# Byford Rail Extension
## Armadale Precinct DA5 Drainage Strategy

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# 1. Introduction

The purpose of this drainage strategy technical memo is to describe the overarching drainage design strategy that feeds into the CI-200 Package – civil design of Armadale Station Precinct to support the Development Application 5 (DA5). This stormwater management strategy supports DA5 in relation to the area north of Neerigan Brook and South of the Northern Viaduct Abutment. The extent of DA5 is bisected by Armadale Road.

## 1.1 Project Locality

The site is located in Armadale, Western Australia and bounded by Aragon Ct, 26/24 Aragon Ct, Neerigen St to the South, Railway Ave to the West, Streich Avenue to the East and the Northern Viaduct Abutment to the North.


**Figure 1. Project Locality**

## 1.2 Site Layout

The Precinct consists of two landscaped areas that site underneath the Byford Rail Extension viaduct structure. The two areas are bisected by Armadale Road but connected via a principle shared path and shared path bridge which runs in a north-south direction following the Byford Rail Extension rail line. Figure 2 shows the layout of the proposed design and the extents of the area shown by the dashed yellow line.


Figure 2. Site Layout

The existing stormwater runoff from the rail is contained within the rail reserve and infiltrated at site. Armadale Road drainage is contained within the road reserve via pit and pipe drainage and conveys overland flows to the west down Armadale Road and to the north via Streich Avenue. In rare events breakout flows from Neerigen Brook spillout to the North and South and flow into the DA5 area. These breakout flows run underneath Armadale Road into the rail reserve via two existing RCP culverts.

## 1.3 Catchment Plan

Drainage catchments for the Armadale Precinct (DA5 area) are as shown below in Figure 4.


Figure 3. Catchment Plan

The DA5 area can be broken up into 3 distinct catchments, namely South East, South West and North. The existing drainage regime for these areas deals with the stormwater runoff from any impervious areas locally by use of the low fraction impervious and significant clearance to groundwater. Runoff from the rail is contained within the rail reserve.

| Name | Area (ha) |
|---|---|
| North Catchment | 1.7 |
| South East Catchment | 0.2 |
| South West Catchment | 0.7 |

## 1.4 Existing Drainage Infrastructure

The City of Armadale (COA) have a network of pit and pipe infrastructure outside of the rail corridor which typically conveys flows from east to west to follow the natural topography. The drainage network typically runs along kerb lines and/or shoulders of local roads to the west of the rail corridor. Runoff from within the rail corridor is predominantly managed by a network of open earth channels and cross track culverts, typically discharging into the COA drainage network at select locations.

Armadale Road is owned and operated by MRWA and stormwater runoff is managed by traditional pit and pipe network and convey away from the project site to the west.

# 2. Design Criteria

## 2.1 Design Strategy

The overarching drainage strategy has the following primary objectives:

- Apply Water Sensitive Urban Design Principles
- Reduce post development runoff rates to predevelopment conditions
- Meet the SWTC requirements
- Meet the PTA Specification 8880-450-090: Design of Drainage for PTA Infrastructure

The design seeks to treat the first flush in bio-retention areas wherever possible and limit the discharge off-site to the predevelopment discharge rates as determined by hydrologic modelling. The predevelopment condition has been considered as the original unimproved site, i.e. 100% pervious.

## 2.2 Design Criteria Order of Precedence

The following design criteria and design guidelines have been considered during the development of the Armadale (DA5) Drainage Strategy. The order in which these are listed below denotes the order of precedence as it applies to the strategy.

4.1.1 SWTC-BRE-PTAWA-PM_RPT-00007

4.1.2 8880-450-090 – Specification: Design of Drainage for PTA Infrastructure

4.1.3 A Guide to Water Sensitive Urban Design for Public Transport Infrastructure in Western Australia

4.1.4 Stormwater Management Manual of Western Australia

4.1.5 Australian Runoff Quality: A Guide to WSUD

4.1.6 Adoption Guidelines for Stormwater Biofiltration Systems produced by the CRC for Water Sensitive Cities

## 2.3 SWTC -BRE-PTAWA-PM-RPT-00007

Relevant extracts from the SWTC are reproduced below:

4.1.7 2.1.3-10 - The maximum storage depth of bio-retention areas in the station precinct where accessible to public shall be limited to 300mm with 1 in 4 batter slopes.

4.1.8 2.1.3-9 - The maximum emptying time for infiltration drainage system within the station precinct and outside the rail reserve shall comply with the Stormwater Management Manual for Western Australia.

**Table 1 – Extract from Stormwater management manual chapter 9 Table 5**

| AEP | 1EY | 0.5EY | 0.2EY | 10% | 5% | 2% | 1% |
|---|---|---|---|---|---|---|---|
| Maximum Emptying Time in days | 0.5 | 1 | 1.5 | 2 | 2.5 | 3 | 3.5 |

4.1.9 2.1.3-8 Provision of bio-retention filter media shall comply with the Stormwater Management Manual for Western Australia

4.1.9.1 Typically, filter media consists of a sandy loam with a saturated hydraulic conductivity between 50 and 300 mm/hr

4.1.10 9.2.3-1 All rainwater run-off from roofed and platform paved areas shall be collected and be disposed of onsite via soakwells or via the local area stormwater drainage system where the NOP actions make this possible.

4.1.11 8880-450-090 - Specification: Design of Drainage for PTA Infrastructure

Relevant extracts from the Specification are reproduced below:

### 4.1.12 Table 7 Drainage Annual Exceedance Probabilities for Outside the Rail Reserve

**Table 2 – Extract from 880-450-090 (Table 7: Drainage Annual Exceedance Probabilities for Outside the Rail Reserve)**

| Item | Situation | AEP % |
|---|---|---|
| 1 | Major system check: TWL to property and railway building floor levels with 300mm freeboard | 1 |
| 2 | Stormwater drainage contained in principal shared path (PSP) corridor: PSP crossfall shall be away from the rail reserve.<br>For larger storms and major storm overland flow paths, and where discharge into PTA rail reserve is unavoidable, this shall be communicated with and accepted by the PTA prior to construction. | 20 |
| 3 | Water Corporation main / branch drains. | |
| 4 | Kerb overtopping. | 20 |
| 5 | Drainage basins and sumps | 10 |
| 6 | Swales and open drains. | 20 |
| 7 | Gutter flow spread limits. | 20 |
| 8 | Piped system with 150 mm of freeboard from HGL to FSL. | 20 |
| 9 | Groundwater level (dry subgrade). | 2 |
| 10 | Drainage system overflows that might cause erosion or scour. | 10 |
| 11 | Drainage basin backwater onto pavement. | 5 |
| 12 | Swales and open drains backwater onto pavement. | 10 |

4.2 2.3.5.13 - No part of the carparks shall be flooded, or inundated, during any storm event smaller than the 10% AEP storm event. The depth of stormwater during the 1% AEP event shall not be more than 200 millimetres in any part of the carpark, at any time, and there shall not be any ponding of stormwater for longer than six hours in any part of the carpark during a 1% AEP storm event

4.3 2.3.5.5 - Stormwater runoff from constructed impervious surfaces generated by the first 15 mm of rainfall from a frequently occurring event shall be retained and/or detained, and treated (if required) at the source as much as practical to meet WSUD requirements

4.4 2.3.17.5 - Infiltration / detention basins shall be designed to include a stormwater biofilter (where treatment of runoff is required) unless otherwise approved by the PTA. Biofilters shall be designed and installed in accordance with the Adoption Guidelines for Stormwater Biofiltration Systems produced by the CRC for Water Sensitive Cities. Where treatment of runoff is not required, basins/flood storage areas shall be designed with vegetative retention/detention systems noting that the root systems of vegetation help to minimise potential soil clogging and maintain infiltration of runoff.

4.5 2.3.15.1 - Any discharge into existing drains shall be compensated to reduce peak flows to pre-development flows or limits acceptable to the controlling authorities

4.6 2.3.15.2 a. - Infiltration into natural surface: If the soil permeability is adequate and no adverse environmental or community effects will result from standing water up to 96 hours, the run-off shall be managed in open drains and swales to infiltrate. Drain blocks at regular intervals and based on hydraulic calculations can be used to maximise infiltration. Excess run-off shall be treated by passing through a vegetated detention basin or approved treatment system. In the sites with potential high-risk pollution (e.g. fuel filling or storage areas, station open carparks, open train and other vehicle depot), first flush runoff should have appropriate treatment before infiltrating to groundwater or discharging to downstream environment when infiltration is not feasible.

# 3. Hydrologic Input Data

The Hydrologic input data used in the drainage strategy is described in this section.

## 3.1 Rainfall

Rainfall data is extracted from the Bureau of Meteorology and is provided as Intensity-Frequency-Durations (IFD).

### 3.1.1 Intensity Frequency Duration

Rainfall IFDs were extracted from the Bureau of Meteorology Website for the site location. The IFDs have been republished below.

Table 3 - IFD Extraction Data and Location

| IFD Design Rainfall Depth (mm) | | | | |
|---|---|---|---|---|
| Issued: | 14912-23 | | | |
| Location Label: | | | | |
| Requested coordinate: | Latitude | -32.155 | Longitude | 116.013 |
| Nearest grid cell: | Latitude | 32.1625 (S) | Longitude | 116.0125 (E) |

Table 4 - IFD Data

| Duration in min | 63.20% | 50% | 20% | 10% | 5% | 2% | 1% |
|---|---|---|---|---|---|---|---|
| 1 | 1.82 | 2.01 | 2.64 | 3.09 | 3.55 | 4.19 | 4.71 |
| 2 | 3.21 | 3.51 | 4.51 | 5.23 | 5.98 | 7.01 | 7.86 |
| 3 | 4.3 | 4.71 | 6.07 | 7.07 | 8.09 | 9.52 | 10.7 |
| 4 | 5.19 | 5.7 | 7.39 | 8.62 | 9.89 | 11.7 | 13.1 |
| 5 | 5.93 | 6.54 | 8.52 | 9.96 | 11.4 | 13.5 | 15.2 |
| 10 | 8.57 | 9.48 | 12.5 | 14.6 | 16.8 | 19.9 | 22.4 |
| 15 | 10.3 | 11.4 | 15.1 | 17.7 | 20.3 | 24 | 26.9 |
| 20 | 11.7 | 12.9 | 17 | 19.9 | 22.9 | 27 | 30.3 |
| 25 | 12.8 | 14.1 | 18.5 | 21.7 | 25 | 29.5 | 33.1 |
| 30 | 13.7 | 15.2 | 19.9 | 23.3 | 26.7 | 31.5 | 35.4 |
| 45 | 16.1 | 17.7 | 23 | 26.9 | 30.9 | 36.6 | 41.1 |
| 60 | 18 | 19.7 | 25.5 | 29.8 | 34.3 | 40.6 | 45.7 |
| 90 | 21 | 22.9 | 29.5 | 34.4 | 39.6 | 47.1 | 53.3 |
| 120 | 23.4 | 25.5 | 32.7 | 38.2 | 44 | 52.5 | 59.7 |
| 180 | 27.2 | 29.6 | 37.9 | 44.3 | 51.2 | 61.5 | 70.4 |
| 270 | 31.7 | 34.4 | 44.1 | 51.6 | 59.9 | 72.3 | 83.1 |
| 360 | 35.3 | 38.3 | 49.1 | 57.5 | 66.8 | 81 | 93.4 |
| 540 | 40.9 | 44.4 | 57 | 66.9 | 77.8 | 94.6 | 109 |

### 3.1.2 Pre-burst

Median Pre-burst Rainfall depths were applied to rainfall data to account for ARR Data Hub storm losses.

The conversion from storm Initial losses to burst initial losses is shown in Equation 1.

$$\text{Burst initial loss} = \text{Storm initial losses} - \text{Pre} - \text{burst rainfall (for Burst initialloss} \geq 0)$$

Equation 1

The median pre-burst rainfall data is published in Table 5.

Table 5 - Median Pre-burst rainfall depths

| min (h)\AEP(%) | 50 | 20 | 10 | 5 | 2 | 1 |
|---|---|---|---|---|---|---|
| 60 (1.0) | 6.4 | 6.3 | 6.3 | 6.3 | 5.6 | 5.1 |
| 90 (1.5) | 7.3 | 7.5 | 7.6 | 7.7 | 7.9 | 8 |
| 120 (2.0) | 3.4 | 4.7 | 5.6 | 6.5 | 6.5 | 6.5 |
| 180 (3.0) | 3.2 | 3.8 | 4.3 | 4.7 | 5.1 | 5.4 |
| 360 (6.0) | 1.9 | 2.1 | 2.2 | 2.3 | 3.1 | 3.7 |
| 720 (12.0) | 0.6 | 0.7 | 0.7 | 0.7 | 1.9 | 2.8 |
| 1080 (18.0) | 0.2 | 0.2 | 0.2 | 0.2 | 1 | 1.5 |
| 1440 (24.0) | 0 | 0 | 0 | 0 | 0.4 | 0.7 |
| 2160 (36.0) | 0 | 0 | 0 | 0 | 0 | 0 |
| 2880 (48.0) | 0 | 0 | 0 | 0 | 0 | 0 |
| 4320 (72.0) | 0 | 0 | 0 | 0 | 0 | 0 |

## 3.2 Losses

Initial and continuing losses have been extracted from the ARR Data Hub for use in hydrologic modelling. The values are shown in Table 6.

Table 6 - Storm Losses

| ID | Value |
|---|---|
| Storm Initial Losses (mm) - Pervious | 26 |
| Storm Continuing Losses (mm/h) - Pervious | 6 |
| Storm Initial Losses (mm) - Impervious | 1 |
| Storm Continuing Losses (mm/h) - Impervious | 0 |
| Antecedent Moisture Condition (AMC) | 3 – Rather Wet |
| Soil Type/Classification | C – Slow Infiltration rates |

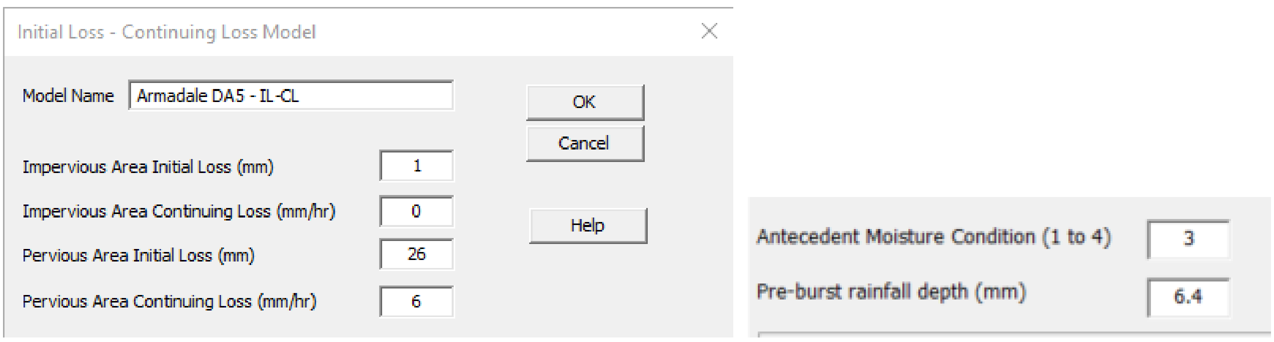


Figure 4: Depression Storage and Soil Classification

# 4. Storm Water Design Strategy

## 4.1 DA5 Area

The stormwater strategy for the DA5 area is to:

1. Maintain post development flow to pre-development levels by use of best management practices.
2. Treat First flush prior to discharge.
3. Manage runoff from LGA, PTA and MRWA assets separately where possible.

The main assets producing impervious area are the Viaduct structure, the PSP including the PSP bridge and footpaths.

## 4.2 DA5 Drainage Strategy

### 4.2.1 Viaduct Drainage

Runoff from the viaduct is collected on structure via concrete channels and grated inlets. The flow is conveyed to downpipes which run down the viaduct columns.

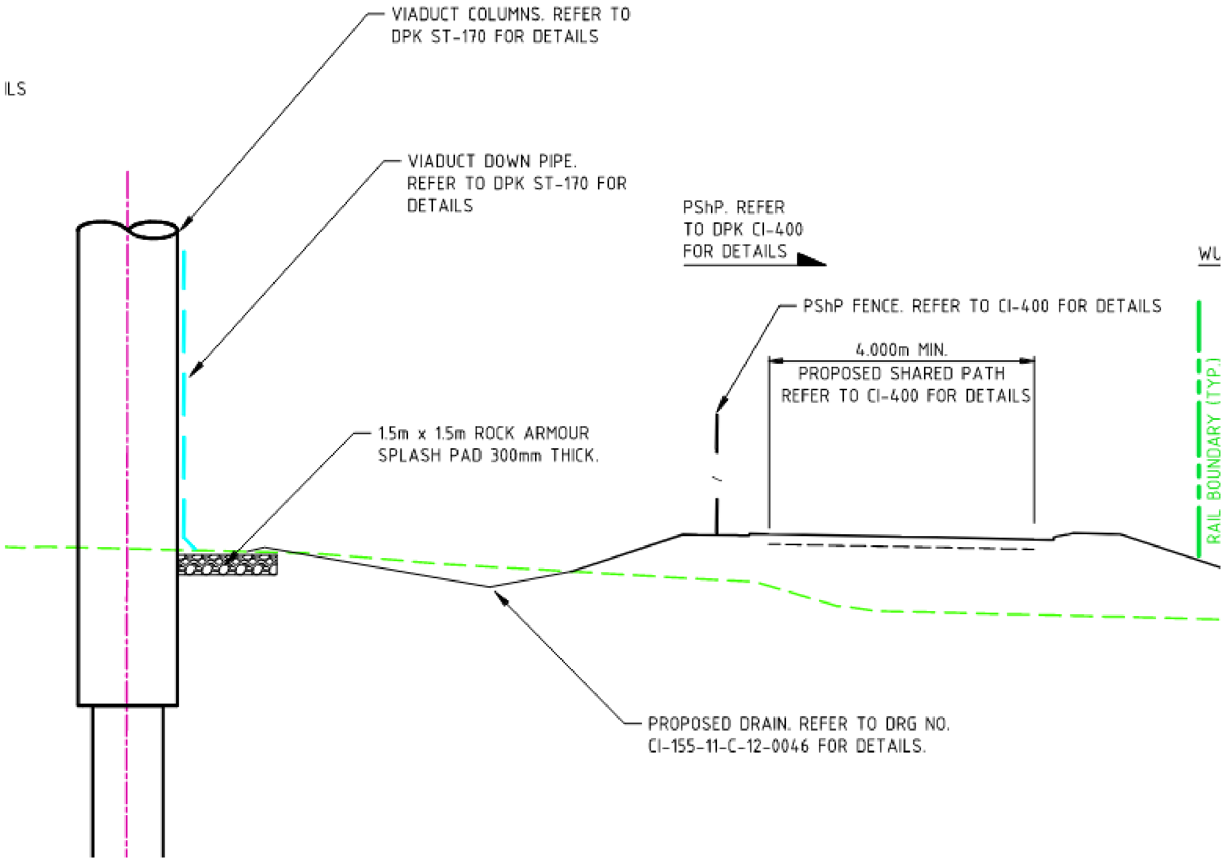


Figure 5 – Indictive Discharge from Viaduct to splash and shallow swale

The figure below provides a less engineered alternative splash pad layout to blend in with the surrounding landscape design. Showing column downpipe runoff discharging onto a splash pad made up natural creek like stone which is then connected to treatment swale.

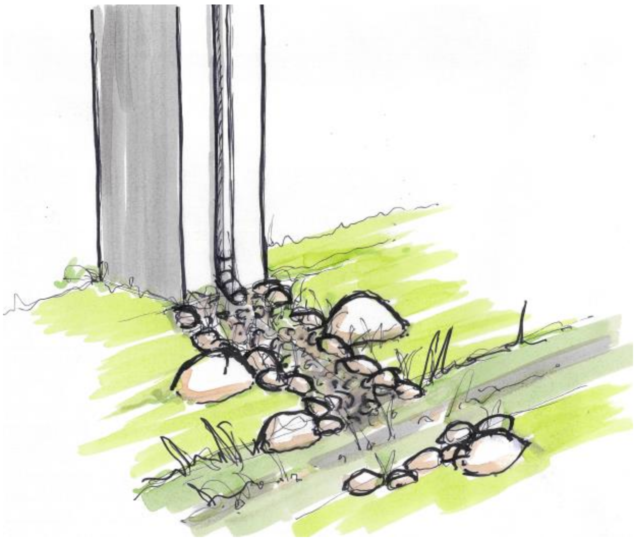

Figure 6– Possible Splash Pad and Swale design Integration.

### 4.2.2 WSUD and Landscape Design Integration

Water Sensitive Urban Design (WSUD) is key to providing effective ways to minimise impacts of the rail development on waterways within the site. The site will provide treatment of storm water and overland water flow pollutants onsite for treatment via vegetated swales and planting. The design and implementation of WSUD systems will be jointly coordinated with Civil, Hydrology and Landscape.

These will be reflected in the design drawings noting size and volume of treatment areas, choice of filtration media and selection of specific plant species for the job. The aim is to provide drainage entities that are more focused on a 'natural' aspect rather than hard engineered and should blend into the landscape. The use of local materials (stone and boulders) and vegetation will assist providing this creek like outcome for stormwater flows from viaduct downpipes and connect with the Hills character the Armadale Station is apt to reflect. This will also manage flows directing stormwater where it needs to go and mitigate any erosion.

Figure 10 below shows Precedence imagery showing creek like entities to manage stormwater flow from viaduct downpipes to treatment swales and basins.

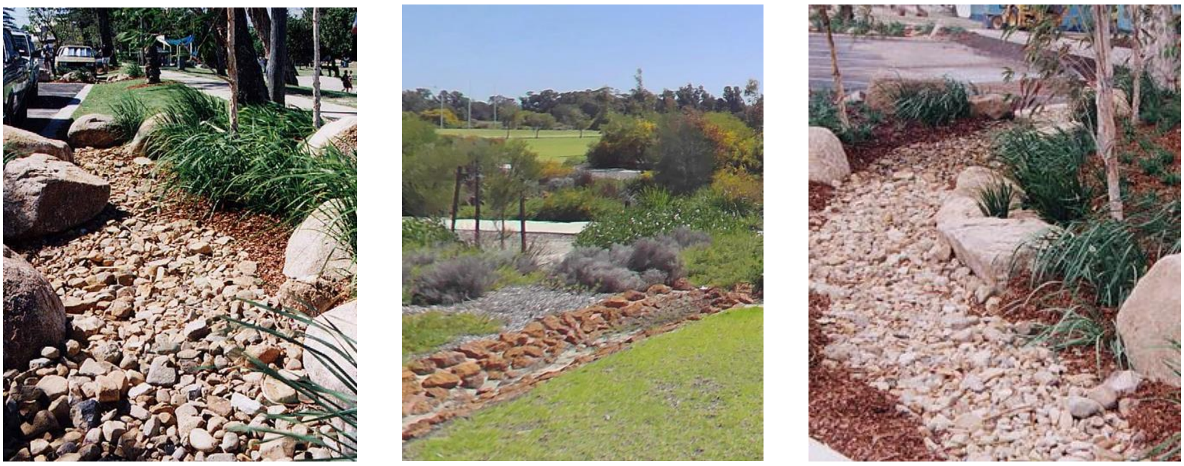

Figure 7 Precedence imagery showing creek like entities

The first flush volume can be accounted for in the design of the treatment swales through use of drain blocks.

### 4.2.3 Runoff Assessment

The viaduct structure is considered as an impervious surface, discharging runoff down to ground level at every column support.

Runoff generated at ground level is calculated based on the landscaping coverage as shown in the LA-230 package. Coverage is mostly pervious with small areas of impervious area primarily from footpaths, PShP.

The catchment areas and flows for the 1% AEP Storm event have been calculated in DRAINS and results shown in Table 7 below for information. For each section of viaduct, the critical duration is 5 mins for a catchment area of 435m2 and a fraction impervious of 1.0.

The viaduct drainage discharges via splash pads to the Western overland flow path discussed in section 4.2.5.1.

Table 7 – DA5 Viaduct Runoff Data

| Start Column | End Column | Catchment Area (m2) | 1% AEP Flow (L/s) | Discharge Location |
|---|---|---|---|---|
| North Abutment | 1 | 435 | 22 | Western Overland Flow Channel |
| 1 | 2 | 435 | 22 | Western Overland Flow Channel |
| 2 | 3 | 435 | 22 | Western Overland Flow Channel |
| 3 | 4 | 435 | 22 | Western Overland Flow Channel |
| 4 | 5 | 435 | 22 | Western Overland Flow Channel |
| 5 | 6 | 435 | 22 | Western Overland Flow Channel |
| 6 | 7 | 435 | 22 | Western Overland Flow Channel |
| 7 | 8 (North of Armadale Rd) | 435 | 22 | Western Overland Flow Channel |
| 8 | 9 (Armadale Rd Median) | 435 | 22 | Soakwell in Median |
| 9 | 10 (South of Armadale Rd) | 435 | 22 | Soakwell |
| 10 | 11 (South of PShP) | 435 | 22 | Overland flow discharging to Neerigen Brook |

### 4.2.4 Storage Requirements

Runoff from both the viaduct and area below is mitigated to control post-development flow rates to a predevelopment level. Storage volumes for this section are solely based on attenuation requirements for localised runoff only, runoff for the PShP is not considered as it is managed separately (considered as MRWA drainage).

Neerigen Brook

It should be noted that Neerigen Street floods during the 1% AEP event due to the Minnawarra Park Ponds breaching. The existing railway embankment acts as a flood protection bund to the immediate downstream properties with the Neerigen Street reserve providing temporary attenuation. Refer Figure below.

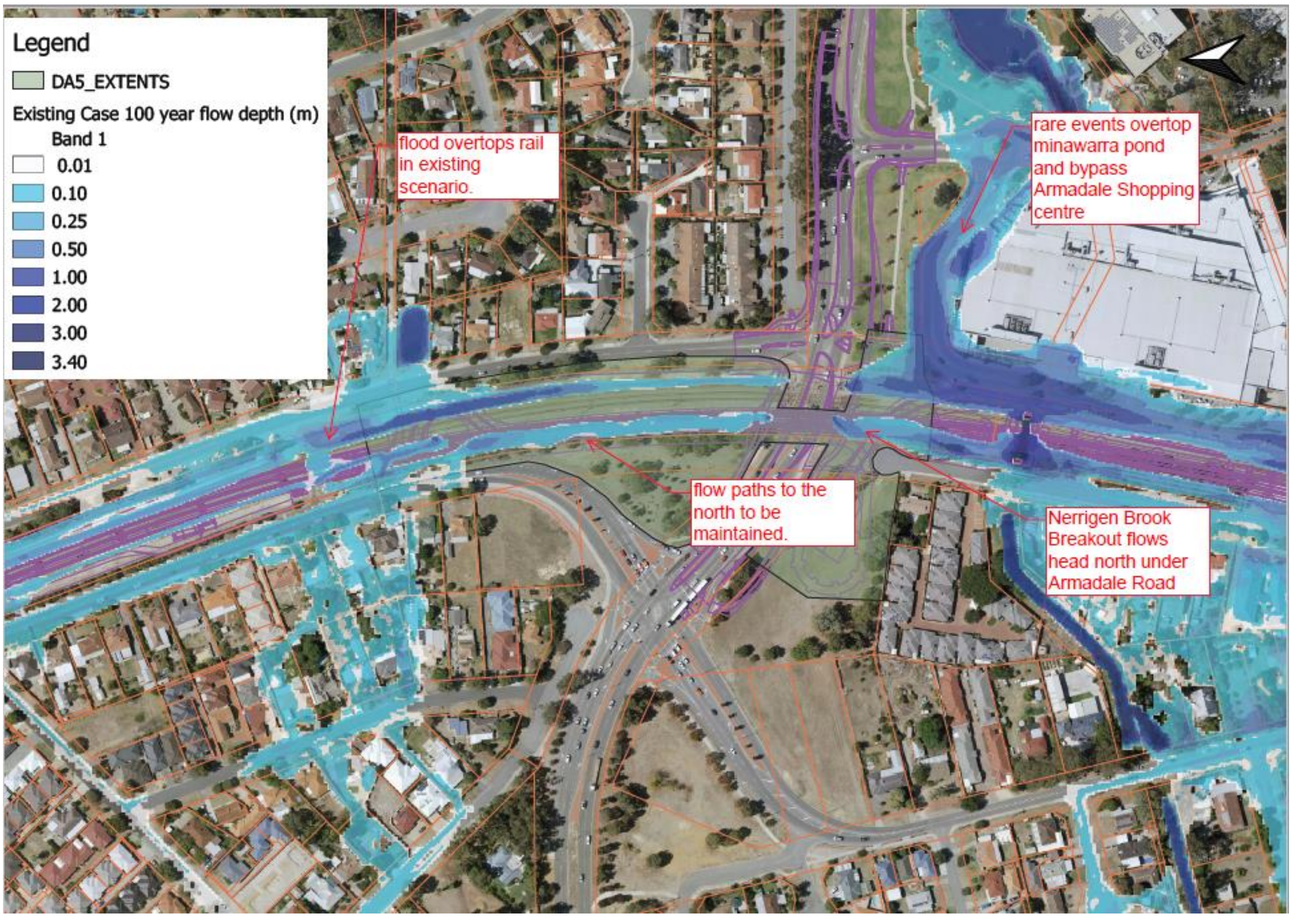


Figure 8 Existing 1% AEP Flood Map

For further flood and hydrological information refer to R30-MET-RPT-CI-160-00001 Linewide-Flood & Hydrology Report.

### 4.2.5 Overland Flow Paths

It is important to maintain existing overland flow paths to prevent adverse flooding outcomes in other areas. The two existing flow paths run north either side of the existing rail embankment.

#### 4.2.5.1 Western Overland Flowpath

The peak flow in the 1% AEP event under Armadale Rd on each side is 0.14m3/s and combined with Viaduct drainage and local catchment produces a peak flow of 0.323 $m^3/s$.

A typical trapezoidal channel with a 1m base and 1in8 side slopes requires a depth of 0.5m making allowance for a 100mm freeboard.

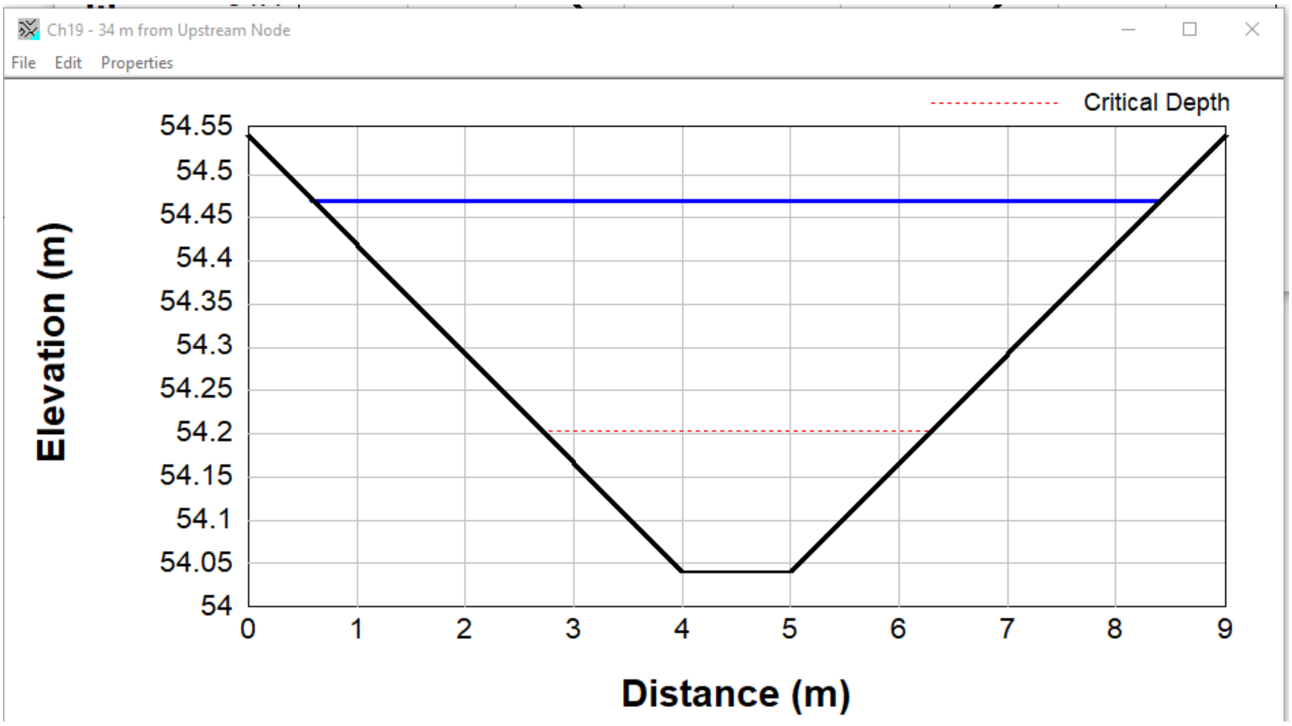


Figure 9 Typical Section for Western Overland Flow Path

Culverts under proposed footpaths where the footpaths intersect the overland flow paths to deal with the minor flooding event.

#### 4.2.5.2 Eastern Overland Flowpath

The Eastern Overland flowpath does not received viaduct drainage and as a result requires a shallower overland flow channel.

A peak flow of 0.236 $m^3/s$ for a typical trapezoidal channel with a 1m base and 1in8 side slopes requires a depth of 0.25m allowing for a 100mm freeboard.

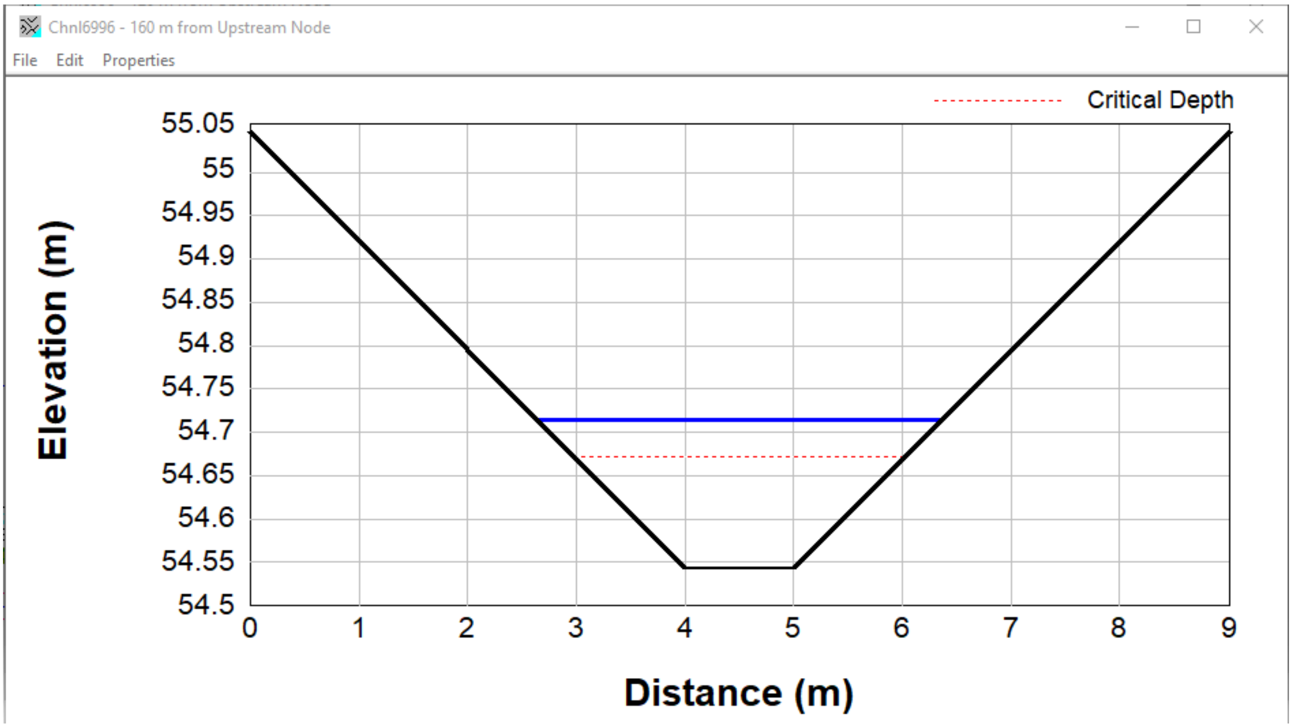


Figure 10 Typical Section for Eastern Overland Flow Path

### 4.2.6 Fraction Impervious (FI)

The fraction impervious has been calculated as Total Impervious Area (TIA) via GIS. Impervious areas were assessed using aerial imagery in the existing case and proposed design drawings in the developed case.

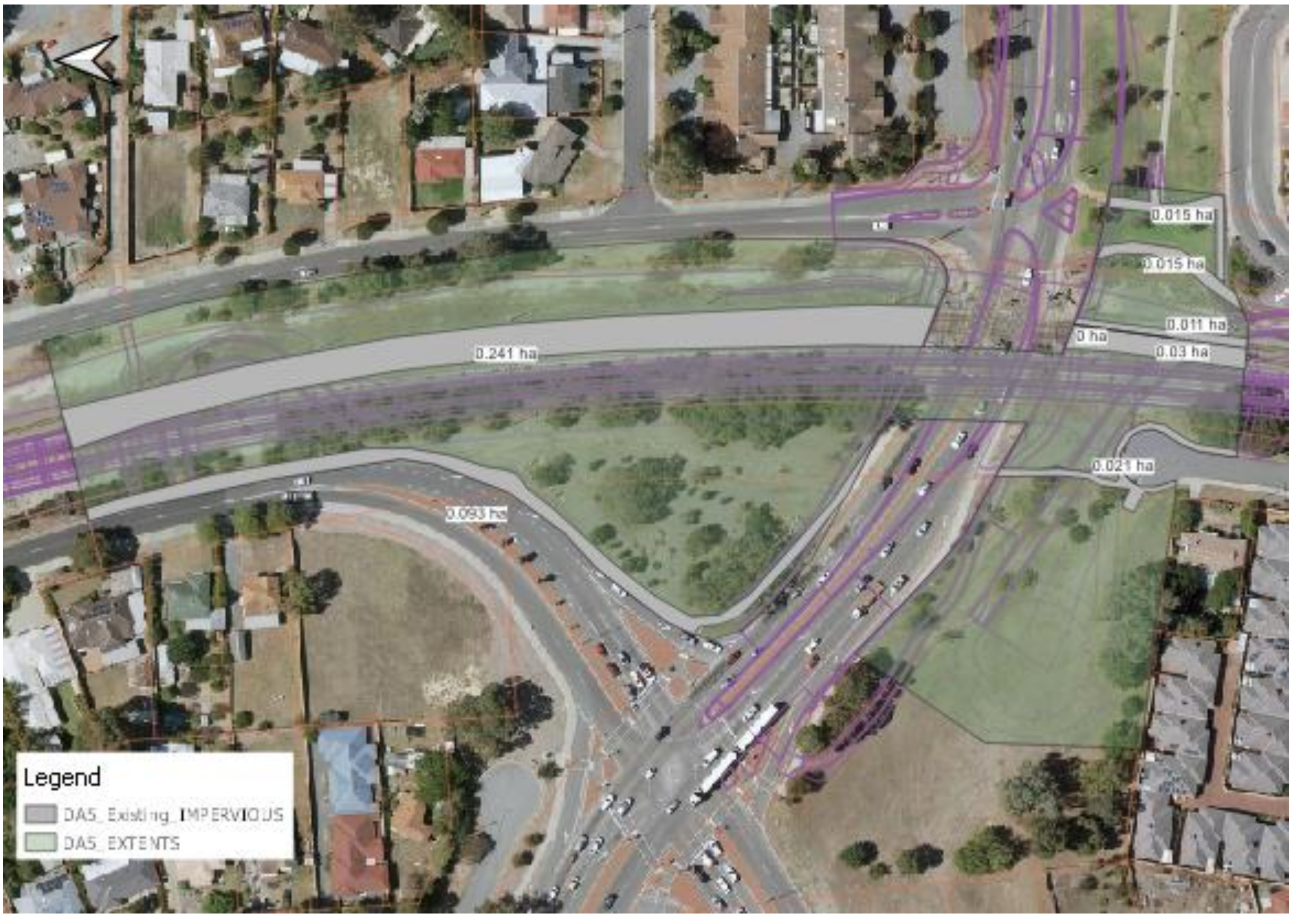


**Figure 11. Existing DA5 Area Fraction Impervious**

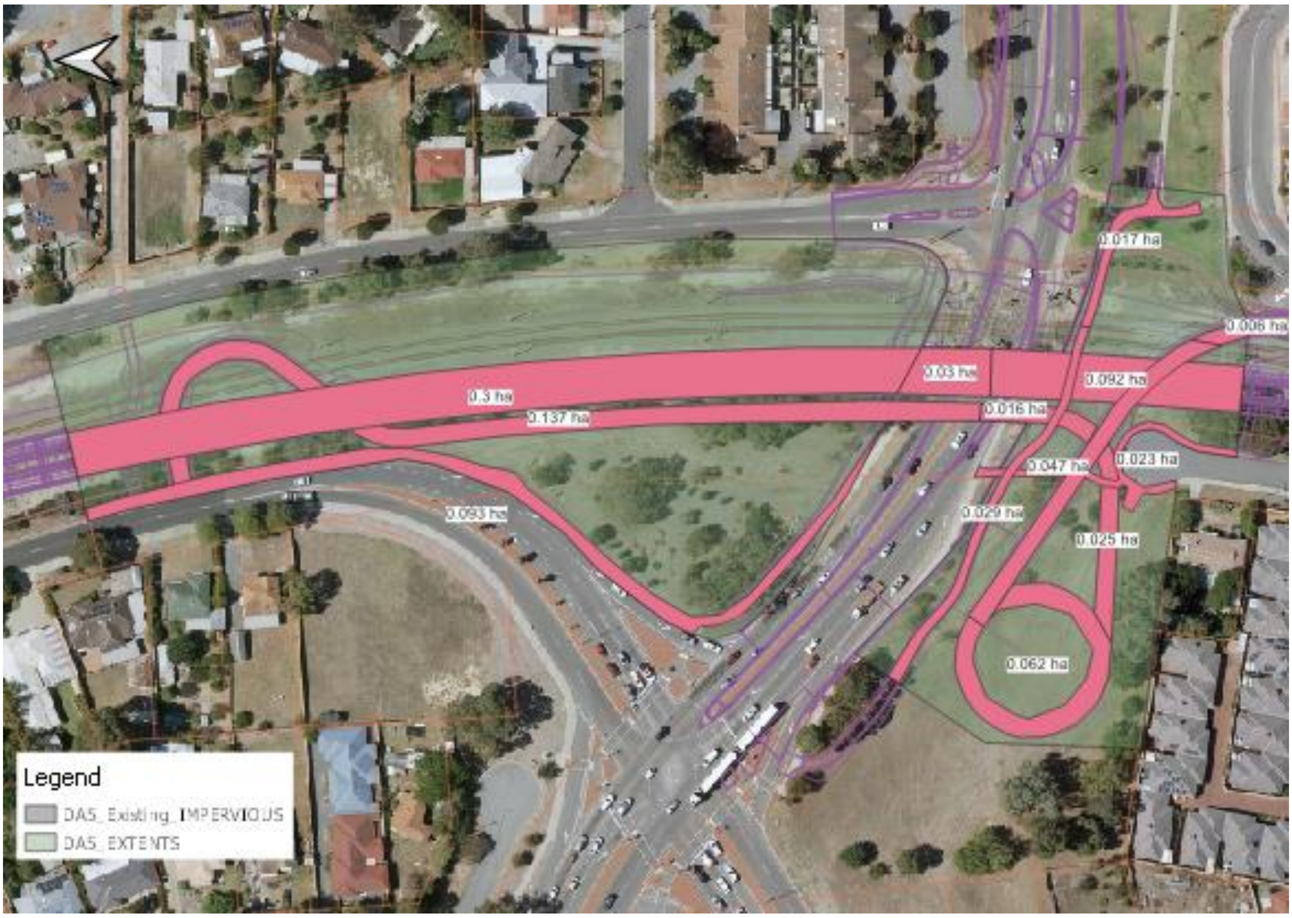


Figure 12. Developed DA5 Area Fraction Impervious

For the purposes of initial design and drainage storage volume assessment, a conservative approach has been undertaken, whereby pre-development catchments have been assessed as completely pervious. This will ensure sufficient on-site storage is achieved in order to limit post development run-off to predevelopment peak flows. During detailed design development within the station precinct, this catchment assessment will be refined to more accurately reflect pre-development conditions.

Table 8 Fraction Impervious Assessment

| Catchment | Area (ha) | Existing Impervious Area (ha) | Design Impervious Area (ha) | Existing FI | Design FI |
|---|---|---|---|---|---|
| South East | 0.2 | 0.043 | 0.023 | 0.22 | 0.12 |
| South West | 0.7 | 0.051 | 0.253 | 0.07 | 0.36 |
| North | 1.7 | 0.334 | 0.611 | 0.20 | 0.36 |

### 4.2.7 Required Storage Volumes

The storage volumes required refer to any volume of water detained on site. This can be achieved via above ground storage e.g., detention basins/swales, infiltration, or underground storage. Based on PTA drainage requirements, storage areas are designed predominantly as bio-retention basins, in order to achieve water quality objectives. These will be designed as far as possible to have a maximum water storage depth of 300mm, with a controlled high-level outlet, and underlying bio-retention media, allowing for percolation of low flow events. Larger flows would move towards tiered bio-retention basins and ultimately into existing stormwater drainage outlet pipe connections as per the existing condition piped outlets for major flows. The storage volume requirements for on-site detention basins as shown below is based on post-development catchment runoff into bio-retention/detention basins for major events up to the 1%AEP, whilst allowing for pre-development peak flow values for this equivalent event.

The required volumes can also be combined to be managed in a larger detention system if this is necessary.

Table 9 Required Storage Volumes

| *Area* | *Post-Development Detention Storage Volume ($m^3$)* | *First Flush Volume ($m^3$)* |
|---|---|---|
| South East | n/a | 4 |
| South West | 9 | 38 |
| North | 21 | 92 |

As the first flush requirement exceed the pre-post development storage requirement, it is sufficient to provide first flush storage only.

For each Pier the required first flush volume is 6.5m3.

### 4.2.8 Pre and Post Development Hydraulic Outputs

Pre-development, post-development unmitigated and post-development mitigates critical design flow result output are published in Figure 13 and Figure 14. The figures below show the minimal increase in peak flow and a slight increase in volume.

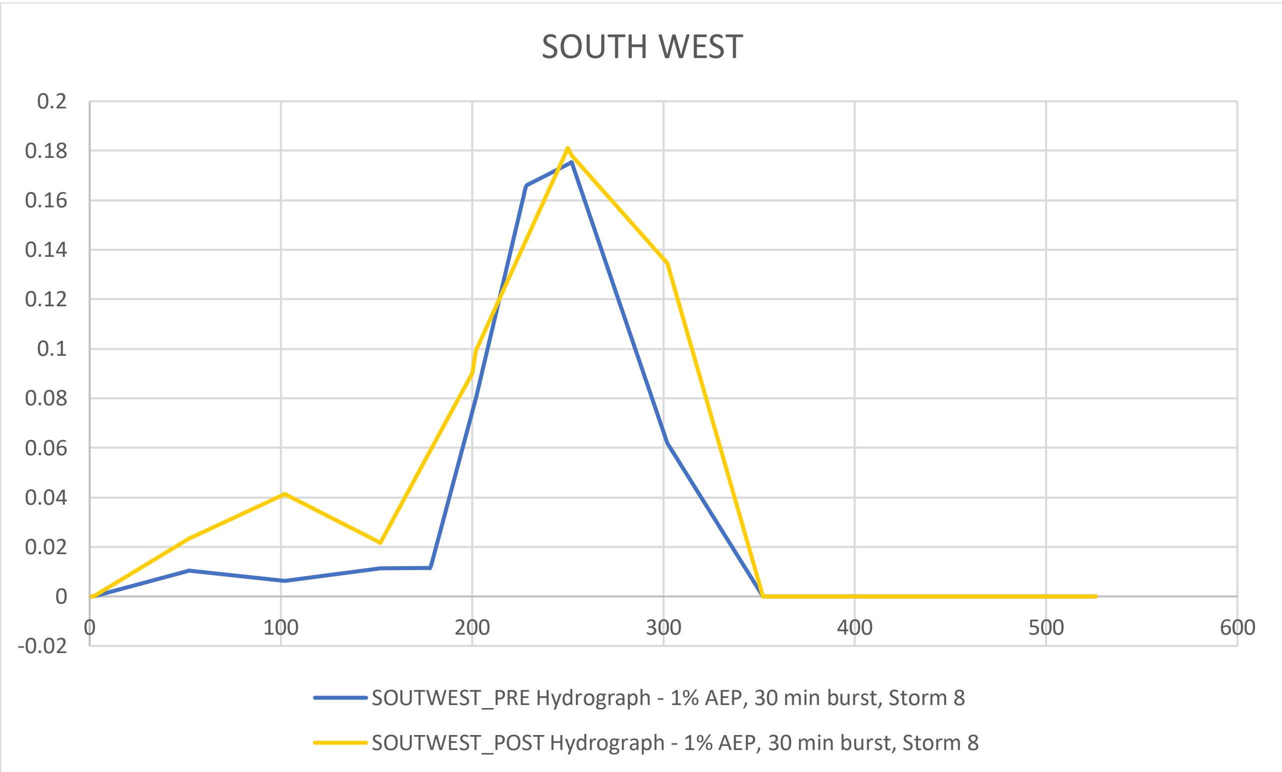


Figure 13 – Pre and Post-Development Critical Duration Runoff 1% AEP South West Catchment

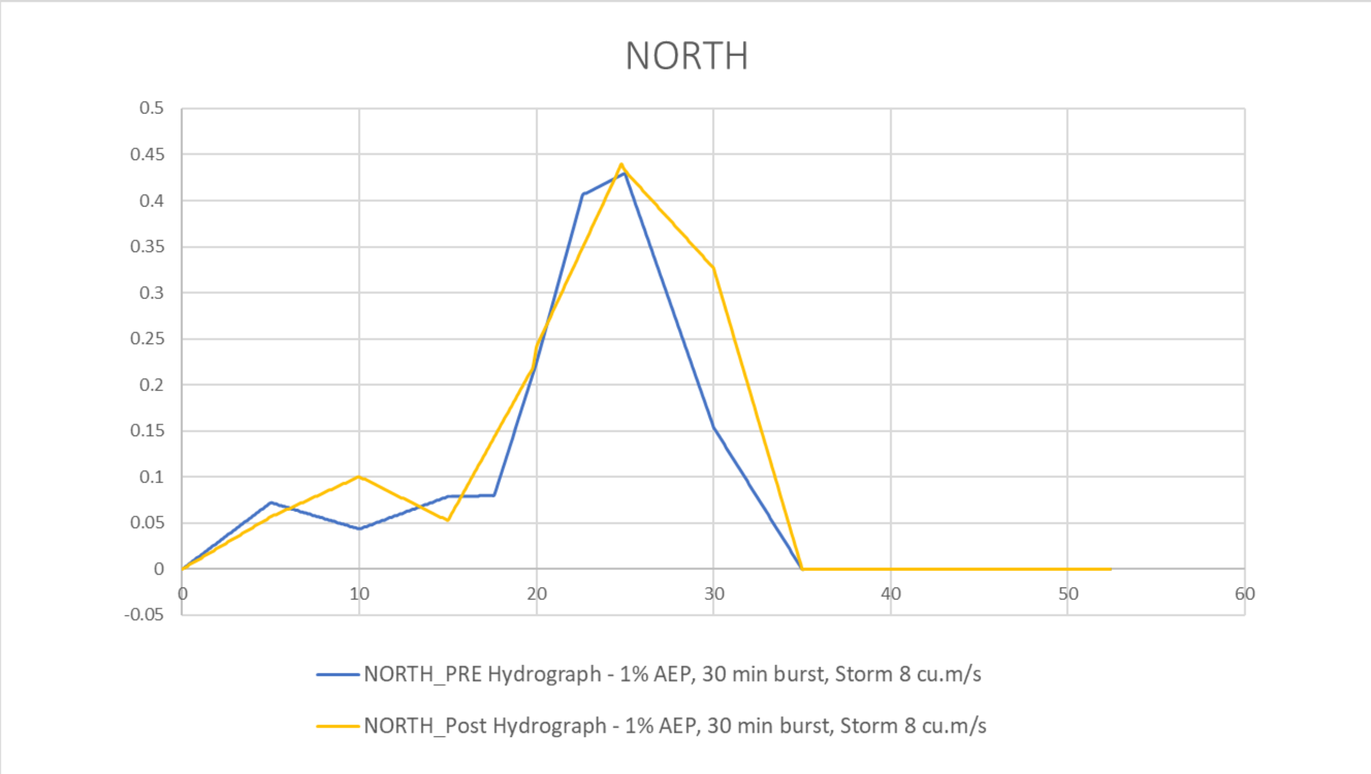


**Figure 14 – Pre and Post-Development Critical Duration Runoff 1% AEP North Catchment**

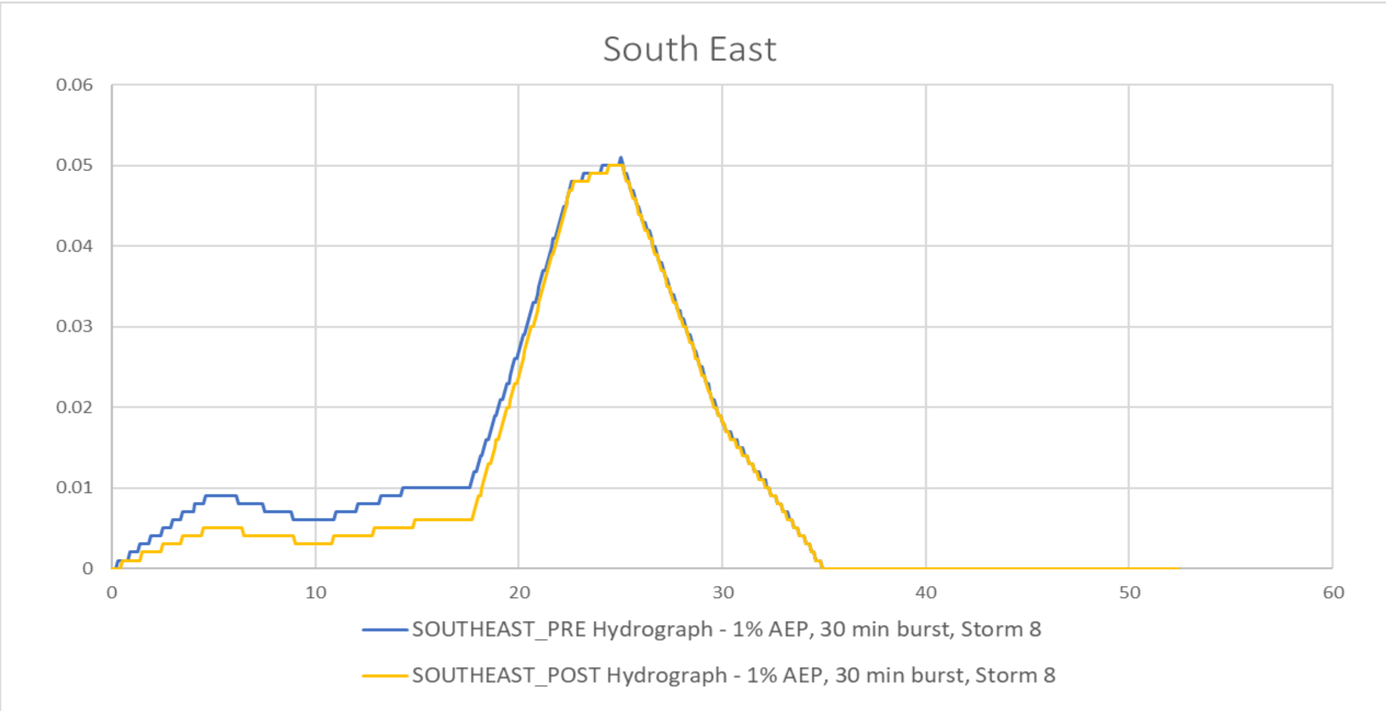


**Figure 15 – Pre and Post-Development Critical Duration Runoff 1% AEP North Catchment**

### 4.2.9 Treatment Volumes

The first 15mm of stormwater runoff from impervious areas will be treated in overland flow paths. Table 9 shows the volumes required to treat the first flush.

Table 10 Storm Losses

| Water Treatment Volumes | | |
|---|---|---|
| Area | First Flush Volume ($m^3$) | First Flush Area (ha) |
| South East | 4 | 0.023 |
| South West | 38 | 0.253 |
| North | 92 | 0.611 |

Figure 16 below shows the design schematic of proposed culverts and overland flow paths.

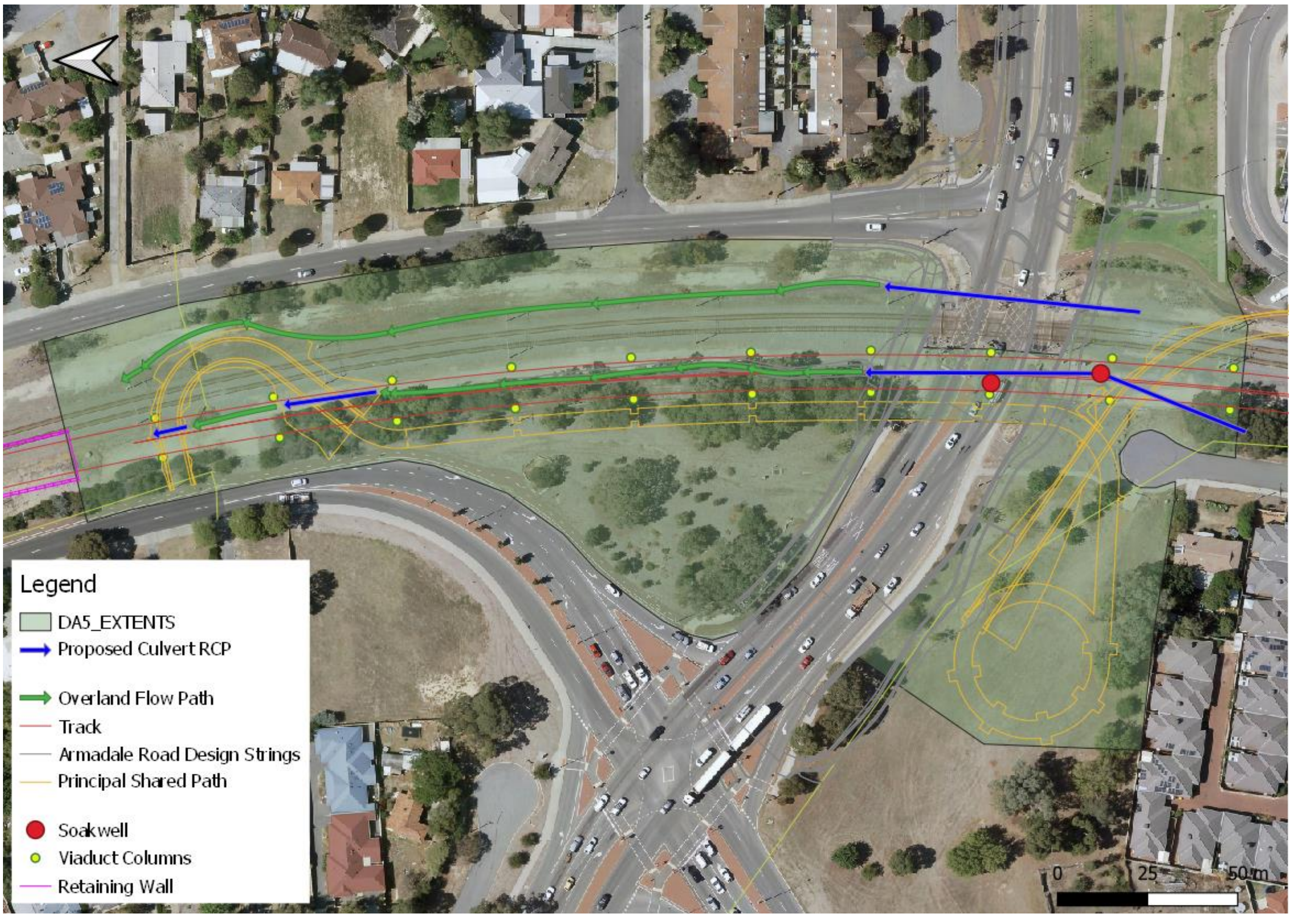


Figure 16. Design Schematic

# 5. Conclusion

The overall stormwater strategy applies water sensitive urban design principles and safely conveys water via surface drainage to above ground storage systems. The design maintains pre-development runoff rates by utilising stormwater overland flow paths with check dams.

The design maintains existing overland flow paths to minimise afflux in rare events.