| | Annual Exceedance Probability (AEP) | | | | | | |
|---|---|---|---|---|---|---|---|
| Duration | 63.2% | 50%# | 20%* | 10% | 5% | 2% | 1% |
| 1 min | 1.14 | 1.26 | 1.68 | 1.99 | 2.32 | 2.79 | 3.19 |
| 2 min | 2.00 | 2.21 | 2.88 | 3.35 | 3.82 | 4.39 | 4.81 |
| 3 min | 2.64 | 2.92 | 3.82 | 4.46 | 5.11 | 5.93 | 6.56 |
| 4 min | 3.17 | 3.50 | 4.60 | 5.40 | 6.22 | 7.32 | 8.19 |
| 5 min | 3.61 | 4.00 | 5.28 | 6.22 | 7.20 | 8.56 | 9.66 |
| 10 min | 5.24 | 5.81 | 7.75 | 9.23 | 10.8 | 13.1 | 15.1 |
| 15 min | 6.40 | 7.09 | 9.48 | 11.3 | 13.2 | 16.2 | 18.6 |
| 20 min | 7.32 | 8.11 | 10.8 | 12.9 | 15.1 | 18.4 | 21.1 |
| 25 min | 8.10 | 8.98 | 12.0 | 14.2 | 16.6 | 20.1 | 23.1 |
| 30 min | 8.79 | 9.74 | 12.9 | 15.3 | 17.9 | 21.5 | 24.6 |
| 45 min | 10.5 | 11.6 | 15.3 | 18.0 | 20.9 | 24.8 | 28.0 |
| 1 hour | 11.9 | 13.1 | 17.2 | 20.1 | 23.2 | 27.2 | 30.5 |
| 1.5 hour | 14.1 | 15.6 | 20.2 | 23.4 | 26.7 | 30.9 | 34.2 |
| 2 hour | 15.9 | 17.5 | 22.5 | 26.0 | 29.4 | 33.9 | 37.2 |
| 3 hour | 18.8 | 20.6 | 26.3 | 30.1 | 33.8 | 38.6 | 42.3 |
| 4.5 hour | 22.0 | 24.1 | 30.6 | 34.9 | 39.0 | 44.5 | 48.6 |
| 6 hour | 24.6 | 26.9 | 34.1 | 38.8 | 43.3 | 49.4 | 54.1 |
| 9 hour | 28.6 | 31.2 | 39.5 | 45.0 | 50.3 | 57.8 | 63.6 |
| 12 hour | 31.6 | 34.6 | 43.8 | 50.1 | 56.1 | 64.8 | 71.7 |
| 18 hour | 36.2 | 39.6 | 50.5 | 58.0 | 65.4 | 76.3 | 85.1 |
| 24 hour | 39.6 | 43.4 | 55.7 | 64.2 | 72.7 | 85.4 | 95.8 |
| 30 hour | 42.3 | 46.5 | 59.9 | 69.4 | 78.8 | 92.9 | 105 |
| 36 hour | 44.6 | 49.1 | 63.5 | 73.7 | 84.0 | 99.2 | 112 |
| 48 hour | 48.4 | 53.3 | 69.3 | 80.7 | 92.3 | 109 | 123 |
| 72 hour | 53.9 | 59.5 | 77.6 | 90.5 | 104 | 122 | 137 |
| 96 hour | 58.2 | 64.1 | 83.5 | 97.1 | 111 | 130 | 145 |
| 120 hour | 61.8 | 68.0 | 88.0 | 102 | 116 | 134 | 149 |
| 144 hour | 65.0 | 71.4 | 91.5 | 105 | 118 | 137 | 151 |
| 168 hour | 68.0 | 74.5 | 94.5 | 108 | 120 | 138 | 151 |

**Table 4. 2016 IFD Rainfall Depths (mm)**

## 2.3 Climate Change Impacts on Rainfall Intensity

Climate change estimates for Tasmania have been developed through the Climate Futures Project. Subsequently the ClimateAsyst tool (http://climateasyst.pittsh.com.au/app/) was developed. Figure 4 shows a screen shot from the tool depicting the percentage change in the 1% AEP, 24 hour rainfalls between the base and target periods in the vicinity of the Sheepwash Creek catchment.

Rainfall (24hr intensity) - Precipitation - 100yr ARI - 24hr - 2085 (2070 to 2099) percentage difference vs Base Period

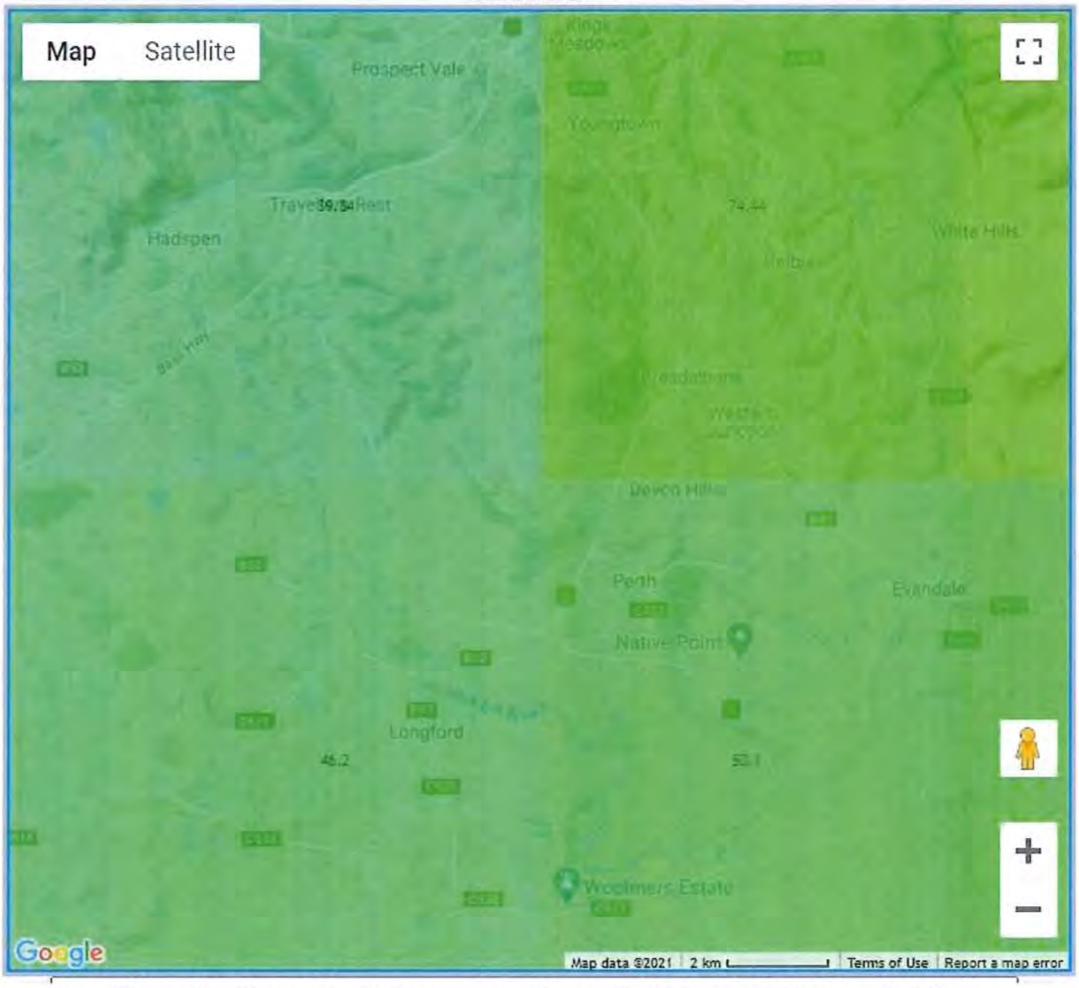


**Figure 4. ClimateAsyst Percentage change in 100 year 24 hour rainfalls**

The percentage increase in 24 hour rainfalls between the base period (1961-1990) and the protected period (2070 – 2099) is derived from the bottom right hand grid square in Figure 4 which covers the catchment area. The grid value has a projected increase of 50.1%, which is quite high in comparison to the interim climate change factor of 16.3% (RCP 8.5 to 2090) given by the ARR Data Hub.

The projected climate change percentage change values vary around Tasmania from negative values i.e., a reduction in 24 hour rainfall totals to increases of over 100% such as in Scamander area.

The factor of 50.1% was applied to the 1% AEP rainfall and the increased hyetograph was applied through the RORB model to generate hydrographs.

## 2.4 Adopted Loss and Routing Parameters and RORB Model Performance

The main purpose of the RORB model in this study is for the generation of hydrographs for the 2D hydrodynamic model. First the 'rural' version of the model (with no development or dam) was calibrated to peak value of the 1% AEP flood obtained from the FFA.

We also wanted to use RORB to corroborate the flood peak estimates developed from FFA. To this end we selected several potential regional methods available in RORB which predict peak flows leaving the model network. These methods employ a principal routing parameter, Kc, which is estimated from the catchment area. For 'm', which accounts for non-linearity in the network, the default recommended value of m = 0.8 was adopted.

A range of storms were applied using various regional methods within RORB but selected from the most analogous geographical areas. The loss parameters suggested by the ARR Data Hub (Initial Loss = 19mm, Continuing Loss = 5.2 mm/hr) were initially adopted with a view to modifying these within the most appropriate regional model. The selection of values was limited to values from the range typically experienced in Tasmania.

The RORB Manual formula, proved to be the most appropriate. The regional formula in this method is as follows:

In initial FIT runs, RORB suggests, as one option, the use of a first trial value calculated as follows:

$$k_c = 2.2A^{0.5} (Qp / 2)^{0.8-m}$$

where A is the catchment area, (km$^2$), and Qp is the (maximum) peak discharge of hydrograph(s) (m$^3$/s). It should be noted that the term Qp has a value of unity when m is at its recommended value of 0.8. Kc is calculated for an entire catchment area.

Figure 5 shows the results of a batch run for flows estimated at the Drummond Street culverts for the rural catchment model. Storm durations of 1 to 36 hours were applied to the model. For the reporting site at Drummond Street the largest 1% AEP peak was produced by the 12 hour duration storm. The peak value was 10.23 m$^3$/sec was achieved with only minor adjustment of Initial and Continuing Loss (IL and CL), the adopted values being IL = 20 mm and CL = 3.03 mm/hour.

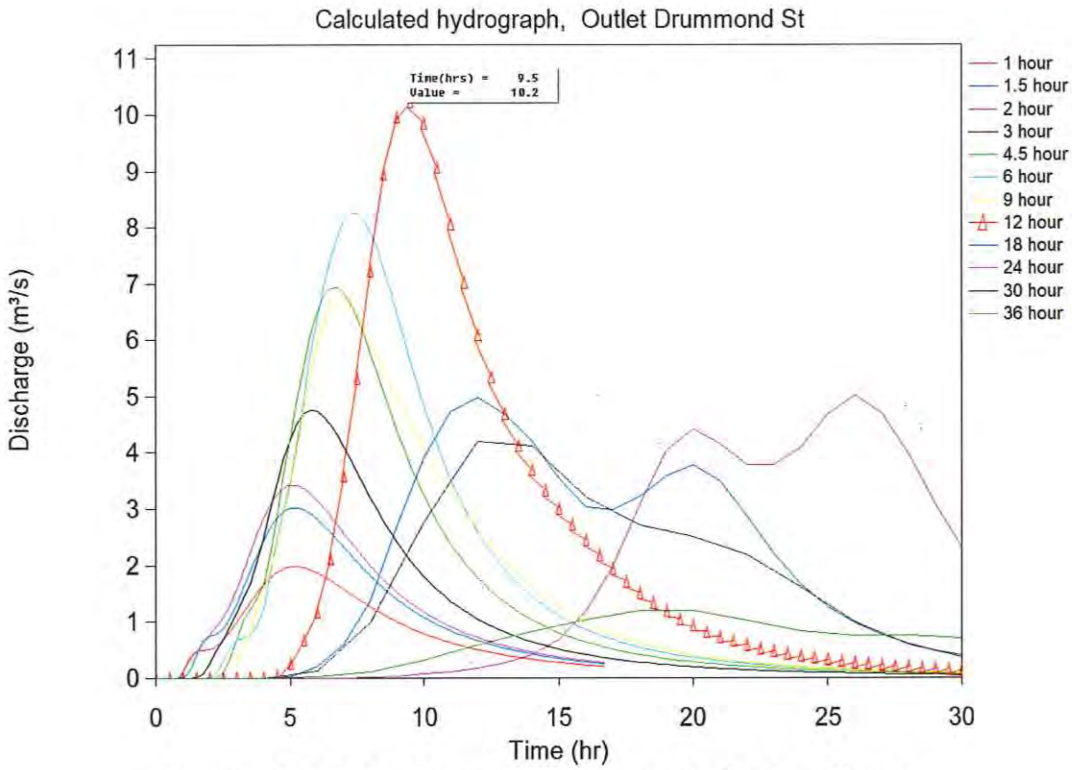


**Figure 5. Modelled hydrographs at Drummond St Culvert (Rural)**

Table 5 shows the values for the RORB routing parameters Kc, m and the loss parameters for initial and continuing losses used in generating the 5%, 2%, 1% and 1% AEP climate change hydrographs.

| AEP in Years | $K_c$ | m | IL (mm) | CL (mm/hr) | $Q_{Peak}$ ($m^3/s$) |
|---|---|---|---|---|---|
| 5% | 6.51 | 0.8 | 20 | 2.05 | 7.13 |
| 2% | 6.51 | 0.8 | 20 | 2.50 | 8.82 |
| 1% | 6.51 | 0.8 | 20 | 3.03 | 10.23 |
| 1% Climate Change | 6.51 | 0.8 | 20 | 3.03 | 21.97 |

**Table 5. RORB routing and loss parameters Sheepwash Creek (Rural)**

The last three columns show the IL and CL values required to calibrate the peak flows to the average peak flood discharges generated from FFA, and the peak flows at Drummond Street. For the 1% AEP Year Climate Change, IL, and CL the 1% AEP year values were adopted and the rainfall depth for the 12 hour duration event was increased by 50.1% to 104.4 mm with an Areal Reduction Factor (ARF) of 0.97.

The model required only minimal adjustment to the continuing loss parameters to produce flood peaks at Drummond Street, which are very close to the average flood frequency values at that location for the 1%, 2% and 5% AEP events.

The IL and CL parameters shown in Table 5 are well within their expected ranges.

The favourable comparison of FFA peaks with the RORB peaks means that we can confidently apply the RORB generated hydrographs into the 2D domain of the hydraulic model for estimating the 1% AEP flood surface.

## 2.5 Hydrograph Generation

Hydrographs were generated at five locations for input into the hydraulic model and one to check the RORB model flow at Drummond Street.

### 2.5.1 The Dams

The original lower MacKinnon dam was added to the model and assumed to be full. Some time was also spend looking at the recently constructed upper 'detention' dam. This dam was formed out of the borrow pit used to provide material for the highway embankments, and its primary use appears to be a supplementary irrigation storage for the MacKinnon property. Most of the upstream catchment flows past the detention dam. It is only diverted into the storage when the bank across the channel diverts flows which have not passed through 3x DN900 bypass pipes. There appears to be only 500mm depth of storage before the inflow short circuits to the outlet spillway.

Normally, with an effective a detention basin, the storage is online and all outflows up to the spillway level are controlled by a carefully designed outlet structure. The flow control arrangement in the current configuration is unlikely to work as they are too ill defined. If the inlet channel blocks with vegetation or the diversion bank erodes, then most flow will bypass. As it is not a Council asset, there is no guarantee it will be maintained and it could easily be modified without NMC's knowledge. Therefore, for the purposes of this study it was assumed it does not provide detention.

If NMC decides detention is desirable above West Perth due to increased climate change flows, we recommend that furth investigation into the benefits of storage rights in the lower larger dam be considered, with modifications, so it will act as a detention basin.

It was noted in the 2016 flood study that the 'Significant' consequence category of the original dam, which has been in place since it was constructed, may underestimate the potential downstream impact of the dam. Similar findings are contained in the Midland Highway Perth Link Roads Concept Design Report (Pitt and Sherry, 2017) which recommended a 'High B' consequence category be assigned. The report states that 'in the event of a Sunny Day Failure there will be potential risk to human life measured in terms of Population at Risk (PAR) of ≥100 to <1000.' It is recommended an updated dam safety emergency plan be sought for the dams. This will provide confidence to NMC that the dams, which may severely impact Perth if they fail, is being appropriately monitored and maintained.

### 2.5.2 Highway Culverts

Culvert works were undertaken in conjunction with the construction of the Perth bypass. A 'frog' culvert with 1.3m x 1.18m internal dimensions is located on the creek line. A secondary 3.35m x 2.87m internally dimensioned stock underpass is located offline with the inlet approximately 280 metres south-west of the frog culvert inlet. In the modelled scenarios these were firstly treated as a merged structures placed on the alignment of the frog culvert with no blockages. The reasons for this decision were as follows:

- Without calibration the adopted peak flow rates discharging to the culverts are estimates. Calibration could provide evidence that the adopted flow rates are resultant flooding is higher that those adopted here. The merged culverts provide a flow rate through the urban area which would allow for conservative decisions to be made based on the information currently available
- The highway is not a detention basin or a dam. It is not subject to the same standards and has not been certified as such. It cannot therefore be relied upon to provide detention on its upstream side
- NMC has no jurisdiction or maintenance oversight over the highway or the culverts. It is possible that piping erosion to occur, which would allow flows through the highway embankment outside the culverts. In addition, scour of the steep embankments, or debris washed in from the upper catchment, could result in blockage of the frog culvert. This may result in the stock culvert with the large opening acting as the primary means of discharge.

A second lot of maps were produced which modelled the 'frog' culvert and stock underpass in their actual locations. Comparing the two variations of the 1% AEP flood maps, where the culverts are modelled separately, there is an increase in flooding upstream of the highway. There is little to no reduction in the flood footprint between the highway and Phillip Street, and there is a marginal reduction in the footprint downstream of Phillip Street.

The 1% AEP climate change maps show a similar pattern, however the footprint between the highway and Phillip Street increases when the culverts are modelled separately due to flows from the stock underpass breaking out from the open drain located on the downstream side of the highway.

### 2.5.3 Urbanisation

In this round of modelling some of the subcatchments that have been urbanised with increased impermeable area percentages have been modified accordingly. Some RORB subcatchment links were also represented as lined or piped. The model linkages have been slightly modified to reflect the highway and urban development, and the lower MacKinnon Dam has been included, refer to Figure 6.

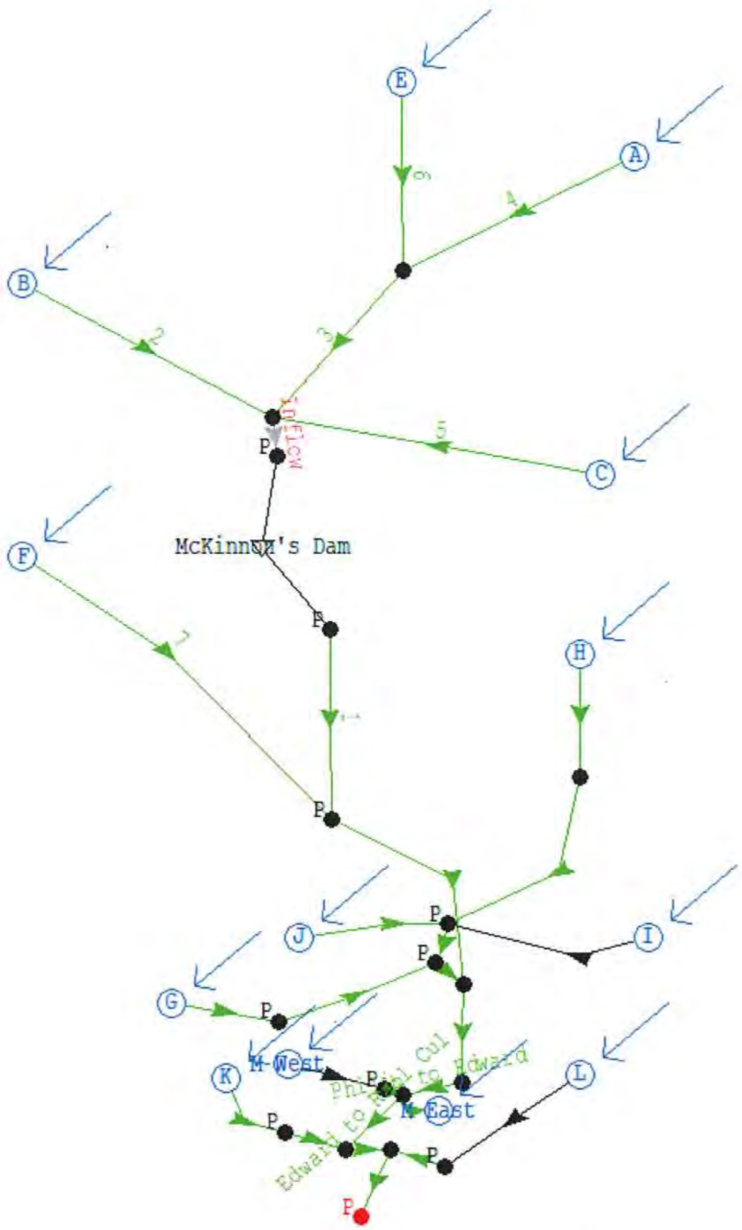


**Figure 6. RORB model network for the Urbanised Catchment with the Dam**

The resultant peak outflow at Drummond Street is marginally reduced for the 12 hour event due to the minor effects of detention at the dam, which is assumed full, and the increased runoff due to urbanisation does not compensate for this. The flood peak of the 12 hour 1% AEP event is 10.1 $m^3$/sec as shown Figure 7.

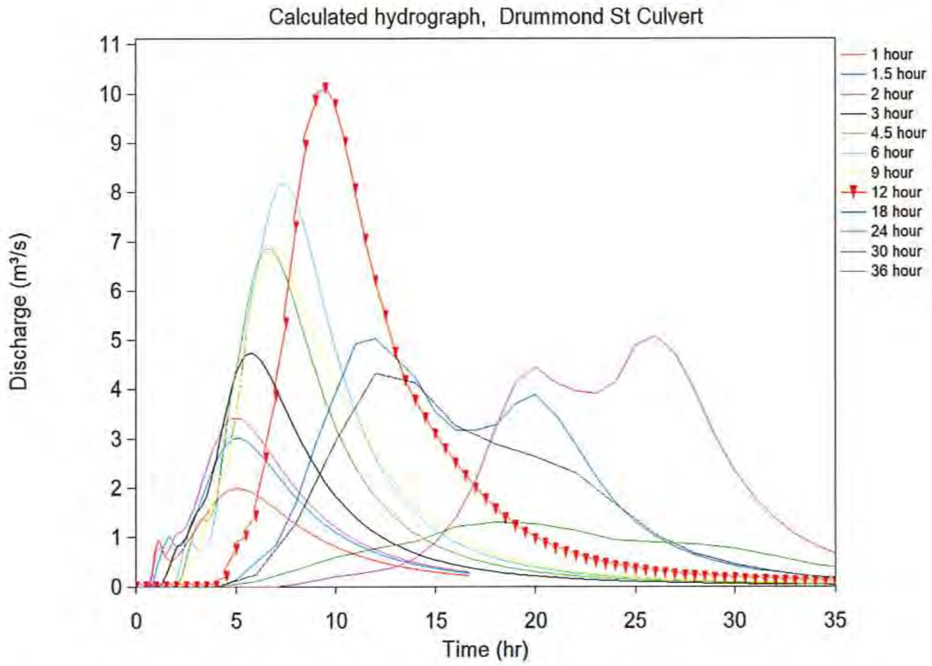


**Figure 7. Modelled hydrographs at Drummond St Culvert with Urbanisation & Dam**

The subcatchment site descriptions in Table 6 correlate with the catchment delineation in Figure 2 and the RORB network in Figure 6. The numbers in column 1 of Table 6 relate to the RORB output hydrograph reference number. The last two columns indicate the coordinates of the approximate physical location of the input boundaries in the 2D model domain. Input hydrographs for current and climate change conditions are presented in Figures 8 and 9.

- Hydrograph 5 is applied just above the main culvert under the highway,
- Hydrograph 8 is applied below the highway but upstream of Philip Street
- Hydrograph 9 is applied in the creek parallel to Cromwell Street
- Hydrographs 10 & 11 are applied either side of the railway above Drummond Street

| RORB hydrograph no. | Site Description | East | North |
|---|---|---|---|
| 5 | Dam & Cat F | 513398 | 5398960 |
| 8 | Cat G, J, I & H | 513600 | 5398644 |
| 9 | M (West & East) | 513713 | 5398028 |
| 10 | Cat K | 512569 | 5397488 |
| 11 | Cat L | 513681 | 5397441 |

**Table 6. RORB Hydrograph references**

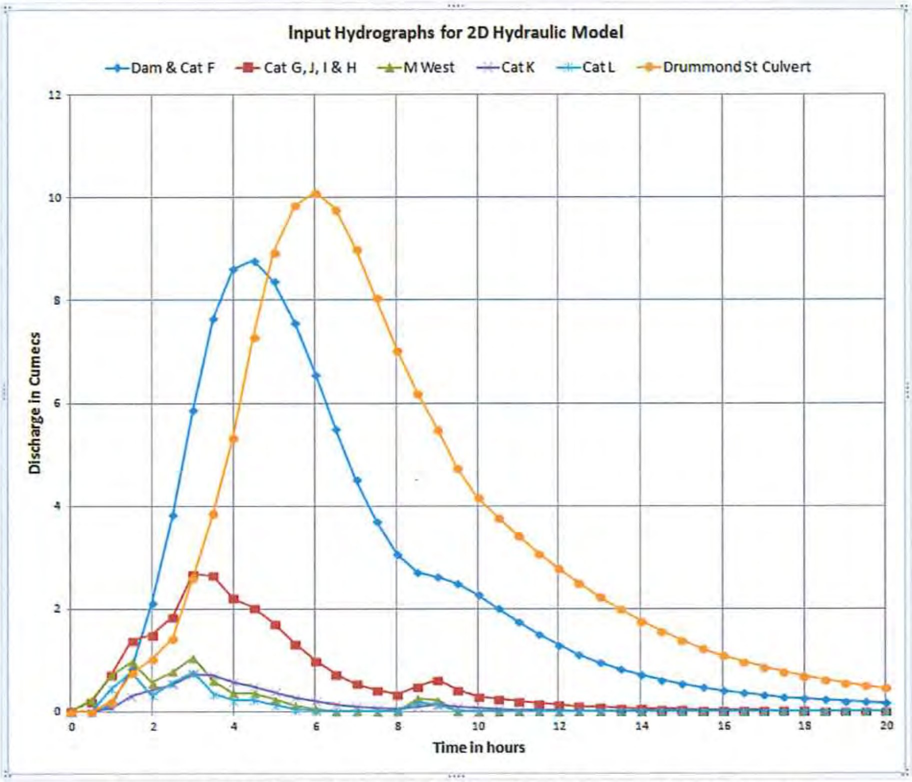


Figure 8. 1% AEP Flood Hydrographs (Current Conditions)

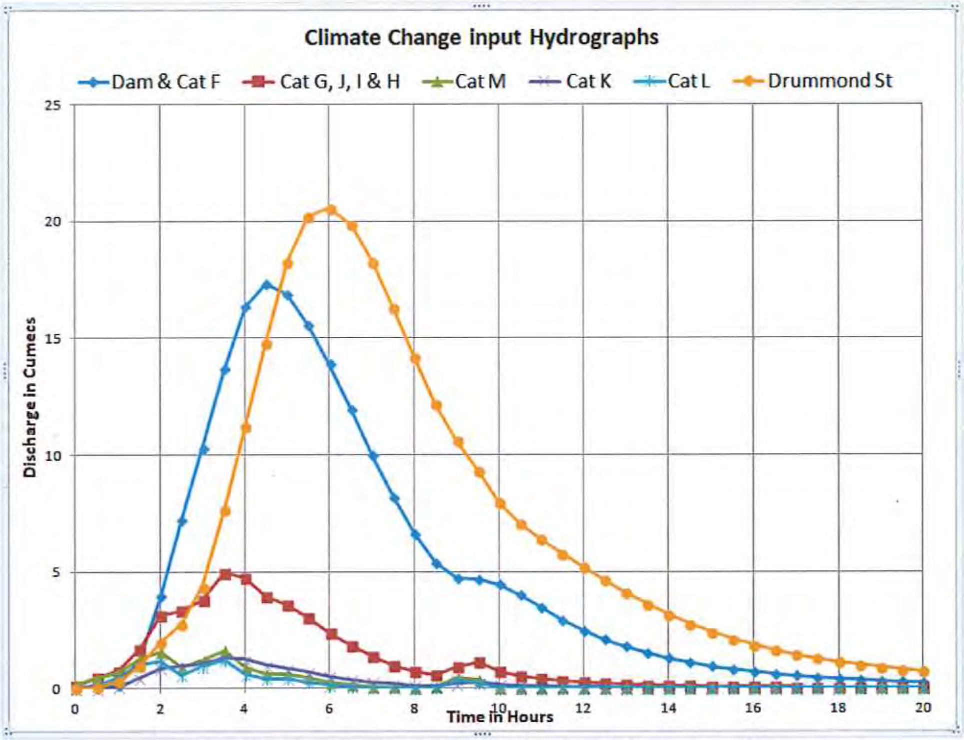


Figure 9. 1% AEP Flood Hydrographs (Climate Change)

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## 2.4 Potential Detention Dam

Before moving to the hydraulic simulations, we investigated MacKinnon's Dam as a potential location for a detention dam. A significant reduction in the dams out flow is potentially available to protect West Perth if the dam is modified as follows:

- Install 3 x DN700 pipes in one of the abutments at an elevation of 170.65 m AHD.
- Raise the dam and spillway 2.2 metres to provide a spill level of 180.00 m AHD

This theoretical detention basin reduces the climate change inflow for the 1% AEP 12 hour flood from 16.43 $m^3$/sec at the dam to 8.97 $m^3$/sec:

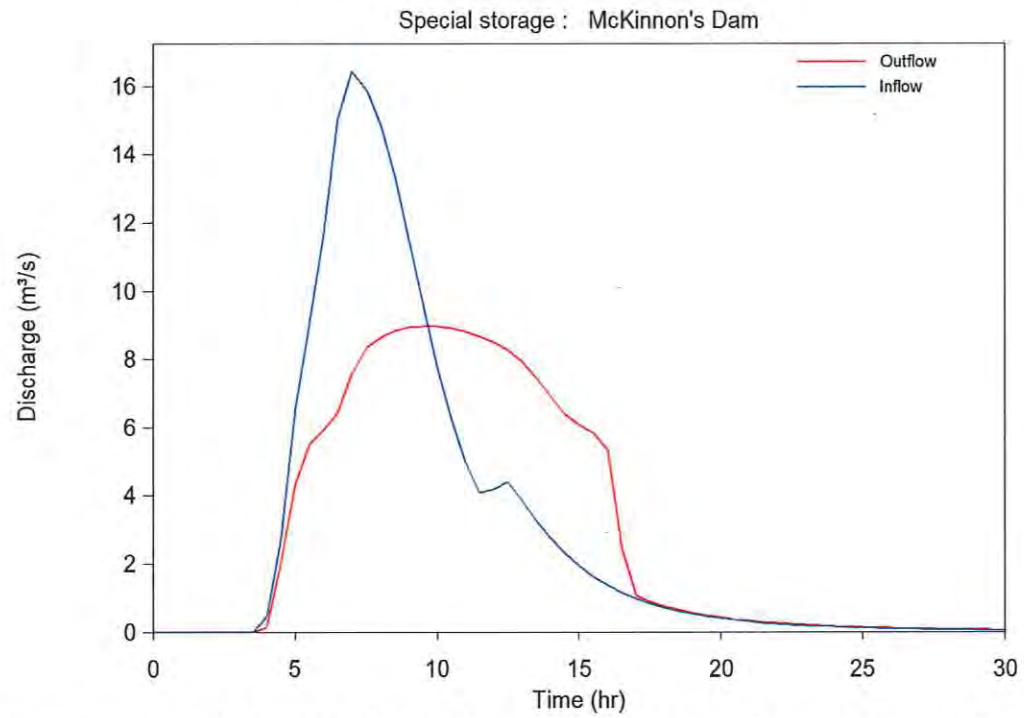


**Figure 10. Potential 1% AEP (Climate Change) Flow Reduction from MacKinnon's Dam**

RORB outputs are as follows:

```
*** Calculated hydrograph,  Dam Inflow

                              Hydrograph
                                Calc.
Peak discharge,m³/s           16.43
Time to peak,h                 7.00
Volume,m³                  3.33E+05
Time to centroid,h             9.26
Lag (c.m. to c.m.),h           3.14
Lag to peak,h                 0.885

Results of routing through special storage McKinnon's Dam
 Peak elevation=  179.31 m
 Peak outflow =     8.97 m³/s (pipe flow)
 Peak storage =     9.37E+04 m³
```

If Council determined that acquiring a drainage easement or the structure for the purpose of modifying the dam for flood mitigation purposes was a serious option, then this first pass analysis could be refined.

It is anticipated that an array of pipe diameters and invert elevations would be trialled combined with options to raise the embankment and spillway crest height creating more dynamic storage. A range of storm durations should also be considered to optimise the design and costs.

## 3. Hydrodynamic Model Overview

In this study flood maps have been produced using two dimensional (2D) analysis for flood impacts for the 1% AEP and climate change 1% AEP flood scenarios.

The model chosen for the analysis was the ISIS2D hydrodynamic model. The software has two different analytical engines available:

- The Alternating Direction Implicit (ADI) was employed to calculate the 1% AEP year current and 1% AEP Climate Change flood surface through the application of hydrographs generated by RORB.
- The Total Variation Diminishing scheme (TVD) can be applied to steep catchments and dam break analysis but has not been employed on this project.

Both ADI and TVD solve the Saint-Venant equations, representing conservation of mass and momentum; the difference is that the ADI solver is designed for subcritical flows, and the TVD solver is capable of modelling both sub and supercritical flows. The TVD solver is ideally suited to situations where supercritical flows are likely to occur, and modelling these accurately is important. Examples are in modelling dam breach, very steep catchments, or flow down spillways.

The 2D model domain was set up with 2 metre square grid and most of the culverts were represented as 1D embed elements as it was considered likely that they would overtop in some flows. Where bridges/culverts do not overtop, they can be represented by a gap in the embankment. The new highway bridge at the bottom end of the model has been represented as a slot, linked 1D elements in the 2D domain represented all the other culverts.

The model time-step is partially determined by the grid size and the velocity of flow, the Courant Number describes the condition that should be met which is described below. In our case we used a 0.25 second time-step for the 2D model and 0.25 second time-step for the linked 1D model.

## 3.1 Model Health & Courant Number

In mathematics, the Courant–Friedrichs–Lewy (CFL) condition is a necessary condition for convergence while solving certain equations numerically by the method of finite differences. It arises in the numerical analysis of explicit time integration schemes, when these are used for the numerical solution. As a consequence, the time step must be less than a certain time in many explicit time-marching computer simulations, otherwise the simulation will produce incorrect results

In ISIS2D the program authors recommend that under Alternating Direction Implicit (ADI) simulation approach the Courant Number does not exceed a value of 8 for most of the time.

Figure 11 shows a plot of Courant number versus time in seconds for the 1% AEP flood simulation for current conditions. The Courant number does not rise above 1.24 out of 120000 seconds where the time step was 0.25 seconds. The model can is therefore healthy and stable.

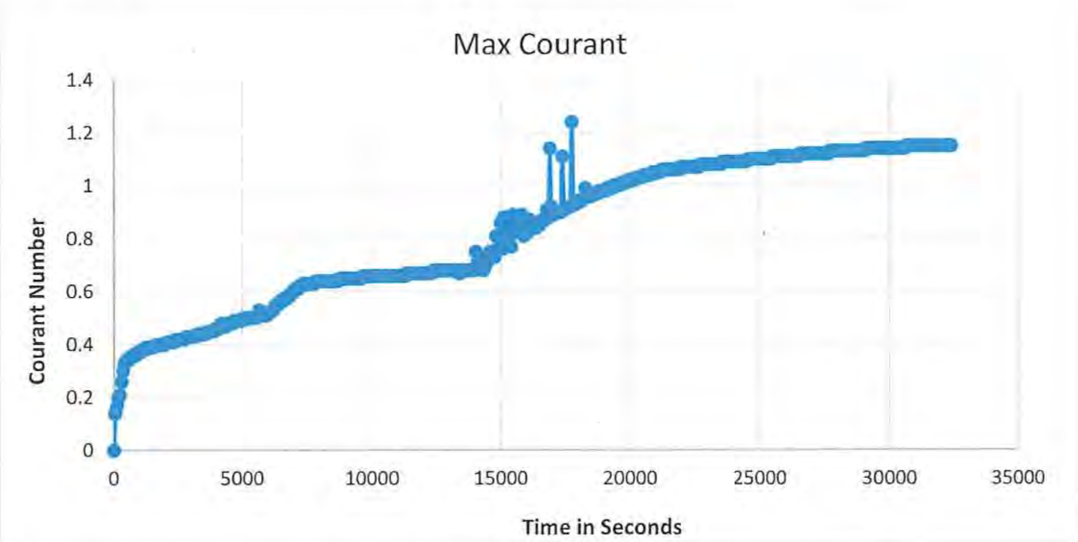


**Figure 11. Maximum Courant No. 1% AEP Flood Simulation**

## 3.2 Catchment Roughness

In both one dimensional and especially 2D modelling the critical parameters in the calibration of a model are the floodplain hydraulic roughness represented by Manning's 'n' representing the surfaces and vegetation types present on the flood plain.

To achieve a calibration, it is also preferable to have captured peak flood levels along the waterway for a flood event of known discharge. For Sheepwash Creek we currently have neither peak flood levels nor the associated discharge. However, Council has installed a gauge and will gather peak levels in the event of a large flood. So, at some future date a check can be carried out.

The estimated flood plain roughness values developed for the large-scale study of the Longford-Hadspen basin were adopted. The values were derived from model calibration for that project model. The Longford-Hadspen Manning's 'n' values provide a good estimate for West Perth and Sheepwash Creek.

The roughness parameters adopted for West Perth initially are as shown in Table 8.

| Land surface | Manning's n |
|---|---|
| Pasture | 0.03 |
| Trees and scrub | 0.1 |
| River | 0.046 |

**Table 7. Adopted roughness parameters**

## 3.3 Flood Mapping

The principal objective of this study was the production of flood maps using two-dimensional (2D) analysis to delineate the flood surfaces for the 1% and climate change 1% AEP flood scenarios. The resulting maps are provided in Appendix C.

### 3.3.1 General

The maps show the extent of floods likely to be experienced in up to 1% AEP Climate Change event for the median or 50% probability; it is based on flows shown in Table 5.

The maps may be used for emergency management assessments as the best information available at the time of publication. For general purposes the 1% CC to 1% AEP flood levels can be described here as the median flood discharge value +/- 0.3 metres; users may also be able to access the GIS flood layers if authorised by Council's senior officers.

### 3.3.2 Flood Frequencies

An AEP or annual exceedance probability is the probability on average that a given flood height will be equalled or exceeded in any one year. Another term is ARI or Average Recurrence Interval; this is the average period between events of a nominated size. Table 8 shows the chance of a given AEP event occurring in a nominated period:

| Annual Exceedance Probability (AEP) | 20 Year Period | 50 Year Period |
|---|---|---|
| 5% (20 Year ARI) | 64% | 92% |
| 2% (50 Year ARI) | 33% | 64% |
| 1% (100 Year ARI) | 18% | 39% |
| 0.5% (200 Year ARI) | 10% | 22% |

**Table 8. Probability of flood magnitude being exceeded in a 20 or 50 year period**

### 3.3.3 Flood Discharge Values

It should be noted that the AEP or ARI associated with a particular discharge will change with time, due to additional recorded data altering the flood frequency estimate or through climate change. However, the flood level associated with a particular discharge and depicted on the maps will only change if flood plain conditions change as a result of flood plain modification, vegetation increase or decrease or further calibration data becoming available.

Further calibration data for peak flood levels with an associated peak flow estimates from a gauging station could produce different modelled levels.

Table 9 presents the peak flow estimates for the selected range of AEPs:

| AEP | ARI in Years | Average Peak Flood Discharges Generated from Flood Frequency Analysis & RORB Modelling |
|---|---|---|
| | | Sheepwash Creek @ Drummond Street |
| 5% | 20 | 7.14 |
| 2% | 50 | 8.88 |
| 1% | 100 | 10.23 |
| 1% CC | 100 (yr. 2085) | 21.97 |

**Table 9. Average peak flood discharges generated from Flood Frequency Analysis & RORB modelling**

### 3.3.4 Flood Surface

Flood surface levels can be determined from direct measurement by surveying the levels in the aftermath of a flood and then assigning an AEP to the flood surface, or by hydraulic modelling with a hydrodynamic model. Both approaches require FFA or hydrological modelling to determine the flood's AEP.

Figure 12 is taken from the modelling interface. It shows both the 1% AEP and 1% AEP Climate Change flood surfaces, which are mid and light blue respectively, and hydrograph input locations. The 1% AEP surface comes close to several houses but only as a shallow depth. Houses built with a minimum freeboard of 300mm above surrounding ground level or as advised by the Building Code of Australia (BCA), would probably not suffer inundation of their floors unless by wave action created by vehicles or wind. However, the 1% AEP CC flood surface will challenge several properties unless some form of flood mitigation is instigated.

**Figure 12. 1% AEP and 1% AEP Climate Change Flood Surfaces & Hydrograph Input Locations**

Mitigation could take the form of flood detention as discussed in Section 2.6, or channel and culvert enhancement along Sheepwash Creek. It may be that both forms of mitigation will be required. Section 3.4 will examine the current afflux at some of the culverts to illustrate the potential for mitigation by culvert enhancement.

Hopefully Council will continue to refine the map as more information becomes available, but for now this is the best estimate available for the various flood surfaces.

## 3.4 Head Loss at Culverts

This section looks at the head loss and or afflux at Philip Street, Edward Street, Youl Road, on the creek alignment, and Drummond Street.

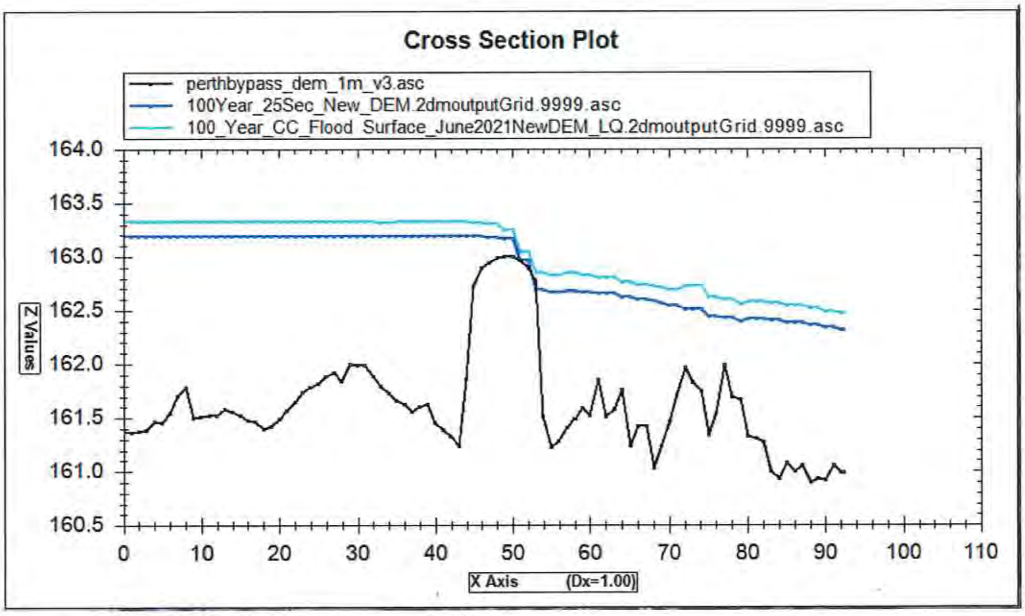


**Figure 13. Philip Street Cross Section**

The Philip Street overtops in both the 1% AEP and 1% AEP CC floods with only 1.51 $m^2$ waterway area. Table 10 indicates about 500mm head loss across the road:

| Existing Philip Street Culverts (2 x DN900 and 1230 x 900mm) | | | |
|---|---|---|---|
| | Upstream | Downstream | Head Loss |
| 1% AEP CC Flood Surface | 163.32 | 162.82 | 0.50 |
| 1% AEP Flood Surface | 163.19 | 162.66 | 0.53 |
| Change in Elevation | 0.13 | 0.16 | - |

**Table 10. Flood levels across Phillip Street**

Edward Street overtops in the 1% AEP CC event and has a 430 mm head loss upstream to down steam across the road. A larger culvert would probably raise the 1% AEP flood surface downstream and reduce it upstream.

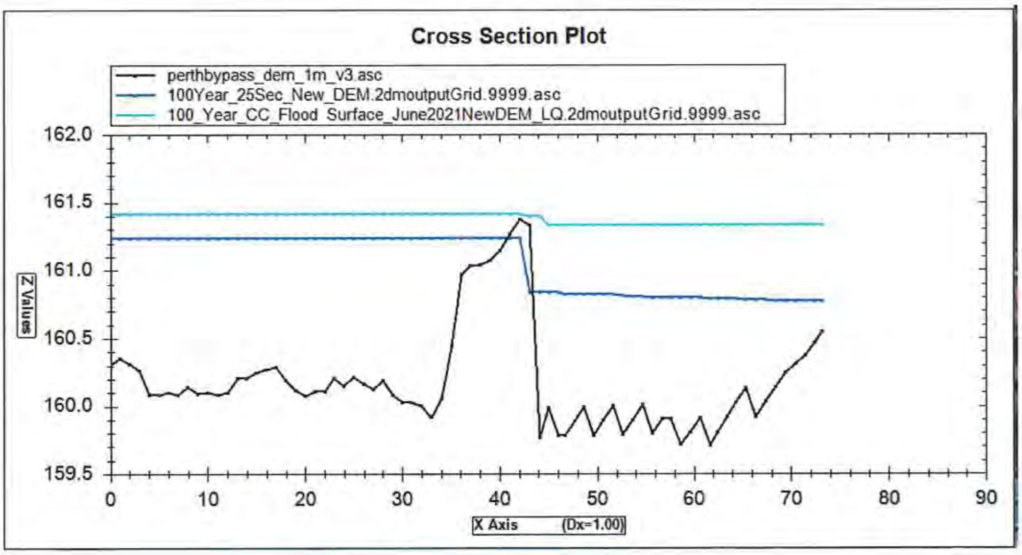


**Figure 14. Edward Street Cross Section**

| Edward Street Culvert (3 x DN1050) | | | |
|---|---|---|---|
| | Upstream | Downstream | Head Loss |
| 1% AEP CC Flood Surface | 161.41 | 161.33 | 0.08 |
| 1% AEP Flood Surface | 161.24 | 160.81 | 0.43 |
| Change in Elevation | 0.17 | 0.52 | - |

**Table 11. Flood levels across Edward Street**

At Youl Road and the railway on the original Creek alignment a significant drop of about 800mm occurs in both flood events. Upgrading Edward Street alone may increase the upstream flood level. As the Effra Court subdivision is upstream of this road, it would be well worth investigating the benefits of upgrading the culverts at this location.

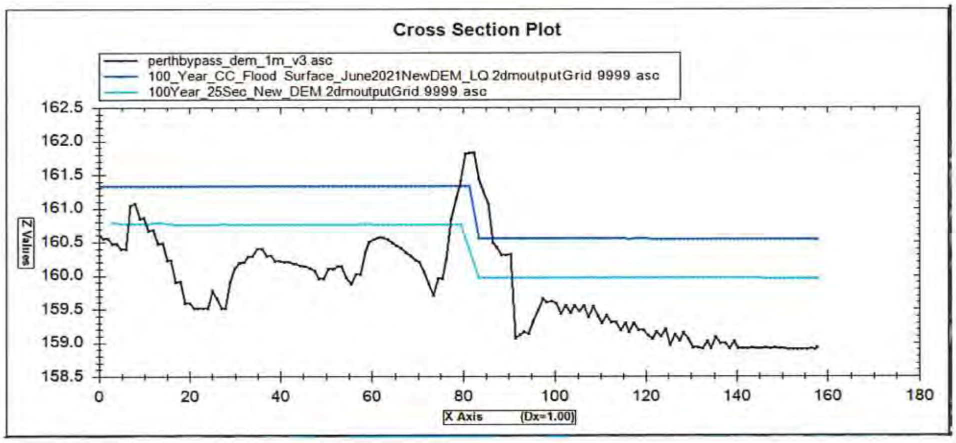


**Figure 15. Old Railway Culverts at Youl Road Cross Section**

| Youl Road Culverts on Original Creek Alignment (2 x DN600) | | | |
|---|---|---|---|
| | Upstream | Downstream | Head Loss |
| 1% AEP CC Flood Surface | 161.33 | 160.54 | 0.79 |
| 1% AEP Flood Surface | 160.76 | 159.96 | 0.8 |
| Change in Elevation | 0.57 | 0.58 | - |

**Table 12. Flood levels across Edward Street**

At Drummond Street the analysis indicates that for the 1% AEP and the 1% AEP CC the head loss across the road is of the order of 630 mm and 980mm respectively, see Table 13.

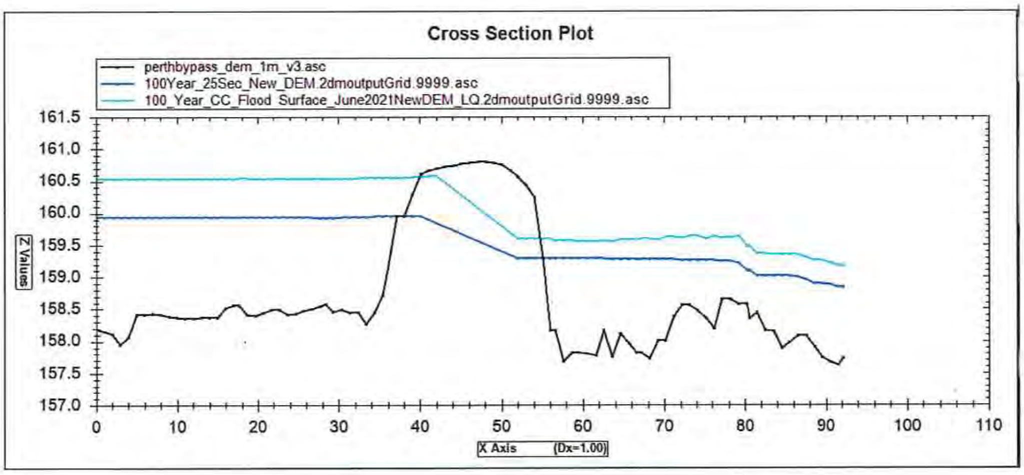


**Figure 16. Drummond Street Cross Section**

| **Drummond St Culverts (3 x DN1200)** | | | |
|---|---|---|---|
| | Upstream | Downstream | Head Loss |
| 1% AEP CC Flood Surface | 160.54 | 159.56 | 0.98 |
| 1% AEP Flood Surface | 159.92 | 159.29 | 0.63 |
| Change in Elevation | 0.62 | 0.27 | - |

**Table 13. Flood levels across Drummond Street**

There appears to be significant potential benefit in investigating the upgrading of these culverts.

# 4. Conclusions, Findings and Recommendations

1. It is recommended that the flood maps produced by this study are incorporated into the Planning Scheme flood hazard layer and are adopted to aid and facilitate emergency management.
2. As per the recommendation in the 2016 report, the hazard rating of the original McKinnon's dam should be confirmed. This will provide confidence to NMC that the dam, which may severely impact Perth if it fails, is being appropriately monitored and maintained. Dam failure currently poses a risk to existing and future west Perth development which has not yet fully been considered.
3. Council should further investigate the acquisition of dynamic storage in the lower MacKinnon Dam for flood detention and/or investigate the upgrading of culverts at Philip Street, Edward Street, Youl Road and Drummond Street. This will reduce the elevation of the current 1% AEP flood surface, reduce the impact of the 1% AEP climate change flood, and increase the relative current freeboard of existing residential properties.
4. Installation of a guide bank on the highway culvert outlet at the northern end of Perth, which would allow flow to be better directed into the creek channel, should also be considered.
5. We recommend Council continue with the gauging station upstream of Philip Street. This will have several potential benefits including: capturing data to enable calibration to future floods, capturing annual flood peaks to facilitate flood frequency analysis, emergency management.
6. To predict and measure the impacts of floods we recommend the collection of individual residential and commercial floor levels of critical infrastructure to be associated with the property address and as a GIS layer. These can be combined with the flood map data for planning assessments and for emergency management purposes with respect to flood warning and response.
7. To facilitate the capture of flood peak data for future flood studies within the municipality, we recommend that static flood gauges are installed upstream and downstream of the major bridges and that post flood surveys are carried out following major flood events and the data recorded for future use in conjunction with flood discharge data.

## 5. References


1. *Australian Rainfall and Runoff: A Guide to Flood Estimation*, Institute of Engineers, 1998.
2. *Australian Rainfall and Runoff, Revision Project 6: Loss Models for Catchment Simulation* July 2016.
3. Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors) *Australian Rainfall and Runoff: A Guide to Flood Estimation*, Commonwealth of Australia (Geoscience Australia), 2019.
4. Courant, R.; Friedrichs, K.; Lewy, H. (September 1956) [1928], *On the partial difference equations of mathematical physics*, AEC Research and Development Report, NYO-7689, New York: AEC Computing and Applied Mathematics Centre – Courant Institute of Mathematical Sciences.
5. ISIS2D User Manual CH2MHILL and on line reference.
6. Midland Highway Perth Link Roads Concept Design Report, Pitt and Sherry, 2017.
7. *Open-Channel hydraulics*, Ven Te Chow, McGraw-Hill, 1959.
8. The RORB Version 6, User Manual, Monash University by E.M. Laurenson, R.G. Mein, and R.J. Nathan 2010.

# APPENDIX A – RFFE OUTPUT

RESULTS FROM ARR RFFE 2015 MODEL
Datetime: 2021-03-07 16:53
Region name: Tasmania
Region code: 2
Site name: West Perth
Latitude at catchment outlet (degree) = -41.576805639
Longitude at catchment outlet (degree) = 147.1632938
Latitude at catchment centroid (degree) = -41.557611267
Longitude at catchment centroid (degree) = 147.162779707
Distance of the nearest gauged catchment in the database (km) = 20.71
Catchment area (sq km) = 8.76
Design rainfall intensity, 1 in 2 AEP and 6 hr duration (mm/h): 4.487118
Design rainfall intensity, 1 in 50 AEP and 6 hr duration (mm/h): 8.191507
Shape factor of the ungauged catchment: 0.72

ESTIMATED FLOOD QUANTILES:

| AEP (%) | Expected quantiles (m^3/s) | 5% CL m^3/s | 95% CL m^3/s |
|---|---|---|---|
| 50 | 1.29 | 0.560 | 3.04 |
| 20 | 1.99 | 0.860 | 4.69 |
| 10 | 2.53 | 0.950 | 6.62 |
| 5 | 3.09 | 0.990 | 9.30 |
| 2 | 3.88 | 1.00 | 14.1 |
| 1 | 4.53 | 0.990 | 18.8 |

DATA FOR FITTING MULTI-NORMAL DISTRIBUTION FOR BUILDING CONFIDENCE LIMITS:

1 Mean (loge flow) = 0.522
2 St dev (loge flow) = 0.510
3 Skew (loge flow) = 0.135

Moments and correlations:

| No | Most probable | Std dev | Correlation | | |
|---|---|---|---|---|---|
| 1 | 0.522 | 0.540 | 1.000 | | |
| 2 | 0.510 | 0.387 | -0.330 | 1.000 | |
| 3 | 0.135 | 0.169 | 0.150 | -0.440 | 1.000 |

This is the end of output file.

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# APPENDIX B – FLOOD MAPS


1% AEP Flood Extent

Depth (m)

0.0 - 0.1, 0.1 - 0.2, 0.2 - 0.3, 0.3 - 0.5, 0.5 - 1, 1.0 - 1.2, 1.2 - 2, 2.0 - 3.7

Frog Culverts

Stock Underpass

N

0 100 200 300 400

Metres

Scale 1:3,000 @ A1

Coordinate System GDA 1994 MGA Zone 55

Base data from theLIST, © State of Tasmania

Base image Esk Mapping & GIS 2020

# SHEEPWASH CREEK FLOOD MAP

**1% Annual Exceedance Probability (AEP)**

**Based on Modelling 2nd June 2023**

**Frog culvert capacity modelled online**

**Stock underpass capacity modelled offline**

NORTHERN MIDLANDS COUNCIL

HYDRODYNAMICA

Map Created by: L Cornwell

Map Version: 5

Hydraulic Modelling by: S Ratcliffe

Map Date: 7th June 2023

ESK SPATIAL


1% AEP Climate Change Flood Extent

Depth (m)

0.0 - 0.1 | 0.1 - 0.2 | 0.2 - 0.3 | 0.3 - 0.5 | 0.5 - 1 | 1.0 - 1.2 | 1.2 - 2 | 2.0 - 3.7

Frog Culverts

Stock Underpass

N

0 | 100 | 200 | 300 | 400

Metres

Scale 1:3,000 @ A1

*Coordinate System GDA 1994 MGA Zone 55*
*Base data from theLIST, © State of Tasmania*
*Base image Esk Mapping & GIS 2020*

# SHEEPWASH CREEK FLOOD MAP

**1% Annual Exceedance Probability (AEP) Climate Change**
**Based on Modelling 3rd June 2023**
**Frog culvert capacity modelled online**
**Stock Underpass capacity modelled offline**

NORTHERN MIDLANDS COUNCIL

HYDRODYNAMICA

Map Created by: L Cornwell
Map Version: 5
Hydraulic Modelling by: S Ratcliffe
Map Date: 7th June 2023

ESK SPATIAL

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**Our Ref:** 45018

18th May 2023

Ms Claire Hynes
Delegate (Chair)
TASMANIAN PLANNING COMMISSION

Via Email: tpc@planning.tas.gov.au

Dear Ms Hynes

**Northern Midlands Local Provision Schedule**
**Draft amendment 03-2022 and permit-11-0056**
**Lot 1 Drummond Street, Perth.**

I refer to the commission direction letter 3rd of April. The applicant is to provide the commission with a full copy of the Pitt & Sherry Concept Design Report and an assessment against the relevant zone and codes consistent with the requested amendment to rezone the land to General Residential, including Natural Assets Code.

a) Please see the attached scanned copy of the Pitt and Sherry Report.

b) It is noted that the original submission was made under the Interim Scheme and the original submission includes an assessment against the relevant codes and clauses of the interim scheme. The Natural Assets codes is not within the interim scheme. The attached priority habitat overlay map shows that the two areas subject to being rezoned to General Residential are not within the overlay.

   The Northerm Midlands Council have transitioned to the Tasmanian Planning Scheme since the lodgement of the rezoning. Below is an assessment of the relevant zone and codes of the Tasmanian Planning Scheme for the proposed subdivision.

## 8.6 Development Standards for Subdivision

### 8.6.1 Lot Design

#### Objective:

That each lot:
(a) has an area and dimensions appropriate for use and development in the zone;
(b) is provided with appropriate access to a road;
(c) contains areas which are suitable for development appropriate to the zone purpose, located to avoid natural hazards; and
(d) is orientated to provide solar access for future dwellings

| Acceptable Solutions | Performance Criteria |
|---|---|
| **A1**<br>Each lot, or a lot proposed in a plan of subdivision, must:<br>a) Have an area no less than 450m² and:<br>i. Be able to contain a minimum area of 10m x 15m with a gradient not steeper than 1 in 5 clear of: | **P1**<br>Each lot, or proposed in a plan of subdivision, must have sufficient useable area and dimensions suitable for its intended use, having regard to:<br>a) the relevant requirements for development of buildings on the lots;<br>b) the intended location of buildings on the lots;<br>c) the topography of the site; |

| Acceptable Solutions | Performance Criteria |
|---|---|
| a. all setbacks required by the clause 8.4.2 A1 and A2 and A3, and 8.5.1 A1 and A2; and<br>b. easements of other title restrictions that limit or restrict development; and<br>ii. existing buildings are consistent with the setback required by clause 8.4.2 A1 and A2 and A3, and 8.5.1 A1 and A2; or<br>b) be required for public use by the Crown, a council or State authority;<br>c) be required for the provisions of Utilities; or<br>d) be for the consolidation of a lot with another lot provided each lot is within the same zone | d) the presence of any natural hazards;<br>e) adequate provision of private open space; and<br>f) the pattern of development existing on established properties in the area; and<br>g) must be no more than 15% smaller than the minimum applicable lot size required by clause NOR-S6.8.2 A1 (a) |

### Comment:

**A1** is met with Lot 1 and Lot 2 are larger than 450m² and can contain a 10 x 15m area clear of all setback requirements as both lots are vacant.

| Acceptable Solutions | Performance Criteria |
|---|---|
| **A2**<br>Each lot, or a lot proposed in a plan of subdivision, excluding for public open space, a riparian or littoral reserve or Utilities, must have a frontage not less than 12m. | **P2**<br>Each lot, or proposed in a plan of subdivision, excluding for public open space, a riparian or littoral reserve or Utilities, must be provided with a frontage or legal connection to a road by right of carriageway, that is sufficient for the intended use, having regard to:<br>a) the width of frontage proposed, if any<br>b) the number of other lots which have the land subject to the right of carriageway as their sole or principal mean of access;<br>c) the topography of the site;<br>d) the functionality and useability of the frontage;<br>e) the ability to manoeuvre vehicles on the site; and<br>f) the pattern of development existing on established properties in the area,<br><br>and is not less than 3.6m wide. |

### Comment:

**A2** is met with Lot 1 and 2 having frontage larger than 12m, with Lot 1 having access to Napoleon Street and Lot 2 having access to Main Road/Arthur Street.

| Acceptable Solutions | Performance Criteria |
|---|---|
| **A3**<br>Each lot, or a lot proposed in a plan of subdivision, must be provided with a vehicular access from boundary of the lot to a road in accordance with the requirements of the road authority. | **P3**<br>Each lot, or proposed in a plan of subdivision, must be provided with reasonable vehicular access to a boundary of a lot or building area on the lot, if any having regard to:<br>a) the topography of the site<br>b) the distance between the lot or building area and the carriageway;<br>c) the nature of the road and the traffic;<br>d) the anticipated nature of vehicles likely to access the site; and<br>e) the ability for emergency services to access the site. |

Comment:

**A3 is met:** With access to be provided to the boundary in accordance with the road authority requirements.

| Acceptable Solutions | Performance Criteria |
|---|---|
| **A4**<br>Any lot in a subdivision with a new road, must have the long axis of the lot between 30 degrees west of true north and 30 degrees east of true north. | **P4**<br>Subdivision must provide for solar orientation of lots adequate to provide solar access for future dwellings, having regard to:<br>a) the size, shape and orientation of the lots;<br>b) the topography of the site;<br>c) the extent of overshadowing from adjoining properties;<br>d) any development on the site;<br>e) the location of roads and access to lots; and<br>f) the existing pattern of subdivision in the area. |

Comment:

**P4** is met as both lots will have adequate solar orientation. There is no overshadowing from adjoining properties and no development is proposed on the site.

## 8.6.2 Roads

Objective:
That the arrangement of new roads within a subdivision provides for;

a) safe, convenient and efficient connections to assist accessibility and mobility of the community;
b) the adequate accommodation of vehicular, pedestrian, cycling and public transport traffic; **and**
c) the efficient ultimate subdivision of the entirety of the land and of surrounding land.

| Acceptable Solutions | Performance Criteria |
| --- | --- |
| **A1**<br>The Subdivision includes no new roads | **P1**<br>The arrangement and construction of roads within a subdivision must provide an appropriate level of access, connectivity, safety, convenience and legibility for vehicles, pedestrians and cyclists, having regard to:<br>a) any road network plan adopted by council;<br>b) the existing and proposed road hierarchy;<br>c) the need for connecting roads and pedestrian path, to common boundaries with adjoining land, to facilitate future subdivision potential;<br>d) maximising connectivity with the surrounding road, pedestrian, cycling and public transport networks;<br>e) minimising the travel distance between key destinations such as shops and services and public transport routes;<br>f) access to public transport;<br>g) the efficient and safe movement of pedestrians, cyclists and public transport;<br>h) the need to provide bicycle infrastructure on new arterial and collector roads in accordance with the *Guide to Road Design Part 6A: Paths for Walking and Cycling 2016;*<br>i) the topography of the site; and<br>j) the future subdivision potential of any balance lots on adjoining or adjacent land. |

Comment:

**A1 is met:** there are no new roads proposed

## 8.6.3 Services

Objective:
That the subdivision of land provides services for the future use and development of the land.

| Acceptable Solutions | Performance Criteria |
| --- | --- |

| A1 | P1 |
|---|---|
| Each lot, or a lot proposed in a plan of subdivision, excluding for public open space, a riparian or littoral reserve or Utilities, must have a connection to a full water supply service. | A lot, or a lot proposed in a plan of subdivision, excluding for public open space, a riparian or littoral reserve or Utililites, must have a connection to a limited water supply service, having regard to:<br>(a) flow rates;<br>(b) the quality of potable water<br>(c) any existing or proposed infrastructure to provide the water service and its location<br>(d) the topography of the site; and<br>(e) any advice from a regulated entity |

Comment:

**A1** is met as both lots will be connected to existing Water mains.

| Acceptable Solutions | Performance Criteria |
|---|---|
| **A2**<br>Each lot, or a lot proposed in a plan of subdivision, excluding for public open space, a riparian or littoral reserve or Utilities, must have a connection to a reticulated sewerage system. | **P2**<br>No Performance Criterion |

Comment:

**A2** is met as both lots will be connected to Sewer mains located within the road reserve of Napoleon and Drummond Street.

| Acceptable Solutions | Performance Criteria |
|---|---|
| **A3**<br>Each lot, or a lot proposed in a plan of subdivision, excluding for public open space, a riparian or littoral reserve or Utilities, must be capable of connecting to a public stormwater system. | **P3**<br>Each lot, or proposed in a plan of subdivision, must be capable of accommodating an on-site stormwater management system adequate for the future use and development of the land, having regards to:<br>a) the size of the lots<br>b) topography of the site<br>c) soil conditions;<br>d) any existing buildings on the site;<br>e) any area of the site covered by impervious surfaces; and<br>f) any watercourse on the land. |

Comment:

**P3** is met. Each lot is capable of having sufficient area to accommodate onsite stormwater management.

## C7.7.2 Subdivision within a priority vegetation area.

### Objective:

a) works associated with subdivision will not have an unnecessary or unacceotable impact on priority vegetation; and

b) future development likely to be facilitated by subdivision is unlikely to lead to an unnecessary or unacceptable impact on priority vegetation.

| Acceptable Solutions | Performance Criteria |
|---|---|
| **A1**<br>Each lot, or a lot proposed in a plan of subdivision, must:<br>a) be for the pruposes of creating separate lots for existing buildings;<br>b) be required for public use by the Crown, a council or State authority;<br>c) be required for the provisions of Utilities; or<br>d) be for the consolidation of a lot or;<br>e) not include any works (excluding boundary fencing), building area, bushfire hazard management area, service or vehicular access within a priority vegetation area. | **P1.1**<br>Each lot, or a lot proposed in a plan of subdivision, within a priority vegetation area must be for:<br>(a) subdivision for an existing use on the site, provided any clearance is contained within the minimum area necessary to be cleared to provide adequate bushfire protection, as recommended by the Tasmanian Fire Service or an accredited person;<br>(b) subdivision for the construction of a single dwelling or an associated outbuilding;<br>(c) subdivision in the General Residential Zone or Low Density Zone Residential Zone;<br>(d) use or development that will result in significant long-term social and economic benefits and there is no feasible alternative location or design;<br>(e) subdivision involving clearing of native vegetation where it is demonstrated that on-going existing management cannot ensure the survival of the priority vegetation and there is little potential for long-term persistence; or<br>(f) subdivision involving clearance of native vegetation that is of limited scale relative to the extent of priority vegetation on the site.<br><br>**P1.2**<br>Works association with subdivision within a priority vegetation area must minimise adverse impacts on priority vegetation, having regard to:<br>a) the design and location of any works, future development likely to be facilitated by the subdivision, and any constraints such as topography or land hazards;<br>b) any particular requirements for the works and future development likely to be facilitated by the subdivision;<br>c) the need to minimise impacts resulting from bushfire hazard management measures through siting and fire-resistant design of any future habitable buildings;<br>d) any mitigation measured implemented to minimise the residual impacts on priority vegetation;<br>e) any on-site biodiversity offsets; and<br>f) any existing cleared areas on the site. |

Comment:

**P1.1**

a) Bushfire report states no increase in risk and no clearance is required for subdivision. Any future development will be subject to a different application and bushfire report.
b) The subdivision will likely result in a single dwelling and associated outbuildings. Any further development will be subject to a separate application.
c) The subdivision is within General Residential.
d) Use and development will result in long term social and economic benefits to the area of Perth, due to the Highway bypass of Perth creating land suitable for further residential development.
e) No clearing is proposed as part of the development.
f) No clearing is proposed as part of the development.

**P1.2**

a) There are no constraints for work to facilitate subdivision. The works required are water and sewer connections. No clearing is required as the connection will be located around the existing access.
b) Lots to be connected to reticulated mains are required.
c) Bushfire has no increase of risk and no clearing is required as part of the application.
d) No mitigation measures are proposed.
e) No on-site biodiversity offsets are proposed.
f) The site of Lot 2 has been used existing for low grazing and has existing cleared areas on the site. Any future dwelling can be located within cleared areas on site but will be subject to a separate application.

Yours faithfully

PDA Surveyors

Allan Brooks

**PDA Surveyors, Engineers & Planners**

# Midland Highway
# Perth Link Roads
# Concept Design Report

transport | community | mining | industrial | food & beverage | energy

| | |
|---|---|
| **Prepared for:** | VEC SHAW Joint Venture |
| **Client representative:** | Owen Cavanough |
| **Date:** | 21 November 2017<br>Rev01 |

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Appendix D: Geotechnical Report

Prepared by: ............................................ Ross Mannering — Date: 21 November 2017

Reviewed by: ............................................ Rob Casimaty — Date: 21 November 2017

Authorised by: ............................................ Andy Turner — Date: 21 November 2017

| Revision History | | | | | |
|---|---|---|---|---|---|
| Rev No. | Description | Prepared by | Reviewed by | Authorised by | Date |
| 00 | Concept Design Report | R. Mannering | R. Casimaty | A. Turner | 17/11/2017 |
| 01 | Updated Concept Design Report | R. Mannering | R. Casimaty | A. Turner | 21/11/2017 |

# 1. Introduction

The Perth Link Roads project involves the construction of a new alignment for the Midland Highway that passes along the southern and western outskirts of the Perth Township. In the south, the works match into the existing Highway on the western side of the South Esk River Bridge, and to the north, the works match into the recently constructed Perth to Breadalbane project. The design developed by the Department of State Growth provides grade separated interchanges at Illawarra Main Road and at northern extent of the project. A roundabout is provided to the west of the South Esk River bridge to maintain a southern ingress and egress to the Perth township.

The Department of State Growth is delivering the Perth Link Roads project using a Design and Construct procurement model. This Concept Design Report has been prepared to provide an overview of the key aspects of the Tender Design.

# 2. Roads

## 2.1 Geometry

### 2.1.1 Design Speed

The tender design is compliant with the design speed requirements in Table 3010.22 of the Tender Documents.

### 2.1.2 Horizontal Alignment

The northern and eastern approaches to the Southern Roundabout are required to match into the existing Midland Highway with the tie-in works to be constructed under traffic. To minimise the extent of the tie-ins works and hence traffic disruption, the alignment of the northern and eastern approaches to the Southern Roundabout have been modified so the horizontal position of the Southern Roundabout is moved approximately 15m to the south west. The relocation reduces the length of the tie-in works on each approach by approximately 70m and 40m respectively, significantly cutting back the time required to construct the tie-ins and hence minimising potential for traffic management issues.

In optimising the design of the western interchange, the start of the Westbound Ramp from the South has been moved to the west by approximately 40m. This adjustment has reduced the deck area for Railway Underpass No. 1, reducing the bridge construction cost.

### 2.1.3 Vertical Alignment

In developing the Tender Design, it was established that the Principal's Reference Design did not provide the required vertical clearance of 5.75m over the rail track at Railway Underpass No.1 and No.2. This Tender Design now provides sufficient vertical clearance through a combination of raising the vertical alignment of the Midland Highway, southbound on ramp and minimising the structural depth of the bridge. The combination of these two design optimisation processes has required the Midland Highway alignment at Railway Underpass No. 1 to be raised by approximately 700mm. The vertical alignment has also been lowered on the southern approach to Railway Underpass No. 1, reducing the height of the embankment in the vicinity of Sheepwash Creek.

In addition to addressing the clearance issue for the two railway underpasses, the vertical alignment of all the other road links through the Western Interchange have been optimised to minimise the extent of imported embankment required. This takes into account the structural depth and clearance requirements for each bridge structure.

The VEC Shaw JV construction methodology for the Perth Link Roads project involves sourcing the embankment material required for the project through expansion of Mackinnon's dam which is located on the western side of the Midland Highway near the northern extent of the project. To facilitate expansion of the dam whilst mitigating dam break risk for the Perth Township, the vertical alignment of the Midland Highway has been raised in the vicinity of the Northern Interchange. The modification to the vertical alignment has resulted in the centreline of the Tender Design being approximately 1.0m higher at the low point near chainage 3,800 (MC00) relative to the Principal's Reference Design. In conjunction with raising of the Midland Highway vertical alignment, the vertical alignment of the northbound off ramp and Northern Roundabout No. 2 has been raised accordingly to minimise the impact in the event of a dam break.

A gravity sewer and rising main cross Eskleigh Main Road at chainage 250 (MC50). Review of the existing levels on these services provided with the reference design indicates that these utilities services could be retained in their current location rather than be relocated if the vertical alignment of Eskleigh Road was raised. Accordingly, the vertical alignment of Eskleigh Road has been raised by approximately 700mm in the vicinity of the water and sewer main to provide clearances compliant with Taswater requirements.

### 2.1.4 Cross Section

The typical cross sections adopted in the Tender Design are consistent with the Principal's Reference Design except for the following locations:

- Where the Midland Highway and the Westbound Ramp from the North are in cut near Mackinnon's Hill. In this location 3:1 batters have been adopted to a depth of 1.8m instead of 0.9m. This is based on review of the geotechnical information available, which indicates that there is potential for material between 0.9m and 1.8m below existing levels not to be stable at a slope of 0.33:1.
- For Mountford Access No. 1 from chainage 180 to 720 MCX0 where 3.5m lanes and 1.0m sealed shoulders have been provided due to the Mountford Access No. 1 pavement being used as a temporary detour for Illawarra Road. Increased pavement thickness will also be provided.

To minimise the extent of imported embankment required for the project and to utilise the relatively high volume of topsoil that will be generated through stripping, embankments constructed north of the Illawarra Main Road detour that are designed to have a finished embankment slope of 2:1 will be constructed from rock fill at a slope of 1.5:1. The stripped topsoil will then be used to establish the 2:1 surface. The rock fill will be sourced from expansion of Mackinnon's Dam. Slope stability analysis has confirmed the suitability of this embankment construction methodology; details are provided in Geotechnical Report in Appendix D.

### 2.1.5 Sight Distance

Consistent with the Principal's Reference Design, median widening and shoulder widening has been provided where stopping sight distance for the design speed specified in Table 3010.22 cannot be achieved due to the presence of bridge structures and traffic barriers. Table 1 indicates the locations where median and shoulder widening has been provided to ensure adequate manoeuvre width is provided.

Table 1: Median and Shoulder Widening for Bridge Structures and Traffic Barriers

| Road Name | Start Chainage | End Chainage | Width (m) | Shoulder/Median |
|---|---|---|---|---|
| Midland Highway | 1,460 | 2,820 | 5.0 | Median |
| | 4,050 | 4,430 | 4.0 | Median |
| | 1,540 | 1,775 | 2.5 | Shoulder |
| | 1,775 | 2,270 | 3.5 | Shoulder |
| | 2,270 | 2,685 | 4.0 | Shoulder |
| | 2,685 | 2,870 | 2.5 | Shoulder |

| Road Name | Start Chainage | End Chainage | Width (m) | Shoulder/Median |
|---|---|---|---|---|
| | 4,030 | 4,310 | 2.5 | Shoulder |
| Westbound Ramp from South | 20 | 480 | 2.5 | Shoulder |
| Westbound Ramp from North | 335 | 935 | 4.0 | Shoulder |
| | 935 | 1,045 | 2.5 | Shoulder |
| Southbound Ramp | 940 | 1,670 | 4.0 | Shoulder |
| | 1,610 | 1,760 | 4.0 | Shoulder |
| | 1,760 | 2,010 | 2.5 | Shoulder |
| Northbound Ramp | 815 | 965 | 2.5 | Shoulder (MCBO) |
| | 0 | 535 | 2.5 | Shoulder |

Note – the above chainages include the length of tapers

## 3. Pavements

The pavements for the project have been designed in accordance with the requirements of Section 3040 of the Project Specification and *VicRoads Code of Practice 500.22 – Selection and Design of Pavements and Surfacings* including *Department of State Growth Supplement June 2017*. The pavement designs have been influenced by the following factors which specifically relate to this project:

- A substantial length of the Southern Link is in cut
- The majority of the Western Interchange and a substantial proportion of the Western Link is in fill
- A section of the Western Interchange at Mackinnon's Hill is in cut in hard rock.

Table 2 indicates the Design Traffic and Design Reliability Level that has been adopted for the purpose of the pavement design based on Advice to Tenderers No. 10.

Table 2: Design Traffic Loadings

| Location | Design Traffic (ESAs) | Design Reliability Level (%) |
|---|---|---|
| New Highway, South of Western Interchange | $1.6 \times 10^7$ | 97.5 |
| New Highway, through Western Interchange | $9.0 \times 10^6$ | 97.5 |
| New Highway, North of Western Interchange | $1.5 \times 10^7$ | 97.5 |
| Western Interchange – Southbound Ramp | $8.6 \times 10^6$ | 97.5 |
| Western Interchange – Westbound Ramp from South | $6.2 \times 10^6$ | 97.5 |
| Western Interchange – Northbound Ramp | $6.0 \times 10^6$ | 97.5 |
| Western Interchange – Westbound Ramp from North | $4.6 \times 10^6$ | 97.5 |
| Illawarra Main Road | $1.1 \times 10^7$ | 97.5 |
| Northern Interchange – Northbound Off Ramp | $1.0 \times 10^6$ | 97.5 |
| Northern Interchange – Southbound On Ramp | $2.5 \times 10^5$ | 97.5 |
| Northern Interchange – Existing Southern Access Road | $6.8 \times 10^5$ | 90 |

| Location | Design Traffic (ESAs) | Design Reliability Level (%) |
|---|---|---|
| Northern Interchange – Main Road Perth | $1.0 \times 10^6$ | 90 |
| Eskleigh Road | $2.0 \times 10^5$ | 90 |
| Unsealed Private Access Roads (Mackinnon, Einoder) | $2.0 \times 10^5$ | 90 |
| Illawarra Main Road New Cul de Sac | $2.0 \times 10^5$ | 90 |
| New Southern Roundabout | $1.6 \times 10^7$ | 97.5 |
| Main Road (MC20) | $1.6 \times 10^7$ | 97.5 |
| Main Road Connection (MC40) | $1.6 \times 10^7$ | 97.5 |
| New Northern Roundabout No. 2 | $6.8 \times 10^5$ | 90 |
| Northern Interchange – Southbound Off Ramp (MCG0) | $1.0 \times 10^6$ | 90 |
| New Northern Roundabout No. 1 | $1.0 \times 10^6$ | 90 |
| Existing Eastern Service Road (MCM0) | $1.0 \times 10^6$ | 90 |
| Existing Midland Highway (MCN0) | $1.0 \times 10^6$ | 90 |

Based on review of the geotechnical information supplied with the tender documents, consideration of the vertical alignment and the strategy for the generation of the imported embankment required for the project, the subgrade CBR values indicated in Table 3 have been adopted for the pavement design. The rock source to be utilised for the imported embankment for the project also provides the opportunity to place a high strength capping layer. This will facilitate the adoption of the maximum CBR of 10 permitted by the tender documents.

Table 3: Subgrade CBR

| Location | String Name | Start Chainage | End Chainage | Subgrade CBR |
|---|---|---|---|---|
| New Highway, South of Western Interchange | MC00 | 0<br>715 | 715<br>2,070 | 5<br>10 |
| New Highway, through Western Interchange | MC00 | 2,070 | 2,810 | 10 |
| New Highway, North of Western Interchange | MC00 | 2,820 | 4,550 | 10 |
| Western Interchange – Southbound Ramp | MCB0 | 810 | 2,080 | 10 |
| Western Interchange – Westbound Ramp from South | MCA0 | 0 | 685 | 10 |
| Western Interchange – Northbound Ramp | MCD0 | 0 | 800 | 10 |
| Western Interchange – Westbound Ramp from North | MCC0 | 0 | 1,210 | 10 |
| Illawarra Main Road | MCB0 | 0 | 810 | 4 |
| Northern Interchange – Northbound Off Ramp | MCJ0 | 0 | 420 | 10 |
| Northern Interchange – Southbound On Ramp | MCH0 | 0 | 460 | 10 |
| Northern Interchange – Existing Southern Access Road | MCQ0 | 0 | 320 | 10 |
| Northern Interchange – Main Road Perth (Northbound On Ramp) | MCT0 | 320 | 665* | 10 |

| Location | String Name | Start Chainage | End Chainage | Subgrade CBR |
|---|---|---|---|---|
| Eskleigh Road | MC50 | 0 | 440 | 5 |
| Unsealed Private Access Roads (Mackinnon, Einoder) | MCV0<br>MCX0<br>MCX5 | 0<br>0<br>0 | 1530<br>730<br>1295 | 5<br>5<br>5 |
| Illawarra Main Road New Cul de Sac | N/A | N/A | N/A | 5 |
| New Southern Roundabout | N/A | N/A | N/A | 5 |
| Main Road (MC20)<br>(Midland Highway) | MC20 | 0<br>130 | 130<br>240 | 5<br>4 |
| Main Road Connection (MC40) | MC40 | 0 | 240 | 5 |
| New Northern Roundabout No. 2 | N/A | N/A | N/A | 10 |
| Northern Interchange – Southbound Off Ramp (MCG0) | MCG0 | 0 | 500 | 10 |
| New Northern Roundabout No. 1 | N/A | N/A | N/A | 10 |
| Existing Eastern Service Road (MCM0) | MCM0 | 0 | 100 | 10 |
| Existing Midland Highway (MCN0) | MCN0 | 0 | 240 | 10 |

* Matches to MC00 at chainage 5215 (approx. 335m north of MCT0 chainage 665)

Using the design traffic loadings in Table 2 and the subgrade CBR values in Table 3, the pavement configurations in Table 4 have been adopted for the pavements across the project.

Table 4: Pavement Configurations

| Location | String Name | Start Chainage | End Chainage | Pavement Configuration |
|---|---|---|---|---|
| New Highway, South of Western Interchange | MC00 | 0 | 715 | Class 1: 200mm<br>Class 3: 160mm<br>Class 4: 160mm<br>Type A Capping: If required |
| | | 715 | 2070 | Class 1: 200mm<br>Class 3: 160mm<br>Class 4: 100mm<br>Type A Capping: 300mm |
| New Highway, through Western Interchange | MC00 | 2070 | 2810 | Class 1: 200mm<br>Class 3: 150mm<br>Type A Capping: 300mm |
| New Highway, North of Western Interchange | MC00 | 2820 | 4550 | Class 1: 200mm<br>Class 3: 160mm<br>Class 4: 100mm<br>Type A Capping: 300mm |
| Western Interchange – Southbound Ramp | MCB0 | 810 | 2080 | Class 2: 200mm<br>Class 3: 150mm<br>Type A Capping: 300mm |

| Location | String Name | Start Chainage | End Chainage | Pavement Configuration |
|---|---|---|---|---|
| Western Interchange – Westbound Ramp from South | MCA0 | 0 | 685 | Class 2: 200mm<br>Class 3: 120mm<br>Type A Capping: 300mm |
| Western Interchange – Northbound Ramp | MCD0 | 0 | 800 | Class 2: 200mm<br>Class 3: 110mm<br>Type A Capping: 300mm |
| Western Interchange – Westbound Ramp from North | MCC0 | 0 | 1210 | Class 2: 190mm<br>Class 3: 110mm<br>Type A Capping: 300mm |
| Illawarra Main Road** | MCB0 | 570 | 810 | Class 1: 200mm<br>Class 3: 150mm<br>Class 4: 100mm<br>Type A Capping: 110mm* |
| Northern Interchange – Northbound Off Ramp | MCJ0 | 0 | 420 | Class 2: 170mm<br>Class 3: 100mm<br>Type A Capping: 300mm |
| Northern Interchange – Southbound On Ramp | MCH0 | 0 | 460 | Class 2: 140mm<br>Class 3: 100mm<br>Type A Capping: 300mm |
| Northern Interchange – Existing Southern Access Road | MCQ0 | 0 | 320 | Class 2: 170mm<br>Class 3: 100mm<br>Type A Capping: 300mm |
| Northern Interchange – Main Road Perth (Northbound On Ramp) | MCT0 | 320 | 665*** | Class 2: 170mm<br>Class 3: 100mm<br>Type A Capping: 300mm |
| Eskleigh Road | MC50 | 0 | 440 | Class 2: 140mm<br>Class 3: 190mm<br>Type A Capping: If required |
| Unsealed Private Access Roads (Mackinnon, Einoder) | MCV0<br><br><br><br>MCX0<br><br><br><br>MCX5 | 0<br><br><br><br>0<br><br><br><br>0 | 1530<br><br><br><br>730<br><br><br><br>1295 | Class 2: 140mm<br>Class 3: 190mm<br>Type A Capping: If required<br><br>Class 2: 140mm<br>Class 3: 190mm<br>Type A Capping: If required<br><br>Class 2: 140mm<br>Class 3: 190mm<br>Type A Capping: If required |
| Illawarra Main Road New Cul de Sac | N/A | N/A | N/A | Class 2: 140mm<br>Class 3: 190mm<br>Type A Capping: If required |

| Location | String Name | Start Chainage | End Chainage | Pavement Configuration |
|---|---|---|---|---|
| New Southern Roundabout | N/A | N/A | N/A | Class 1: 200mm<br>Class 3: 160mm<br>Class 4: 160mm<br>Type A Capping: If required |
| Main Road (MC20)<br>(Midland Highway) | MC20 | 0<br><br><br><br><br>130 | 130<br><br><br><br><br>240 | Class 1: 200mm<br>Class 3: 160mm<br>Class 4: 160mm<br>Type A Capping: If required<br><br>14mm Type H DGA: 40mm<br>20mm Type SI DGA: 110mm<br>20mm Type SF DGA: 75mm<br>Class 4: 100mm |
| Main Road Connection (MC40) | MC40 | 0 | 240 | Class 1: 200mm<br>Class 3: 160mm<br>Class 4: 160mm<br>Type A Capping: If required |
| New Northern Roundabout No. 2 | N/A | N/A | N/A | Class 2: 170mm<br>Class 3: 100mm<br>Type A Capping: 300mm |
| Northern Interchange – Southbound Off Ramp (MCG0) | MCG0 | 0 | 500 | Class 2: 170mm<br>Class 3: 100mm<br>Type A Capping: 300mm |
| New Northern Roundabout No. 1 | N/A | N/A | N/A | Class 2: 170mm<br>Class 3: 100mm<br>Type A Capping: 300mm |
| Existing Eastern Service Road (MCM0) | MCM0 | 0 | 100 | Class 2: 170mm<br>Class 3: 100mm<br>Type A Capping: 300mm |
| Existing Midland Highway (MCN0) | MCN0 | 0 | 240 | Class 2: 170mm<br>Class 3: 100mm<br>Type A Capping: 300mm |

* Minimum CBR 6 capping as opposed to CBR 10

** Dense graded asphalt overlay to be adopted on Illawarra Main Road between chainage 0 and 570 MCB0 subject to pavement investigations during detailed design.

*** Matches to MC00 at chainage 5215 (approx. 335m north of MCT0 chainage 665)

The geotechnical information supplied with the tender documents did not include any investigations of the condition of the existing Illawarra Main Road pavement. For the tender design it has been assumed that the existing Illawarra Main Road pavement between chainage 0 and 570 MCB0 will receive a dense graded asphalt regulation layer of a minimum 40mm thickness. However, the appropriateness of this treatment would be verified during the detailed design and if necessary modified to a granular overlay. Shoul d it be necessary to place a granular overlay to strengthen the pavement, the extent of dense graded asphalt wearing course would be refined to only include the area in the vicinity of the Pateena Road junction.

Effective management of traffic during the tie-in of the works to the South Esk River Bridge near the Southem Roundabout will be a critical component of managing the expectations of the travelling public during the works. To minimise the duration of the tie-in works and hence traffic disruption, an asphalt pavement has been specified for the Midland Highway extending approximately 90m west from the South Esk River Bridge. A copy of the CIRCLY output for the asphalt pavement design as required by Clause 3040.06(c) is provided in Appendix A.

## 4. Bituminous Surfacing

Table 3010.021 of the tender documents indicates the expected 2019 traffic volumes and percentage of heavy vehicles for the various road elements of the Perth Link Roads project. The traffic data from the tender documents is replicated in Table 5 below.

Table 5: Design Traffic Volumes

| Location | Design Traffic Vehicles (Vehicles/Day) Year 2019 | %HV |
|---|---|---|
| New Highway, South of Western Interchange | 9,278 (two way) | 16 |
| New Highway, through Western Interchange | 6,242 (two way) | 14 |
| New Highway, North of Western Interchange | 11,846 (two way) | 11 |
| Western Interchange – Southbound Ramp | 1,429 (one way) | 27 |
| Western Interchange – Westbound Ramp from South | 1,610 (one way) | 17 |
| Western Interchange – Northbound Ramp | 2,679 (one way) | 10 |
| Western Interchange – Westbound Ramp from North | 2,936 (one way) | 7 |
| Illawarra Main Road | 8,151 (two way) | 12 |
| Northern Interchange – Northbound Off Ramp | 259 (one way) | 17 |
| Northern Interchange – Southbound On Ramp | 370 (one way) | 3 |
| Northern Interchange – Existing Southern Access Road | 2,038 (two way) | 3 |
| Northern Interchange – Main Road Perth | 2,972 (two way) | 3 |
| Eskleigh Road | 500 (two way) | 3 |

In July 2016, Austroads released Technical Report AP-T310-16, Selection and Design of Initial Treatments for Sprayed Seal Surfacings. The Report is an update of the design of initial treatments for sprayed seal surfacing and supersedes AP-T68-06, Update of the Austroads Sprayed Seal Design Method (Austroads 2006b).

Table 3.1 from AP-T310-16 replicated below provides guidance on the selection of initial seals taking into account traffic volumes, average seasonal temperature and equivalent heavy vehicles.

For cool climates and equivalent heavy vehicle percentages less than 25% Austroads recommends the adoption of the following initial seal types for traffic volumes between 200-2000 vehicles per lane per day:

- AMC5 (S/S)
- CRS67 (D/D).

For traffic volumes greater than 2,000 vehicles per lane per day Austroads recommends the following initial seal types:

- AMC6 (S/S or D/D)
- CRS67 (D/D), or
- Modified emulsion (D/D).

Table 3.1: Preliminary guide to the selection of initial seals

| EHV | Average seasonal temperature | Low traffic (<200 v/l/d)[3] | Medium traffic (200–2000 v/l/d)[3] | High traffic (> 2000 v/l/d) |
|---|---|---|---|---|
| | | Typical binder[5] and seal type | | |
| Low (< 25%) | Cool[4] | AMC4 (S/S)<br>CRS67 (S/S)<br>CRS60 (D/D) | AMC5 (S/S)<br>CRS67 (D/D) | AMC6 (S/S, D/D)<br>CRS67 (D/D)<br>Modified emulsion (D/D) |
| | Warm | AMC5 (S/S)<br>CRS67 (S/S) | AMC6 (S/S)<br>CRS67 (D/D) | AMC7 (S/S, D/D)<br>CRS67 (D/D)<br>Modified Emulsion (D/D)<br>PMB[1][2] (S/S, D/D) |
| | Hot | AMC6 (S/S) | AMC7 (S/S) | AMC7 (S/S)<br>PMB[1][2] (S/S, D/D) |
| High (≥ 25%) | Cool[4] | AMC5 (S/S)<br>CRS67 (S/S, D/D) | AMC6 (S/S)<br>PMB[1][2] (S/S, D/D)<br>CRS67 (D/D) | CRS67 (D/D)<br>Modified emulsion (D/D) |
| | Warm | AMC6 (S/S)<br>CRS67 (S/S) | AMC7 (S/S)<br>PMB[1][2] (S/S, D/D)<br>Modified Emulsion (D/D) | AMC7 (S/S)<br>PMB[1][2] (D/D)<br>Modified emulsion (D/D) |
| | Hot | AMC7 (S/S) | AMC7 (S/S)<br>PMB[1][2] (S/S, D/D) | AMC7 (S/S)<br>PMB[1][2] (D/D) |

1 *Guidance for cutting practice for PMB sprayed seals is found in Section 4.7, AP-T235-13, Austroads (2013a).*
2 *Care must be taken when using PMBs (especially SBS) because it is difficult to achieve sufficient adhesion to the base and aggregate wetting. To date successful trials have been undertaken only with crumb rubber binders (Austroads 2014), as yet other binders are unproven in this application, pending further trials.*
3 *When low or medium traffic is coupled with high stress situations such as intersections, turning lanes, and grades, consider following guidance for the 'high traffic' category instead.*
4 *Sealing in cool and damp conditions increases risk of seal failure, consideration should be given to postponing works if possible until weather conditions have improved.*
5 *Cutback bitumen grades nominated in the table are based on typical pavement materials used in Australia. Adjustment to the proportion of cutter oil content may be required for very porous (less cutter oil) or tightly bonded (more cutter oil) pavement surfaces. For tightly bonded surfaces (including stabilised pavements), pavement surface preparation is essential to achieving an adequate bond, particularly when emulsions, low cutter content cutback bitumen and PMB grades are used.*

Table 6 indicates the estimated lane volumes on the Midland Highway assuming a 50/50 split between north and south bound traffic and that 60% of vehicles travel in the left lane and 40% in the right lane.

Table 6: Midland Highway Traffic Volumes

| Location | AADT | Northbound | Southbound | Left Lane | Right Lane |
|---|---|---|---|---|---|
| New Highway, South of Western Interchange | 9,278 | 4,639 | 4,639 | 2,783 | 1,856 |
| New Highway, through Western Interchange | 6,242 | 3,121 | 3,121 | 1,873 | 1,248 |
| New Highway, North of Western Interchange | 11,846 | 5,923 | 5,923 | 3,554 | 2,369 |

Appendix C of VicRoads Code of Practice RC 500.22 Selection and Design of Pavements and Surfacings also provides guidance on the selection of initial seal treatments for roads constructed clear of traffic with over 2,000 vehicles per lane per day. A copy of Appendix C from the Code of Practice is included in Appendix B of this Report. The Code of Practice recommends that roads sealed between October and March be sealed with a prime and 14/7 HSS2 or XSS seal.

Considering that the New Highway South of the Western Interchange and North of the Western Interchange account for a significant proportion of the bituminous surfacing area required for the project and are expected to carry over 2,000 vehicles per day in the left lane, the Tender Design allows for the application of a 14/7 HSS2 seal with a Polymer Modified (S35E) Binder to all sealed roads, except for the areas where dense graded asphalt wearing course will be applied as shown on the Tender Design Drawings. The advantage of the proposed HSS2 seal and PMB is that it provides a more robust seal than a prime and single coat seal which will provide superior construction quality and reduce maintenance requirements for the Department post expiry of the defects liability period. It should be noted that this proposed treatment complies with the requirements of Supplementary Notice No. 5 which mandates the placement of a 14/7 seal for a substantial portion of the Western Link.

Dense graded asphalt wearing course would be provided in areas which will experience high breaking or turning forces from heavy vehicles as well as to bridge decks. The areas where dense graded asphalt will be applied are shown on the Tender Design Drawings. Based on the traffic volumes across the various road elements for which dense graded asphalt wearing course is to be applied a variety of Type N and Type H dense graded asphalt could be adopted. For construction efficiency purposes the Tender Design allows for the application of Type H dense graded asphalt for all dense graded asphalt weari ng course.

# 5. Drainage

## 5.1 Overall strategy

The design of the stormwater drainage has been developed to provide a safe motoring environment in accordance with Austroads standards. The design of the cross drainage incorporates culverts located to provide positive drainage to all areas of the works and sized to mitigate potentially adverse impacts. Longitudinal drainage will comprise channels and pipes. The design of the cross drainage and longitudinal drainage will be closely coordinated with the design of the highway vertical alignment to ensure that sufficient cover is provided to all culverts and pipes, headwalls are appropriately protected where they are located within clear zones, and that the integrated drainage system operates adequately under gravity to prevent ponding or siltation without requiring more than normal maintenance.

### 5.1.1 Flood Plain Mapping and Dam Break Assessment of Sheepwash Creek and West Perth Flood Plain

The VEC SHAW Joint Venture propose to source a substantial proportion of the material required to construct the embankments by expanding Mackinnon's Dam which is located on the western side of the Highway, north of the Perth township. To assess the feasibility of expanding the dam, the VEC SHAW Joint Venture engaged IPD Consulting to review the impact of a 1 in 100 year flood event, assess sunny day dam break inundation and review the dam's consequence category. A copy of the assessment is included in Appendix C.

In modelling the dam expansion, the assessment undertaken by IPD Consulting incorporated relocation of the stock underpass from chainage 3,950 (MC00) to chainage 3,800 (MC00) in addition to the raising of the vertical alignment discussed in Section 2.1.3. The relocation results in Mackinnon's land upstream of the Highway being used as detention storage during a 1 in 100 year flood event. Relocation of the stock underpass and reliance on the upstream land for detention storage significantly minimises flooding in the Perth Township during a 1 in 100 year flood event. Figure 2 from IPD Consulting's report is included below to demonstrate the resultant reduction in flooding.

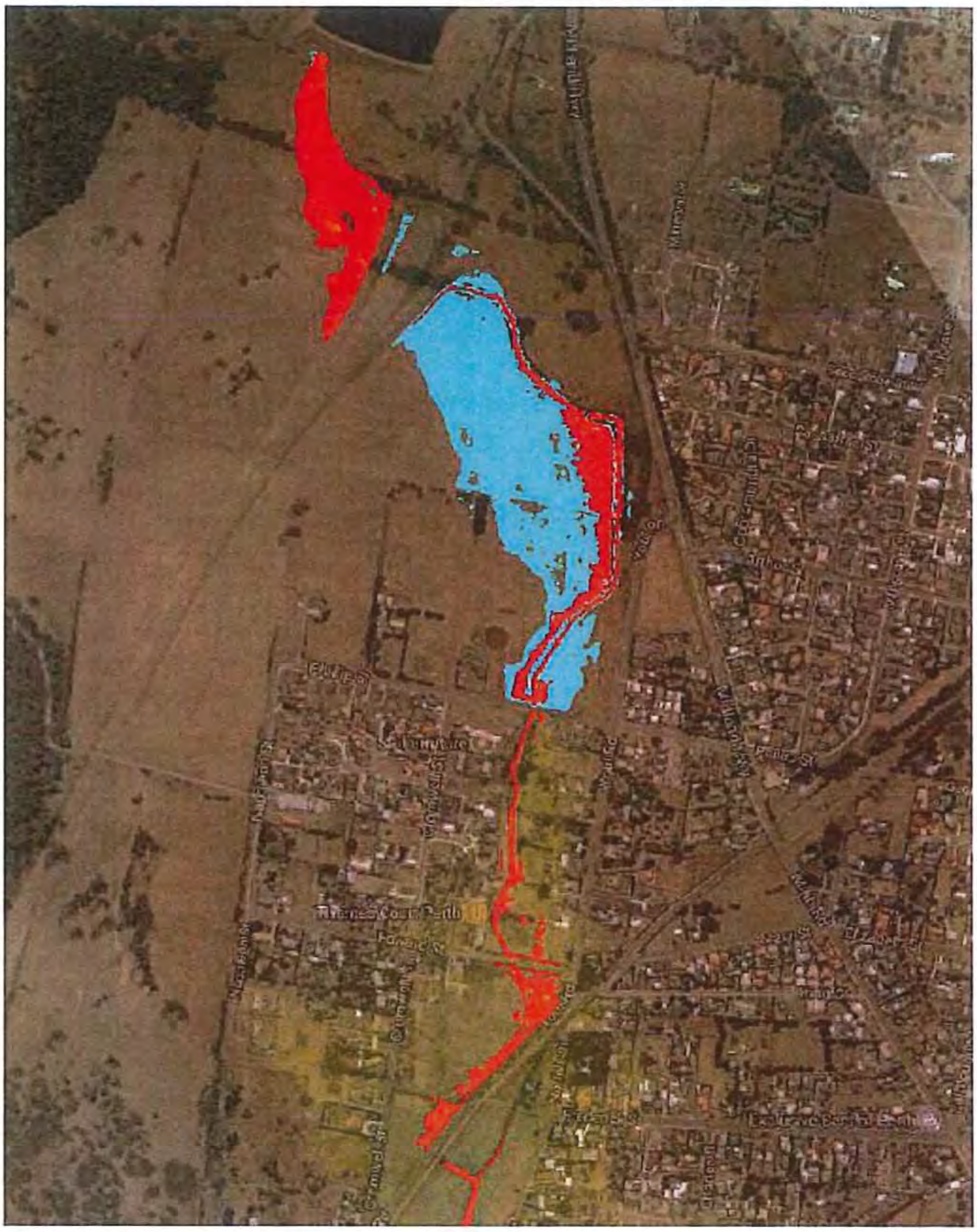

Figure 2 - 1in100 AEP Flood (Blue Without Highway Development, Red With Highway Development)

The VEC SHAW Joint Venture has discussed the stock underpass relocation with Mackinnon and has approval for the relocation. A copy of the agreement is available upon request. Relocation of the northern stock underpass also reduces its length by approximately 70m making it far more suitable for use by stock.

Raising of the vertical alignment and relocation of the northern stock underpass also provides a significant reduction in inundation through the Perth Township during a sunny day dam break event. This is achieved whilst ensuring that the Midland Highway will remain operational.

In assessing Mackinnon's dam, IPD Consulting have determined that a consequence Category of High B as defined by the ANCOLD Guidelines on the Consequence Categories for Dams (2012) should apply regardless of whether the dam is expanded or not.

## 5.2 Road Surface Drainage and Aquaplaning

Aquaplaning occurs when the depth of water on a road surface creates a lift under tyres and centrifugal forces affect traction. Therefore, the road pavement must not only provide sufficient skid resistance, but a road surface grade that allows rainwater to drain off the road surface before it accumulates to a critical depth. In this case, the critical aquaplaning depth is 4mm.

A check on the Tender Design has been undertaken based on the proposed bituminous surfacing types, and confirms that the water depths on all road surfaces are within acceptable limits.

## 5.3 Scour Protection for Surface Drains, Open Channels and Culvert Outlets

The key parameter for the design of scour protection measures is bed shear stress. Shear stresses can be determined from 1D hydraulic model outputs and 2D modelling.

Grass and rock-lined channels and table drains have been assessed using ChannelDesigner[1], which estimates roughness values and shear stresses according to the procedures described in HEC15 – The Design of Roadside Channels with Flexible Linings. Notably, the roughness of grass and rock lined channels increases as the depth of flow decreases relative to the height of the grass or size of rock, to the extent that Manning's n roughness values are often much higher than anticipated and the conveyance of the channel is significantly decreased. Manning's n values of about 0.1 for grass-lined table drains are not unrealistic, and may be appropriate for conditions in which maintenance (i.e. grass mowing) is infrequent and where the soils and climate encourage grass growth.

The extent of scour protection required has been determined from 2D models by mapping peak bed shear stresses. Based on the findings of the analysis, the extent of rock lining required is shown on the Tender Design drawings.

Shear stresses at discharge points at the stock and frog culverts under the Link Roads are relatively benign and scour is prevented by the concrete invert and stock path.

Rock dispersing aprons have been specified for culvert requiring scour protection at outlets. The dispersing aprons are shown on the Tender Design drawings.

## 5.4 Frog Culverts and Stock Underpass Culverts

### 5.4.1 Frog Culverts

The Department of State Growth's Green and Golden Frog Management Guideline[2] (2015) is provided to manage the habitat affected by the construction of the Perth Link Roads. The main habitat areas are associated with Sheepwash Creek. The Project Specification requires oversize culverts that have more than sufficient flood capacity to provide linkages between habitats for frogs.

This project includes two frog linkages:

- Frog culvert chainage 1,490 (MC00) on Sheepwash Creek
- Frog culverts chainage 4,110 (MC00) located near the Northern Interchange.

[1] Proprietary software developed by Martin Jacobs.
[2] Provided in the Tender Documents, Volume 4, Part G

### 5.4.2 Stock Underpass Culverts

This project includes two stock underpasses:

- Chainage 1,470 (MC00) near Sheepwash Creek, which comprises 1 x 3,600 x 3,000 RCBC and passes under the main carriageway
- Chainage 3,800 (MC00) near the Norther Interchange, which comprises 1 x 3,600 x 3,000 RCBC and passes under the northbound off-ramp, the main carriageway and southbound on-ramp.

The stock underpass near the Northern Interchange has been relocated to the south by approximately 160m relative to the Principal's Reference Design. The advantages of the relocation are as follows:

- The overall length of the stock underpass is reduced by approximately 70m providing a construction saving for this element
- The stock underpass in the Principal's Reference Design was approximately 110m long and may result in stock resisting passage through the underpass
- The relocated position still minimises flooding in the Perth Township for a 1 in 100 year flood event.

## 5.5 Systematic Design of Culverts and Channels

### 5.5.1 Modelling Strategy

The systematic design of culverts and channels has been facilitated by the development of a comprehensive DRAINS model. The purpose of the model is to include all the catchments and drainage paths affected by the proposed road and to route flows through the proposed drainage system. The design criteria have been applied with the aim of producing a compliant design.

### 5.5.2 Catchment Delineation and Parameterisation

Sub-catchments were delineated by reference to the following terrain information:

- Approximate catchment boundaries from contours and catchments from TheList mapping
- 1m DEM, where it was available (the DEM coverage did not extent to the extremities of some catchments)
- Road design model

Catchment parameters were defined as follows:

- Fraction impervious varied according to catchment and interpreted from aerial photographs
- Runoff coefficients:
  - $C_{10}$ pervious areas = 0.10
  - $C_{10}$ impervious areas = 0.90
- Time of concentration calculated in accordance with the requirements of the Tender Documents.

### 5.5.3 Estimation of Flows

The Intensity Frequency Distributions (IFD) used in the DRAINS model were sourced from the on-line BoM tool for 2016 IFD.

DRAINS estimates stormwater runoff flows by the Extended Rational Method.

DRAINS routes flows through links, so that flows can be estimated at every component of the drainage system. DRAINS can also be used to estimate the sizes of culverts, pipes and channels.

### 5.5.4 Culvert Blockages

Culverts were modelled with a blockage factor of 50% in accordance with the requirements of the Tender Documents.

### 5.5.5 Culvert Capacity

The culvert sizes proposed in the Principal's Reference Design have been retained in the Tender Design except for the crossing of Illawarra Road, west of the Western Interchange at chainage 930 (MCB0) and 680 (MCA0). The Principal's Reference Design proposed that the culvert crossing of the Northbound and Southbound Ramps be a DN600 RCP and that the crossing of the Westbound Ramps be a DN900 RCP. Analysis of this catchment which has an area of approximately 27 hectares indicates that the culverts proposed in the Principal's Reference Design would result in the water overtopping Illawarra Road in a 100 year ARI.

To mitigate this risk recognising that the road levels in this area constrained by bridge clearance requirements for the Western Interchange a short distance to the east, the Tender Design incorporates a wide table drain on the northern side of the Northbound Ramp within the road reservation. The widened table drain which provides approximately 1,700m$^3$ of detention storage restricts the capacity of the culverts required to twin DN900 RCPs under both the North and South Bound Ramps and the Westbound Ramps.

Analysis of the existing DN600 culvert under the rail track south of Railway Underpass No. 1 indicates that the culvert has insufficient capacity for a 100 year ARI based on existing conditions. The culvert and detention system arrangement proposed above will limit flows in a 100 year ARI event to this culvert to approximately pre-development conditions avoiding increased flooding risk for the rail line and also the Mountford Nominees Pty Ltd property located downstream of the rail culvert.

### 5.5.6 Southern Roundabout

The Tender Design incorporates a modified drainage layout for the Southern Roundabout. The layout adopts table drain grated pits on the northern approach to the roundabout as level constraints do not make it practical to adopt a culvert at the transition from table drain to kerb and gutter. The drainage layout has also been developed to provide RCP sizes that are of sufficient size to connect to standard Department of State Growth drainage structures. It is for this reason that RCPs are provided on both the southern and northern sides of the Midland Highway. The outfall of the longitudinal system has been designed so that the outlet drain from the DN900 is clear of the existing gravity water and sewer pipes.

### 5.5.7 Northern Interchange

The drainage layout at Northern Roundabout No. 1 is consistent with the layout of the Principal's Reference Design. Kerb scuppers and batter drains have been provided at the southern end of the kerb and gutter on the existing Midland Highway connection to the roundabout to discharge water from the kerb and gutter into the adjacent open drainage system. Kerb scuppers have also been provided on the Existing Southern Access Road at the eastern end of the kerb and gutter which passes under the Northern Underpass.

As mentioned previously, the levels on the Northbound Off Ramp, Northern Roundabout No. 2 and Northbound On Ramp have been raised to protect the Main Midland Highway alignment and the Perth Township for a dam break scenario accounting for enlargement of the Mackinnon Dam. In raising these levels, the cross fall of Northern Roundabout No. 2 has been modified. Accordingly, the Tender Design incorporates a modified longitudinal drainage layout for Northern Roundabout No. 2.

## 6. Geotechnical

The geotechnical information supplied with the Tender Documents has been reviewed as part of developing the Tender Design. A Geotechnical Report outlining the findings of the review including geotechnical requirements that have been integrated into the Tender Design is included in Appendix D.

# 7. Service Relocations

Details of the services works required to facilitate the road construction are outlined by location below. For each location, specific details regarding each service type is provided. The locations which require service locations include:

- The Southern Roundabout
- New Midland Highway chainage 670 to 800 (MC00)
- Western Interchange
- New Midland Highway chainage 3,740 (MC00)
- Northern Interchange

## 7.1 Southern Roundabout

### 7.1.1 Telstra

There is an existing Telstra cable which crosses the Midland Highway approximately 20m west of the South Esk River Bridge at chainage 220 (MC20). There are pits either side of the road crossing and the pit on the southern side of the Highway will require raising to suit the new levels. The Telstra cable will need to be potholed prior to installation of the road safety barrier to avoid impacting on the Telstra cable during driving of the posts.

### 7.1.2 Sewer

There are gravity and rising sewer mains in the vicinity of the Southern Roundabout. Connection of Eskleigh Road to the new roundabout requires a section of the DN225 AC sewer rising main to be relocated. A proposed alignment for the relocation is shown on the Tender Drawings. Provision has been made for a scour valve along the new section of rising main and an air valve where it connects into the existing rising main on the southern side of Eskleigh Road. It is not anticipated that the rising main crossing of the Midland Highway will be impacted by the works. However, potholing investigations will need to be undertaken to confirm this during the detailed design phase.

The existing gravity sewer main crosses Eskleigh Road at approximately chainage 250 (MC50) and the Midland Highway at chainage 170 (MC20). To avoid the need to relocate the sewer main for Eskleigh Main Road the vertical alignment has been lifted to ensure adequate cover to the main is provided. This vertical alignment modification also has the added benefit of avoiding the need to relocate the adjacent water main. It is not anticipated that the gravity sewer main crossing of the Midland Highway will be impacted by the works based on the levels provided in the Principal's Reference Design.

A scour line also crosses the Midland Highway at approximate chainage 200 (MC20) and crosses Eskleigh Main Road at approximate chainage 400 (MC50). An air valve on the scour line in the vicinity of chainage 430 (MC50) will most likely require raising to suit the new road levels. It is not anticipated that the Midland Highway crossing will be impacted by the works. However, potholing investigations will need to be undertaken to confirm this during the detailed design phase.

### 7.1.3 Water

As indicated above, raising of the vertical alignment of Eskleigh Road in the vicinity of chainage 250 (MC50) is expected to avoid the need to relocate the existing water main.

It is not anticipated that the water main crossing of the Midland Highway will be impacted by the works based on the levels provided in the Principal's Reference Design.

## 7.2 Chainage 670 to 800 (MC00)

### 7.2.1 Electricity

An existing overhead electricity line crosses the new Midland Highway near the access to Glen Ireh Estate at chainage 670 (MC00). From a safety in design perspective it is recommended that this overhead crossing be relocated underground, minimising the potential for the electricity cables to be impacted during construction as well as following completion of the works when the new Highway is open to traffic.

### 7.2.2 Telecommunications

An existing Telstra cable crosses the Highway at chainage 800 (MC00). The vertical alignment and associated tables drains in this location require the cable to be lowered to provide adequate cover. Whilst the scope of the relocation works will need to be coordinated with Telstra, it is expected that new pits will be installed on the north eastern side of the Highway and the south western side of the Glen Ireh Access & Service Road.

## 7.3 Western Interchange

### 7.3.1 Electricity

Relocation underground of the existing overhead electricity line which is located parallel to Illawarra Road is an important component of the service relocation works at the Western Interchange. Relocation underground will require the installation of a stay on the existing pole at chainage 1440 ( MCB0) and a road crossing under Illawarra Main Road. The new underground alignment will be positioned adjacent to the Westbound Ramp (from the South) before passing under the two railway underpasses and connecting to an existing overhead electricity pole near the new cul-de-sac. In the vicinity of the railway underpasses the underground electricity cable will be positioned underneath the shared path between an existing Telstra cable and the western abutments of the rail underpasses. It is proposed to divert the underground electricity cable around the western side of the cul-de-sac to avoid the cable being positioned under the new pavement.

The new underground electricity alignment will cross two new water mains being installed by TasWater. Liaison with TasWater will be required to confirm the as constructed installation depths of the water mains so that adequate separation between the water mains and the underground electricity are provided.

### 7.3.2 Telecommunications

As discussed in Section 7.3.1, there is an existing Telstra cable located on the northern side of the railway line. This cable is not expected to be impacted by the construction of the shared path.

To the south of the railway line there is an optic fibre cable. Construction of the reinforced earth walls associated with the railway underpasses will require the installation of reinforcing straps over the optic fibre cable. Liaison with Telstra will be required to confirm that the cable can be retained in its current location.

### 7.3.3 Water

TasWater are intending to modify their water infrastructure in the vicinity of the Western Interchange so that it is compatible with the Principal's Reference Design. Informed by the water main realignment information provided, the Tender Design has been developed so that it compatible with the modifications planned to be undertaken by TasWater. As constructed information will need to be obtained from TasWater to inform the detailed design and supplemented with potholing information where required. A critical component of the detailed design will be integration of the reinforced earth walls associated with the Westbound Ramp (From North) Overpass.

Where redundant water mains are located within the extent of the earthworks they will be removed.

## 7.4 Chainage 3740 (MC00)

### 7.4.1 Telecommunications

Telstra and NBN cables cross the new Midland Highway at approximate chainage 3740 (MC00). Informed by level information supplied by State Growth for these cables, the open drains on either side of the Highway at the crossing have been modified to ensure that adequate cover is provided for these cables.

## 7.5 Northern Interchange

### 7.5.1 Telecommunication

Construction of Northern Roundabout No. 1 requires relocation of Telstra and NBN cables. The relocation works will need to be completed prior to commencement of the roundabout.

### 7.5.2 Electricity

An existing overhead electricity pole on the eastern side of Northern Roundabout No. 1 requires relocation clear of the works. Similarly to the Telstra and NBN cables, the pole will need to be relocated prior to construction of the roundabout.

# 8. Street Lighting

Street lighting is required for all roundabouts and for merge and diverge lanes from the gore area continuously to the end of the taper. The lighting levels of AS1158 Part 1 for Category V5 are to be satisfied.

To determine street lighting quantities **pitt&sherry** has developed a concept street lighting layout. The layout has been developed using 12.0m frangible poles with 3.0m outreach arms and 250W luminaires where possible to provide the same appearance as the street lights on the Perth to Breadalbane project. The concept layout requires the installation of 80 of these streetlights.

To adequately illuminate the Southern Roundabout a 20m high pole with four 3.0m outreach arms is required. This pole would be mounted in the centre of the roundabout island.

The street lighting assessment has also identified that under structure lighting will be required for the Southbound Ramp Underpass, Westbound Ramp Underpass and Westbound Ramp Overpass. Due to its relatively short length, under structure lighting is not required for the Northern Underpass.

For the detailed design **pitt&sherry** would finalise the street lighting layout before providing the information to TasNetworks in digital format to enable them to undertake the electrical design component. Details of TasNetworks' electrical design would then be integrated onto the roadworks drawings so that the full scope of the civil works component of the street lighting is accurately depicted on the drawings. A key component of TasNetworks' electrical design input will be selection of appropriate power supply points.

# 9. Bridges

## 9.1 General

This section provides details of the tender design for the following:

- Southbound Ramp Underpass
- Westbound Ramp Underpass
- Westbound Ramp Overpass
- Northern Underpass
- Railway Underpass No.1
- Railway Underpass No.2
- Box culvert structures at Ch. 1,465, 1,470, 3,800 and 4,100

The tender design is in accordance with AS5100:2017, adopting two lanes of SM1600 and HLP400 loading as required in the tender brief.

The design details for each bridge are provided in the tender design drawings as part of this submission. Key dimensions for the six bridges are as follows:

Table 7: Bridge Design Details

| Bridge | Span Length | Beam Skew | T-Beam Size (excluding deck) |
|---|---|---|---|
| Southbound Ramp Underpass | 17.4m | 30 degrees | 750mm |
| Westbound Ramp Underpass | 17.4m | 30 degrees | 750mm |
| Westbound Ramp Overpass | 17.4m | 30 degrees | 750mm |
| Northern Underpass | 21.6m | 14 degrees | 1000mm |
| Railway Underpass No.1 | 18.5m | 32* degrees | 750mm |
| Railway Underpass No.2 | 19.5m | 28 degrees | 750mm |

* Consistent with the Principal's Reference Design

The design details for each box culvert are provided in the tender design drawings as part of this submission Key dimensions for the box culverts are as follows:

Table 8: Box Culvert Key Dimensions

| Box Culvert | Culvert Size | Culvert Length | Max Fill over |
|---|---|---|---|
| Ch. 1,465 | 3,600mm wide by 3,000mm high | 40.6m | 3.6m |
| Ch. 1,470 | 1,500mm wide by 1,200mm high | 78.4m | 7.5m |
| Ch. 3,800 | 3,600mm wide by 3,000mm high | 40.6m | 1.0m |
| Ch. 4,100 | 1,500mm wide by 1,200mm high | 90.64m | 10.6m |

## 9.2 The Design Process

We understand the importance of communication between the designer and contractor in a D&C contract. As described elsewhere in our submission, **pitt&sherry** and VEC have a long history of working together on D&C contracts, in particular on bridges.

We have read and understood the requirements of the tender brief with respect to the design process as described in Section 1170.04.

In particular, the structures design process will include:

- Contributing to the monthly design report for the project;
- Attending the design coordination meetings;
- Undertaking design verification at Preliminary and Detailed design stages as required in the tender brief Section 1170.05;
- Proof engineering of all bridges and culverts including foundations by an independent engineer (Rare) in accordance with Section 1170.06;
- Submission of the bridge design details to the Superintendent at preliminary and detailed design stage in accordance with section 1170.07.

The above will form part of the design management strategy for the project in accordance with Section 2030.

## 9.3 Safety in Design

**pitt&sherry** have successfully integrated Safety in Design as part of our design thinking over the past two years, and this thinking has informed many aspects of our tender design as described below.

The fully integral bridge designs eliminate the need for bearings and joints, which reduces safety risks to future inspection and maintenance personnel who would need to inspect and replace these components.

Another aspect of this tender designs that improves safety is that all bridges are single span. As a result, no piers are required, which eliminates the additional risk of inspection of these piers, and eliminates the risk of pier collision by train or vehicles.

At the reinforced soil wingwalls, we have allowed to extend the concrete panels up an additional 900mm above finish surface level behind the walls, which removes the need for a safety barrier on top of the wingwalls, and provides an improved barrier protection.

As the preliminary and detailed design progresses, there will be more opportunities to incorporate safety features into the design.

## 9.4 Integral Bridges

The tender brief emphasises the importance of minimising bridge maintenance of its design life, and one of the best ways to do this is to make bridges integral where possible, eliminating the need for the maintenance and replacement of expansion joints and bearings.

Our design approach has been to make all six bridges integral, including the Southbound and Westbound Ramp Underpass bridges, which were semi-integral in the reference design. Making all bridges integral with a maximum skew of 30 degrees means there are no bearings or deck joints. As a result, there is no need for the provision of access for inspection of these items, and no ongoing costs for maintenance and eventual replacement of the bearings and deck joints.

Using fully integral bridges does mean there is additional deck area on the Southbound Underpass, Westbound Underpass and Westbound Overpass that does not form part of the roadway. This deck area can be left simply as a concrete finish, or there may be opportunities to use these spaces for vegetation etc. upon award there will be the opportunity to discuss such opportunities with the Principal.

Note that for the Westbound Overpass bridge, there is the possibility to increase the skew on this bridge significantly more than 30 degrees, resulting in reduced unused bridge deck area and abutment length. This design has not been submitted as our preferred design, because the design would be a substantial departure from the intent of the Tender Brief with respect to integral bridges. However, if our tender was successful, there are opportunities to seek this alternative design with the Principal as a way of providing some additional costs savings for the construction of this bridge.

## 9.5 Bridge Superstructure

All bridges have been designed using conventional Super T (T- Roff) precast beams of varying depths (refer Table 7).

As all bridges have relatively short span lengths, other beam types including precast planks and Superplanks could be used just as effectively. While T-Beams have been submitted as the preferred design, if our tender was successful, there are opportunities to review alternative beam designs with State Growth as a way of providing some additional cost savings for the project.

All bridges on the bridge have been designed with medium level barriers to AS5100:2017. The desing has included a risk assessment for the off-structure barriers, and regular level barriers are appropriate. As a result, the design includes a transition to TL4 Thrie beam barriers on the approaches.

## 9.6 Bridge Substructure

All the tender designs have reinforced soil structures at the abutments. The primary function of these walls is to retain fill – they do not carry bridge live loads. Galvanised metal straps have been adopted at the abutments for increased durability when compared to other strap types. The reinforced soil panel shape and pattern will be similar to those recently constructed on the Devon Hills bridge that is referenced in the tender brief.

Bridge dead and live loads are supported by steel piles. The steel piles supporting the bridge superstructure shall be sleeved to allow for the correct amount of rotation and translation of the integral abutments. The pile sleeves also keep the pile loads independent of the reinforced soil walls.

At the railway underpass bridges, additional train collision protection has been provided behind the reinforced soil walls as shown on our drawings.

Please refer to the geotechnical section of this concept design report for further information on the bridge substructures.

## 9.7 Bridges Over Rail

TasRail is a key stakeholder in this project as two main structures, Railway Underpass 1 & 2 traverse the railway line. During the tender design phase, the design team has had two meetings with TasRail to discuss the project, look at options, and discuss the impacts the project will have on TasRail.

On this basis it is considered that the tender designs for Railway Underpass 1 & 2 are likely to meet the requirements of TasRail.

## 9.8 Box Culverts

The design has optimised the location and hydrology in the tender design to obtain the most cost effective box culvert solutions – refer to the hydraulics section in this design report for further details.

The design has used insitu concrete base slabs in accordance with the tender brief. The base slab in stock underpasses will be designed for exposure class C1. Precast crown units will be used.

## 9.9 Other Structure Design Details

The bridge design drawings show varying asphalt thicknesses from 50-100mm to suit the vertical curvature and super elevation across the bridge decks. It is noted that the tender brief requires an unfactored 100mm thick asphalt thickness to allow for future asphalt thickness increases, and the loading from this asphalt depth has been allowed for in the tender design.

The design includes the earthquake requirements for bridge structures under the new standard AS5100:2017, and our tender designs comply with these requirements.

The design allows for the durability requirements for the site to ensure that a 100 year design life will be achieved.

The design has included a lighting review, and three bridges: the Southbound Ramp Underpass, Westbound Ramp Underpass, and the Westbound Ramp Overpass require lighting to be installed to the underside of the bridge, due to their large bridge widths.

## 9.10 Design Opportunities

The tender design of each structure is in accordance with the Tender Brief Section 3050 – Structures. In some cases, the design process has identified parts of the tender brief or the relevant State Growth specifications that provide an opportunity to make changes to the specified design requirements. These will provide a significant cost saving and/or design improvement.

These opportunities have been provided to State Growth separately as part of the tender submission, but the key items have been summarised in the table below where relevant to the structures:

Table 9: Bridge Design Opportunities

| Component | Description |
|---|---|
| Piling | All structures adopt UC piles enabling the same process and equipment to be used at each structure. This includes the eastern abutment on the Northern Underpass which with higher level basalt, is proposed to have UC sections stood in mass concrete and hence keeping all other abutment details the same. |
| Reinforced Soil Walls | Reinforced soil walls will be designed by **pitt&sherry** reducing cost and complexity in managing the design process. The amount of walls has been optimised by increasing batter slopes to 1.5:1 near the bridge structures reducing wall lengths allowing embankment fill to spill around wall terminations. Wall heights will continue approx. 900mm above finished surface level to act as a pedestrian barrier at the top of the wingwalls. |
| Integral Abutments | All structures will include integral abutments in the design, eliminating the need for maintenance and replacement of expansion joints and bearings. |

| Component | Description |
|---|---|
| Superstructure | The Southbound Ramp Underpass, Westbound Ramp Underpass and Westbound Ramp Overpass have all been designed to have the same length beams, simplifying construction and minimising construction duration. |

## 9.11 Bridge Summary

The bridge tender designs provided have been well developed using the design teams extensive experience in designing bridges for State Growth. These key advantages of these bridges designs are that they provide low maintenance, low risk bridges for the Principal.

# 10. Signs and Linemarking

The signs and linemarking design for the Tender Design is consistent with the Principal's Reference Design and compliant with the requirements of the Tender Documents. Preliminary sizing calculations for direction and reassurance signs were performed to assist with pricing.

# 11. Accommodation Works

The scope of accommodation works has been based on the information supplied in the Principal's Reference Design. Relocation of the stock underpass near the Northern Interchange has been agreed with Mackinnon. Evidence of the agreement is available.

# 12. Alternative Design

The VEC SHAW Joint Venture tender submission includes a proposed alternative involving the provision of a single northbound lane on the Midland Highway from the Southern Roundabout to chainage 2980 (MC00) where the Northbound Ramp from Illawarra Road connects to the Midland Highway. Under this alternative full width earthworks would be completed in excavation; however, the width of the embankment would generally only be constructed to accommodate a single northbound lane and shoulder. In the vicinity of the bridge structures the embankment would be constructed full width due to the reinforced earth walls. The full width of the bridge abutments and deck for Railway Underpass No. 1, the Southbound Ramp Unde rpass and Westbound Ramp Underpass would also be constructed to avoid having differing aged bridge components. The northbound pavement would only be constructed to sufficient width to accommodate the single northbound lane and shoulder. The advantage of this alternative design is that it minimises the quantity of imported embankment material and pavement required which results in a corresponding cost saving. Provision of full width embankment in the vicinity of Railway Underpass No. 1, the Southbound Ramp Underpass and Westbound Ramp Underpass minimises the complexity of widening in the embankment in the future.

Table 3040.063 of the tender documents indicates that the Annual Average Daily Traffic on the Midland Highway south of the Western Interchange and through the Western Interchange is expected to be 9,278 vehicles per day and 6,242 vehicles per day respectively. Allowing 20 years of 1.5% compound traffic growth the traffic volumes in 2029 are estimated to be 12,496 and 8,407 vehicles per day respectively. A general traffic engineering rule of thumb is that peak hour traffic volumes are typically 10% of the Annual Average Daily Traffic volume. Applying this rule, the peak hour volumes in 2029 on the Midland Highway south of the Western Interchange and through the Western Interchange are expected to be in the order of 1,250 and 840 vehicles per hour respectively. Assuming a 50% northbound/50% southbound split, the northbound peak hour traffic volumes are expected to be in the order of 625 and 420 vehicles per hour respectively. The traffic lanes on the Midland Highway north of the Southern Roundabout could be expected to have a traffic carrying capacity in the order of 1,800 vehicles per hour per lane. Based on this, a single northbound lane could be expected to have more than adequate traffic capacity to cater for the expected traffic demand in 2029.

Based on the traffic volume assessment it would be feasible to also reduce the Midland Highway to a single southbound lane through the Western Interchange and south of the Western Interchange. However, as the Southbound Ramp from the Western Interchange is expected to carry 1,429 vehicles per day in 2029 of which 27% are heavy vehicles, adopting a single southbound lane may be undesirable as it would reduce the efficiency of the merge.

## 13. Summary

The VEC Shaw JV Tender Design that has been developed in collaboration with **pitt&sherry** provides the following key features:

- Minimises the length of tie-in works to be constructed under traffic at the Southern Roundabout
- Minimises the need for service relocations through optimisation of design levels
- Provides improved flood protection for the Perth Township
- Optimises pavement configurations based on the sourcing of rock to make up for the project's significant earthworks imbalance thereby minimising construction traffic and potential for damage to public roads
- Provides bituminous surfacing treatments appropriate for the expected traffic volumes, minimising future maintenance demands
- Minimises traffic disruption during construction through constructing a temporary detour to Illawarra Main Road
- Facilitates construction of the major earthworks without the need to travel on public roads
- Provides low maintenance bridge structures through the adoption of integral abutments.

The alternative design described in Section 12 would reduce costs through the associated reduction in imported embankment and pavement quantities. The reduction in traffic capacity on the Midland Highway has been demonstrated to be acceptable.

# Appendix A

## CIRCLY Output for Asphalt Pavement Tie-in

**HB17353.txt**

CIRCLY Version 5.0u (8 April 2013)

Job Title: HB17353 Asphalt Pavement

Damage Factor Calculation

Assumed number of damage pulses per movement:
One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

ID: HB17353b Title: New Highway, South of Western Interchange

| Load No. | Load ID | Movements |
|---|---|---|
| 1 | ESA75-Full | 1.60E+07 |

Details of Load Groups:

| Load No. | Load ID | Load Category | Load Type | Radius | Pressure/ Ref. stress | Exponent |
|---|---|---|---|---|---|---|
| 1 | ESA75-Full | SA750-Full | Vertical Force | 92.1 | 0.75 | 0.00 |

Load Locations:

| Location No. | Load ID | Gear No. | X | Y | Scaling Factor | Theta |
|---|---|---|---|---|---|---|
| 1 | ESA75-Full | 1 | -165.0 | 0.0 | 1.00E+00 | 0.00 |
| 2 | ESA75-Full | 1 | 165.0 | 0.0 | 1.00E+00 | 0.00 |
| 3 | ESA75-Full | 1 | 1635.0 | 0.0 | 1.00E+00 | 0.00 |
| 4 | ESA75-Full | 1 | 1965.0 | 0.0 | 1.00E+00 | 0.00 |

Layout of result points on horizontal plane:
Xmin: -3000 Xmax: 5000 Xdel: 5
Y: 0

Details of Layered System:

ID: HB17353A2 Title: Perth Link Road New Highway, South of Western Interchange CBR4

| Layer No. | Lower i/face | Material ID | Isotropy | Modulus (or Ev) | P.Ratio (or vvh) | F | Eh | vh |
|---|---|---|---|---|---|---|---|---|
| 1 | rough | DG14TH80ho | Iso. | 4.90E+03 | 0.40 | | | |
| 2 | rough | DG20TSI80H | Iso. | 5.30E+03 | 0.40 | | | |
| 3 | rough | DG20TSF80H | Iso. | 5.15E+03 | 0.40 | | | |
| 4 | rough | Gran_80 | Aniso. | 8.00E+01 | 0.35 | 5.92E+01 | 4.00E+01 | 0.35 |
| 5 | rough | Sub_CBR4 | Aniso. | 4.00E+01 | 0.45 | 2.76E+01 | 2.00E+01 | 0.45 |

Performance Relationships:

| Layer No. | Location | Performance ID | Component | Perform. Constant | Perform. Exponent | Traffic Multiplier |
|---|---|---|---|---|---|---|
| 1 | bottom | DG14TH80ho | ETH | 0.003269 | 5.000 | 1.100 |
| 2 | bottom | DG20TSI80H | ETH | 0.003151 | 5.000 | 1.100 |
| 3 | bottom | DG20TSF80H | ETH | 0.003839 | 5.000 | 1.100 |
| 5 | top | Sub_2004 | EZZ | 0.009300 | 7.000 | 1.500 |

Reliability Factors:
Project Reliability: Austroads 97.5%

| Layer No. | Reliability Factor | Material Type |
|---|---|---|
| 1 | 0.67 | Asphalt |
| 2 | 0.67 | Asphalt |
| 3 | 0.67 | Asphalt |
| 5 | 1.00 | Subgrade (Austroads 2004) |

Details of Layers to be sublayered:
Layer no. 4: Austroads (2004) sublayering

Results:

| Layer | Thickness | Material | Load | Critical | CDF |
|---|---|---|---|---|---|

**HB17353.txt**

| No. | | ID | ID | Strain | |
|---|---|---|---|---|---|
| 1 | 40.00 | DG14TH80ho | ESA75-Full | -1.24E-05 | 2.04E-05 |
| 2 | 110.00 | DG20TSI80H | ESA75-Full | -4.84E-05 | 2.24E-02 |
| 3 | 75.00 | DG20TSF80H | ESA75-Full | -1.22E-04 | 8.51E-01 |
| 4 | 100.00 | Gran_80 | | n/a | n/a |
| 5 | 0.00 | Sub_CBR4 | ESA75-Full | 3.46E-04 | 2.37E-03 |

# Appendix B

# Appendix C from VicRoads Code of Practice 500.22

## Appendix C - Guide to Selection of Initial Seal Treatments on Pavements Constructed Clear of Traffic

**Appendix C**

| Period when Initial Treatment is Applied | Opening to Traffic within 12 months of Application of Initial Bituminous Surfacing (BS) Treatment for roads and highways with >2000 vehicles/lane/day | | | Opening to Traffic more than 12 months after first sprayed BS treatment |
|---|---|---|---|---|
| | Opening from October to March | Opening from April to May | Opening from June to September Should be avoided. The following options may be considered in some circumstances. | |
| October to March | Prime & Size 14/7 HSS2 or XSS seal | Prime & Size 14 seal with a polymer modified binder followed by a Size 7 emulsion seal (consider polymer modification) at 1-2 weeks before opening depending on condition of surface and weather. | Prime & Size 14 seal using a polymer modified binder, followed by a Size 7 polymer modified emulsion seal at 1-2 weeks before opening depending on condition of surface and weather. | In most circumstances it is undesirable to seal and have long periods without traffic. In some circumstances it may be desirable to apply holding treatments such as a prime and size 7 seal to protect a prepared pavement surface from construction traffic prior to applying the final surfacing treatment. In such cases specialist advice should be sought. |
| April to May | Size 7 emulsion primerseal followed by a Size 14/7 HSS2 or XSS 1-2 weeks prior to opening. | Size 10 primerseal followed by a Size 7 emulsion seal (consider polymer modification) at 1-2 weeks before opening. Apply a Size 14/7 HSS2 or XSS final seal in 1 to 3 years. | Size 10 primerseal followed by a Size 7 polymer modified emulsion seal at 1-2 weeks before opening. Apply a Size 14/7 HSS2 or XSS final seal in 1 to 3 years. | |
| June to September Should be Avoided. Delay pavement preparation until October. | Size 7 emulsion primerseal followed by a Size 14/7 HSS2 or XSS seal 1-2 weeks prior to opening. | Avoid, postpone pavement preparation and sealing works until October | | |

*Notes to Appendix C*

- *Location of works, Weather and pavement conditions can vary the treatments suggested and this guide should only be used to assist with programming of works and determining potentially suitable treatments which should be confirmed prior to application.*
- *Specialist advice should be obtained to confirm the appropriate selection of the most appropriate treatments. In some cases as described above a HSS2 or XSS seal may not be necessary and could be substituted with a lesser treatment if traffic volumes and characteristics are sufficient to justify. Further guidance can be found in Appendix A of the Update of Double/Double design for Austroads Sprayed Seal Design Method (AP-T236/13)*
- *All Size 7 seals applied as a second or third application are applied at the base rate of application unless designed as a Double/Double seal.*
- *Hatched areas of Table – There are significant risks of poor performance and works should not be planned to occur during these periods. Avoid, postpone pavement preparation and sealing works until October. Specialist advice should be sought.*

# Appendix C

## Mackinnon's Dam – Flood & Dam Break Inundation Review

pitt&sherry ref: HB17353H003 CONCEPT DESIGN REPORT 31P REV01/RM/bc

9 November 2017

Shaw Contracting Pty Ltd
776 Whitemore Road
Whitemore, 7303

Doc ref: 1647_Rev C

Attn: Joe Luttrell, Chief Executive Officer

Dear Joe,

**Mackinnon's Dam (ID 7569)**
**Flood & Dam Break Inundation Review**

The following report reviews the likely impact of the new Breadalbane – Perth Highway (Perth bypass stage 2 works) on the flooding of the Perth area during a 100AEP storm event, and also the impacts during a dam break of MacKinnon's Dam, which lies slightly to the north of Perth.

The review has undertaken the following specific tasks;

- Review the impact of the 100AEP design flood
- Review the Sunny Day Dam break inundation
- Review the Dams Consequence Category

## 1.1 Background

### *1.1.1 Dam Details*

Construction of Mackinnon's dam was complete in March 2010, with initial planning and engineering investigations undertaken as early as 2005. The dams' pertinent details, as highlighted in the As-Constructed report is summarised below in Table 1.

| | |
|---|---|
| Year Constructed | 2010 |
| Dam Capacity | 206 ML |
| Catchment | 1 km$^2$ |
| Embankment Type | Earthfill |
| DPIPWE Dam ID | 7569 |
| Upstream Batter | 3.0 (H): 1 (V) |
| Downstream Batter | 3.0 (H): 1 (V) |
| Maximum Height | Approximately 10.25 m |
| Crest | Approximately 4.0 m wide |
| Spillway | Overtopping spillway channel cut into right abutment and left abutment |

**IPD Consulting Pty Ltd**
Infrastructure Planning & Design Mobile: 0419 574 975 Email: mwalters@ipdconsulting.com.au P.O. Box 1371, Launceston TAS 7250 www.ipdconsulting.com.au ABN 96 121 714 878

Table 1 - Mackinnon's Dam; Pertinent Details

Mackinnon's Dam is located approximately 2 km's north of the township of Perth, as shown in figure 1, in the north of Tasmania. The dam impounds Sheepwash Creek, a tributary of the South Esk River.

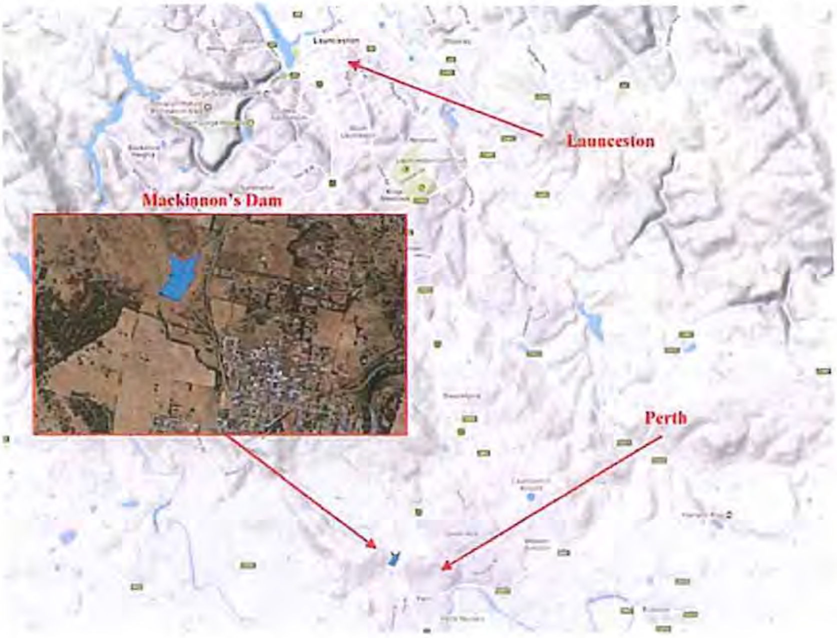


Figure 1 - Mackinnon's Dam Location (Google, 2017)

### *1.1.2 Current Consequence Category*

We understand that the Mackinnon's dam is currently categorised and accepted as a storage with a 'Significant' hazard category, and that this hazard category was recommended by ARM in accordance with the (now superseded) ANCOLD *Guidelines on Assessment of the Consequences of Dam Failure* (May 2000).

The consequence category of Mackinnon's dam is complicated by the fact that it was undertaken in accordance with the now superseded May 2000 guidelines, compared to the current October 2012 guidelines which determines a Consequence Category rather than a Hazard Category.

It is difficult to make a direct comparison from the May 2000 to the October 2012 guidelines but in effect a dam with a "Significant" Hazard Category is to some degree equivalent to a dam with a High C Consequence Category under the revised guidelines.

### 1.2 Review the impact of the 100AEP design flood

We have reviewed the impact of the new highway on the likely flood depths during the 1:100 AEP design flood for Sheep wash Creek.

Sheep wash creek passes through Perth, and crosses numerous roads / culverts and its general alignment is adjacent to residential properties.

The modelling considered the size and location of both the proposed "frog culvert" and stock underpass as part of the new highway design.

The Frog Culvert is a low-level stormwater culvert which is configured in a manner which allows the passage of native animals under the highway, similar to works which have already been undertaken on Stage 1 of the highway works. The frog culvert is also the primary culvert for passing the base flows within Sheep wash Creek, and is located at IL165.70.

The stock underpass is located at a higher elevation (RL166.25), and has been modelled to ensure no flow passes through this structure up to the 1:100 AEP event. To achieve this a small 250mm bund was modelled around the entrance to the stock underpass. In the field the stock underpass entry will be in cut and hence this additional height is not considered significant.

We assessed a range of frog culvert sizes to determine the impact downstream, with the outflow hydrograph provided in Appendix D. There was not a significant advantage to move away from the current 1200 x 1500 size nominated.

The hydrographs used for this modelling work is attached in Appendix D.

The results of this modelling work indicate that all flows up to the 1:100AEP pass through only the frog culvert, and that the stock underpass is not required. The peak outflow from the Frog Culvert is in the order of 1.6m3/s, for the current design (1500 x 1200).

As a base scenario, in comparing the 1:100-year AEP, it is evident that allowing the frog culvert to act as a hydraulic control the downdraw of the event, the impact downstream is reduced. It must be noted that all downstream culverts have assumed to be upgraded as per the recommendations made in Hydrodynamica's *Stormwater Assessment and Recommendations, for Northern Midlands Council – West Perth*, dated September 2015.

### 1.3 Review the Sunny Day Dam break inundation

#### *1.3.1 Overview*

To review the consequence category and associated dam break inundation area, we have undertaken a dam break assessment considering both the existing conditions (ie no highway) and the proposed highway alignment.

#### *1.3.2 Dam Breach Hydrographs*

A dam breach hydrograph has been developed in accordance with the Froehlich method (CDSB, 2010) using the staged storage curve developed by IPD. Hydrographs have been developed for Sunny Day Failure (SDF) conditions only. All hydrographs are attached as Appendix D.

It is noted that the construction of an additional 200ML storage upstream (Refer Appendix E) of the existing dam has been considered, and as the proposed storage is forded by existing material (rock) and is connected to the existing dam via a new spillway, we have considered this storage is not subject to failure via overtopping or piping failure and therefore for the storage volume is not released instantaneously in the dam break event. The existing dam's spillway is at RL178, while the new storage has a connection channel/spillway at RL177.50.

#### *1.3.3 Inundation*

The dam break inundation has been modelled with results shown in Appendix B.

The new highway provides a benefit to the dam break flooding depths and flood extent.

Typical flood depths are reduced to below 300mm in most cases and velocities are not considered excessive, through the urban zones.

## 2 Review the Dams Consequence Category

#### *2.1.1 Overview*

We have undertaken a review of the Dam's Consequence category and provide the following comments from our review.

#### *2.1.2 Total Infrastructure costs*

It is considered that a SDF dam break would have a MEDIUM impact on downstream infrastructure. Downstream maintenance may be required at major culverts, private access roads and fence lines particularly in locations with high velocities.

It is noteworthy that due to the slight changes in severity definitions for each of the criteria under Total Infrastructure Costs, that they are not 100% comparable. However, the assessment of severity of damages losses conducted by ARM in 2005, given their severity levels applied indicate a MEDIUM severity to Residential and Infrastructure. This indicates that 4 to 49 Residential houses will be damaged or destroyed in an event. Further the event will result in damages of between $1M to $10M (refer *ANCOLD Guidelines on Assessment of the Consequences of Dam Failure* (May 2000)).

#### *2.1.3 Impact on Dam Owners Business*

It is considered that a dam break would have a MINOR impact on the dam owners' business due to their inability to provide water to key stakeholders.

Although not 100% comparable, this was generally in accordance with previous hazard assessment undertaken by ARM in 2005.

#### *2.1.4 Health and Social Impacts*

It is considered that a dam break would have a MINOR impact from a health and social perspective.

Although not 100% comparable, this was generally in accordance with previous hazard assessment undertaken by ARM in 2005.

#### *2.1.5 Environmental Impact*

It is considered that a dam break would have a MINOR impact from an environmental perspective as the discharge will likely be representative of current water quality and any environmental damage (expected to be erosion related) would recover in less than a year.

Although not 100% comparable, this was generally in accordance with previous hazard assessment undertaken by ARM in 2005.

#### *2.1.6 Population at Risk (PAR)*

Based on the inundation maps included, it is anticipated that in the event of a Sunny Day Failure there will be potential risk to human life measured in terms of Population at Risk (PAR) of ≥100 to <1000 in accordance with the ANCOLD *Guidelines on the Consequence Categories for Dams* (2012). It IPD's interpretation of the guidelines PAR "*includes all people who would be directly exposed to flood waters assuming they took no action to evacuate*".

### *2.1.7 Consequence Category Assessment*

Based on the consequence category assessment and the assumptions above, it is recommended a consequence category of **HIGH B** be assigned to Mackinnon's dam going forward. This assessment is regardless of whether works are conducted on the dam or not.

Further to this assessment however, it is noted that the PAR (Population at Risk) assessment was the key criteria which has pushed the dam from a High C to a High B assessment. In reviewing the PAR and the old assessment it appears that the assessment began looking at the related flood depths and flood velocity of the dam break, in determining if a PAR was present. This method is in error in our view if using the PAR methodology, and what is being done is in fact moving into a PLL (Potential Loss of Life) assessment.

With that said, the previous process has merit as the flood depths and velocities are low, and hence we are confident that if a full risk assessment process was undertaken in accordance with the guidelines, then the original HIGH C assessment would still stand.

I would be happy to discuss any aspect of the above. If you require any further information or clarification on any aspect of the above please don't hesitate to contact me on Mob: 0419 574 975 or Email: mwalters@ipdconsulting.com.au

Yours faithfully
IPD Consulting Pty Ltd

**Mark Walters**
Director – Civil Engineer

Doc ref: 1647_Rev C

## Appendix A – Inundation Maps for 1:100AEP

Phillip Street: 1:100 AEP


Edward Street: 1:100 AEP

Doc ref: 1647_Rev C

## Appendix B – Inundation Maps (Dam Break)

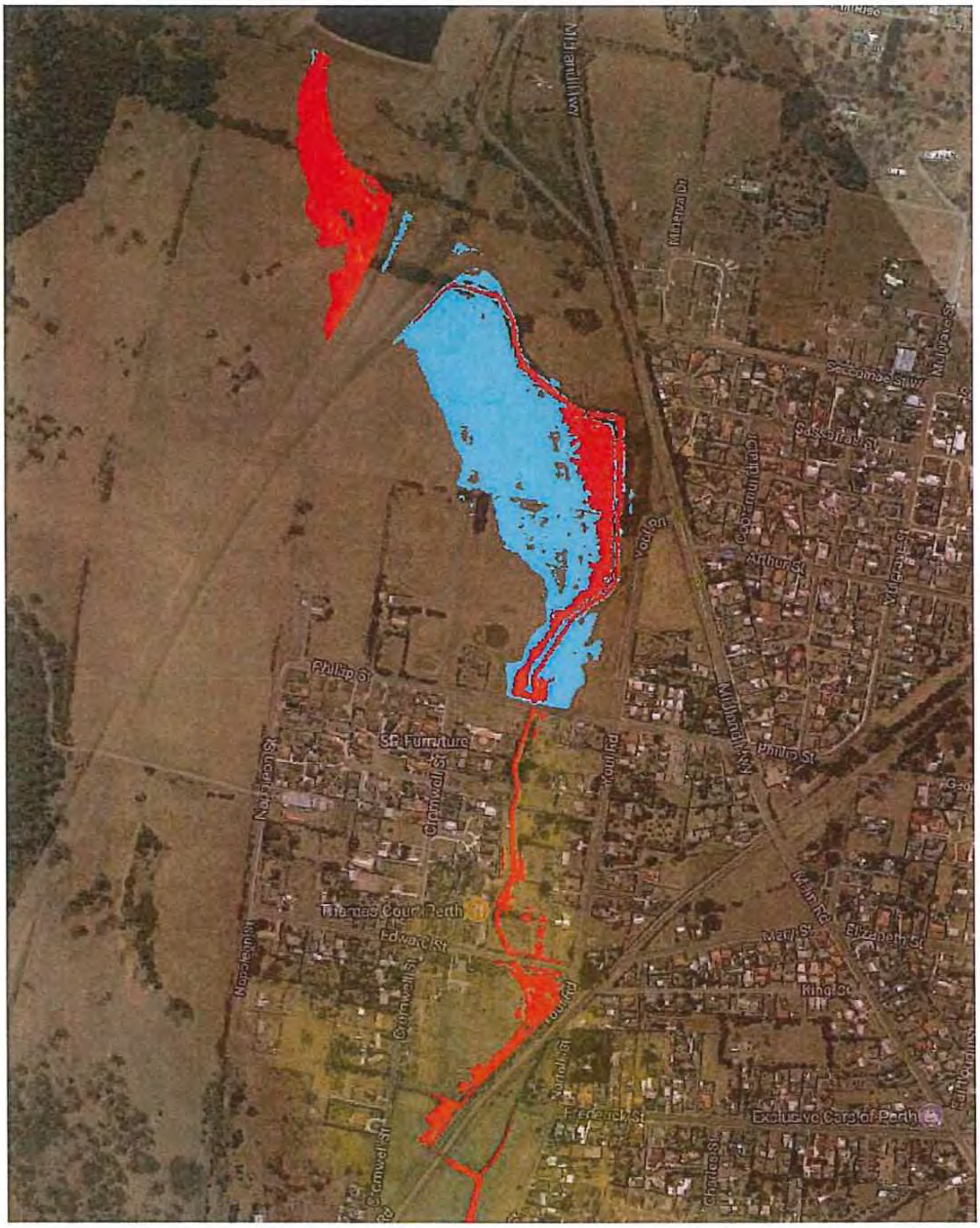


Figure 2 - 1in100 AEP Flood (Blue Without Highway Development, Red With Highway Development)

Doc ref: 1647_Rev C


Figure 3 - SDF of 200ML dam with Highway

Figure 4 - SDF 200ML without Highway

Doc ref: 1647_Rev C

## Appendix C – ANCOLD Consequence Category Assessment

| | |
|---|---|
| **Applicant Name** | Shaw Contracting Pty Ltd |
| **Stream Name** | Sheepwash Creek |
| **Estimated Capacity at FSL** | 200 ML (Existing) |
| **Dam ID. No. (If existing dam)** | 7569 |
| **Dam Height (metres)** | 10.25 M |
| **Location** | 390 Illawara Road, Longford, Tasmania |

| Damage and Loss | Estimate | Severity Level: Minor | Medium | Major | Catastroph |
|---|---|---|---|---|---|
| **B1 TOTAL INFRASTRUCTURE COSTS** | | | | | |
| Residential | $10M-$100M | . | YES | . | . |
| Commercial | <$10M | YES | . | . | . |
| Community Infrastructure | $10M-$100M | . | YES | . | . |
| Dam repair or replacement cost | <$10M | YES | . | . | . |
| ***Total Infrastructure cost severity level*** | **MEDIUM** | | | | |
| **B2 IMPACT ON DAM OWNER'S BUSINESS** | | | | | |
| Importance of the system, need to replace the dai | Restrictions needed during dry periods | YES | . | . | . |
| Effect on services provided by owner | Minor difficulties in replacing services | YES | . | . | . |
| Effect on continuning credibility | Some reaction but short lived | YES | . | . | . |
| Community reaction and political implications | Some reaction but short lived | YES | . | . | . |
| Impact on financial viability | Able to absorb in one financial year | YES | . | . | . |
| Value of water in the storage | Can be absorbed in one financial year | YES | . | . | . |
| ***npact on dam owner' business severity level*** | **MINOR** | | | | |
| **B3 HEALTH AND SOCIAL IMPACTS** | | | | | |
| Human health | <100 people affected | YES | . | . | . |
| Loss of services to the community | <100 people affected | YES | . | . | . |
| Cost of emergency management | <1,000 person days | YES | . | . | . |
| Dislocation of people | <100 person months | YES | . | . | . |
| Dislocation of businesses | <20 business months | YES | . | . | . |
| Employment affected | <100 jobs lost | YES | . | . | . |
| Loss of heritage | Local facility | YES | . | . | . |
| Loss of recreational facility | Local facility | YES | . | . | . |
| ***Health and Social severity level*** | **MINOR** | | | | |
| **B4 ENVIRONMENTAL IMPACTS** | | | | | |
| Area of impact | < 1 km2 | YES | . | . | . |
| Duration of impact | < 1 year | YES | . | . | . |
| Stock and fauna | Discharge from dambreak would not contaminate water supplies used by stock and fauna. | YES | . | . | . |
| Ecosystems | Discharge from dambreak is not expected to impact on ecosystems. Remediation possible. | YES | . | . | . |
| Rare and endangered species | Species exist but minimal damage expected. Recovery within one year. | YES | . | . | . |
| ***Environmental impacts severity level*** | **MINOR** | | | | |
| ***Highest severity level*** | **MEDIUM** | | | | |

**Reasons for recommending a consequence category (refer ANCOLD Guidelines On The Consequence Categories For Dams October 2012) MUST include comments on the PAR (both permanent and itinerant), buildings, roads, other infrastructure and the natural environment downstream of the dam and the potential impacts arising from a dam break:** ***(**Note** Provide photographs to support reasons for recommending consequence category)***

**Population at Risk (PAR)** ≥100 to <1,0( **CONSEQUENCE CATEGORY =** **High B**

PAR includes all those persons who would be directly exposed to flood waters within the dam break affected zone if they took no action to evacuate

**Note 1: With a PAR in excess of 100, it is unlikely damage will be minor, similarly with a PAR in excess of 1,000 it is unlikely damage will be classified as med**

**Note 2: Change to 'High C' where there is a potential of one or more lives being lost**

| | |
|---|---|
| **Completed By** | |
| **Date** | |

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## Appendix D – Model Hydrographs

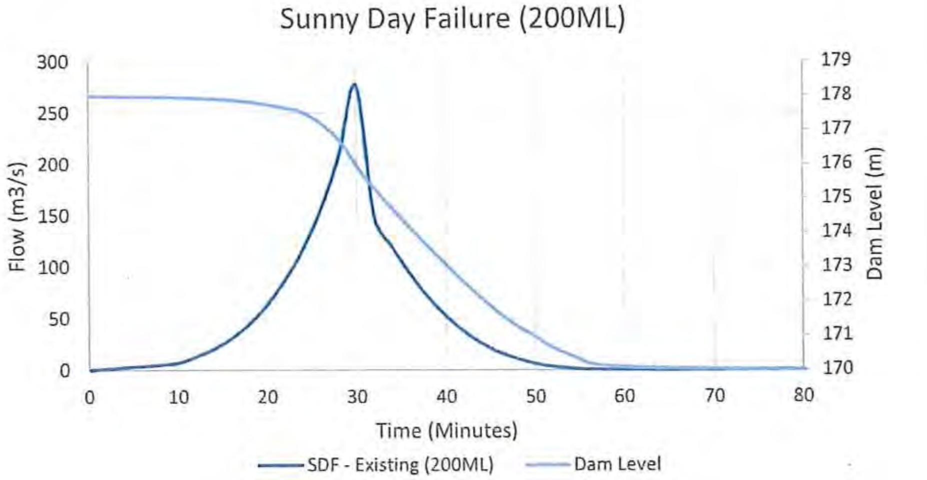


Figure 5 - Sunny Day Failure (SDF), 200ML Dam Breach Hydrograph

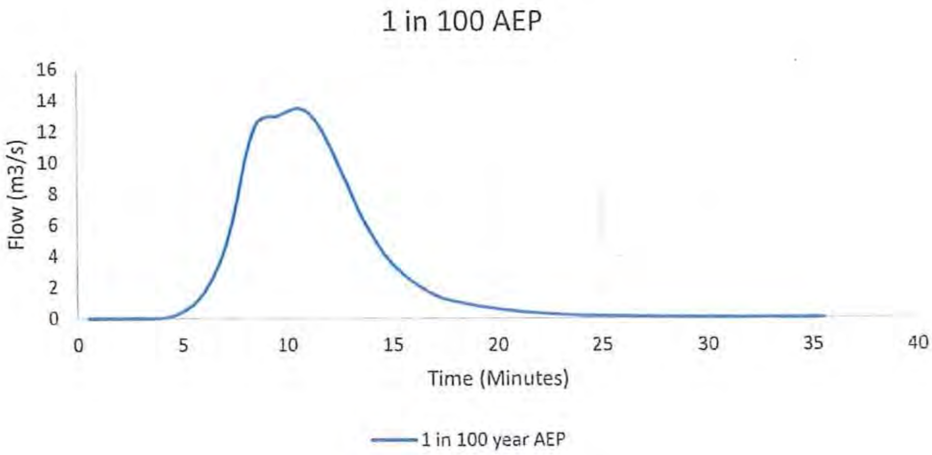


Figure 6 - 1:100 AEP dam flow hydrograph

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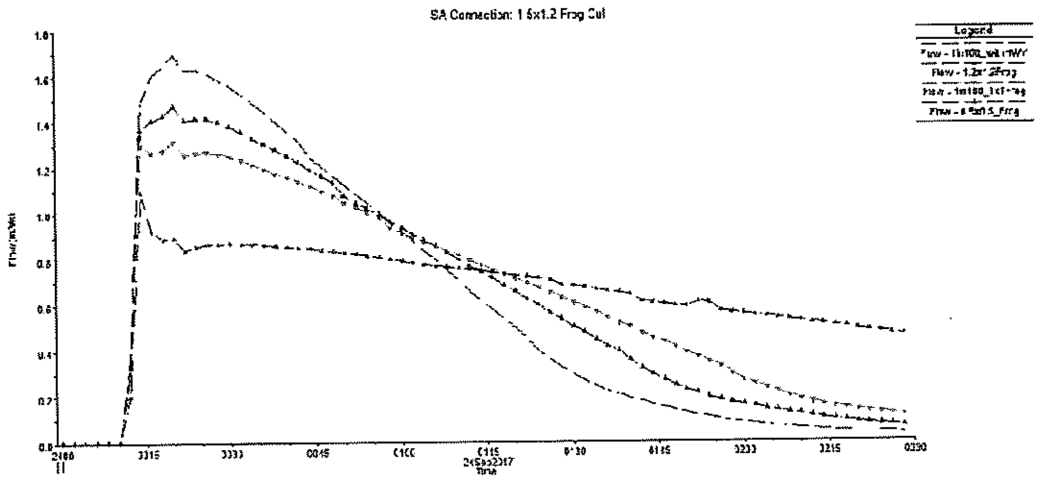
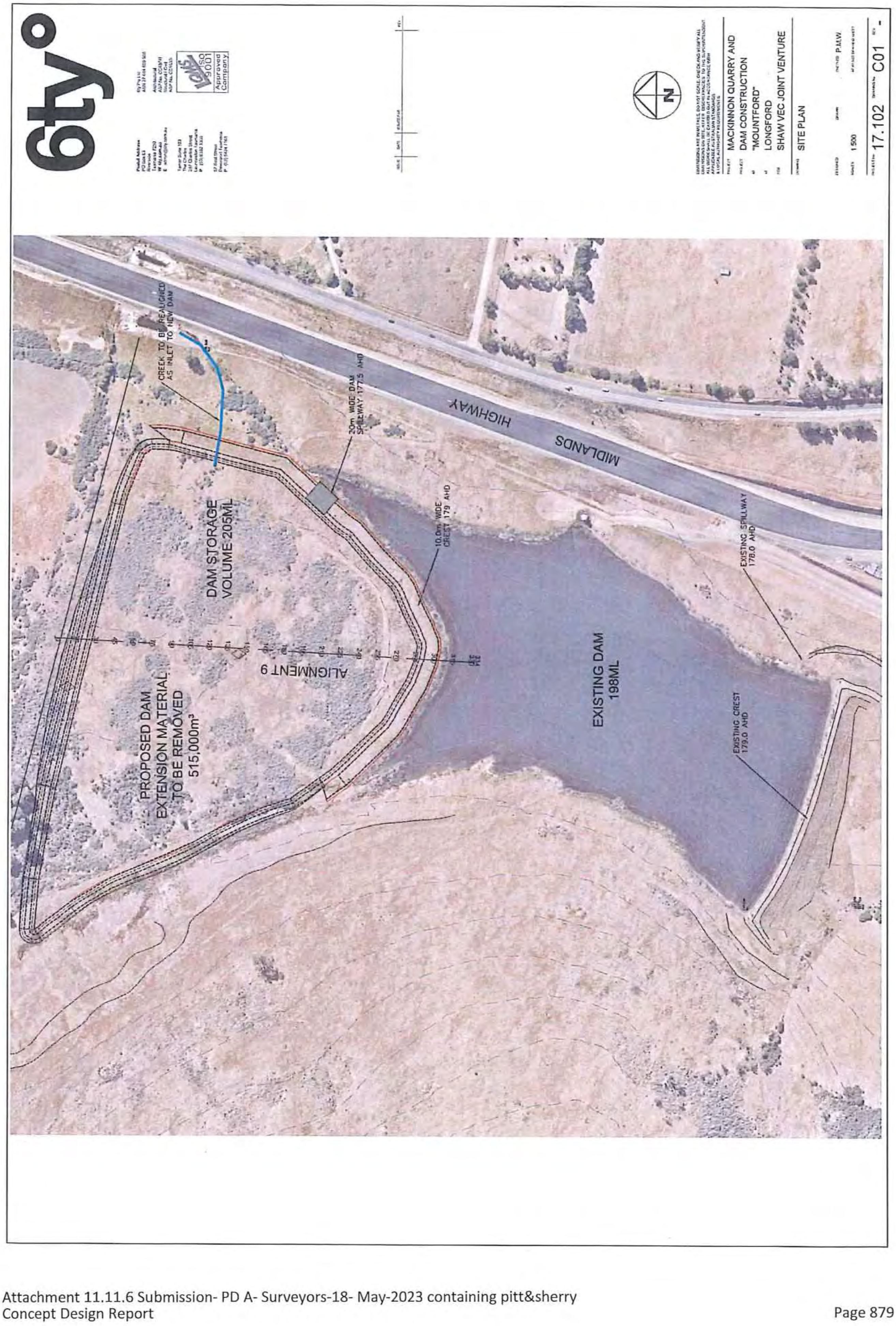
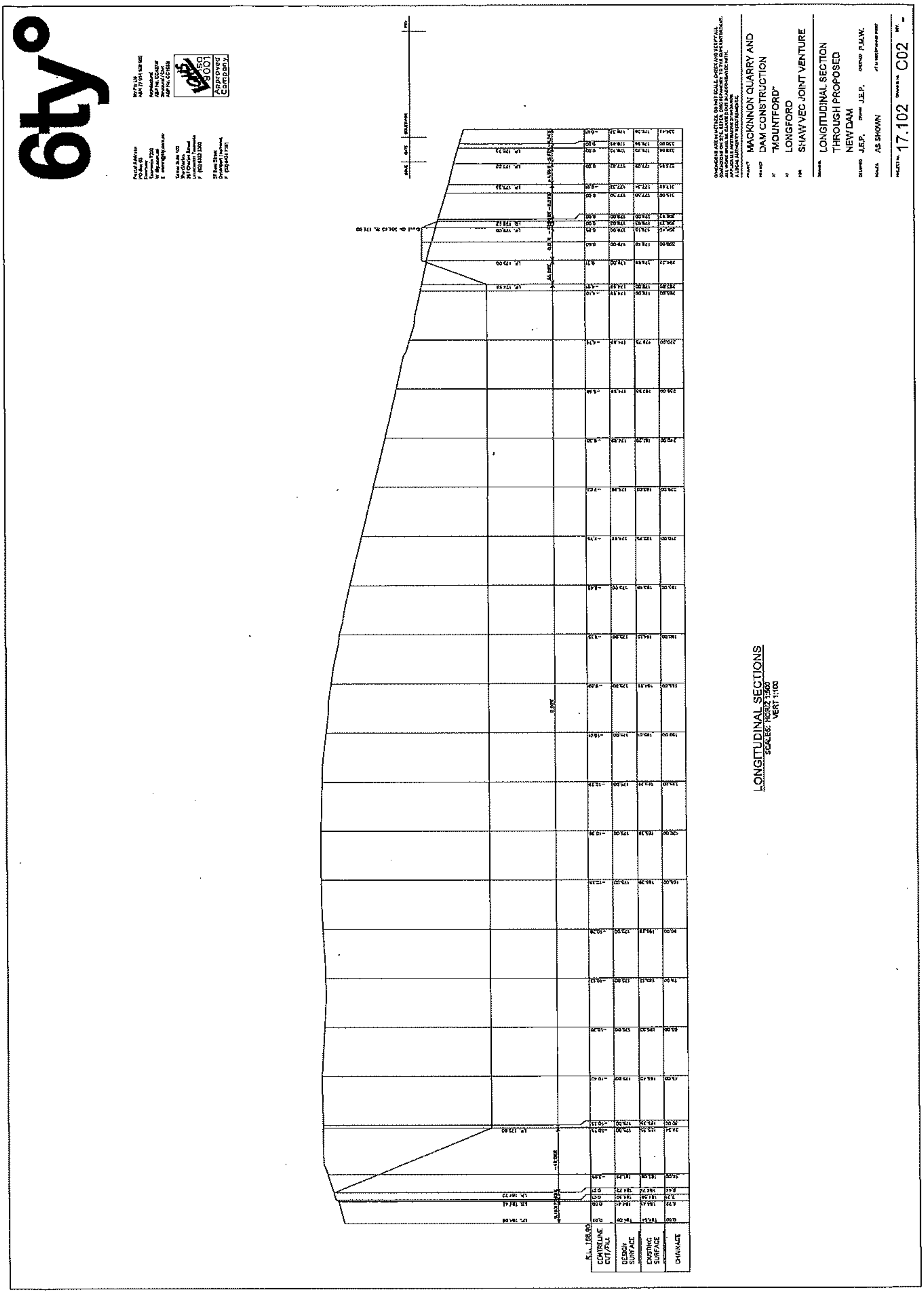



Figure 7 - 1:100 AEP outflow hydrograph from Frog Culvert vs differing sizes

**Appendix E – Proposed new Storage**


6ty°
MACKINNON QUARRY AND
DAM CONSTRUCTION
"MOUNTFORD"
LONGFORD
SHAW VEC JOINT VENTURE
SITE PLAN
1:500
P.M.W
17.102
C01
MIDLANDS
HIGHWAY
CREEK TO BE REALIGNED AS INLET TO NEW DAM
20m WIDE DAM SPILLWAY 177.5 AHD
10.0m WIDE CREST 179 AHD
DAM STORAGE VOLUME 205ML
ALIGNMENT 9
PROPOSED DAM EXTENSION MATERIAL TO BE REMOVED 515,000m³
EXISTING DAM 198ML
EXISTING SPILLWAY 178.0 AHD
EXISTING CREST 179.0 AHD


6ty°
LONGITUDINAL SECTIONS
MACKINNON QUARRY AND DAM CONSTRUCTION
"MOUNTFORD"
LONGFORD
SHAW VEC JOINT VENTURE
LONGITUDINAL SECTION THROUGH PROPOSED NEW DAM
AS SHOWN
17.102
C02
CENTRELINE CUT/FILL
DESIGN SURFACE
EXISTING SURFACE
CHAINAGE

# Appendix D

# Geotechnical Report

# Perth Link Roads Design Report: Geotechnical and Hydrology Report

transport | community | mining | industrial | food & beverage | carbon & energy

| | |
|---|---|
| **Prepared for:** | **VEC SHAW Joint Venture** |
| **Client representative:** | **Owen Cavanough** |
| **Date:** | **13 November 2017**<br>**Rev00** |

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|---|---|---|---|
| Prepared by: | ..................................................... Dejin Hu | Date: | 13 November 2017 |
| Reviewed by: | ..................................................... Daiquan Yang | Date: | 13 November 2017 |
| Authorised by: | ..................................................... Andrew Turner | Date: | 13 November 2017 |

**Revision History**

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|---|---|---|---|---|---|
| 00 | Geotechnical and Hydrology Report | D. Hu | D. Yang | A. Turner | 13/11/2017 |
| | | | | | |



**pitt&sherry** ref: HB17353D012 REP 31P REV00/DH/bc

# Definitions and Abbreviations

| Term | Description |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| CASS | Coastal Acid Sulfate Soils |
| CBR | California Bearing Ratio |
| EDR | Electronic Data Room |
| FoS | Factor of Safety |
| ICP | Initial Capital Projects |
| RE | Reinforced Earth |
| RFP | Request for Proposal |
| RW | Retaining Wall |
| SCI | Site Conditions Information |
| SiD | Safety in Design |
| SLS | Serviceability Limit State |
| ULS | Ultimate Limit State |
| | |
| | |

# 1. Introduction

This report was prepared for VEC SHAW Joint Venture by **pitt&sherry** for tender design and submission of proposed Perth Link Roads project based on relevant reports and information provided in the tender documents, which are listed in Section 1.1below.

The report summarises and presents the geotechnical and hydrological interpretations, assessments and geotechnical design advice for bridge foundation and retaining structure and pavement design.

The proposed structures included in the project consist of the followings:

- Northern Interchange – Underpass
- Western Interchange – Westbound Underpass
- Western Interchange – Southbound Underpass
- Western Interchange – Westbound Overpass
- Western Interchange – Railway Underpass No.1
- Western Interchange – Railway Underpass No.2
- Northern Culvert (MC00 - CH3800)
- Northern Culvert (MC00 - CH4100)
- Southern Culvert (MC00 - CH1465)
- Southern Culvert (MC00 - CH1480).

## 1.1 Reference Material

### 1.1.1 Geotechnical and Hydrogeological Investigations

The Department of State Growth commissioned GHD to conduct a geotechnical investigation for this project in May 2017. The geotechnical investigations generally consisted of test pits and bores, with in-situ and laboratory tests. Dynamic Cone Penetration (DCP) tests were normally undertaken adjacent to test pits. Standard Penetration Test(SPT) were conducted at regular depth intervals within the soil profile during borehole drilling. The laboratory tests included:

- California Bearing Ratios (CBRs)
- Atterberg limits tests
- Soil Gradings
- Moisture content tests
- Standard compaction tests
- Point load test (PLT) on rock core samples.

The results of these investigations were included in the Preliminary Geotechnical Investigation Factual Report (GI), which was provided with the tender documents.

There was no standpipe installation presented in the GI report.

### 1.1.2 Standards and Specifications

The following resources were also referenced for bridge foundation, culvert foundation, earth retaining wall design:

- *AS 1170.4-2007 Structure design actions – Earthquake actions in Australia*
- *AS 1597.2-2013 Precast reinforced concrete box culvert*
- *AS 2159-2009 Piling – Design and installation*
- *AS 4678-2002 Earth-retaining structures*
- *AS 5100.3-2004 Bridge design Part 3: Foundation and soil-support structures*
- *American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications (2007–2010)*
- *VicRoads Bridge Technical Note: BTN1999/010* and other Bridge Technical Notes as applicable.

## 1.2 Structure of Report

Section 2 of this report describes geotechnical and hydrological ground condition assessment and design criteria generally applied for design of bridge foundations, culvert foundations and retaining walls (RWs) of this project and associated design methodologies.

Section 3 summarises and presents specific geotechnical models and design parameters, design results and construction recommendations.

# 2. Geotechnical and Hydrological Ground Condition Assessment

This section summarises the geotechnical and hydrological site conditions encountered during geotechnical investigation.

## 2.1 Surface Geology

A review of the geological map sheet 5039, scale 1:25 000, Longford, Geological Survey of Tasmania (2010) has indicated the following surface geological units in the project area:

- QW: Quaternary aeolian deposits
- TQaF: Tertiary undifferentiated siliceous pebble gravels, and
- Jd: Jurassic Dolerite.

## 2.2 Geotechnical Site Conditions

The ground profile encountered during the geotechnical investigations was generally consistent with the geological maps descriptions.

The ground conditions for each structure location is summarised in the following sections for different foundation design.

### 2.2.1 Subgrade Assessments

The soil laboratory test results and in-situ DCP and SPT test results included in GI factual report have been used to assess the subgrade conditions for pavement design.

Subgrade CBRs within and across site ranges widely from 1% to 17%, though the majority of test results lie in the range from 4% to 17%. There were significant variations in the test results because the tested materials encountered within subgrade profiles include a wide range of mixed materials from clay to sand to gravel. There are four lab test results that indicated the natural materials have low CBR (1% to 2.5%) and high swelling index (4.5% - 6%). Two of them were from embankment foundation, and two of them were from where 2.0m below the proposed road surface. Those test results are located within the southern part of the project, i.e. from BH04 (south abutment of Westbound Ramp Underpass Bridge). If these materials are proposed to be used as subgrade, then they require subgrade treatment measures.

The samples tested in selected test locations and depths may not fully represent the typical natural subgrade materials.

The natural subgrade materials encountered during the geotechnical investigations included clay, sand, gravel and weathered rock materials.

The bedrock encountered in the project area included Dolerite in the northern section of the project and Basalt rock in the southern section of the road alignments. The rock depth varies from 0.7m and 13.0m below the existing ground surface. The road pavement may found onto weathered Dolerite between chainage 2740 and 3240 (MC00).

## 2.3 Hydrological Site Conditions

There was no standpipe installation reported in the GI factual report. The review of online groundwater database (http://wrt.tas.gov.au/ groundwater-info) indicated 2 historic groundwater bores (ID3794 and ID3855). The static water levels detected in the bores are generally less than 5m below the ground level. The groundwater levels may vary seasonally and be affected by nearby waterways.

## 2.4 Design Requirements

### 2.4.1 Bridge Foundations

Design requirements for bridge foundations are provided in:

- The standards and specifications listed in Section 2.5.1
- Midland Highway Perth Link Roads, Request for Tender, No. 2400, Section 3050
- The GI factual report
- The Principal's Reference Design and its road design package, as provided by the Department of State Growth.

### 2.4.2 Culvert Foundations

Design requirements for culvert foundations are provided in:

- Midland Highway Perth Link Roads, Request for Tender, No. 2400, Section 3050.

### 2.4.3 Retaining Walls

Design requirements for retaining walls are provided in:

- Midland Highway Perth Link Roads, Request for Tender, No. 2400, Section 3050.

## 2.5 Design Criteria

General design criteria are summarised in this section.

### 2.5.1 Bridge Foundations

The bridge foundation design criteria have been adopted in general accordance with the Australian Bridge Design Standard AS 5100.3, the recommended design guides referred to within AS 5100.3 and State Growth Standard Specifications.

Based on RFT No.2400, Section 3050 and discussions with the structures design team, these design criteria have been adopted:

- Maximum allowable vertical settlements of 20 mm for major structures
- Allowable horizontal displacements of maximum rotation of H/150 for the upper structures.

Driven piles have been adopted for all bridge foundations except the northern abutment of the Northern Underpass due to construction constraints, the lateral capacity required and the ground conditions. Pile foundations were designed in accordance with AS 2159-2009. A pad foundation has been adopted for the northern abutment of the Northern Underpass due to the presence of high level rock.

### 2.5.2 Culvert Foundations

The culvert foundation design criteria have been adopted in general accordance with:

- AS 5100.3
- The recommended design guides referred to within AS 5100.3
- State Growth Standard Specifications.

Maximum allowable vertical settlements of 20 mm for all minor structures and maximum differential settlement of 10 mm have been adopted in compliance with RFT No. 2400, Section 3050.

### 2.5.3 Retaining Walls

Retaining wall design criteria have been adopted in general accordance with:

- RFT No. 2400, Section 3050
- *AS 1170.4-2007 Structure design actions – Earthquake actions in Australia*
- *AS 4678-2002 Earth-retaining structures*
- *AS 5100 – Bridge Design*
- *VicRoads Bridge Technical Notes.*
- Maximum allowable vertical settlements of 20 mm for all minor structures and maximum differential settlement of 10 mm have been adopted in compliance with RFT NO. 2400, Section 3050.

#### *Load Combinations*

In accordance with AS 1170.4-2007 and AS 4678-2002, each earth retaining member has been designed for the more severe of the following two cases:

1. Static earth pressure plus live load surcharge
2. Earthquake earth pressure.

## 2.6 Design Methodology

### 2.6.1 Bridges

The proposed bridge foundations consist of a driven steel pile solution with pile groups for the abutments.

Driven piles were designed in accordance with AS 2159–2009 to support the axial, lateral and torsional loads applied by the bridge. Bored pile or pad foundation may be adopted for Northern Underpass bridge northern abutment foundation. A geotechnical reduction factor of $\phi_g$=0.64 was adopted for driven pile and $\phi_g$=0.48 for bored pile.

The contribution of the top 1 m, whichever is greater, of soil was ignored under ultimate conditions. The geotechnical design has considered variable ground conditions with/without socketed length into weathered rock where it presents at shallow depth.

Serviceability Limit State (SLS) and Ultimate Limit State (ULS) design loads including axial, lateral and moment loadings from the superstructures for individual piles were provided by **pitt&sherry**'s Bridge Engineers to assess foundation reactions using RSPile 1.0 software. These loads will be re-assessed and revised during detailed design. The influence of the pile group interaction and pile cap will also need to be re-assessed during detailed design.

### 2.6.2 Safety in Design

Safety in Design (SiD) analyses, particularly for construction safety, have been integral to the design process during Concept (RFP Phase) development. The main site-specific geotechnical hazard for bridge abutments and foundations are associated with the complicated subsoils and rock conditions and their behaviour during construction. The variations of rock strength properties have been considered in the design process. During construction, the Contractor will need to appoint a Geotechnical Engineer to oversee critical construction areas and ensure construction works are carried out in accordance with the design intent. Work Method Statements will need to document the construction sequences and constraints such as allowed equipment.

Safety in construction considerations for retaining wall design include:

- For benching into existing slopes, excavation will need to be carried out in a controlled manner to minimise disturbance to the existing slope. Benching will need to be monitored by a Geotechnical Engineer or representative
- For any excavation, temporary cover may be required during winter works or periods where significant rainfall is predicted. Periodic inspections will need to be carried out during excavation by an experienced Geotechnical Engineer or representative
- Movements of reinforced earth (RE) walls, reinforced slopes and affected slopes/grounds/embankments will need to be monitored during construction with pre-installed instruments as specified in the geotechnical design. Monitoring will be conducted at specified frequencies for each type of instrument, and the monitoring data will need to be made available to the Geotechnical Engineer for assessment and interpretation
- The concrete panel facing for RE walls and additional temporary hand rails will need to be installed ahead of the wall construction to act as fall protection
- Handrails and other fall protection measures is required for the construction of reinforced slopes.

# 3. Geotechnical Design Assessments and Recommendations

## 3.1 Bridge Foundation

This Project comprises the construction of five new bridges as follows:

- Western Interchange – Railway Underpass No.1
- Western Interchange – Railway Underpass No.2
- Western Interchange – Southbound Underpass
- Western Interchange – Westbound Underpass
- Western Interchange – Westbound Overpass
- Northern Interchange – Underpass.

### 3.1.1 Railway Underpass No. 1

Railway Bridge No 1 over existing railway requires construction of an approximately 23.4 m long and 34.6 m wide bridge structure, which includes single span supported by abutment foundations.

*Geotechnical Conditions*

A geotechnical investigation for the construction of Rail Bridge No.1 was completed by GHD in 2017 which was used in foundation design.

The soil and rock profiles were interpreted based on boreholes available in this region including those data provided in the GI factual report for this project.

The design profiles at the bridge foundation locations listed in Table 1 and Table 2 were developed based on borehole data from BH10 and BH12, drilled at the bridge abutments.

The general ground profiles at the bridge location consist of residual soil overlying weathered Basalt rock. During drilling, bedrock was encountered along the bridge alignment at approximately RL154.4m. It appears the rock surface levels are generally consistent at both abutments of this bridge.

Table 1: Geotechnical conditions for north abutment

| RL (m) | Description | $\gamma$ (kN/m$^3$) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 159.6 | Topsoil | 18 | - | - | - |
| 159.3 | Clay (CH) (St-VSt) | 19 | 80-150 | - | 24-45 |
| 154.4 | Basalt (HW-MW) | 24 | | 2.5-24 | 250-2400 |

Note: $E_h$ – horizontal elastic modulus = $0.75E_v$ (vertical elastic modulus)

Table 2: Geotechnical conditions for south abutment

| RL (m) | Description | $\gamma$ (kN/m$^3$) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 159.9 | Topsoil | 18 | - | - | - |
| 159.6 | Sandy Clay (CH) (St-H) | 19 | 120-250 | - | 36-75 |
| 154.4 | Basalt (HW-MW) | 24 | | 2.5-72 | 250-7200 |

Note: $E_h$ – horizontal elastic modulus = $0.75E_v$ (vertical elastic modulus)

### *Foundation Design Results*

A summary of the driven pile embedment for 310UC158 steel columns, pile vertical capacities under ultimate loads and the estimated vertical settlement of the pile top at serviceability loads are listed in Table 3 for abutment foundations. The pile design has mainly considered the vertical loads. Due to the integral bridge configuration, the pile horizontal deflections are expected to be minimal.

Table 3: Pile design output – Rail Bridge No.1

| Foundation Location | Pile Dimensions | Estimated Pile Embedment Length (mbgl*) | Vertical Capacity (ULS) (kN/Pile) | Estimated Settlement (SLS) (mm) |
|---|---|---|---|---|
| North abutment | 310UC158 | 10.0 | 2200 | <10 |
| South abutment | 310UC158 | 8.0 | 2000 | <10 |

* mbgl – metre below ground level

## 3.1.2 Railway Underpass No. 2

### *Geotechnical Conditions*

The design profiles at the bridge foundation locations listed in Table 4 and Table 5 were developed based on borehole data from BH11 and BH13, drilled at the bridge abutments.

The general ground profiles at the bridge location consist of residual soil overlying weathered Basalt rock. During drilling, bedrock was encountered along the bridge alignment between RL152.1m and RL153.6m.

It appears the rock surface levels are slightly deeper on the south abutment of this bridge.

Table 4: Geotechnical conditions for north abutment

| RL (m) | Description | $\gamma$ (kN/m$^3$) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 159.9 | Topsoil | 18 | - | - | - |
| 159.2 | Sandy Clay (CH) / Sandy Silt (St-H) | 19 | 100-200 | - | 30-60 |
| 153.6 | Basalt (XW-MW) | 24 | | 1.0-7.2 | 100-720 |

Note: $E_h$ – horizontal elastic modulus = 0.75$E_v$ (vertical elastic modulus)

Table 5: Geotechnical conditions for south abutment

| RL (m) | Description | $\gamma$ (kN/m$^3$) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 160.5 | Topsoil/silty Sand | 18 | - | - | - |
| 159.6 | Sandy Clay (CH) (St-H) | 19 | 60-200 | - | 18-60 |
| 152.1 | Basalt (XW-MW) | 24 | | 1.0-7.2 | 100-720 |

Note: $E_h$ – horizontal elastic modulus = 0.75$E_v$ (vertical elastic modulus)

### *Foundation Design Results*

A summary of the driven pile embedment for 310UC158 steel columns, pile vertical capacities under ultimate loads and the estimated vertical settlement of the pile top at serviceability loads are listed in Table 6 for abutment foundations. The pile design has mainly considered the vertical loads. Due to the integral bridge configuration, the pile horizontal deflections are expected to be minimal.

Table 6: Pile design output – Rail Bridge No 2

| Foundation Location | Pile Dimensions | Estimated Pile Embedment Length (mbgl*) | Vertical Capacity (ULS) (kN/Pile) | Estimated Settlement (SLS) (mm) |
|---|---|---|---|---|
| North abutment | 310UC158 | 11.0 | 2100 | <10 |
| South abutment | 310UC158 | 13.0 | 2200 | <10 |

* mbgl – metre below ground level

## 3.1.3 Southbound Ramp Underpass Bridge

### *Geotechnical Conditions*

The design profiles at the bridge foundation locations listed in Table 7 and Table 8 were developed based on borehole data from BH05 and BH06, drilled at the bridge abutments.

The general ground profiles at the bridge location consist of residual soil overlying weathered Basalt rock. During drilling, bedrock was encountered along the bridge alignment between RL150.2m and RL146.5m. It appears the rock surface levels are much deeper on the south abutment than north abutment of this bridge.

Table 7: Geotechnical conditions for north abutment

| RL (m) | Description | $\gamma$ (kN/m$^3$) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 160.5 | Topsoil | 18 | - | - | - |
| 160.1 | Sandy Clay (CH) / Sand(SP) (St-VSt) | 19 | 60-120 | - | 18-36 |
| 150.2 | Basalt (XW-SW) | 24 | | 1.0-24 | 100-2400 |

Note: $E_h$ – horizontal elastic modulus = 0.75$E_v$ (vertical elastic modulus)

Table 8: Geotechnical conditions for south abutment

| RL (m) | Description | $\gamma$ (kN/m$^3$) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 160.4 | Topsoil / Sandy silt | 18 | - | - | - |
| 159.9 | Sandy Clay (CH) / Sandy Silt (ML) (St-H) | 19 | 100-250 | - | 30-75 |
| 146.5.1 | Basalt (XW-SW) | 24 | | 1.0-72 | 100-7200 |

Note: $E_h$ – horizontal elastic modulus = 0.75$E_v$ (vertical elastic modulus)

### *Foundation Design Results*

A summary of the driven pile embedment for 310UC158 steel columns, pile vertical capacities under ultimate loads and the estimated vertical settlement of the pile top at serviceability loads are listed in Table 9 for abutment foundations. The pile design has mainly considered the vertical loads. Due to the integral bridge configuration, the pile horizontal deflections are expected to be minimal.

Table 9: Pile design output – Southbound Ramp Underpass Bridge

| Foundation Location | Pile Dimensions | Estimated Pile Embedment Length (mbgl*) | Vertical Capacity (ULS) (kN/Pile) | Estimated Settlement (SLS) (mm) |
|---|---|---|---|---|
| North abutment | 310UC158 | 14 | 2300 | <10 |
| South abutment | 310UC158 | 14.5 | 2500 | <10 |

* mbgl – metre below ground level

## 3.1.4 Westbound Ramp Underpass Bridge

Westbound Ramp Underpass Bridge requires construction of an approximately 19.0m long and 53.5m wide bridge structure, which includes a single span supported by abutment foundations.

### *Geotechnical Conditions*

The design profiles at the bridge foundation locations listed in Table 10 and Table 11 were developed based on borehole data from BH03 and BH04, drilled at the bridge abutments.

The general ground profiles at the bridge location consist of upper soil layers overlying weathered Dolerite rock. During drilling, bedrock was encountered along the bridge alignment at between RL154.1m and RL153.2m. It appears the rock surface levels are generally consistent at the abutment locations of this bridge.

Table 10: Geotechnical conditions for north abutment

| RL (m) | Description | $\gamma$ (kN/m$^3$) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 165.9 | Topsoil/Silty Sand (MD) | 18 | - | - | - |
| 164.1 | Sandy Clay (CH) (F-Vst) | 19 | 60-120 | - | 18-36 |
| 158.8 | Sandy Silt (ML) (VSt-H) | 24 | 100-200 | - | 30-60 |
| 154.1 | Dolerite (XW-MW) | | - | 1.0-24 | 100-2400 |

Note: $E_h$ – horizontal elastic modulus = 0.75$E_v$ (vertical elastic modulus)

Table 11: Geotechnical conditions for south abutment

| RL (m) | Description | $\gamma$ (kN/m³) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 165.0 | Silty Sand(L-MD) | 18 | - | - | - |
| 162.5 | Sandy Clay (CH) (St-Vst) | 19 | 50-120 | - | 15-36 |
| 156.1 | Sandy Silt (ML) (VSt-H) | 20 | 100-200 | - | 30-60 |
| 153.2 | Dolerite (XW-SW) | 24 | - | 1.0-7.2 | 100-720 |

Note: $E_h$ – horizontal elastic modulus = $0.75E_v$ (vertical elastic modulus)

### *Foundation Design Results*

A summary of the driven pile embedment for 310UC158 steel columns, pile vertical capacities under ultimate loads and the estimated vertical settlement of the pile top at serviceability loads are listed in Table 12 for abutment foundations. The pile design has mainly considered the vertical loads. Due to the integral bridge configuration, the pile horizontal deflections are expected to be minimal.

Table 12: Pile design output – Westbound Ramp Underpass Bridge

| Foundation Location | Pile Dimensions | Estimated Pile Embedment Length (mbgl*) | Vertical Capacity (ULS) (kN/Pile) | Estimated Settlement (SLS) (mm) |
|---|---|---|---|---|
| North abutment | 310UC158 | 13.0 | 1800 | <10 |
| South abutment | 310UC158 | 14.0 | 2000 | <10 |

* mbgl – metre below ground level

## 3.1.5 Westbound Ramp Overpass Bridge

Westbound Ramp Overpass Bridge requires construction of an approximately 19.0m long and 53.5m wide bridge structure, which includes a single span supported by abutment foundations.

### *Geotechnical Conditions*

The design profiles at the bridge foundation locations listed in Table 13 and Table 14 were developed based on borehole data from BH08 and BH09, drilled at the bridge abutments.

The general ground profiles at the bridge location consist of upper soil layers overlying weathered Basalt rock. A layer of weathered Dolerite was encountered in BH09 at a depth between 10.90m and 12.20m. During drilling, bedrock was encountered along the bridge alignment at between RL148.7m and RL150.3m. It appears the rock surface levels are generally consistent at the abutment locations of this bridge.

Table 13: Geotechnical conditions for north abutment

| RL (m) | Description | $\gamma$ (kN/m³) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 161.2 | Topsoil/Silty Sand (L) | 18 | - | - | - |
| 160.2 | Sandy Clay (CH) (St-Vst) | 19 | 80-120 | - | 24-36 |
| 155.7 | Sandy Silt (ML) (VSt-H) | 19 | 100-200 | - | 30-60 |
| 150.3 | Dolerite (XW-SW) | 24 | - | 1.0-72 | 100-7200 |
| 149.0 | Basalt(RS-SW) | 20-24 | - | 0.5-72 | 50-7200 |

Note: $E_h$ – horizontal elastic modulus = $0.75E_v$ (vertical elastic modulus)

Table 14: Geotechnical conditions for south abutment

| RL (m) | Description | $\gamma$ (kN/m³) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 160.4 | Topsoil | 18 | - | - | - |
| 160.0 | Sandy Clay (CH) (St-Vst) | 19 | 80-120 | - | 24-36 |
| 154.4 | Sandy Silt (ML) (St-H) | 20 | 60-200 | - | 18-60 |
| 148.7 | Basalt(XW-SW) | 24 | - | 1.0-72 | 100-7200 |

Note: $E_h$ – horizontal elastic modulus = $0.75E_v$ (vertical elastic modulus)

### *Foundation Design Results*

A summary of the driven pile embedment for 310UC158 steel columns, pile vertical capacities under ultimate loads and the estimated vertical settlement of the pile top at serviceability loads are listed in Table 15 for abutment foundations. The pile design has mainly considered the vertical loads. Due to the integral bridge configuration, the pile horizontal deflections are expected to be minimal.

Table 15: Pile design output – Westbound Ramp Overpass Bridge

| Foundation Location | Pile Dimensions | Estimated Pile Embedment Length (mbgl*) | Vertical Capacity (ULS) (kN/Pile) | Estimated Settlement (SLS) (mm) |
|---|---|---|---|---|
| North abutment | 310UC158 | 11.0 | 1600 | <10 |
| South abutment | 310UC158 | 14.0 | 1400 | <10 |

* mbgl – metre below ground level

## 3.1.6 Northern Underpass Bridge

Northern Underpass Bridge requires construction of an approximately 21.6m long and 22.5 m wide bridge structure, which includes a single span supported by abutment foundations.

### Geotechnical Conditions

The design profiles at the bridge foundation locations listed in Table 16 and Table 17 were developed based on borehole data from BH01 and BH02, drilled at the bridge abutments.

The general ground profiles at the bridge location consist of upper soil layers overlying weathered Dolerite rock. During drilling, bedrock was encountered along the bridge alignment at between RL158.8m and RL166.6m. It appears the rock surface levels are much deeper at the southern abutment than northern abutment of this bridge.

Table 16: Geotechnical conditions for north abutment

| RL (m) | Description | $\gamma$ (kN/m$^3$) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 167.4 | Topsoil | 18 | - | - | - |
| 167.2 | Sandy Clay (CH)(St) | 19 | 50 | - | 15 |
| 166.6 | Dolerite (HW-SW) | 24 | - | 1.0-72 | 100-7200 |

Note: $E_h$ – horizontal elastic modulus = 0.75$E_v$ (vertical elastic modulus)

Table 17: Geotechnical conditions for south abutment

| RL (m) | Description | $\gamma$ (kN/m$^3$) | $S_u$ (kPa) | UCS(MPa) | $E_v$ (MPa) |
|---|---|---|---|---|---|
| 165.9 | Topsoil | 18 | - | - | - |
| 165.4 | Dolerite (HW-SW) | 19 | 60-200 | - | 18-60 |
| 158.8 | Sandy Clay (CH) (St) | 24 | - | 1.0-72 | 100-7200 |

Note: $E_h$ – horizontal elastic modulus = 0.75$E_v$ (vertical elastic modulus)

### Foundation Design Results

Due to the high strength Dolerite encountered in BH02 at a depth of 0.8m below existing ground surface, pad footing has been adopted for northern abutment foundation. A summary of the proposed foundation details is presented in Table 18. For the southern abutment, the pile embedment for 310UC158 steel columns, pile vertical capacities under ultimate loads and the estimated vertical settlement of the pile top at serviceability loads are listed in Table 18 for abutment foundations. The pile design has mainly considered the vertical loads. Due to the integral bridge configuration, the pile horizontal deflections are expected to be minimal.

Table 18: Pile design output – Northern Underpass Bridge

| Foundation Location | Pile Dimensions | Estimated Pile Embedment Length (mbgl*) | Vertical Capacity (ULS) (kN/Pile) | Estimated Settlement (SLS) (mm) |
|---|---|---|---|---|
| North abutment | Pad footing | 2.6m (founding depth) | 3MPa (allowable bearing capacity) | <10 |
| South abutment | 310UC158 (driven pile) | 9.0 | 1800 | <10 |

* mbgl – metre below ground level

### *Construction Recommendations*

The above designs have been carried out based on an assumption that the piles will be driven into weathered rock materials. The development of the pile capacities should be monitored and confirmed during pile driving. Based on knowledge of the local geological conditions and published information, the rock surface may vary significantly from the above discussed design assumptions. Allowance should be made for additional pile length to be found into weathered rock and achieve the design capacities. Pile driveability should be assessed including selection of appropriate driving equipment prior to construction.

To minimise load transfer between piled foundation and RE wall abutments it is recommended that permanent sleeves are installed around piles. Sleeve annulus should be chosen such that lateral displacement (including shrinkage in integrated pier structures) will not result in pile-sleeve contact.

## 3.2 Culvert Foundation

This Project comprises the construction of four new culverts as follows:

- Southern Culvert (MC00 - CH1465)
- Southern Culvert (MC00 - CH1480)
- Northern Culvert (MC00 - CH3800)
- Northern Culvert (MC00 - CH4100)

### *Geotechnical Conditions*

There was only one test pit excavated near each culvert location during the geotechnical investigation completed by GHD in 2016. The ground profile encountered in the referenced test pit at each culvert location has been adopted for respective foundation design.

### *Allowable Bearing Capacity*

A summary of the founding depth/materials and ground improvement requirements are presented in Table 19 for the culvert foundations.

Table 19: Allowable bearing capacity and founding materials

| Location | Structure dimensions | Foundation materials/Depth | Ground improvement | Reference test holes |
|---|---|---|---|---|
| MC00 CH1465 | 3600x3600 | F-St Clay/1.2m | Compacted granular fill to a depth of 1.2m below existing ground surface | TP07 |
| MC00 CH1470 | 1500x1200 | F-St Clay/0.8m | Compacted granular fill to a depth of 0.8m below existing ground surface | TP07 |
| MC00 CH3955 | 3600x3000 | VSt Clay/0.3m | Compacted granular fill to a depth of 0.3m below existing ground surface | TP14 |
| MC00 CH4100 | 1500x1200 | VSt Clay/0.5m | Compacted granular fill to a depth of 0.5m below existing ground surface | TP14 |

### *Settlement*

The estimated settlement for the culvert structure is likely to be less than **20mm** after ground improvement.

### *Design Considerations*

It was necessary to incorporate the following components into the design:

- The culvert foundation should be founded into competent materials to minimise differential settlement under traffic loads.
- The footing slab should be designed to achieve buoyancy stability for full hydrostatic conditions.

## 3.3 Retaining Walls

The overall and external stabilities and bearing capacity of the reinforced earth (RE) walls have been assessed during development of the tender design.

The RE walls would need to be designed by a specialist designer from **pitt&sherry** during the detailed design in terms of internal stability, straps and non-structural panel face panels.

### *Geotechnical Conditions*

The ground conditions for bridge abutment RE wall are presented in Table 21 for each bridge.

### *Design Loading and Stability Checks*

Table 20: Design loading and stability checks

| | |
|---|---|
| **Design Loading** | The loading combination has considered a load factor between 1.0 and 1.5, with a material factor between 0.5 and 1.0. Seismic peak ground acceleration values for input into retaining wall global stability checks have been determined. An acceleration coefficient of 0.08 was adopted for seismic stability assessment.<br>A traffic surcharge of 20 kPa has been applied at the carriageway locations in accordance with bridge design standard requirements.<br>The RE wall is expected to be maximum height of 13.6 m. It has assumed the ratio of reinforcement length/retaining height of approximately 0.7-0.9. |
| **Global Stability Checks** | Static with a long-term factor of safety (FoS) of 1.50.<br>The global stability checks were undertaken using Rocscience's Slide 7.0 software. The RE wall were treated as a block with sufficient internal stability. Only long-term stability was assessed. No stability checks were completed for short-term and temporary construction batters. These checks would be required during detailed design. |
| **External Stability Checks** | The external stability check includes assessing sliding along the retaining base, overturning and the foundation bearing capacity. The analysis was completed using the program MSEW (Version 3.1) and External Stability of RE Wall (Version 1.0) in accordance with AASHTO 2007–2010. |
| **Geotechnical Assessment Results** | The results of this analysis indicate that a FoS of greater than 1.5 for long term loading conditions is likely to be achieved for global stability. The sliding and overturning stabilities are satisfactory under combined loading condition as above. |

### *Design Output*

Ground improvement may be required to achieve the design bearing capacities for the proposed RE wall for the bridge abutments. Using compacted granular materials with/without geogrid has been considered for ground improvement for this project.

The design output for each abutment RE wall are summarised in Table 22 including proposed ground improvements.

Table 21: RE wall design output summary

| Wall Location | Wall height (m) | Minimum steel strap length (m) | Estimated ground pressure (kPa) | Founding material / depth (m) | Ground improvement |
|---|---|---|---|---|---|
| Rail Bridge No 1 | 8.7-9.9 | 7-9 | 225-250 | St Clay / 1.2-1.6 | Ground replacement with compacted engineering fill reinforced with geogrid to founding depth |
| Rail Bridge No 2– Northern west abutment | 8.5 | 7 | 220 | St-VSt Clay / 1.0 | Ground replacement with compacted engineering fill to founding depth |
| Rail Bridge No 2– Southern east abutment | 8.5 | 8 | 210 | St Clay / 1.2 | Ground replacement with compacted engineering fill reinforced with geogrid to founding depth |
| South Ramp Underpass Bridge | 12.2 | 9 | 325 | St Clay / 1.7 | Ground replacement with compacted engineering fill reinforced with geogrid to founding depth |
| West Ramp Underpass Bridge | 13.6 | 12 | 340 | MD Silty Sand / 1.5 | Ground replacement with compacted engineering fill reinforced with geogrid to founding depth<br>foundation to be compacted<br>large amount of settlement is expected after wall construction, pre-loading may be required |
| North Underpass Bridge | 12.3 | 10 | 320 | XW Rock / 1.0 | NIL |
| West Overpass Bridge | 8.6 | 7 | 220 | St-VSt Clay / 1.2 | Ground replacement with compacted engineering fill reinforced with geogrid to founding depth |