) and 100 year (24hr)) and treat stormwater pollutants to at least best practice (W1 and W2 combined). .... Options 2 and 3 - Provide some flood storage (given culvert capacities downstream) and provide sediment pond/wetland treatment. .... Option 4 - Provide some flood storage given downstream culvert capacities and sediment collection	W2 -Option 1 - NWL = 28.2 m AHD (45,400 m^2), TED = 28.7 m AHD (52,500 m^2), 100 yr flood level = 30.5 m AHD, 100 yr flood volume = 135,000 m^3. Pipe and headwall outlet to minimise system NWL given downstream frog pond constraints. .... W2A- Option 2 and 3 - NWL = 28.2 m AHD (22,000 m^2), TED = 28.7 m AHD (26,800 m^2), some flood retardation function .... Option 4 - some flood retardation function, sediment ponds S4 and S5 as below	Flat Site. Outlet system and NWL set by bypass frog pond NWL, rather than the Pakenham Bypass culvert invert levels. Relatively high NWL results in fill required around asset in Option 1 to ensure 100 year protection to surrounding properties.	Frog pond located in bypass reserve retained and outlet design accounts for current inflow requirements.

Drainage Element	Description	Primary Objectives	Characteristics	Topography considerations	Environmental/Cultural Heritage Considerations
Sediment Pond S4	Off line sediment pond treating course sediment developed in local catchment to the west of Vegetated Channel V2	Local catchment course sediment collection to current MWC standards prior to stormwater entry into W2	3000 $m^{3.}$ NWL and TED as per W2	See W2	See W2
Sediment Pond S5	Off line sediment pond treating course sediment developed in local catchment to the east of Vegetated Channel V2	Local catchment course sediment collection to current MWC standards prior to stormwater entry into W2	3500 $m^{3.}$ NWL and TED as per W2	See W2	See W2
Diversion Point	Diversion of small rural catchment to Deep Creek at northern PSP boundary	To ensure W3 only treats PSP runoff and to maintain existing Deep Creek Flow regimes	Use existing outfall to contain rural flows north of the PSP and divert these flows to Deep Creek.	Some minor shaping of the existing diversion drainage line (small eroded tributary) may be required	To be assessed
Retarding Basin/Wetland W3	Major drainage element located to the east of Deep Creek directly north of Princes Highway	Retard peak flows to predevelopment flow rates (2 yr, 100 year (peak) and 100 year (24hr)) and treat stormwater pollutants to at least best practice. Divert sub catchment into Deep Creek at this point as currently occurs.	Option 1, 2, 3 and 4 - NWL = 40.3 m AHD (19,600 m^2) TED = 40.8 m AHD (23,500 m^2) 100 yr flood level = 42.2 m AHD 100 yr flood volume = 53,900 m^3 Pipe and headwall outlet to minimise system NWL, given gas and downstream invert constraints.	Flat Site. Outlet system and NWL set by consideration of both highway culvert invert level and pipe requirements upstream (in regard to crossing twin gas lines). See longitudinal section. No embankment required.	Existing grassland on site has little existing value
Sediment Pond S6	Off line sediment pond treating course sediment developed in local catchment feeding into W3	Local catchment course sediment collection to current MWC standards prior to stormwater entry into W3	3200 $m^{3.}$ NWL and TED as per W3	See W3	See W3

Drainage Element	Description	Primary Objectives	Characteristics	Topography considerations	Environmental/Cultural Heritage Considerations
Retarding Basin/Wetland W4 - Option1 .... Wetland W4A - Options 2 and 3 .... Sediment Ponds S7 and S8 - Option 4	Major drainage element located on the Western Tributary directly north of Pakenham Bypass	Option 1 - Retard peak flows to predevelopment flow rates (2 yr, 100 year (peak) and 100 year (24hr)) and treat stormwater pollutants to at least best practice (W1 and W2 combined). .... Options 2 and 3 - Provide some flood storage (given culvert capacities downstream) and provide sediment pond/wetland treatment. .... Option 4 - Provide some flood storage given downstream culvert capacities and sediment collection	Option 1 - NWL = 28.5 m AHD (39,150 m^2), TED = 29.0 m AHD (46,400 m^2), 100 yr flood level = 29.95 m AHD, 100 yr flood volume = 73,500 m^3. Pipe and headwall outlet to minimise system NWL given downstream frog pond constraints. .... Option 2 and 3 - NWL = 28.0 m AHD (11,000 m^2), TED = 28.5 m AHD (15,000 m^2), some flood retardation function .... Option 4 - some flood retardation function, sediment ponds S7 and S8 as below	Flat Site. Outlet system and NWL set by bypass frog pond NWL, rather than the Pakenham Bypass culvert invert levels. Relatively high NWL results in fill required around asset in Option 1 to ensure 100 year protection to surrounding properties.	Frog pond located in bypass reserve retained and outlet design accounts for current inflow requirements.
Sediment Pond S7	Off line sediment pond treating course sediment developed in local catchment to the north east element W4	Local catchment course sediment collection to current MWC standards prior to stormwater entry into W4	3000 m^3 , NWL and TED as per W4	See W4	See W4
Sediment Pond S8	Off line sediment pond treating course sediment developed in local catchment to the north west element W4	Local catchment course sediment collection to current MWC standards prior to stormwater entry into W4	3000 m^3 , NWL and TED as per W4	See W4	See W4

Drainage Element	Description	Primary Objectives	Characteristics	Topography considerations	Environmental/Cultural Heritage Considerations
Retarding Basin/Wetland W2B - Option2	Major drainage element located on Hancocks Gully directly north of Railway - Option 2	Option 2 - Retard peak flows to predevelopment flow rates (2 yr, 100 year (peak) and 100 year (24hr)) and treat stormwater pollutants to at least best practice. …………………………………… Option 1 - not required, Options 3 and 4, See Retarding Basin/Wetland W2W4 below	Option 2 - NWL = 23.2 m AHD ($50,700 \text{ m}^2$), TED = 23.5 m AHD ($60,000 \text{ m}^2$), 100 yr flood level = 24.95 m AHD, 100 yr flood volume = $142,000 \text{ m}^3$ · Pipe and headwall outlet to minimise system NWL given railway invert level.	Flat Site. Outlet system and NWL set railway invert level. Requires landowner cooperation for implementation.	Frog pond(s) can be located in site.
Retarding Basin/Wetland W4B - Option2	Major drainage element located on Western Tributary directly north of Railway - Option 2	Option 2 - Retard peak flows to predevelopment flow rates (2 yr, 100 year (peak) and 100 year (24hr)) and treat stormwater pollutants to at least best practice. …………………………………… Option 1 - not required, Options 3 and 4, See Retarding Basin/Wetland W2W4 below	Option 2 - NWL = 26.5 m AHD ($42,300 \text{ m}^2$), TED = 26.8 m AHD ($48,400 \text{ m}^2$), 100 yr flood level = 28.35 m AHD, 100 yr flood volume = $101,000 \text{ m}^3$ · Pipe and headwall outlet to minimise system NWL given railway invert level.	Flat Site. Outlet system and NWL set railway invert level. Requires Department of Transport cooperation for implementation.	Frog pond(s) can be located close to site.
Retarding Basin/Wetland W2W4 - Option 3 and 4	Major drainage element which combines Western Tributary flows and Hancocks Gully flows north of the railway in Option 3 and South of the Railway in Option 4. …………………… Not required in Option 1 or 2	Options 3 and 4 - Retard peak flows to predevelopment flow rates (2 yr, 100 year (peak) and 100 year (24hr)) and treat stormwater pollutants to at least best practice.	Option 3 - NWL = 23.2 m AHD ($111,500 \text{ m}^2$), TED = 23.5 m AHD ($124,000 \text{ m}^2$), 100 yr flood level = 25.65 m AHD, 100 yr flood volume = $392,000 \text{ m}^3$ · …………………………………… Option 4 - NWL = 20.15 m AHD ($109,000 \text{ m}^2$), TED = 20.5 m AHD ($117,000 \text{ m}^2$), 100 yr flood level = 22.2 m AHD, 100 yr flood volume = $337,000 \text{ m}^3$	Option 3 - located north of the railway. Site not ideal for retarding basin given topography north of the railway. …………………………………… Option 4 - located south of the railway. Contours good for retarding basin placement. Requires agreement from affected landowners.	To be determined

Drainage Element	Description	Primary Objectives	Characteristics	Topography considerations	Environmental/Cultural Heritage Considerations
Sediment Pond S9 - Option 4	Off line sediment pond treating course sediment developed in potential future development between the railway and Pakenham Bypass	Local catchment course sediment collection to current MWC standards prior to stormwater entry into vegetated channel system	3000 m^3 , NWL and TED to be determined	see W2W4 - Option 3	To be determined
Sediment Pond S10 - Option 4	Off line sediment pond treating course sediment developed in potential future development between the railway and Pakenham Bypass	Local catchment course sediment collection to current MWC standards prior to stormwater entry into vegetated channel system	2000 m^3 , NWL and TED to be determined	see W2W4 - Option 3	To be determined
Sediment Pond S11 - Option 4	Off line sediment pond treating course sediment developed in potential future development between the railway and Pakenham Bypass	Local catchment course sediment collection to current MWC standards prior to stormwater entry into vegetated channel system	2000 m^3 , NWL and TED to be determined	see W2W4 - Option 3	To be determined
Vegetated channels south of Pakenham Bypass - Options 2, 3 and 4	Vegetated channels conveying flow to major drainage elements as shown in the drawing set	Ensure 100 Year capacity within drainage channel. Provide ecological corridor and passive recreation opportunities (bicycle paths etc).	40 metre wide vegetated channel meandering within a 60 metre reserve. See 1304/5 for typical cross section	Location, size and alignment to be determined given the ultimate strategy developed	Very little existing vegetation along drainage reserve.

Drainage Element	Description	Primary Objectives	Characteristics	Topography considerations	Environmental/Cultural Heritage Considerations
Deep Creek Flood Plain - All Options	Drainage Reserve adjacent to Creek as defined by a proposed development line	Ensure existing creek and flood plain ecological values are maintained and enhanced. Ensure flood impact due to development line does not impact on existing development or roads. Ensure adequate fill levels are set adjacent to Reserve	100 m Drainage reserve east of Creek (in general) and a 50 metre reserve west of Creek.	Some reshaping of the flood plain is required to ensure no increase in flood levels - see Stormy Water Solutions report "Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, 5 October 2014".	To be determined
Deep Creek at Ryan Road - All Options	Culvert upgrading and road raising	Ensure 100 Year capacity of Ryan Road culvert system	See Stormy Water Solutions report "Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, 5 October 2014".	See Stormy Water Solutions report "Pakenham East Precinct Structure Plan, Deep Creek Corridor Proposals, 5 October 2014".	To be determined

APPENDIX B Culvert Capacities

Capacity of Deep Creek at Princes Highway

Based on vic Roads Drawing 128769
Assume all culverts flowing under outlet control
and head loss = 0.3 to ensure carriageway not engaged

head loss = $(Ke+Kex)\times V^2/2g + S_f \times L$
$S_f = Q^2n^2/A^2R^{4/3}$

Twin 3700 by 3700 Box Culvert System

W =	3.7	m
D =	3.7	m
Capacity flow/cell =	25	m^3/s
Wetted perimeter =	14.80	m
Area =	13.69	m^2
Hyd radius =	0.925	m
V =	1.83	m/s
Ke =	0.5	
Kex =	1	
n =	0.013	
L=	43.3	
S_f =	0.0006	
Head loss =	0.28	m

Total system Design Flow =	50	m^3/s	
MWC Deep Creek Flood Plain flow 1989	42.0	m^3/s	OK

Capacity of Hancocks gully at Princes Highway			
Based on vic Roads Drawing 42075			
Assume all culverts flowing under outlet control			
Twin 2400 by 2800 Box Culvert System			
Obvert at DS end about 38.9 m AHD			(maybe lower)
Road about 39.6 (min)		(matches drawing levels)	
Ultimate flood level DS will probably be below DS obvert			
Therefore take max head loss across culvert = 450 mm			
to ensure road not engaged			
That is 225 mm head loss across each culvert (carriageway)			
Assume both culverts the same size as detailed in the above drawing			
Allows for about 150 freeboard to culvert headwall top (approx)			
(To be confirmed)			
W =	2.44	m	head loss = $(Ke+Kex) \times V^2/2g + S_f \times L$
D =	2.76	m	$S_f = Q^2n^2/A^2R^{4/3}$
Capacity flow/cell =	11.2	m^3/s	
Wetted perimeter =	10.40	m	
Area =	6.73	m^2	
Hyd radius =	0.647538	m	
V =	1.66	m/s	
Ke =	0.5		
Kex =	1		
n =	0.013		
L=	14		
S_f =	0.0008		
Head loss =	0.223	m	
Total system Capacity =	22.4	m^3/s	
RORB Design Flow =	18.1	$m^3/$	With RB W1 - OK

RORB flow applicable to Option 4 post development (Worst Case)

Capacity of Hancocks Gully culvert system at Freeway

Based on vic Roads Drawing 583249
Assume all culverts flowing under outlet control

TWL based on approx vegetated channel downstream =
28.0 m AHD (approx)

Max HWL @ 200 below min pavement = 28.8 (approx)

head loss = $(Ke+Kex)\times V^2/2g + S_f \times L$
$S_f = Q^2n^2/A^2R^{4/3}$

NB lidar puts min road at 29.0 and this is consiutent with culvert design drawings

1. Five 2400 by 600 Box Culvert System		
W =	2.4	m
D =	0.6	m
Downstream obvert =	28.2	
TWL =	28.2	
Capacity flow/cell =	3.15	m^3/s
Wetted perimeter =	6.00	m
Area =	1.44	m^2
Hyd radius =	0.24	m
V =	2.19	m/s
Ke =	0.5	
Kex =	1	
n =	0.013	
L=	43.3	
S_f =	0.0054	
Head loss =	0.60	m
HWL =	28.80	
Total Flow =	15.75	m^3/s

2. Frog pond connection		
One 375 mm dia		
pipe dia =	0.375	m
RCP pipe radius =	0.1875	m
Design flow =	0.15	m^3/s
Downstream obvert =	26.8	
TWL =	28.0	
Capacity flow/cell =	0.165	m^3/s
Wetted perimeter =	1.18	m
Area =	0.11	m^2
Hyd radius =	0.09375	m
V =	1.49	m/s
Ke =	0.5	
Kex =	1	
n =	0.013	
L=	70	
S_f =	0.0089	
Head loss =	0.79	m
HWL =	28.79	
Total Flow =	0.15	m^3/s

3. Frog Culvert - One 1500 by 900			
W =	1.5	m	
D =	0.9	m	
Asume culvert 50% full			
not running under pressure			
Water Depth	0.55	m	
Drain Base width	1.5	m	
Longitudinal Slope	0.0025	m/m	
side slope of batter	1 in		0.01
Flow Area (A)	0.828	m^2	
ss length	0.55	m	
Wetted Perimeter (P	2.60	m	
Hydraulic Radius (R	0.32	m	
mannings n	0.013		
Capacity (Q)	1.49	m^3/s	
Velocity (V)	1.79	m/s	
Total Flow =	1.49	m	m^3/s

4. Box Culvert - One 3000 by 1200		
W =	3	m
D =	1.2	m
Downstream obvert =	28.25	
TWL =	28.25	
Capacity flow/cell =	8.5	m^3/s
Wetted perimeter =	8.40	m
Area =	3.60	m^2
Hyd radius =	0.428571	m
V =	2.36	m/s
Ke =	0.5	
Kex =	1	
n =	0.013	
L=	43.3	
S_f =	0.0029	
Head loss =	0.55	m
HWL =	28.80	
Total Flow =	8.5	m^3/s

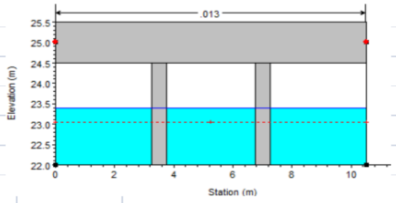
Total system Capacity =	25.9	m^3/s	
RORB Design Flow =	22.4	m^3/s	OK Post development, W1 retarding basins upstream and a vegetated channel along Handcocks gully

RORB flow applicable to Option 4 post development (Worst Case)

Capacity of West Trib culvert system at Freeway													
Based on vic Roads Drawing 583245							TWL based on approx vegetated channel downstream =						
Assume all culverts flowing under outlet control							28.0	m AHD (approx)					
							Max HWL @ 500 below min pavement =					28.8	(approx)
head loss = $(Ke+Kex)\times V^2/2g + S_f \times L$							NB lidar puts min road at 29.5 and this is consiutent with culvert design drawings						
$S_f = Q^2n^2/A^2R^{4/3}$													
1. Ten 2400 by 900 Box Culvert System				**2. Frog pond connections**					**3. Frog Culvert - One 1500 by 900**				
				Two 375 mm dia									
W =	2.4	m		pipe dia =		0.375	m		**W =**		1.5	m	
D =	0.9	m		RCP pipe radius =		0.1875	m		D =		0.9	m	
				Design flow =		0.15	m^3/s						
Downstream obvert =		28.6		Downstream obvert =			28.3		Asume culvert 50% full				
TWL =		28.6		TWL =			28.3		not running under pressure				
Capacity flow/cell =	3	m^3/s		Capacity flow/cell =		0.13	m^3/s		Water Depth	0.55	m		
Wetted perimeter =	6.60	m		Wetted perimeter =		1.18	m		Drain Base width	1.5	m		
Area =	2.16	m^2		Area =		0.11	m^2		Longitudinal Slope	0.0025	m/m		
Hyd radius =	0.327273	m		Hyd radius =		0.09375	m		side slope of batter	1 in	0.01		
V =	1.39	m/s		V =		1.18	m/s		Flow Area (A)	0.828	m^2		
Ke =	0.5			Ke =		0.5			ss length	0.55	m		
Kex =	1			Kex =		1			Wetted Perimeter (P	2.60	m		
n =	0.013			n =		0.013			Hydraulic Radius (R	0.32	m		
L=	44.5			L=		70			mannings n	0.013			
S_f =	0.0014			S_f =		0.0055			Capacity (Q)	1.49	m^3/s		
									Velocity (V)	1.79	m/s		
Head loss =	0.21	m		Head loss =		0.49	m						
HWL =	28.81			HWL =		28.79							
Total Flow =	30	m^3/s		Total Flow =		0.3	m^3/s		Total Flow =	1.49	m	m^3/s	
Total system Capacity =		31.8	m^3/s										
RORB Design Flow =		26.0	m^3/s	OK	Post development, no retarding basins upstream								
					But diversion to Deep Creek US of Princes highway								

RORB flow applicable to Option 4 post development (Worst Case)

Capacity of Hancocks gully at Princes Highway



Yet to be surveyed
Assume all culverts flowing under outlet control

All assumed design levels and bridge opening
assumptions to be proven by detailed survey
as the functional/detailed design stage of the project

head loss = $(Ke+Kex) \times V^2/2g + S_f \times L$

$S_f = Q^2 n^2 / A^2 R^{4/3}$

Note: Hec Ras analysis indicates a jump in upstream headwater level due to contraction losses
(see below), and this must be accounted for in the
vegetated channel design or any fill required on future development on this land

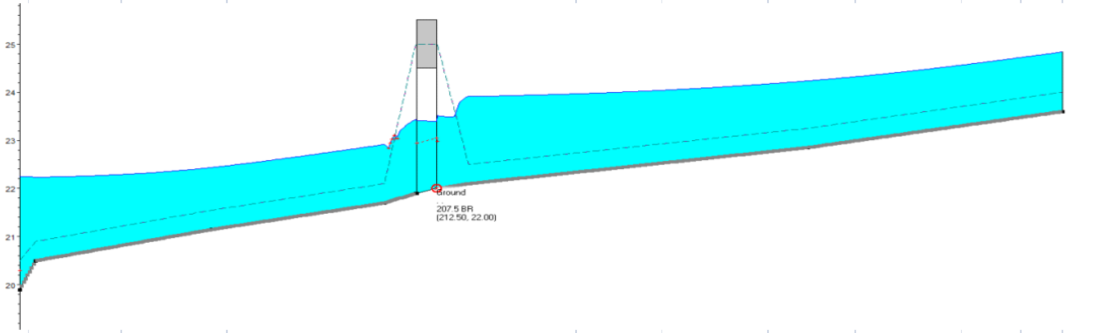
Note: the Hec Ras analysis assumes the culvert IL = 22.0
and (in the ultimate situation) the area under the
bridge will be concrete

Equivalent of triple 3.0 by 2.5 Box Culvert System
(assume culverts not flowing full and flow depth = 1.4 m)

Reach	River Sta	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
		(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
.	532	32.00	23.60	24.84		24.93	0.005485	1.43	24.70	33.39	0.47
.	402	32.00	22.85	24.24		24.30	0.003187	1.20	29.83	35.77	0.37
.	229	32.00	22.10	23.91		23.94	0.000887	0.79	45.41	36.80	0.20
.	213	32.00	22.00	23.51	22.98	23.72	0.000553	2.01	15.89	10.50	0.52
.	207.5	Bridge									
.	202	32.00	21.90	23.43		23.63	0.000536	1.99	16.05	10.50	0.51
.	186	32.00	21.70	22.92		23.02	0.005960	1.47	24.00	33.06	0.49
.	98	32.00	21.15	22.44		22.52	0.004533	1.35	26.38	34.19	0.43
.	8	32.00	20.50	22.23		22.26	0.001086	0.84	42.57	36.80	0.22
.	0	32.00	19.90	22.25	20.29	22.25	0.000054	0.25	135.94	74.50	0.05

Assume culvert 50% full
not running under pressure

Water Depth	1.4	m	Based on Hec Ras analysis
Drain Base width	3	m	
Longitudinal Slope	0.0025	m/m	(say)
side slope of batters	1 in	0.01	
Flow Area (A)	4.2196	m^2	
ss length	1.40	m	
Wetted Perimeter (P)	5.80	m	
Hydraulic Radius (R)	0.73	m	
mannings n	0.013		
Capacity (Q)	13.13	m^3/s	
Velocity (V)	3.11	m/s	



Total Flow = 39.38 m m^3/s

RORB Design Flow = 32.2 m^3/ OK

RORB flow applicable to Option 4 post development (Worst Case)

APPENDIX C Hydrological Modelling

Hydrological Modelling using the RORB model was developed for this study by SWS to estimate flood flows and retarding basin requirements.

RORB is an industry-standard Runoff Routing Model originally developed by Monash University (Laurenson EM and Mein RG) and is used extensively within Australia for applications such as this.

The analysis below considers the 100 Year ARI storm events for all options. Consideration of this event will provide realistic site delineation of the retarding basin areas required to meet all objectives. However development of the functional design of the retarding basins must include clearly defining the current low flow regimes to the downstream receiving bodies. Flow regimes will need to be maintained post development in order to protect the existing ecology and channel morphology of the downstream Deep Creek and Hancocks Gully creek systems. As such, future work must consider the retardation of the 2 Year ARI flow to predevelopment flow rates. Option 4 is the only analysis below which considered this requirement at this stage. This aspect of the design must be formulated during the functional design process for the strategy ultimately developed by Council.

It should be noted that the hydrological analysis is considered preliminary only at this stage. It has been undertaken to ensure realistic land take estimates for this aspect of the design. The final RORB model developed for the chosen strategy may differ from those models detailed below.

The RORB model for Options 1, 2 and 3 were developed in February 2013. The results are replicated below. The RORB model for Option 4 was developed in February 2014, and given a slight review of the catchment, the RORB parameters and flow results may differ slightly from the 2013 analysis results detailed below.

C.1 Option 1

Figure C.1 details the RORB model setup for Option 1, which considers development within the PSP area only. Tables C.1 and C.2 detail the tabulation of the RORB model setup (i.e. catchment areas, fraction imperviousness and reach lengths etc). Pakenham rainfall intensities were utilised.

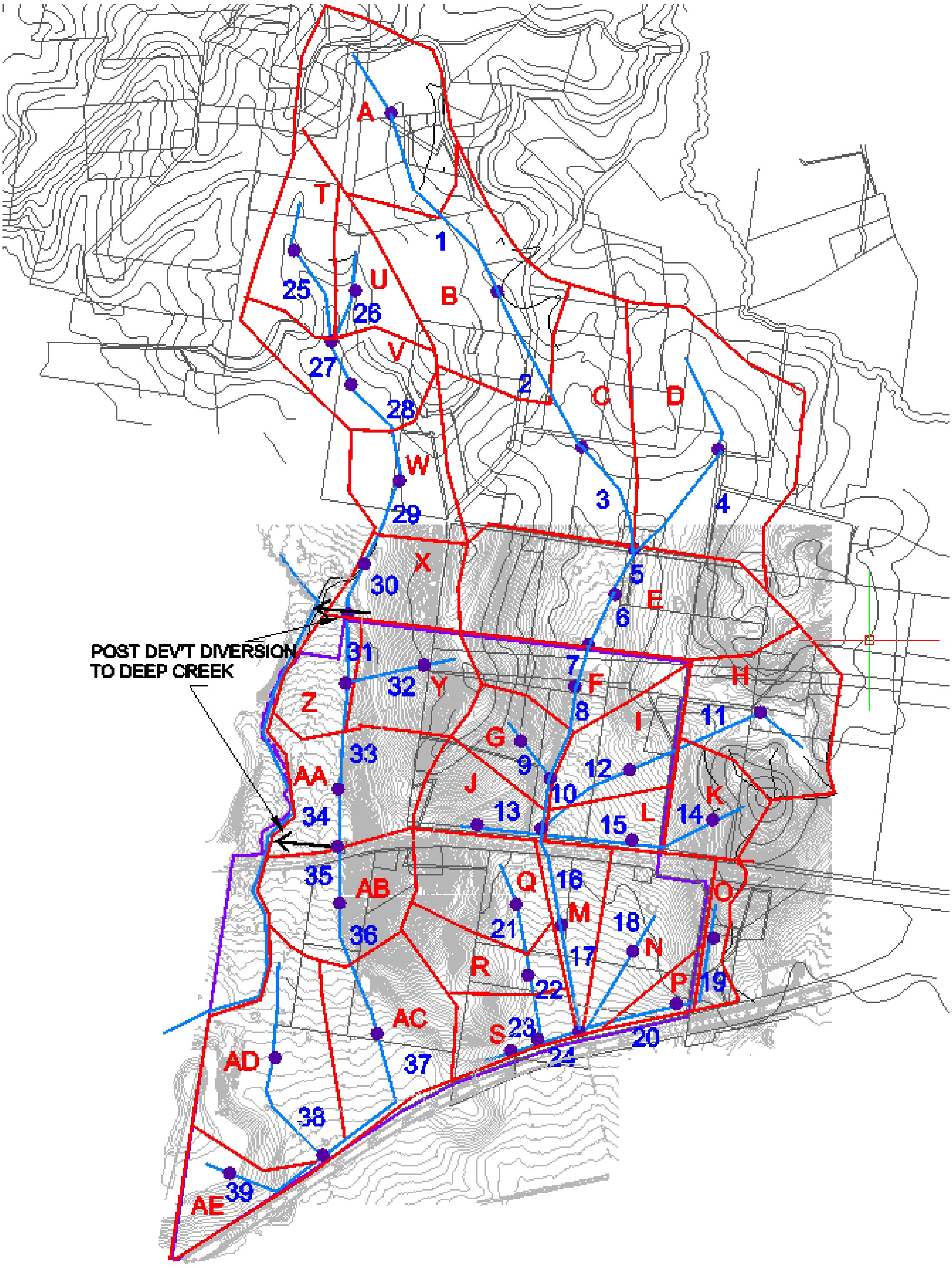


Figure C.1 **Option 1 - RORB Model**

Sub Area	Area (ha)	Area (km^2)	Fraction	Fraction		
			Imperviousness	Imperviousness		
			(predevelopment)	(developed)		
A	72	0.72	0.05	0.05		
B	79	0.79	0.05	0.05		
C	106	1.06	0.05	0.05		
D	107	1.07	0.05	0.05		
E	99	0.99	0.05	0.05		
F	35	0.35	0.05	0.60		
G	25	0.25	0.05	0.60		
H	58	0.58	0.05	0.05		
I	36	0.36	0.05	0.60		
J	25	0.25	0.05	0.60		
K	27	0.27	0.05	0.05		
L	19	0.19	0.05	0.60		
M	24	0.24	0.05	0.60		
N	46	0.46	0.05	0.60		
O	12	0.12	0.05	0.05		
P	9	0.09	0.05	0.60		
Q	37	0.37	0.05	0.60		
R	27	0.27	0.05	0.60		
S	24	0.24	0.05	0.60		
T	34	0.34	0.05	0.05		
U	24	0.24	0.05	0.05		
V	40	0.40	0.05	0.05		
W	35	0.35	0.05	0.05		
X	30	0.30	0.05	0.10		
Y	38	0.38	0.05	0.60		
Z	23	0.23	0.05	0.60		
AA	51	0.51	0.05	0.60		
AB	46	0.46	0.05	0.50	interface UGB and hilltop dev't	
AC	59	0.59	0.05	0.50	interface UGB and hilltop dev't	
AD	73	0.73	0.05	0.50	interface UGB and hilltop dev't	
AE	24	0.24	0.05	0.50	interface UGB and hilltop dev't	
Total:	**1343**	**13.43**	**0.05**	**0.33**		

Table C.1 **Option 1 - RORB Model Catchment Area Definition**

					100 year/ 5 Year	100 year/ 5 Year
Reach	**Length (m)**	**Length (km)**	**Fall (m)**	**Slope (%)**	**Reach Type**	**Reach Type**
					(undeveloped)	(developed)
1	1170	1.17			NATURAL	NATURAL
2	970	0.97			NATURAL	NATURAL
3	700	0.70			NATURAL	NATURAL
4	730	0.73			NATURAL	NATURAL
5	230	0.23			NATURAL	NATURAL
6	320	0.32			NATURAL	NATURAL
7	240	0.24			NATURAL	NATURAL
8	530	0.53			NATURAL	NATURAL
9	280	0.28	22.00	7.9	NATURAL	PIPED
10	290	0.29			NATURAL	NATURAL
11	800	0.80	16.00	2.0	NATURAL	EX/UNLINED
12	590	0.59	8.00	1.4	NATURAL	PIPED
13	360	0.36	16.00	4.4	NATURAL	PIPED
14	550	0.55	15.00	2.7	NATURAL	EX/UNLINED
15	515	0.52	4.50	0.9	NATURAL	PIPED
16	530	0.53			NATURAL	NATURAL
17	600	0.60			NATURAL	NATURAL
18	535	0.54	5.00	0.9	NATURAL	PIPED
19	450	0.45	14.50	3.2	NATURAL	EX/UNLINED
20	550	0.55	5.00	0.9	NATURAL	PIPED
21	420	0.42	5.00	1.2	NATURAL	PIPED
22	370	0.37	2.50	0.7	NATURAL	PIPED
23	150	0.15	1.00	0.7	NATURAL	PIPED
24	240	0.24	1.00	0.4	NATURAL	PIPED
25	540	0.54			NATURAL	NATURAL
26	330	0.33			NATURAL	NATURAL
27	285	0.29			NATURAL	NATURAL
28	600	0.60			NATURAL	NATURAL
29	515	0.52			NATURAL	NATURAL
30	300	0.30			NATURAL	NATURAL
31	380	0.38	3.50	0.9	NATURAL	PIPED
32	460	0.46	28.00	6.1	NATURAL	PIPED
33	590	0.59	5.00	0.8	NATURAL	PIPED
34	340	0.34	3.00	0.9	NATURAL	PIPED
35	320	0.32	2.50	0.8	NATURAL	PIPED
36	770	0.77	7.00	0.9	NATURAL	PIPED
37	880	0.88	3.50	0.4	NATURAL	PIPED
38	630	0.63	3.00	0.5	NATURAL	PIPED
39	550	0.55	7.50	1.4	NATURAL	PIPED

Table C.2 Option 1 - RORB Model Reach Definitions

RORB is based on the following equation relating storage (S) and discharge (Q) of a watercourse.

$S = k \times Q^m$ where $k = K_c \times K_r$

The values K_c and m are parameters that can be obtained by calibration of the model using corresponding sets of data on rainfall for selected historical flows. If historical flows are unknown, values can be estimated from regional analysis or by values suggested by Australian Rainfall and Runoff (AR&R). In this case, flow gauging information was not available. However a regional parameter set has been developed by Melbourne Water for the South East region of Melbourne. This relationship is detailed below.

$K_c = 1.55A^{0.55} = 6.4$

m = 0.8

Initial loss = 10 mm

Pervious area runoff coefficient, 100 year = C_{perv} = 0.6 mm

Stormy Water Solutions checked estimated (dummy) predevelopment RORB flows against other rational method calculations and the DSE regional flow estimate graphs in 2013. This check was done to ensure the K_c value adopted was valid for its application in this area. A K_c value of 6.4 resulted in design predevelopment flows less than typically expected for a catchment of this size. A K_c value of 5 gave better results and was adopted for the analysis of Option 1 in the 1013 analysis.

The post development model assumes:

- The increase in fraction imperviousness as detailed in Table C.1,
- The change in reach types as detailed in Table C.2,
- Four, 4 retarding basins/wetland (W1, W2, W3, and W4) incorporating Stage/Storage/Discharge relationships as detailed in Appendix D,
- Diversion of the 100 Year ARI into Deep Creek at the end of Reach 30 (Figure C.1), and
- Diversion of the 100 Year ARI into Deep Creek at the end of Reach 34 (Figure C.1).

Table C.3 details the RORB results for Option 1. As detailed:

- The peak 100 Year flow from the future development does not exceed the predevelopment flow rate at all retarding basin outlet points,
- The 24 hour 100 Year flow from the future development does not exceed the predevelopment flow rate for a storm of this duration at retarding basin outlet points, and
- Each retarding basin can store the difference between the expected post development and predevelopment 24 hour 100 Year flow volume.

As such, the analysis suggests the retarding basin/wetland initiatives detailed in Option 1 can ensure no increase in downstream 100 Year ARI flood flows and no increased flood effect within the KWRFPD during a 24 hour 100 Year ARI flood event.

	Pre - development 100 Year Peak Flow [2] (m³/s)	Pre - development 24 hour, 100 Year Flow (m³/s)	Post Development 100 Year Flow [3] (m³/s)	100 Year Water Level in Retarding Basin[3] (m AHD)	Predevelopment 24 hour, 100 Year Hydrograph Volume (m³)	Post development 24 hour, 100 Year Hydrograph Volume (m³)	Difference between post development and Pre Development 24 hour hydrograph Volume (m³)	Maximum Active Flood Storage in Retarding Basin (m³)
Hancocks Gully at Northern PSP boundary	12.5		12.5					
Upstream End of Reach 10	12.9		13.2					
Hancocks Gully at Princes Highway - Location of Retarding Basin Wetland W1	16.9	14.8	14.2	40.70	544,000	580,000	36,000	86,700
Hancocks Gully at Freeway- Location of Retarding Basin Wetland W2	19.0	16.9	15.0	30.50	685,000	748,000	63,000	135,000
Western Tributary Flow into Deep Creek at Northern PSP boundary[1]	4.3		4.3					
Western Tributary Flow into Deep Creek at Princes Highway[1] - Location of Retarding Basin Wetland W3	3.6	3	2.1	42.20	89,000	119,000	30,000	53,900
Western Tributary at Freeway[1]- Location of Retarding Basin Wetland W4	6.3	5.3	4.8	29.95	161,000	212,000	51,000	73,500

1 - Assumes most of the flow from Subareas T - X does not directly enter Deep Creek
2 - Predevelopment Critical Duration at all locations = 9 hours (100 yr)
3 - Post Development Critical Duration = 9- 12 hours (100 yr)

Pre - development RORB model - PREDEV_FEB_2013.CAT
Post - development RORB model - POSTDEV_FEB_2013.CAT

Table C.3 RORB Results – Option 1

C.2 Option 2

Figure C.2 details the RORB model setup for Option 2. Option 2 details a possible alternative where some elements are located directly downstream of the PSP boundary. Option 2 would require agreement from affected landowners and the Department of Transport (DoT) on the land affected by potential future railway stabling infrastructure.

Option 2 adds catchment areas AF (25.5 ha, F_{imp} = 0.7) and AG (152.5 ha, F_{imp} = 0.4), and Reaches 24a (770 m, natural), 24b (600 m, piped), 40 (390 m, drowned) and 41 (200 m, piped).

Option 2 allows for four retarding basins/wetland systems (W1, W2, W3, and W4) incorporating Stage/Storage/Discharge relationships as detailed in Appendix E.

Due to the increased catchment area Kc has been increased to 5.5 to provide consistent flow values at equivalent locations at Option 1.

Apart from the changes above all other post development assumptions are as described in Section 6.1 above.

Table C.4 details the RORB results for Option 2. As detailed:

- The peak 100 Year flow from the future development does not exceed the predevelopment flow rate at all retarding basin outlet points,
- The 24 hour 100 Year flow from the future development does not exceed the predevelopment flow rate for a storm of this duration at retarding basin outlet points, and
- Each retarding basin can store the difference between the expected post development and predevelopment 24 hour 100 Year flow volume.

As such, the 2013 analysis suggests the retarding basin/wetland initiatives detailed in Option 2 can ensure no increase in downstream 100 Year ARI flood flows and no increased flood effect within the KWRFPD during a 24 hour 100 Year ARI flood event.

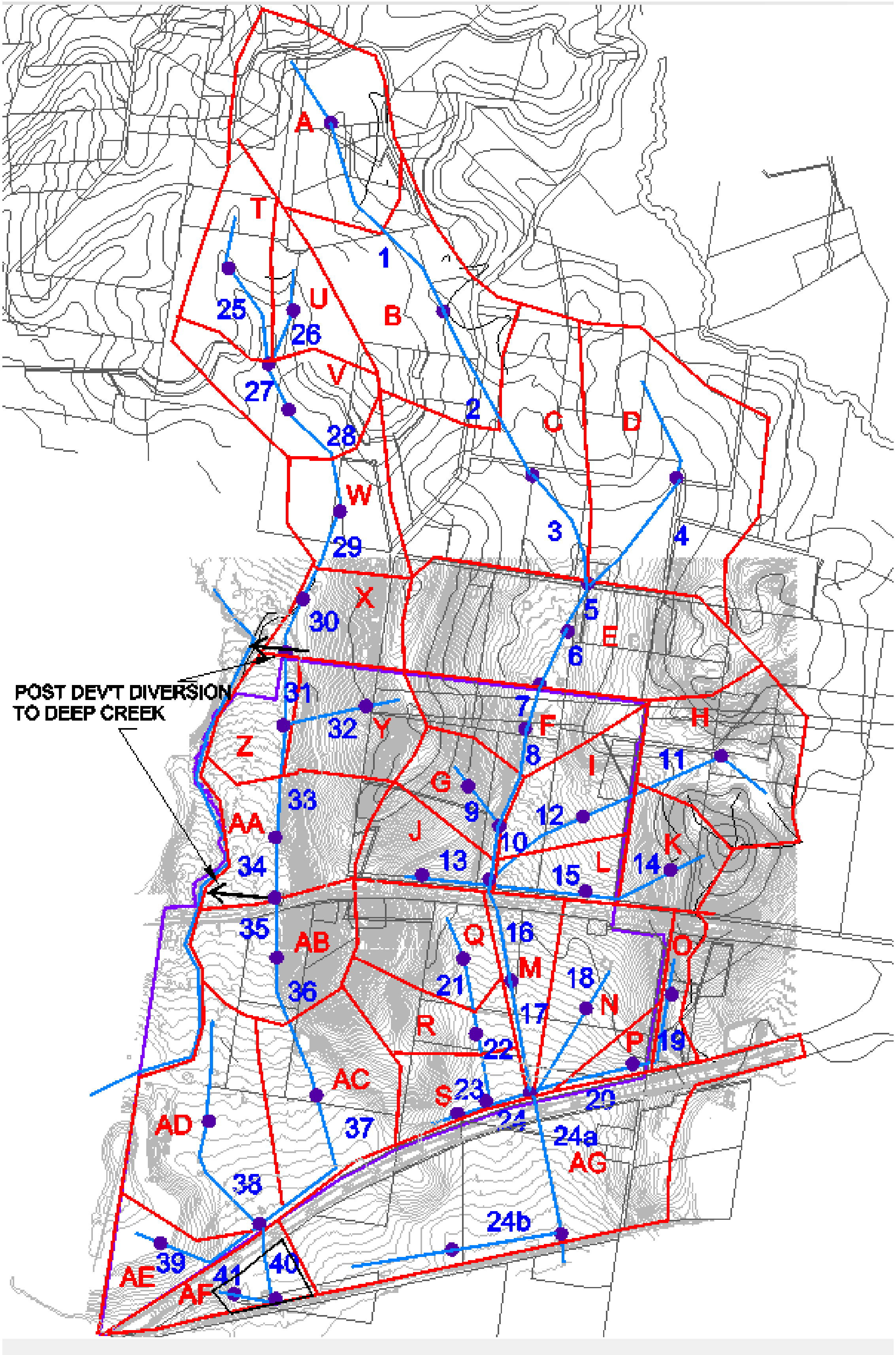


Figure C.2 **Option 2 - RORB Model**

	Pre - development 100 Year Peak Flow [2] (m^3/s)	Pre - development 24 hour, 100 Year Flow (m^3/s)	Post Development 100 Year Flow [3] (m^3/s)	100 Year Water Level in Retarding Basin[3] (m AHD)	Predevelopment 24 hour, 100 Year Hydrograph Volume (m^3)	Post development 24 hour, 100 Year Hydrograph Volume (m^3)	Difference between post development and Pre Development 24 hour hydrograph Volume (m^3)	Maximum Active Flood Storage in Retarding Basin (m^3)
Hancocks Gully at Princes Highway - Location of Retarding Basin Wetland W1	16.9	14.8	14.2	40.70	544,000	580,000	36,000	86,700
Hancocks Gully at Railway- Location of Retarding Basin Wetland W2	21.5	19.9	18.4	24.95	817,000	870,000	53,000	142,000
Western Tributary Flow into Deep Creek at Northern PSP boundary[1]	4.3		4.3					
Western Tributary Flow into Deep Creek at Princes Highway[1] - Location of Retarding Basin Wetland W3	3.8	3	2.1	42.20	89,000	119,000	30,000	53,900
Western Tributary at Railway[1]- Location of Retarding Basin Wetland W4	6.5	5.5	4.8	28.35	175,000	240,000	65,000	101,000

1 - Assumes most of the flow from Subareas T - X does not directly enter Deep Creek
2 - Predevelopment Critical Duration at all locations = 9 hours (100 yr)
3 - Post Development Critical Duration = 9- 12 hours (100 yr)

Pre - development RORB model - PAK_EAST_PRE_FEB_2013_EXTEND.CAT
Post - development RORB model - OPTION2_FEB_2013.CAT

Table C.4 RORB Results – Option 2

C.3 Option 3

Figure C.3 details the RORB model setup for Option 3. Option 3 details a possible alternative where some elements are located directly downstream of the PSP boundary. Option 3 would require agreement from affected landowners and the Department of Transport (DoT) on the land affected by potential future railway stabling infrastructure.

Option 3 adds catchment areas AF (83 ha, F_{imp} = 0.7 and AG (95 ha, F_{imp} = 0.15), and Reaches 40 (640 m, natural), 41 (1000 m, natural) and 42 (400 m, natural).

Option 3 allows for three retarding basins/wetland systems (W1, W3, and W2W4) incorporating Stage/Storage/Discharge relationships as detailed in Appendix F.

Due to the increased catchment area Kc has been increased to 5.5 to provide consistent flow values at equivalent locations at Option 1.

Apart from the changes above all other post development assumptions are as described in Section 6.1 above.

Table C.5 details the RORB results for Option 3. As detailed:

- The peak 100 Year flow from the future development does not exceed the predevelopment flow rate at all retarding basin outlet points,
- The 24 hour 100 Year flow from the future development does not exceed the predevelopment flow rate for a storm of this duration at retarding basin outlet points, and
- Each retarding basin can store the difference between the expected post development and predevelopment 24 hour 100 Year flow volume.

As such, the analysis suggests the retarding basin/wetland initiatives detailed in Option 3 can ensure no increase in downstream 100 Year ARI flood flows and no increased flood effect within the KWRFPD during a 24 hour 100 Year ARI flood event.

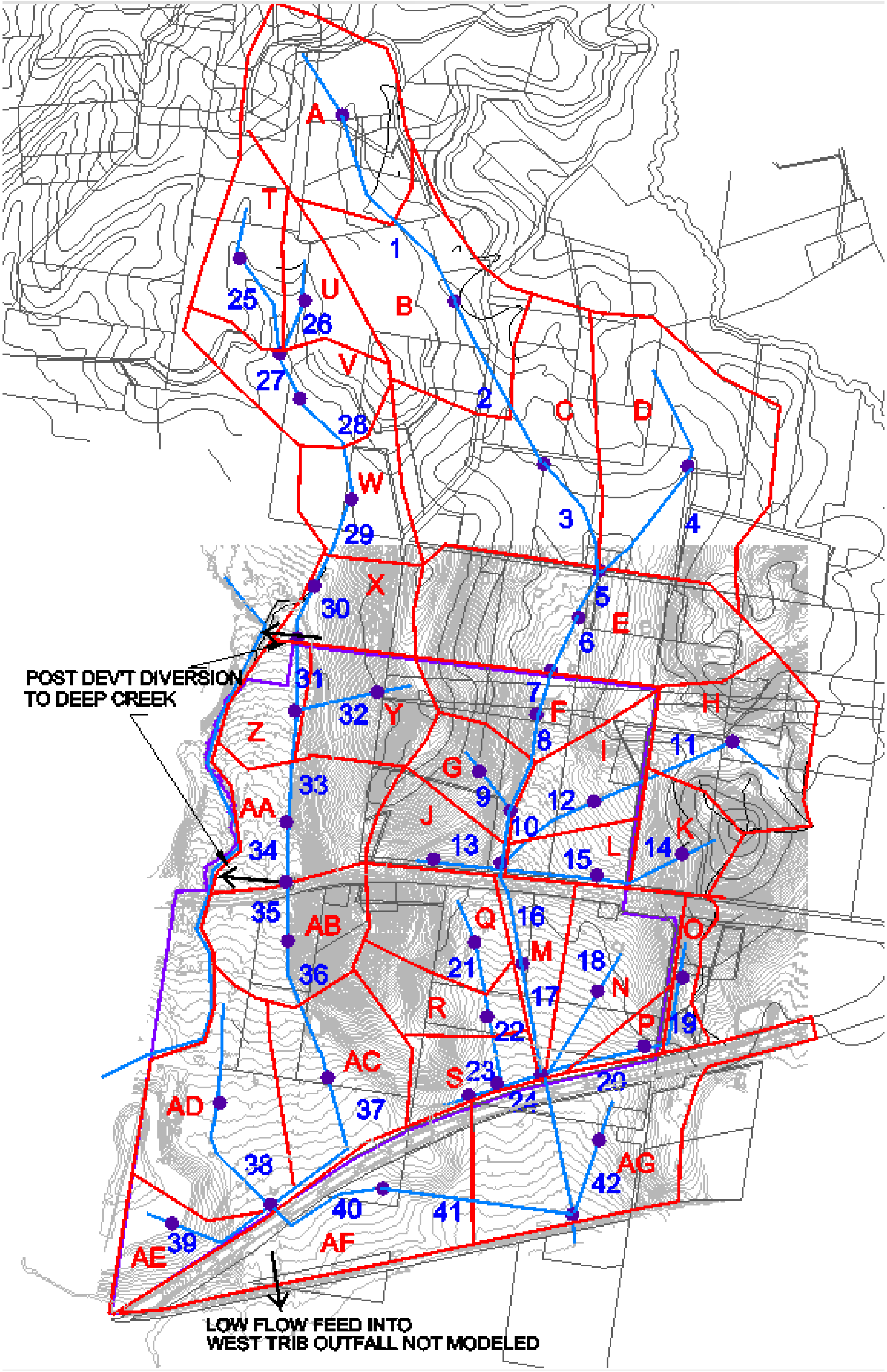


Figure C.3 **Option 3 - RORB Model**

	Pre - development 100 Year Peak Flow [2] (m³/s)	Pre - development 24 hour, 100 Year Flow (m³/s)	Post Development 100 Year Flow [3] (m³/s)	100 Year Water Level in Retarding Basin[3] (m AHD)	Predevelopment 24 hour, 100 Year Hydrograph Volume (m³)	Post development 24 hour, 100 Year Hydrograph Volume (m³)	Difference between post development and Pre Development 24 hour hydrograph Volume (m³)	Maximum Active Flood Storage in Retarding Basin (m³)
Hancocks Gully at Princes Highway - Location of Retarding Basin Wetland W1	16.9	14.8	14.2	40.70	544,000	580,000	36,000	86,700
Hancocks Gully at Railway- Location of Retarding Basin Wetland W2W4	21.5	18.9	17.6	25.65	817,000	1,130,000	313,000	392,000
Western Tributary Flow into Deep Creek at Northern PSP boundary[1]	4.3		4.3					
Western Tributary Flow into Deep Creek at Princes Highway[1] - Location of Retarding Basin Wetland W3	3.8	3	2.1	42.20	89,000	119,000	30,000	53,900

1 - Assumes most of the flow from Subareas T - X does not directly enter Deep Creek
2 - Predevelopment Critical Duration at all locations = 9 hours (100 yr)
3 - Post Development Critical Duration = 9- 12 hours (100 yr)

Pre - development RORB model - PAK_EAST_PRE_FEB_2013_EXTEND.CAT
Post - development RORB model - OPTION3_FEB_2013.CAT

Table C.5 RORB Results – Option 3

C.4 Option 4

Figure C.4 details the RORB model setup for Option 4 (2014 model – slight changes to previous models given this 2014 review/work). Option 4 details a possible alternative where some elements are located downstream of the PSP boundary. Option 4 would require agreement from affected landowners and the Department of Transport (DoT) on the land affected by potential future railway stabling infrastructure.

Option 4 adds catchment areas AF (84 ha, F_{imp} = 0.9), AG (96 ha, F_{imp} = 0.5) and AH (110 ha, F_{imp} = 0.1).

Option 4 allows for three retarding basins/wetland systems (W1, W3, and W2W4) incorporating Stage/Storage/Discharge relationships as detailed in Appendix G. Some flood storage is also allowed for upstream of the Pakenham bypass given the constraints of the existing culvert configurations.

Due to the 2014 review, the regional K_c parameter has been used for this analysis (7.1). The predevelopment fraction imperviousness was increased to 0.1 for this 2014 analysis.

Apart from the changes above all other post development assumptions are as described in Section 6.1 above.

Table C.6 details the RORB results for Option 4. As detailed:

- The peak 100 Year flow from the future development does not exceed the predevelopment flow rate at all outfall points,
- The 24 hour 100 Year flow from the future development does not exceed the predevelopment flow rate for a storm of this duration at outlet points, and
- Each retarding basin can store the difference between the expected post development and predevelopment 24 hour 100 Year flow volume.

As such, the analysis suggests the retarding basin/wetland initiatives detailed in Option 4 can ensure no increase in downstream 100 Year ARI flood flows and no increased flood effect within the KWRFPD during a 24 hour 100 Year ARI flood event.

It should be noted that Option 4 is the worst case scenario in terms of the design flow expected in vegetated channels and at road crossings in the developed scenario. This is because upstream storage provisions are minimised in this option.

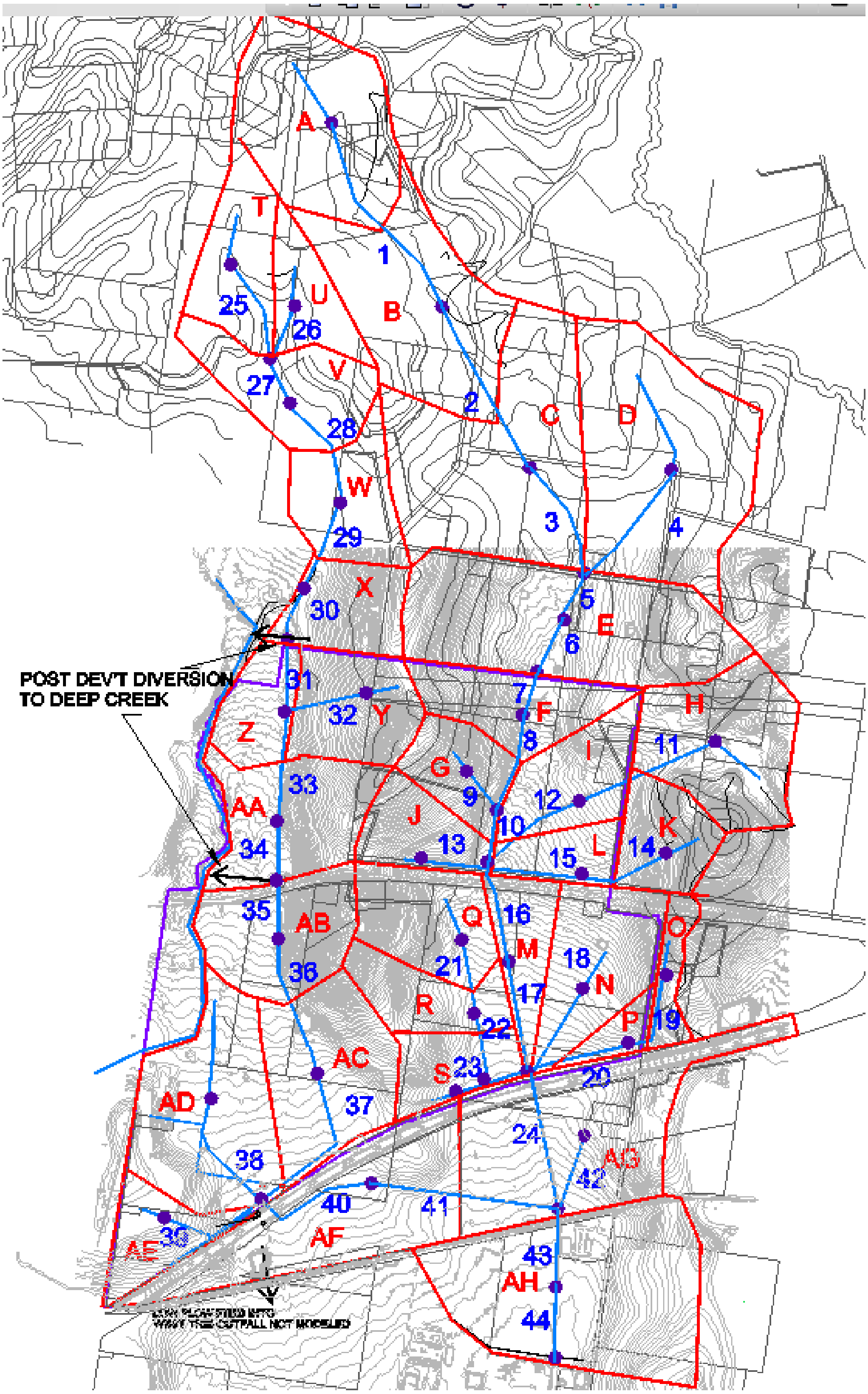


Figure C.4 Option 4 - RORB Model

	Pre - development 100 Year Peak Flow [2]	Pre - development 24 hour, 100 Year Flow	Post Development 100 Year Flow [3]	100 Year Water Level in Retarding Basin[3]	Predevelopment 24 hour, 100 Year Hydrograph Volume	Post development 24 hour, 100 Year Hydrograph Volume	Difference between post development and Pre Development 24 hour hydrograph Volume	Maximum Active Flood Storage in Retarding Basin
	(m^3/s)	(m^3/s)	(m^3/s)	(m AHD)	(m^3)	(m^3)	(m^3)	(m^3)
Hancocks Gully at Princes Highway - Location of Retarding Basin Wetland W1[4]			18.1	40.50				
Hancocks Gully at Pakenham Bypass			22.4					
Hancocks Gully at Railway			32.2					
Hancocks Gully at location of Retarding Basin Wetland W2W4 (Option 4)[5]	30.5	25.7	23.7	22.20	1,150,000	1,310,000	160,000	337,000
Western Tributary Flow into Deep Creek at Northern PSP boundary[1]	4.3		4.9					
Western Tributary Flow into Deep Creek at Princes Highway[1,6] - Location of Retarding Basin Wetland W3	4.1	3.3	2.3	42.30	95,500	123,000	27,500	57,700
Western Tributary Flow at Freeway			26.1					

1 - Assumes most of the flow from Subareas T - X does not directly enter Deep Creek
2 - Predevelopment Critical Duration at all locations = 9 hours (100 yr)
3 - Post Development Critical Duration = 9- 12 hours (100 yr) (except for West Trib at Freeway = 20 mins)
4 - W1 retarding basin design modified in 2014 to allow for off line sediment ponds and reduce flood retention
5 - Higher flow than Option 3 due to model changes, development assumed b/w railway and bypass and less storage effects in the upstream catchment
6 - Slight change from previous model due to model configuration changes

Pre - development RORB model - PREDEV_FEB_2014.CAT
Post - development RORB model - OPTION4_MARCH 2014.CAT

Table C.6 RORB Results – Option 4

APPENDIX D Stage/Storage/Discharge Relationships – Option 1

The RORB model constructed to assess Option 1 incorporated the following Stage/Storage Discharge relationships for Retarding Basin/Wetland systems W1, W2, W3 and W4. These are based on the design extents, wetland normal water levels, batter requirements etc detailed in 1304/1 and the outlet requirements detailed below.

All levels and volumes are at the concept design stage only and are subject to change.

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W1 – Option 1

Number of Outlet Culverts =	4	
Diameter =	1.05	m
Length =	80	m
Upstream IL =	37.5	m AHD
Downstream IL =	37	m AHD
Downstream Obvert =	38.05	m AHD
Long. Slope (1/)=	160	
Mannings n =	0.013	

Wetland NWL = 39.0 m AHD

Wetland TED = 39.5 m AHD

Pit outlet to relatively low pipe. Pipe invert well below NWL

Flow (m^3/s)	Head Water Level (m AHD)	Storage (m^3)	
0	39	0	Wetland Normal Water Level
0.1	39.5	22600	Wetland TED
0.5	39.6	28000	sill weir domination of outflow assumed
4.4	40.0	48000	sill weir domination of outflow assumed
13.7	40.5	75000	culvert outflow dominates
15.0	41.0	106000	culvert outflow dominates

Assumes pit outlet - sill @39.5, invert pipe = 37.5 m AHD

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W2 – Option 1

Number of Culverts =	5	
Diameter =	1.05	m

Length =	80	m
Upstream IL =	28.2	m AHD
Downstream IL =	28	m AHD
Downstream Obvert =	29.05	m AHD
Long. Slope (1/)=	400	
Mannings n =	0.013	

Wetland NWL = 28.2 m AHD

Wetland TED = 28.7 m AHD

Pipe outlet – upstream and downstream IL set to allow feed to existing frog ponds

Flow (m^3/s)	Head Water Level (m AHD)	Storage (m^3)	
0	28.2	0	Wetland Normal Water Level
0.1	28.7	24475	Wetland TED - weir on pipe headwall sets TED
0.5	29.3	57325	sill weir domination of outflow assumed
5.1	29.7	82000	sill weir domination of outflow assumed
13.5	30.2	113575	culvert outflow dominates
16.0	30.7	149575	culvert outflow dominates
18.2	31.2	185575	culvert outflow dominates

<u>Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W3 – Option 1</u>

Number of Culverts =	3	
Diameter =	0.9	m
Length =	65	m
Upstream IL =	40.3	m AHD
Downstream IL =	40	m AHD
Downstream Obvert =	40.9	m AHD
Long. Slope (1/)=	216.666667	
Mannings n =	0.013	

Wetland NWL = 40.3 m AHD

Wetland TED = 40.8 m AHD

Flow (m^3/s)	Head Water Level (m AHD)	Storage (m^3)	
0	40.3	0	Wetland Normal Water Level
0.04	40.8	10775	Wetland TED - weir on pipe headwall sets TED
0.3	41.00	16000	
0.5	41.21	22000	
1.0	41.44	30000	
1.5	41.90	42000	
2.0	42.17	53000	
2.5	42.37	60000	

Accounts forsome backwater effects from Deep Creek in flood events

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W4 – Option 1

Number of Culverts =	4	
Diameter =	0.90	m
Length =	50	m
Upstream IL =	28.5	m AHD
Downstream IL =	28.3	m AHD
Downstream Obvert =	29.20	m AHD
Long. Slope (1/)=	250	
Mannings n =	0.013	

Wetland NWL = 28.5 m AHD

Wetland TED = 29.0 m AHD

Flow (m^3/s)	Head Water Level (m AHD)	Storage (m^3)	
0	28.5	0	Wetland Normal Water Level
0.04	29	21000	Wetland TED - weir on pipe headwall sets TED
2.0	29.10	26000	Pipe half full
3.0	29.57	50000	Pipe running full
3.5	29.65	56000	Pipe running full
4.0	29.74	62000	Pipe running full
6.0	30.22	90000	Pipe running full

APPENDIX E Stage/Storage/Discharge Relationships – Option 2

The RORB model constructed to assess Option 2 incorporated the following Stage/Storage Discharge relationships for Retarding Basin/Wetland systems W1, W2, W3 and W4. These are based on the design extents, wetland normal water levels, batter requirements etc detailed in 1304/2 and the outlet requirements detailed below.

All levels and volumes are at the concept design stage only and are subject to change.

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W1 – Option 2

As with Option 1 – Appendix D

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W2 – Option 2

Number of Culverts =	8	
Diameter =	1.05	m
Length =	45	m
Upstream IL =	23.2	m AHD
Downstream IL =	23	m AHD
Downstream Obvert =	24.05	m AHD
Long. Slope (1/)=	225	
Mannings n =	0.013	

Wetland NWL = 23.2 m AHD

Wetland TED = 23.5 m AHD

Flow (m^3/s)	Head Water Level (m AHD)	Storage (m^3)	
0	23.2	0	Wetland Normal Water Level
0.1	23.5	16600	Wetland TED - weir on pipe headwall sets TED
6.2	24.0	47730	sill weir domination of outflow assumed
13.4	24.5	97605	culvert outflow dominates
19.0	25.0	147480	culvert outflow dominates - flooding outside cut line

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W3 – Option 2

As with Option 1 – Appendix D

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W4 – Option 2

Number of Culverts =	2	
Diameter =	1.05	m
Length =	45	m
Upstream IL =	26.5	m AHD
Downstream IL =	26.3	m AHD
Downstream Obvert =	27.35	m AHD
Long. Slope (1/)=	225	
Mannings n =	0.013	

Wetland NWL = 26.5 m AHD

Wetland TED = 26.8 m AHD

Flow (m^3/s)	Head Water Level (m AHD)	Storage (m^3)	
0	26.5	0	Wetland Normal Water Level
0.1	26.8	13600	Wetland TED - weir on pipe headwall sets TED
0.5	27.0	32000	sill weir domination of outflow assumed
4.0	28.0	77000	culvert outflow dominates
6.4	29.0	148000	culvert outflow dominates - flooding outside cut line

APPENDIX F Stage/Storage/Discharge Relationships – Option 3

The RORB model constructed to assess Option 3 incorporated the following Stage/Storage Discharge relationships for Retarding Basin/Wetland systems W1, W3 and W2W4. These are based on the design extents, wetland normal water levels, batter requirements etc detailed in 1304/3 and the outlet requirements detailed below.

All levels and volumes are at the concept design stage only and are subject to change.

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W1 – Option 3

As with Option 1 – Appendix D

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W3 – Option 3

As with Option 1 – Appendix D

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W2W4 – Option 3

Number of Culverts =	6	
Diameter =	1.05	m
Length =	60	M
Upstream IL =	23.2	m AHD
Downstream IL =	23	m AHD
Downstream Obvert =	24.05	m AHD
Long. Slope (1/)=	300	
Mannings n =	0.013	

Wetland NWL = 23.2 m AHD

Wetland TED = 23.5 m AHD

Flow (m^3/s)	Head Water Level (m AHD)	Storage (m^3)	
0	23.2	0	NWL
0.1	23.5	35300	TED
5.1	24.0	99000	sill weir domination of outflow assumed
10.2	24.5	174500	culvert outflow dominates
14.4	25.0	250000	culvert outflow dominates
18.0	25.5	360000	culvert outflow dominates
21.0	26.0	470000	culvert outflow dominates

APPENDIX G Stage/Storage/Discharge Relationships – Option 4

The RORB model constructed to assess Option 4 incorporated the following Stage/Storage Discharge relationships for Retarding Basin/Wetland systems W1, W3 and W2W4. These are based on the design extents, wetland normal water levels, batter requirements etc detailed in 1304/3 and the outlet requirements detailed below.

All levels and volumes are at the concept design stage only and are subject to change.

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W1 – Option 4

Slight changes to the Option 1 assumptions as the 2014 review looked at providing off line sediment ponds at this location and maximising RB outflow given the highway culvert capacities

Number of Outlet Culverts =	4	
Diameter =	1.20	m
Length =	80	m
Upstream IL =	37.5	m AHD
Downstream IL =	37	m AHD
Downstream Obvert =	38.20	m AHD
Long. Slope (1/)=	160	
Mannings n =	0.013	

Flow (m^3/s)	Head Water Level (m AHD)	Storage (m^3)	
0	39.2	0	Wetland Normal Water Level
0.1	39.5	11000	Wetland TED
0.5	39.6	15000	sill weir domination of outflow assumed
4.4	40.0	32000	sill weir domination of outflow assumed
18.0	40.5	58000	culvert outflow dominates
20.0	41.0	84000	culvert outflow dominates

Assumes pit outlet - sill @39.5, invert pipe = 37.5 m AHD

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W3 – Option 4

As with Option 1 – Appendix D

Stage /Storage /Discharge Relationship – Retarding Basin/Wetland W2W4 – Option 4

2 YEAR CONTROL

Number of Outlet Culverts =	5	
Diameter =	0.75	m
Length =	30	m
Upstream IL =	20.2	m AHD
Downstream IL =	20	m AHD
Downstream Obvert =	20.75	m AHD
Long. Slope (1/)=	150	
Mannings n =	0.013	

100 YEAR CONTROL

Number of Outlet Culverts =	6	
Diameter =	1.20	m
Length =	30	m
Upstream IL =	20.2	m AHD
Downstream IL =	20	m AHD
Downstream Obvert =	21.20	m AHD
Long. Slope (1/)=	150	
Mannings n =	0.013	

Flow (m^3/s)	Head Water Level (m AHD)	Storage (m^3)	
0	20.2	0	Wetland Normal Water Level
0.1	20.5	33900	Wetland TED
0.5	20.6	45900	sill weir domination of outflow assumed
2.3	21	99500	culvert outflow dominates
4.0	21.45	170375	culvert outflow dominates - 2 yr control
14.3	21.5	179215	culvert outflow dominates
21.5	22	284865	
26.8	22.5	407115	

APPENDIX H Stormwater Pollutant Modelling – Current Best Practice

The performance of the stormwater quality management system outlined in Section 5 and detailed in Appendix A was analysed using the MUSIC model, Version 5.

Subareas and fraction imperviousness are as detailed in Appendix C above and are consistent with the RORB model. Sub areas are subject to change given the final development layout, however, provided the criteria of providing wetland systems and sediment ponds is adhered to, the final MUSIC results are not expected to change significantly.

Sediment pond and wetland sizes are as detailed in Appendix A. Wetland areas are taken as the average area between normal water level (NWL) and Top of Extended Detention (TED). Wetland areas also account for areas taken up the sediment ponds. The results are detailed below. Wetland extended detention times are three days, except for wetland systems such as W1 and W2 which incorporate 2 days each as they are systems in series.

Bureau of Meteorology rainfall and evaporation data available in Koo Wee Rup (2004) at 6 minute intervals was utilised. This is the reference gauge defined by MWC for this area of Melbourne.

It is clear that there is room to move in regard to system size, shape and orientation. However, it is considered that the sizes and shapes of drainage and WSUD elements detailed in Appendix A are realistic given:

- The outlet invert level constraints,
- Existing frog pond outlet constraints at the Pakenham Bypass,
- Gas line level constraints in regard to wetland inlet and outlet invert levels and the resulting required normal water levels,
- Batter requirements (1 in 8 batters (minimum) have been allowed from cut lines to the Normal water level of all systems at this stage),
- Stormwater pollutant retention requirements, and
- Flood retardation requirements.

Note that a full water balance of all systems will be required at the functional design stage of the project to ensure water body turnover periods are sufficient to result in self sustaining wetland systems over time.

Tables H.1, H.2, H.3 and H.4 detail the MUSIC results.

Location	W2 outfall at Pakenham Bypass - Point D	W3 outfall to Deep Creek upstream of Princess Highway - Point A	W4 outfall at Pakenham Bypass - Point C
TSS Produced in the catchment (kg/yr)	594,000	109,000	188,000
Residual TSS Load (kg/yr)	119,000	17,000	24,000
TSS Retained on site	80%	84%	87%
TP Produced in the catchment (kg/yr)	1,430	223	402
Residual TP Load (kg/yr)	419	62	99
TP Retained on site	71%	73%	75%
TN Produced in the catchment (kg/yr)	9,890	1,610	2,850
Residual TN Load (kg/yr)	5,090	818	1,400
TN Retained on site	49%	49%	51%

Table H.1 MUSIC Results – Option 1

Location	W2 outfall at Railway - Point F	W3 outfall to Deep Creek upstream of Princess Highway - Point A	W4 outfall at Railway - Point E
TSS Produced in the catchment (kg/yr)	709,000	110,000	218,000
Residual TSS Load (kg/yr)	142,000	17,400	30,700
TSS Retained on site	80%	84%	86%
TP Produced in the catchment (kg/yr)	1,640	223	463
Residual TP Load (kg/yr)	525	62	125
TP Retained on site	68%	73%	73%
TN Produced in the catchment (kg/yr)	11,500	1,580	3,290
Residual TN Load (kg/yr)	6,340	804	1,690
TN Retained on site	45%	49%	49%

Table H.2 MUSIC Results – Option 2

Location	W2W4 outfall at Railway - Point F	W3 outfall to Deep Creek upstream of Princess Highway - Point A
TSS Produced in the catchment (kg/yr)	915,000	109,000
Residual TSS Load (kg/yr)	124,000	16,600
TSS Retained on site	86%	84%
TP Produced in the catchment (kg/yr)	2,110	217
Residual TP Load (kg/yr)	529	60
TP Retained on site	75%	73%
TN Produced in the catchment (kg/yr)	14,800	1,590
Residual TN Load (kg/yr)	7,050	816
TN Retained on site	52%	49%

Table H.3 MUSIC Results – Option 3

Location	W2W4 outfall Downstream of the railway	W3 outfall to Deep Creek upstream of Princess Highway - Point A
TSS Produced in the catchment (kg/yr)	1,020,000	106,000
Residual TSS Load (kg/yr)	184,000	16,300
TSS Retained on site	82%	84%
TP Produced in the catchment (kg/yr)	2,340	223
Residual TP Load (kg/yr)	708	61
TP Retained on site	70%	73%
TN Produced in the catchment (kg/yr)	16,500	1,580
Residual TN Load (kg/yr)	9,050	812
TN Retained on site	45%	49%

Table H.4 MUSIC Results – Option 4

As detailed above, it is clear that as the pollutant reductions meet or exceed the current best practice requirements of 80% TSS, 45% TP and 45% TN, the proposals will protect against adverse effects on the water quality downstream of the development, within the KWRFPD, and Westernport Bay.